The International Levee Handbook

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The International Levee Handbook

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USA Disclaimer

Foreword

Levees, otherwise known as flood embankments or dikes, are a vital part of modern flood risk management. Many of our towns and cities would be uninhabitable without them. Most countries have many existing levees in their river and coastal systems. It is estimated that there are several hundreds of thousands of kilometres of levees in Europe and the USA alone. The maintenance of these levees in both normal and flood conditions is a major task for flood management authorities. Levees are maintained and improved and new levees are built. Yet many of the techniques used do not necessarily take full advantage of the experience developed in other countries. Only by sharing knowledge internationally can we ensure the most efficient, effective and environmentally-sensitive work programmes.

Our national governments realised that there was the need to sponsor the production of a single reference source on good practice in the management and design of levees, drawing on the skills found across Europe and in the USA. The production of this new handbook is very appropriate, and is the fruits of collaboration between the USA, France and the UK, with additional support from Ireland, the Netherlands and Germany.

This handbook is more than a revision or combination of existing documents within participating countries. It represents more than five years work by an international team of experts supported by an international peer review process. The team has put together an extensive handbook on the safety assessment, management, design and construction of levees, which incorporates all the main elements of good practice. While the handbook is not prescriptive it is our belief that appropriate application of the guidance in this handbook will help to underpin long-term improvements in the management and design of levees and will help to promote conservation of natural systems in balance with the proper protection of human life and property.

We have pleasure in presenting this handbook to everyone involved in managing levees and commissioning new levees on which many communities across the world rely to protect them from flooding.

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Contents

1 Introduction

Courtesy BeeldbankVenW.nl, Rijkswaterstaat

CHAPTER 1 CONTENTS

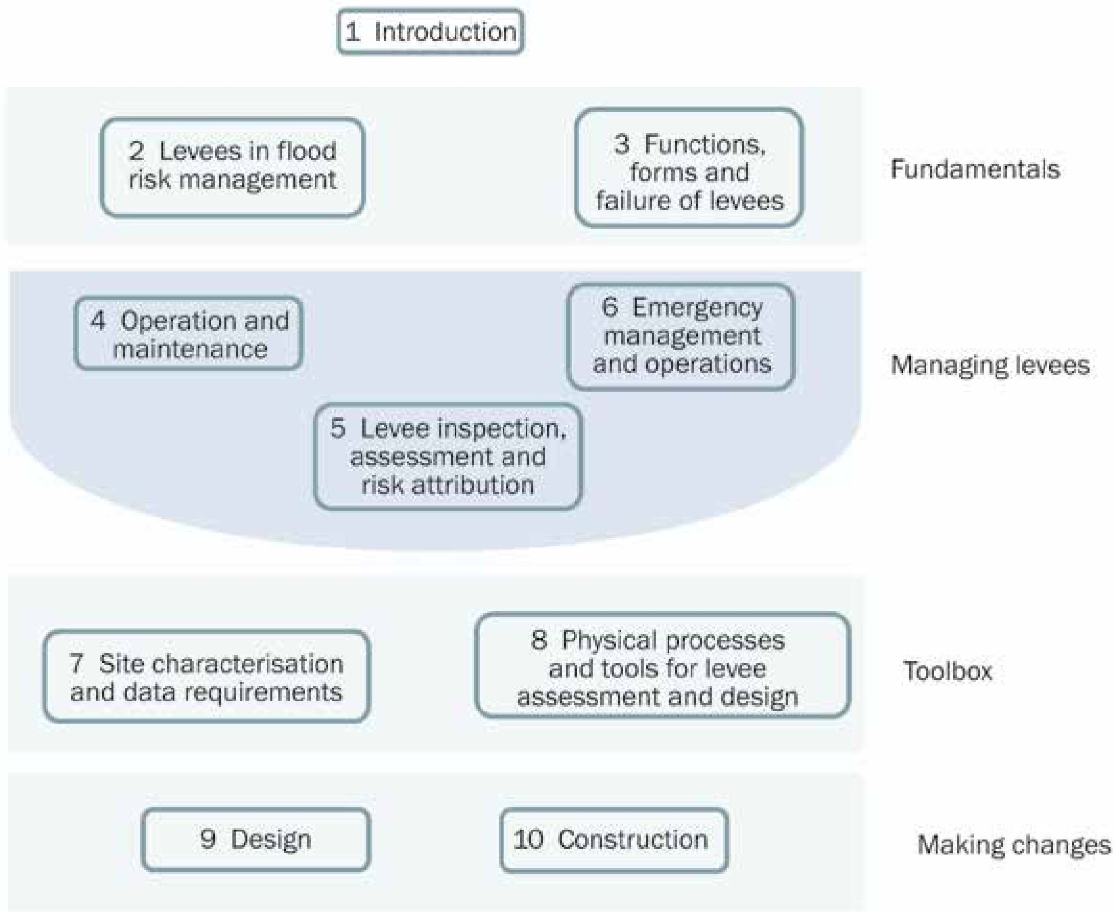
1 INTRODUCTION

Chapter 1 introduces the handbook and presents the motivation and process that led to its development. The chapter gives the reader an overview of its contents and explains how to use it.

The chapter flow chart shows the conceptual links between the technical chapters that follow this introduction. It is repeated at the start of each chapter but is expanded to show more detail of the contents of that chapter. It indicates that the handbook is split into four major parts:

- **fundamentals** – setting out what all users need to appreciate
- **managing levees** – focusing on what managers of existing levees need to know
- **toolbox** – providing detailed technical information (data equations etc) for use by all users
- **making changes** – focusing on the needs of those involved in design and construction of new or improved levees.

This flow chart shows where to find information in the chapter and how it relates to other chapters. Use it in combination with the contents page to navigate the handbook.



1.1 USE OF LEVEES

Levees are raised, predominantly earth, structures (also called dikes, digues or flood defence embankments) that are not reshaped under normal conditions by the action of waves and currents, whose primary objective is to provide protection against fluvial and coastal flood events along coasts, rivers and artificial waterways (Figure 1.1).

Levees form part of flood defence systems that may also include flood walls, pumping stations, gates closure structures, natural features, and other associated structures. In many instances levees have been built up and extended over decades or sometimes centuries. Few of these were originally designed or constructed to modern standards and records of their construction and historical performance may not exist.

Figure 1.1 Typical riverine levee (a) (courtesy USACE) and typical coastal levee (b) (courtesy STOWA)

Despite their apparent simplicity, levees can be surprisingly complex structures. They have generally been constructed by placing locally won fill material onto alluvial flood plains (with all their inherent natural variability). Unlike engineered structures, levees can be irregular in the standard and nature of their construction and can deteriorate markedly over time if they are not well maintained. Furthermore, levees are generally long linear structures that are part of an overall system. Such systems should be considered as chains that are only as strong as the weakest link.

Evidence-based assessment, good design, effective adaptation, good inspection and routine maintenance are vital if levees (particularly those representing the weakest parts of levee systems) are to perform well on the occasions when they are loaded in storm or flood events. It should be noted that levees may stand for much of their lives without being loaded to their design capacity. This can create a false sense of security in the level of protection they will provide.

1.2 BACKGROUND TO THE HANDBOOK

Coastal and riverine flooding continues to produce devastating consequences, in both life and economic losses, around the world. With economic growth, urbanisation and the ensuing concentration of population and property, people are moving in increasing numbers to flood-prone areas in many countries. Where flood protection defenses have been improved, have not been fully tested, or experience infrequent flooding, residents become complacent and less aware of the threat of floods. In such cases, they are hardly prepared for floods and by no means assured of proper actions to take, consequently suffering more serious damage once a flood occurs.

Flood and storm events around the world continue to lead to critical flood defence failures resulting in tragic losses of life and the devastation of large areas. Also, levees have been severely tested by exceptional rainfall events. However, despite their critical importance in mitigating flood risk, interest and investment in levees has tended to be lower than in other critical water retaining infrastructure such as dams. In particular, in many countries, levees have lacked the legal and technical framework necessary to promote an appropriate level of performance.

In September 2008, organisations from six countries (France, Germany, Ireland, the Netherlands, United Kingdom, and the United States of America) expressed a desire in principle to participate in an international project to learn from one another's experiences and to share the effort to produce good practice guidance – The International Levee Handbook (ILH). That desire resulted in several international meetings and workshops, development of a scoping report and ultimately culminating in this handbook.

The principal objective of the handbook is to provide a comprehensive and definitive guide to good practice in the evaluation, design, implementation, maintenance and management of levees. The handbook is a non-prescriptive reference and should be used in conjunction with other relevant national and international codes and manuals. It is not intended to be a prescriptive code of practice for decision making but should be regarded as an important document in decision support and for reference in the application of international codes and manuals.

The handbook has been written by a core team of experts and practitioners from the full range of relevant disciplines drawn from the partner countries. The development of the handbook followed an agreed set of processes that was managed by a technical editorial team, and supported by an executive steering board drawn from national backing groups of the partner countries. Management support was provided by CIRIA (UK) who also prepared the resulting document for final publication. The document was made available to a broader international audience for review and comment during the development process.

1.3 SCOPE

The handbook takes a risk, performance and systems based approach. Any levee will have a primary function of flood management or coastal defence to which performance objectives or standards will apply. All levees will also have various secondary functions, eg environmental, amenity, health and safety, access, which can impose significant performance requirements. The handbook also follows a tiered approach to all aspects of managing and maintaining a levee or levee system such that concepts are applicable to levees in both urban and rural settings.

In drafting the handbook, the author teams considered the various management interventions that are needed to achieve the performance requirements of the levee or levee system over its whole life cycle. So the handbook addresses the assessment of existing riverine, coastal and estuarine flood protection levees (possibly for new or changed performance requirements), their adaptation or replacement, their operation and maintenance, as well as new design and removal. Consideration is also given to the fact that management interventions range from major construction projects carried out by external constructors through to routine maintenance by the involved authorities' own work force.

The handbook does not address levees constructed for purposes other than flood protection. Also, it does not cover the design of other water retaining structures. Associated structures are addressed because they influence the performance of a levee structure or its operation. The handbook also recognises the importance of structures that stabilise levees by managing riverine and coastal morphology such as beaches, dunes and groynes. Where necessary, reference to other management guidance is given for such structures.

1.4 STRUCTURE OF THE HANDBOOK

The handbook contains information that is useful for both existing and newly designed levees, however the structure of the handbook is such that existing levees are treated first followed by newly designed levees. Details about each chapter are presented in the rest of this section. Figure 1.2 presents a high level view of the handbook showing how each chapter contributes information to understanding a levee system as presented by the source-pathway-receptor conceptual model.

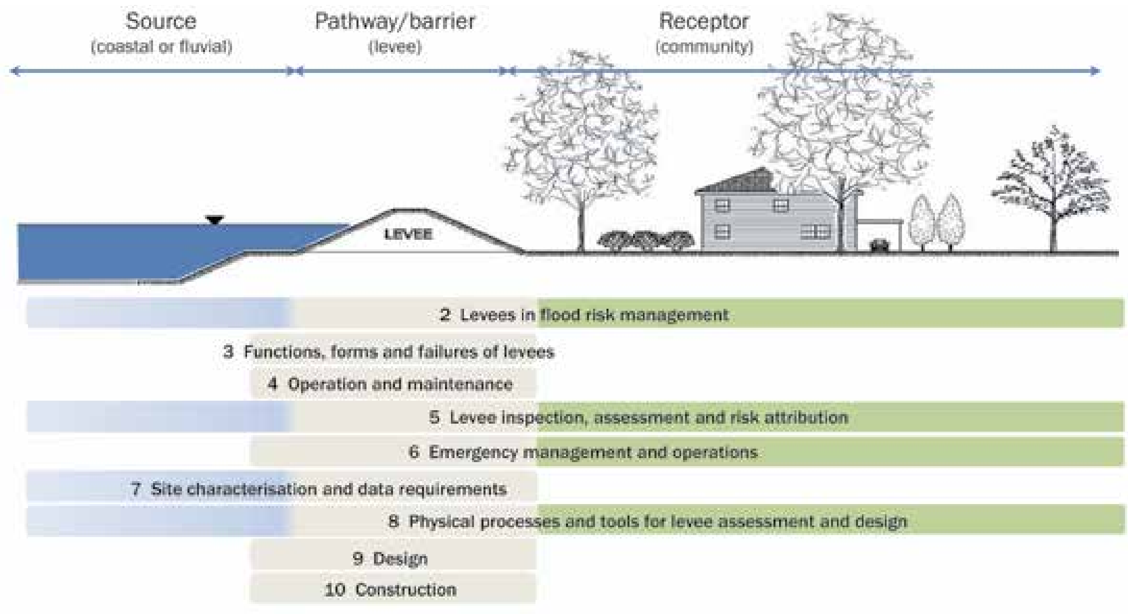


Figure 1.2 Illustration of chapter contents relative to the source-pathway-receptor conceptual model

1.4.1 Use of the handbook

The following features are designed to assist the reader in navigating the handbook:

- **diagram of general structure:** Table 1.1 provides a layout of the structure and contents of the complete handbook. It also suggests a relationship between the main phases of a typical project
- **diagram of content relevance to different users:** Table 1.2 presents an evaluation of the content from different users' perspectives to assist the reader in finding information relevant to their needs
- **high-level contents list:** this is given for the complete handbook at the start of the book
- **detailed contents list:** at the start of each chapter there is a contents list for that chapter only
- **structure of each chapter:** the front end of each chapter includes a detailed contents list for that chapter, an introductory box that describes what is included in that chapter, and a flow chart to demonstrate links with other chapters
- **where am I?** each page tells the reader their current location in the handbook. The chapter number is shown on the navigation bar running down the outer edge of right-hand pages, and the chapter title is given on the page header
- **electronic version:** the complete manual is available to downlaod from CIRIA's website: www.ciria.org

Table 1.1 *Structure and content of the handbook*

	Chapters	Description
Fundamentals	***Chapter 2*** Levees in flood risk management	Sets out the context of flood risk management in which levees and levee management should be seen, discussing the influence of environmental change. It explains roles and responsibilities in flood risk and levee management and why it is important to manage levees throughout their life cycle.
	Chapter 3 Function, form and failure of levees	Introduces an overview of levee functions within a flood risk management system and the multi-functionality of levees. It describes and illustrates the main types of forms of levees and presents the main structures associated with levees. The chapter concludes with a discussion of the processes of levee failure and how these are connected to the forms and functions of levees.
Managing levees	***Chapter 4*** Operation and maintenance	Addresses both operational and maintenance aspects of managing existing levees, including organisational aspects and the management of encroachments and vegetation. Maintenance requirements are described and related to the identification and resolution of defects arising from various deterioration and damaging mechanisms
	Chapter 5 Levee inspection, assessment and risk attribution	Presents levee assessment-related activities and their integration. The chapter provides a tiered approach to the assessment of levee systems including risk analysis, assessment and inspection. Data collection methods and related issues are described including inspections, investigations and monitoring. Data management systems to support levee management are also discussed given the importance of data availability and treatment for assessment activities.
	Chapter 6 Emergency management and operations	Sets out the principles of emergency management detailing preparedness and response and how the management of levees relates to the wider picture. It describes the various emergency intervention techniques including equipment and activities for minimising levee overtopping and damage and for subsequent repair and closure of breaches.
Toolbox	***Chapter 7*** Site characterisation and data requirements	Having described the basic principles of site characterisation for levees and their environment, the majority of the chapter is focused on giving detailed investigation and analysis techniques to establish the hydraulic and geotechnical boundary conditions at levees and also the condition of existing levees. It provides relevant equations and techniques for assessing the hydraulic and morphological conditions. It describes desk study procedures, intrusive and non-intrusive techniques for sampling and field investigation of geotechnical properties as well as relevant laboratory testing techniques and approaches to data interpretation that are suited to levees and the ground that levees are built on. The chapter also explains methods and procedures for determining appropriate parameters for design.
	Chapter 8 Physical processes and tools for levee assessment and design	Provides the engineering and scientific tools for the analysis and design of existing and new levees, embracing both geotechnical and hydraulic engineering disciplines. It details external and internal hydraulic, geotechnical and seismic actions on levees, sets out the physical processes that control the performance of levees, their protection systems, and associated floodwalls and indicates the analytical engineering methods and techniques (from simple equations to numerical techniques and modelling) that best represent the relevant mechanisms. The chapter concludes with a description of methods of assessing levee breach and subsequent inundation.
Making changes	***Chapter 9*** Design	Sets out principles of levee design, roles and responsibilities of those involved in design and the required reports and documentation. The chapter then explains how to determine levee layout and alignment and levee crest levels and geometry. Information on design calculations and detailing including methods of analysing failure mechanisms according to various codes of practice are included with further details on the specifics of design for seepage and internal erosion, surface protection measures and for limiting serviceability changes. The chapter concludes with advice on earthworks materials selection and compaction, and the design of spillways and of the levee earthworks around embedded/associated structures, including crest walls and pipes.
	Chapter 10 Construction	Describes levee preparation for construction concerning organisational aspects, programming and the management of construction risk. It focuses on the specifics of earthworks including the suitability of the soils, their treatment and handling. The stages of construction for earthworks are described for new build, adaptation, repair and decommissioning. The incorporation of non-earthworks structures is also discussed.

1.5 TARGET READERSHIP

Potential users of the handbook include planners, developers, structure owners, asset managers, regulatory bodies engineers, risk analysts, designers, constructors, emergency planners and responders, environmental organisations, educational institutions and the public.

The handbook is written to assist a technically competent practitioner with a broad (but not necessarily expert) knowledge of the field of application to arrive at the best approach for a particular levee or levee system. In this regard the handbook aims to provide information to support decision making rather than to direct it. The handbook will also seek to provide the intelligent client (ie a client with a technical background, but no particular specialist knowledge) with sufficient background information to understand the main issues and general procedures likely to be followed by an experienced practitioner.

The handbook has been written to address two major viewpoints:

1. **The manager** of the operating authority's physical structures who has the overall task of owning, maintaining, upgrading, adding to and disposing of its stock of flood or coastal levees.
2. **The designer** who will tend to focus on the need for, design of and implementation of improvements and new works.

In addition, the handbook provides some useful information for constructors (or other organisations) that may be advising the manager or designer carrying out maintenance, or carrying out new construction work.

Table 1.2 Relevance of chapters for different stakeholders and users

Stakeholder/user	Chapter								
	2 Levees in flood risk management	3 Functions, forms, failures of levees	4 Operation and maintenance	5 Levee inspection, assessment and risk attribution	6 Emergency management and operations	7 Site characterisation and data requirements	8 Physical processes and tools for levee assessment and design	9 Design	10 Construction
Planner	★	★	○	○	★	○	○	*	★
Developer	*	★	*	●	○	●	○	●	*
Structure owner	*	★	★	★	★	●	●	○	★
Asset manager	*	★	★	★	★	●	*	*	*
Regulatory body	*	*	●	★	*	○	○	○	★
Geotechnical engineer	○	★	★	★	★	★	★	★	★
Hydraulics engineer	○	★	★	★	★	★	★	★	★
Risk analyst	*	★	○	★	●	*	*	○	○
Designers	*	★	*	★	○	★	*	★	★
Constructor	●	★	*	○	●	●	●	*	★
Emergency planners and responders	●	★	*	★	★	○	○	○	*
Environmental organisation	*	*	★	○	○	○	○	○	★
Educational institution	*	*	*	*	*	*	*	*	*

Note

The relevance of material to each stakeholder or user group is indicated by the following symbols:

★	High	●	Medium-low
*	Medium-high	○	Low

2 Levees in flood risk management

Courtesy Bart van Eyck, Rijkswaterstaat

CHAPTER 2 CONTENTS

2 LEVEES IN FLOOD RISK MANAGEMENT

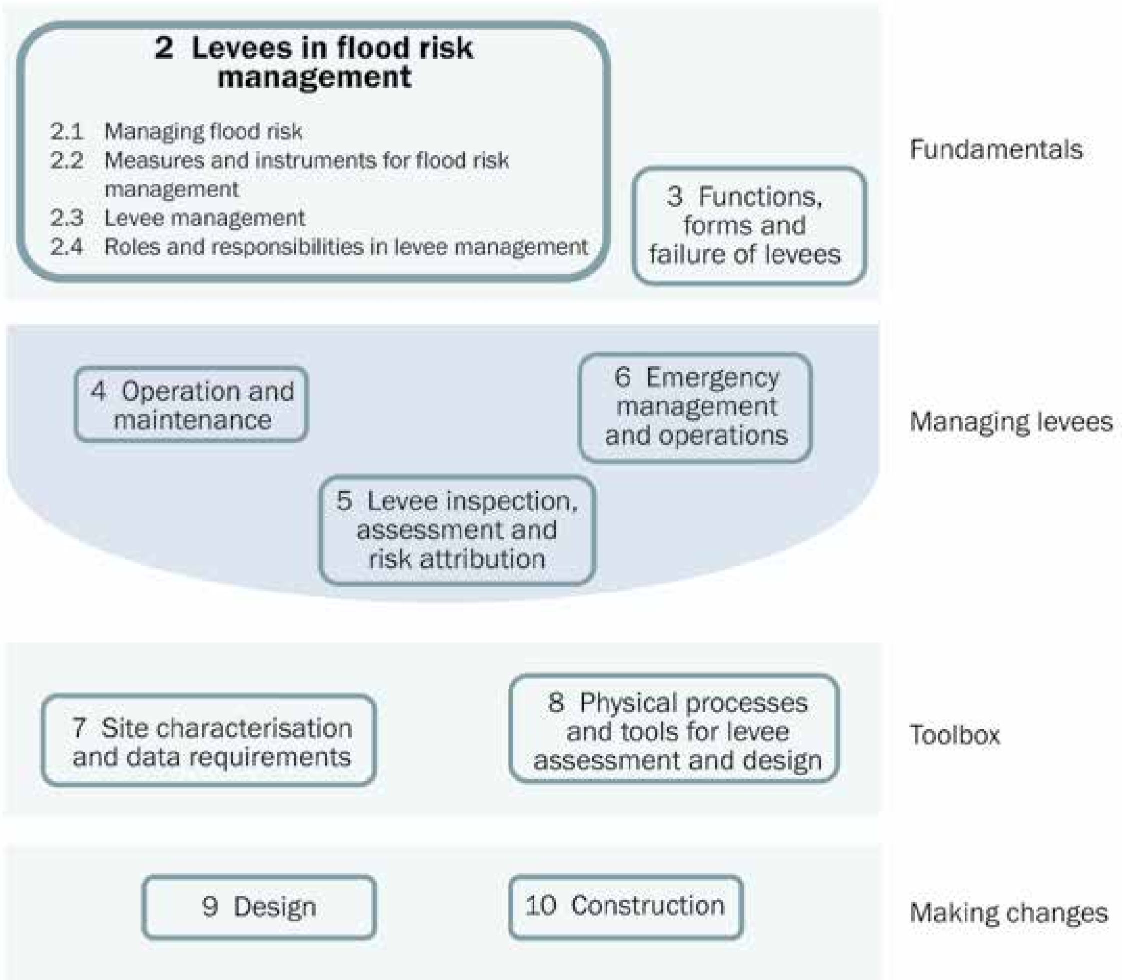
Chapter 2 sets levees in the wider context of flood risk management. General principles for levee management for use throughout the handbook are introduced.

Key outputs to other chapters

- starting point ⇨ **levee management**
- **conceptual and risk management frameworks** ⇨ all chapters

Note: The reader should revisit **Chapters 2 and 3** throughout the levee life cycle for a reminder of important issues.

This flow chart shows where to find information in the chapter and how it relates to other chapters. Use it in combination with the contents page to navigate the handbook.



CHAPTER CONTENTS AND TARGET USERS

This chapter consists of four sections, providing an overview of the management of flood risk systems and levees, and the basic roles and responsibilities of those organisations and individuals involved. All users of this handbook are recommended to read Chapter 2 before continuing to subsequent chapters as it provides an overview of key principles and concepts behind the detailed levee management and design information contained in the remainder of the handbook. As well as introducing the subject, it helps the reader (by suitable cross references) to identify the chapters in the handbook that are most relevant for their requirements.

Managing flood risk

The principles of risk and flood risk management are explained in Section 2.1, setting these in the context of the wider environment. Generic frameworks are illustrated and described for flood systems, flood risk management processes, and flood risk identification, analysis and assessment. Causes of, and responses to, changes in flood risk are also discussed.

Measures and instruments for flood risk management

In the context of developing flood risk management policy, Section 2.2 discusses structural and non-structural flood risk reduction measures, financial and regulatory instruments, and formulating portfolios of options for reducing flood risk.

Levee management

Section 2.3 explains how performance objectives and safety standards for levees are used to deliver flood risk mitigation policy aims and objectives. A framework for the life cycle of a levee is defined. This section also introduces approaches to levee asset management, including assessments and reviews of levee performance and reliability, and techniques for failure analysis.

Roles and responsibilities in flood risk management

Section 2.4 discusses the roles and responsibilities of those involved in managing levees and levee systems, including authorities, regulators, managers, designers, engineers. The importance and means of communication of risks to the wider public and local community is also discussed along with the importance of educating and empowering local communities.

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2.1 MANAGING FLOOD RISK

Flooding is a worldwide phenomenon. Over the last few decades the world has experienced a rising number of devastating flood events. The trend in such natural disasters is increasing. Also, escalations in both the probability and magnitude of flood hazards are expected in places as a result of climate change. Past disasters have triggered many governments to embark on *flood risk management* initiatives, such as flood control schemes (including levees), early warning systems and evacuation planning, with the ultimate aim of defending their inhabitants from the vagaries of nature.

Although this handbook is primarily about the management of levees for flood risk reduction it is appropriate for this chapter to place levees in the wider context of flood risk management. The management of levees must be seen alongside a broader range of activities such as land use planning and emergency preparedness that may help to reduce flood risk. Also, levees do not normally provide flood control on their own. In any particular location on a river, estuary or coast, an individual levee segment will work together with other levees, structures and flood risk reduction measures. This interrelationship of structures and activities is often referred to as a flood risk management system. Sections 2.1 and 2.2 of this chapter provide this broader systems context. Sections 2.3 and 2.4 then move on to more specific material about levee management and communication.

2.1.1 Flood management systems

2.1.1.1 Environmental context of managing flood risk

The structures and components of a flood management system including levees are constructed and managed within an existing environment. Appropriate consideration of the environmental characteristics in which levees are located is central to the satisfactory and sustainable design, construction and/or operation of flood management systems. Such considerations include:

- ecosystems, habitats and species
- geology, ground conditions and foundations
- geomorphological processes and waterway navigation
- hydrology and hydraulic loading on the system
- land use, occupancy, transport, critical infrastructure, and agriculture
- availability of materials
- amenity, access and public safety
- land drainage and flood water storage
- the effects of seasonal climate variability and of climate change
- the effects of human development.

Conflicting interests may need to be taken into account in achieving a solution that is appropriate to the particular location. Section 3.1 discusses these issues in further detail as they relate to levees.

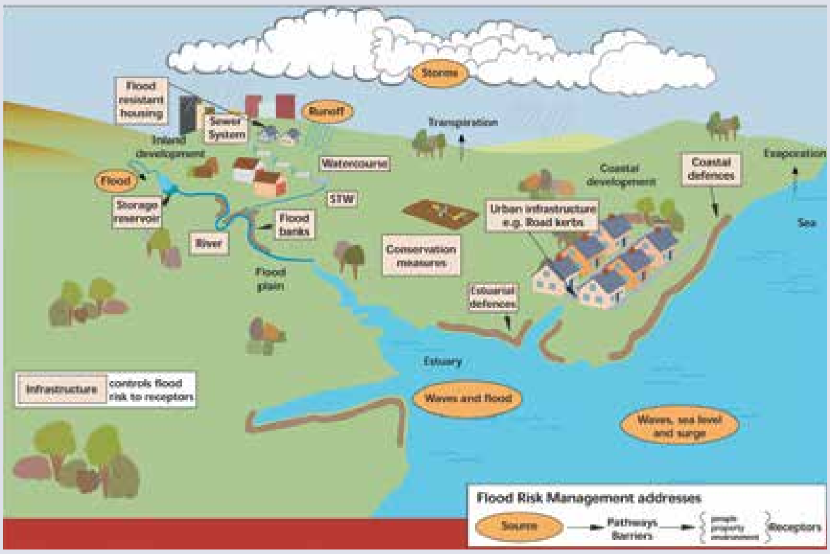
2.1.1.2 Sources, pathways and receptors

Floods, whether small or large, may be considered to be part of the natural behaviour of rivers, lakes, estuaries and the sea. But they may cause harm to society, and that is where the term hazard comes in. A hazard is a physical event or human activity with the potential to result in harm to people and damage to goods and property. In flood risk management, the interest is only in floods that constitute a hazard. A hazard does not automatically lead to a harmful outcome, but identification of a hazard does mean that there is a possibility of harm (or adverse consequences) occurring.

Floods are a complex phenomenon and consist of different *sources* (sea, rivers, lakes etc) of water, and *pathways* through which the flood can impact various types of *receptors*. The complexity of flooding can be illustrated using the source-pathway-receptor model shown in Box 2.1. For a risk to arise (Figure 2.1) there must be:

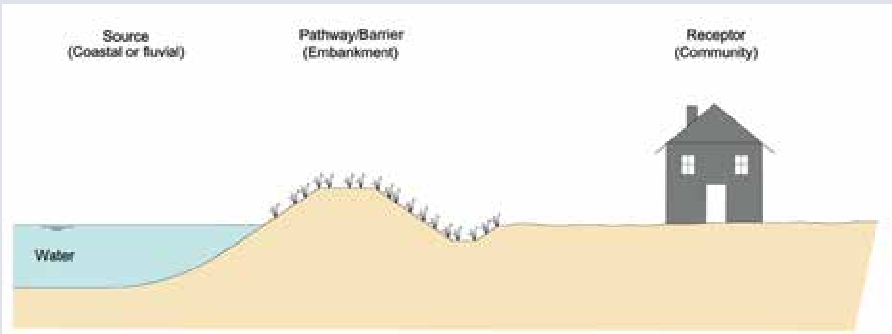
- a hazard that consists of a source or initiator event (ie high rainfall)
- a receptor (eg floodplain, people and properties)
- a pathway between the source and the receptor (ie flood routes including through, over or around flood control structures and the routes by which water spreads in the floodplain).

Box 2.1 The flooding system and the source-pathway-receptor framework



Note
STW = sewage treatment works

Figure 2.1 The flooding system (courtesy M Bramley)



Source	→	Pathway	→	Receptor
eg rainfall, wind, waves, excessive runoff, upstream releases		*eg overtopping, overflow, breach, flood plain indundation*		*eg people, property, environment*

Figure 2.2 Source-pathway-receptor conceptual model (from Morris et al, 2007)

- **sources:** typically, hydraulic loadings that impinge on defences including river levels, flows, waves, tidal and surge water levels and their associated probability of occurrence (singular or jointly) and that depend on driving factors such as meteorology (including wind) and hydrological and hydraulic factors that determine the patterns and volume of rainfall and runoff
- **pathways and barriers:** the behaviour under loading from sources of defences (depending on their nature, extent and condition) and of floodplains (depending on topography and land use)
- **receptors:** the exposure and vulnerability of the people, property and environmental features that may be harmed by a flood.

2.1.2 What is flood risk management?

Given the source-pathway-receptor framework, flood risk management can be seen to be about reducing the probability of floods of a particular severity occurring and/or reducing the magnitude of the impacts should flooding occur. The associated definition of flood risk is given in Box 2.2.

Box 2.2 ***The definition of flood risk***

Flood risk is a function of the ***probability*** of both the flood occurring and of any associated breaches in the flood defence system and the ***consequence*** within the leveed area of the undesirable outcome should a flood event occur (loss of lives, habitat and economic losses due to damages to goods and property). So:

Risk = fn (probability, consequence)

In flood risk management this is normally simplified to:

Risk = probability × consequence

An estimate of the total flood risk will normally require some form of integration across all possible flood events, breaches and consequences (see Section 5.2).

Figure 2.3 shows the relationship between the factors that influence flood risk and probability and consequences.

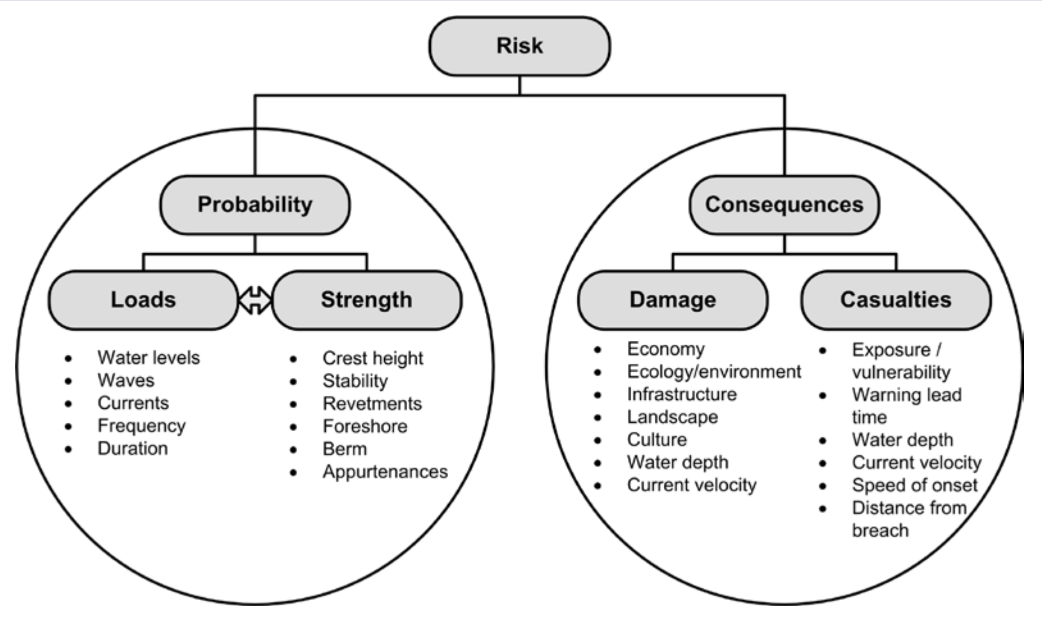


Figure 2.3 ***Governing factors that influence flood risk (courtesy H A Schelfhout)***

All of these factors change over time. For example:

- hydraulic loads will change as the climate changes, with perhaps higher or more erratic rainfall patterns
- levees can become weaker as their structure deteriorates and materials weaken due to climatic variation
- an accidental incident may damage a levee section at some point, reducing its strength
- land use in the area behind a levee may change over time, such as becoming more or less occupied or developed, or changing from agricultural land to industrial use or even residential housing.

Such changes can occur over a wide range of timescales (see Section 2.1.4).

2.1.2.1 The flood risk management process

As depicted in Figure 2.4, the process of flood risk management is about a sequence of first identifying flood risk, then assessing the level of risk, and finally about creating policies and plans to control the risk and to reduce it to an 'acceptable' level.

Figure 2.4 The process of flood risk identification, assessment and control

The process of flood risk management may be characterised in more detail as successively embracing (Figure 2.5) the following:

- **risk identification:** the process of recognising and recording risks
- **flood risk analysis:** which depends on risk identification and estimation, using current tools and methodologies to analyse and combine the likelihood and consequences of flooding (including, for estimates of future flood risk, allowances for future climate change, deterioration and socio-economic change. See Section 5.2)
- **flood risk assessment:** which involves an evaluation of the significance of the risk, an analysis of cost–benefit and formulation of recommendations through options appraisal
- **flood risk treatment:** which involves the combined use of policy and planning instruments (including preventative, control and mitigation) as well as decision making on all aspects of safety with design, implementation and maintenance of structural measures.

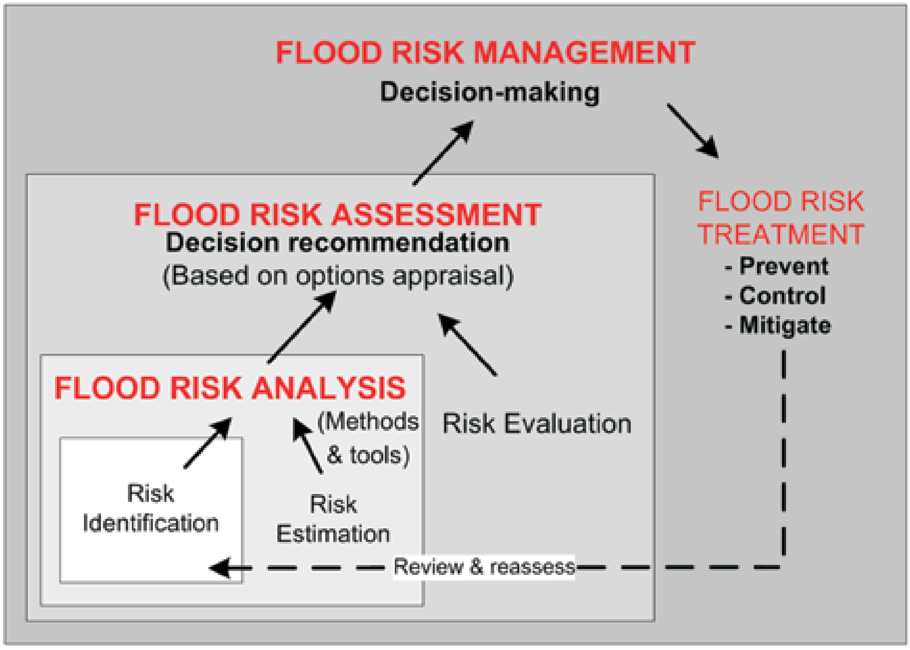


Figure 2.5 Illustration of an approach to flood risk management (from Bowles et al, 1999)

The process in Figure 2.5 is exemplified within the EU Floods Directive 2007, which has successive requirements for European Union (EU) nation states to prepare preliminary flood risk assessments, flood risk maps, and finally flood risk management plans.

Although different countries have the same overall aims for flood risk management and follow the basic process set out in Figure 2.5, the detail of their approaches and the way they are described do vary. For example, the Netherlands uses the 'safety chain' concept (shown in Box 2.3) to describe and evaluate flood risk management policies, and to link it in with the overall approach set out in the EU Floods Directive 2007.

Box 2.3 *A multi-layer safety approach to flood risk management in the Netherlands*

Layer 3: disaster control and evacuation

Layer 2: durable spatial arrangement and flood resistant buildings

Layer 1: prevention of floods by reduction of probabilities and consequences

Figure 2.6

Example of a safety chain approach to flood risk management (from Kolen et al, 2010)

The safety chain (EU Floods Directive 2007)

- **recovery and lessons learned:** returning to normal conditions as soon as possible and mitigating both the social and economic impacts on the affected population
- **emergency response:** developing emergency response plans in the case of a flood including training of operational staff in event and breach management
- **preparedness:** informing the population about flood risks and what to do in the event of a flood. Training operational staff
- **protection:** taking measures, both structural and non-structural, to reduce the likelihood of floods and/or the impact of floods in a specific location
- **prevention:** preventing damage caused by floods by avoiding construction of houses and industries in present and future flood-prone areas, by adapting future developments to the risk of flooding, and by promoting appropriate land use, agricultural and forestry practices.

2.1.2.2 Frameworks for flood risk management

An effective framework for flood risk management will provide a structure within a nation, state or organisation in which policies, processes and procedures should reside and operate. It should include an integrated programme that covers the entire process previously described in Section 2.1.2.1 and also has provision for:

- the context and objectives of the organisation
- the selection of appropriate methods and techniques for flood risk assessment
- decision making on the extent and type of flood risks that are tolerable, and how unacceptable flood risks are to be treated
- identifying and analysing appropriate environmental information and any significant environmental impacts.

As part of this framework, an organisation responsible for managing flood risks should be clear about how the flood risk management relates to its responsibilities, authorities and accountabilities and how it integrates into its organisational processes, including:

- the resources available to conduct the assessments and prepare and implement the management plans
- how the flood risk assessments, maps and management plans will be reported, reviewed, recorded and communicated
- procedures for making the business case for resources for selected interventions to reduce risks
- procedures for managing environmental incidents and ensuring legal compliance (eg environmental permitting).

Issues of organisation and communication are discussed in more detail in Section 2.4.

2.1.2.3 The approach to decision making

Decisions in flood risk management should seek to balance the competing and complementary factors that affect flood risk. Where possible, interventions should be adopted that provide the largest reduction in flood risk for the least societal cost (economic, environmental and social). This balancing act often requires the use of a decision support system (DSS) that highlights different strategic alternatives, and helps to assess their costs and benefits across a range of possible futures (or planning scenarios). However, it must be remembered that although flood risk may be reduced by such an approach, it can never be removed completely. The remaining, or residual, flood risk, needs to be addressed through emergency management processes and procedures (see Section 6.1).

2.1.3 Flood risk management process

This section describes in more detail the flood risk management process set out in Figure 2.5.

2.1.3.1 Identifying flood risks

In the context of this handbook, flood risk (see Box 2.2) results from threats associated with the sea, rivers, lakes and loads to flood risk mitigation structures such as levees. Such threats include:

- very high water levels (including those arising from rapid onset or flash, and long duration floods)
- extreme wave attack
- strong currents.

Risk identification is the process of recognising and recording the risks arising from such threats. The purpose of risk identification is to identify what *might* happen or what situations might arise (often referred to as scenarios) that might affect particular receptors. This process should identify the causes and source(s) of the risk events, situations or circumstances that would have a material impact upon human lives, the environment and the local economy.

The following list gives examples of some of the factors or characteristics that could be included in flood risk identification:

- flood loading conditions (hydro-meteorological events) and their probabilities
- probability of flood inundation without a levee breach (ie loading event overflows or overtops the levee crest)
- levee condition and its probability of failure under load (ie levee reliability)
- characteristics of the floodplain and inundation (depth, velocity, geographical extent etc)
- nature, extent and vulnerability of the receptors (human, environmental, economic) to inundation
- existing risk control mechanisms and measures, and their effectiveness (eg emergency response)
- uncertainty in data and knowledge about the above factors.

To determine these characteristics, knowledge of previous flooding incidents may be used. However, for rare events this may not suffice and, in any case, the circumstances (eg the activities in the floodplain area) may have changed. So, it is necessary to use appropriate predictive calculations to assess the probabilities and magnitudes of all possible floods.

2.1.3.2 Flood risk analysis

Flood risk analysis allows all known contributory factors that make an area vulnerable to inundation to be brought together. It also enables consistent assessment and comparison of potential interventions to influence, control or reduce flood risk. The scope of a levee flood risk analysis should be commensurate with the needs of the decision being informed by the process.

Risk analysis methods are scalable and can vary in their approaches between countries and in their level of effort, detail, and certainty (accuracy). Factors that influence the selection of risk analysis techniques and methods can be described in terms of:

1 2 3 4 5 6 7 8 9 10

- the complexity of the problem and the methods needed to analyse it
- the nature and degree of uncertainty of the risk analysis based on the amount of information available and what is required to satisfy objectives
- the extent of resources required in terms of time and level of expertise, data needs or cost
- whether the method is required to provide quantitative or qualitative output.

The overall degree of uncertainty in the analysis can be determined by combining assessments of uncertainty in different parts of the source-pathway-receptor system. In this handbook, uncertainty is discussed in the following contexts:

- issues in analytical tools (Sections 7.3.13 and 8.11.6)
- risk assessment (Section 5.2.2)
- design (Section 9.5 and 9.10) during construction (Section 10.2).

2.1.3.3 The tiered approach to flood risk analysis

A tiered approach is a risk-based approach in which, following a preliminary risk assessment, the amount of effort put into further investigation and analysis is adjusted according to the severity of the problem and the magnitude of the consequences of failure. As flood risk analysis can be time-consuming and expensive, implementation of a tiered approach can save time and money. This tiered approach can be used in all aspects of the source-pathway-receptor framework (Figure 2.2), with the effort used at each stage to assess each aspect of flood risk being proportionate to its relative importance. An example characterisation is shown in Figure 2.7.

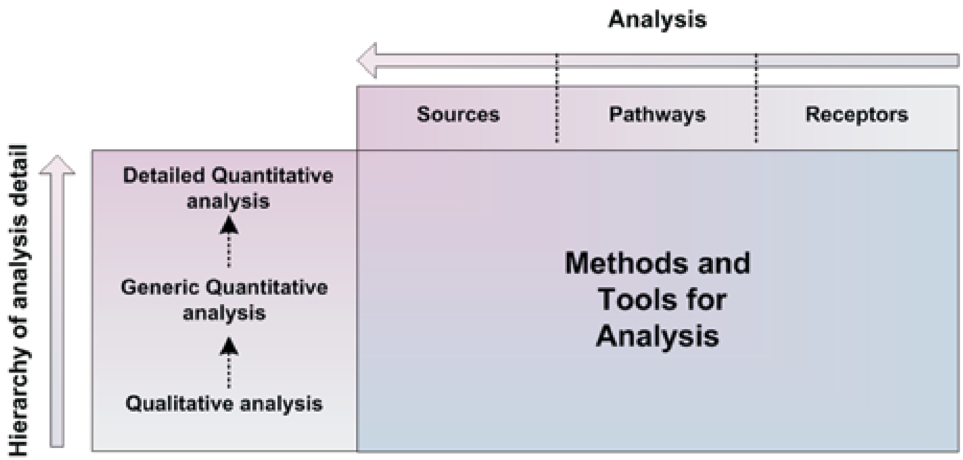


Figure 2.7 An example of a tiered approach to risk analysis (UK approach)

The concept of a tiered approach where the level of detail and associated effort is related to the level of risk can be applied to all aspects of flood management, not just risk analysis.

2.1.3.4 Flood risk evaluation

Risk cannot be entirely eliminated. The flood risk analysis of each possible intervention will determine residual risks, which then need to be evaluated in terms of how acceptable or tolerable they will be to stakeholders. The risk evaluation provides an opportunity to manage levees and flood risk using a framework that is common to all major hazards. Even though there is no one measure of what is 'tolerable', the evaluation stage does allow societal, regulatory, legal, owner and other values and judgments to enter the management and decision making process.

This evaluation should be conducted before comparing and selecting potential measures and instruments (Section 2.2) to reduce flood risks. Formal risk evaluation is not necessary after every periodical risk analysis, but if conducted, it should ideally be performed by an independent team, rather than the one who conducted the analysis.

Risk analysis and how it is applied varies between countries, so it is not surprising that international variations, and even within-country variations, are more evident in risk evaluation than in other stages in the risk assessment process. More information on these aspects of risk evaluation can be found in Section 5.2.11.

2.1.4 Changes in flood risk and associated responses

2.1.4.1 Causes of changes to flood risk

Causes of change to flood risk can either be *drivers*, such as climate change, or *responses* in the form of the measures and/or instruments (see Section 2.2) that are used to control or mitigate flood risks. Both of these affect the whole source-pathway-receptor flooding system and so will affect flood risk, as illustrated in Figure 2.8.

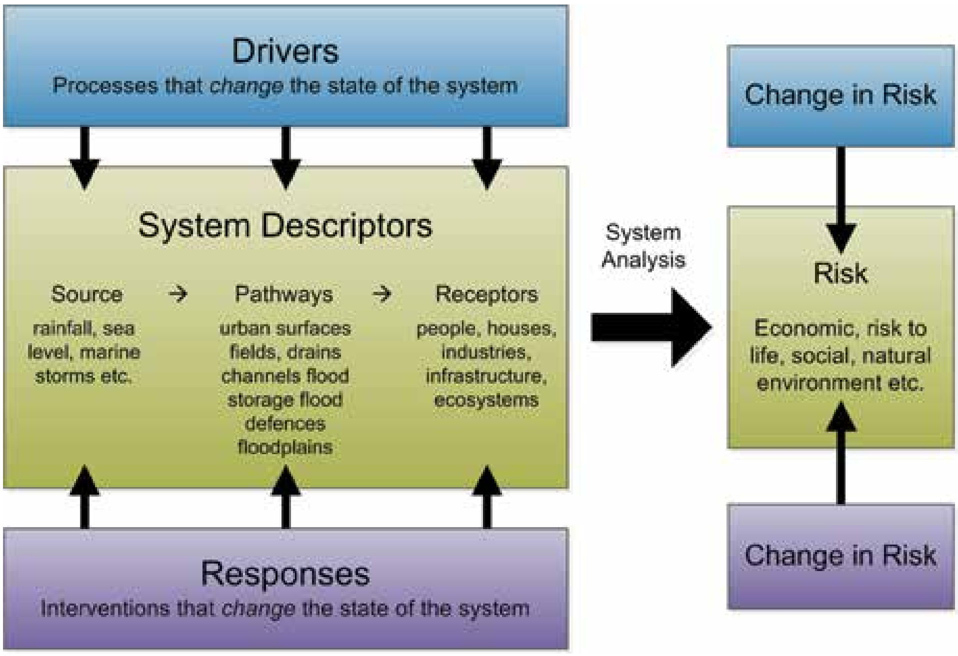
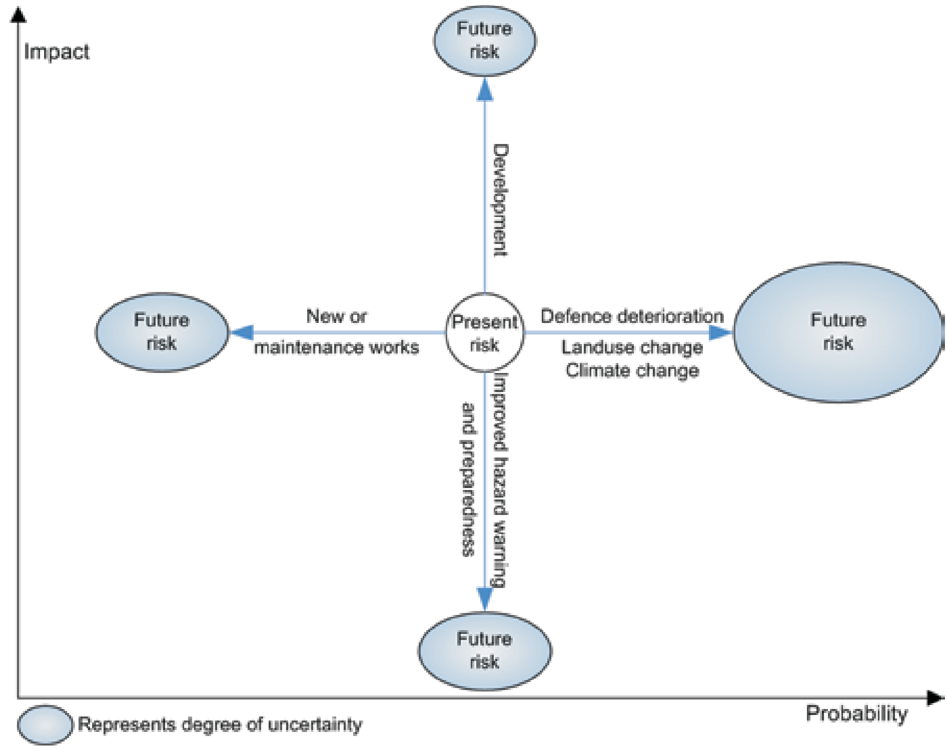


Figure 2.8 Drivers and responses changing the risk in a source-pathway-receptor flood system (Evans et al, 2008)

The resulting change in flood risk can also be viewed within a 'probability – consequence/impact space' as illustrated in Figure 2.9. The figure shows how example risk drivers (Box 2.4) can increase either the probability component of flood risk, such as climate change, defence deterioration, or the land use change. Risk drivers can also affect the impact/consequence component of risk, for example urban development. The extent to which the impact of these various changes is taken into account in the planning, design and management of flood management systems is generally a matter of national policy.

Figure 2.9 also shows how responses can decrease either the probability component of flood risk (eg by defence improvements) or the impact/consequence component (eg by improved hazard warning and preparedness). The various kinds of measures and instruments that can be used to reduce flood risk are discussed in more detail in Section 2.2.



Note
Size of ellipses represents the size of the uncertainty in the risk

Figure 2.9 ***Flood risk expressed in a probability-consequence space, showing ways of modifying it (adapted from Sayers et al, 2003)***

2.1.4.2 Resulting outcome with time

Understanding the resulting influence of these flood risk drivers and responses on flood risk requires assessments over a range of different scenarios, and over a long period of time. The length of time over which changes in flood risk are evaluated is often dictated by national policy, but typically it can be between 50 to 100 years. Once evaluated, the flood risk estimates generated from such assessments should ideally be expressed over common timeframes. For example, in cost–benefit assessment this integration is achieved by determining the present value of the stream of expected annual damages.

However, the prediction of future change in flood risk is inherently uncertain, and this introduces uncertainty into the risk assessment and into the evaluation of corresponding intervention options. Dealing with this uncertainty is one of the main challenges facing flood risk managers.

Chapter 7 describes methods of translating sea level rise to water levels and loadings on levees. How to account for ground subsidence in the design of levees is described in Section 9.12.

Box 2.4 *Some drivers of flood risk*

Source risk drivers

Climate variability and change is a major component of **source** risk drivers. Net warming of the atmosphere in the past several decades has induced many changes in the water cycle including changes in rainfall patterns and intensity, greater frequency and extent of drought, increased atmospheric water vapour, and changes in soil moisture and runoff. Increased land surface saturation and runoff to rivers, lakes and reservoirs results in **higher river discharges**, which can affect:

- levee structures and flood risk through increased water velocities and the potential for scour and erosion of the levee soil
- the likelihood of failure due to an increased level and frequency of hydrodynamic loading
- the likelihood of overtopping due to a reduction in freeboard (the distance between the water level and the crest of the levee).

The warming of the atmosphere and the resultant warming of the sea and widespread melting of snow and sea ice appear to have driven **sea level rise**. A rise in sea level means higher loads (water levels and wave conditions) on flood defence systems and structures. How these loads affect specific structures also depends on:

- the region or location (coast, estuary, river or lake)
- the geometry of the foreland (with potential for a loss of the foreshore) and the hinterland
- the type of hydrodynamic loads (high water level and/or wave attack).

Relative sea level rise (which is particularly important for coastal and estuarine levees) in a particular location is also affected by:

- **isostacy:** the vertical movement of the land mass, which varies across the continents (regional isostacy). Isostacy is, itself influenced by climate change as melting of ice sheets in the polar regions is reducing loads on continental land masses that carry them, and allowing them to rise
- **subsidence:** generally subsidence is caused by local soil conditions, increasing loads, or human activities such as mining or the pumping of water from underground aquifers. Climatic influences are also evident for certain land types and geomorphology (eg settlement by oxidation of peaty soils). However, **increased wave attack** can also arise because of either increased storminess or relative sea level rise increasing water depths and allowing more severe wave conditions to penetrate as far as coastal levees.

Pathway risk drivers

The **deteriorating condition of levees and flood walls** (or their components) due to various physical, chemical and biological mechanisms changes their state and reduces their reliability. This can leave them more vulnerable to failure under flood loading. Even defences that are rebuilt or adapted to a higher elevation can leave people vulnerable to a higher level of flooding impact should the structure(s) overtop or fail (effective communication of flood risk is essential to prevent occupants from forming a false sense of security when a flood defence is modified with increased height or size).

Receptor risk drivers

Land use and occupation can change over time with local and regional development such as:

- the draining of coastal lowlands for agriculture
- population movement and growth and the expansion of residential housing into flood risk areas
- spatial planning policies such as the creation of commercial and industrial enterprises in floodplains and estuaries.

In areas of high development there will often be an associated increase in supporting infrastructure, ie transport links, schools, service industries, utilities, hospitals and retail premises.

Subsidence of the ground surface can also increase potential inundation depths during a flood, resulting in increased harm to the people living in the flooded area and damage to goods and property.

2.2 MEASURES AND INSTRUMENTS FOR FLOOD RISK MANAGEMENT

Physical **measures** and financial and regulatory **instruments** are the means by which either the probability of flooding is reduced or, if inundation does occur, the impacts can be reduced (see Table 2.1). They are usually instigated and managed by flood risk management authorities.

In practice, measures and instruments are closely linked categories. It is virtually impossible to implement any structural measure without appropriate regulatory instruments, without justification for its implementation, and possibly without some financial compensation for those affected by it. Regulatory and financial instruments will influence the behaviour of people as much as they influence the requirement for structural measures.

Table 2.1 *Flood risk management measures and instruments (after FLOODsite, 2009)*

<table>
<tr><th>Goal</th><th>Aim</th><th>Character</th><th>Name</th></tr>
<tr><td rowspan="3">Flood probability reduction</td><td rowspan="2">Flood abatement or flood prevention</td><td>Physical measures</td><td>• conservation tillage
• dams/reservoirs
• reforestation
• restoring meanders in brooks and rivers
• retention in upstream catchment
• retention of water in cities
• wave breakers.</td></tr>
<tr><td>Regulatory instruments</td><td>• wetlands conservation/rehabilitation
• coastal wetland protection.</td></tr>
<tr><td>Flood defence and control</td><td>Physical measures</td><td>• levee construction/strengthening
• flood barrier
• temporary/mobile flood wall/barrier
• coastal sand supply
• bypasses
• connect rivers to existing lakes
• dredging rivers
• embankment relocation/realignment
• compensatory lowering of parts of the floodplain
• removing obstacles to lower hydraulic roughness
• river bed widening.</td></tr>
<tr><td rowspan="5">Flood impact reduction</td><td>Control of flood patterns</td><td>Physical measures</td><td>• compartmentalisation of areas
• detention areas/calamity polders
• floodway
• intentional levee breaching to control flood stage
• ring dikes around villages/towns/cities
• mounds.</td></tr>
<tr><td rowspan="2">Adaptation and regulation of use of flood – prone area</td><td>Physical measures</td><td>• flood proofing of buildings in floodplain.</td></tr>
<tr><td>Regulatory instruments</td><td>• building regulation, including building elevation
• flood resilient building
• land use zoning
• regulations on storage of toxics/chemicals
• adaptation of recreation functions
• adaptation of agricultural practices
• fines for damage increasing behaviour
• subsidies for flood proofing or other measures.</td></tr>
<tr><td>Distribution of flood impacts</td><td>Financial instruments</td><td>• damage compensation/buy-outs
• governmental relief funds
• insurance.</td></tr>
<tr><td>Preparedness</td><td>Communicative instruments</td><td>• risk communication including flood risk maps
• emergency action plans including evacuation plans
• guidance and education for inhabitants
• crisis management
• flood forecasting systems
• flood warning systems
• media information channels (radio/television/internet).</td></tr>
</table>

In using measures and instruments to reduce vulnerability to flooding, account should be taken of the more vulnerable aspects of the system. For example:

- fitting buildings with flood proofing products or raising their ground floor elevation makes them less vulnerable to flood damage
- businesses with a flood protection plan will be less vulnerable to economic losses
- if exposed to the same flood threat, the very young, elderly and infirm are potentially more vulnerable to harm than an adult in good health and will require greater consideration in evacuation planning.

2.2.1 Measures – structural and non-structural

There is a wide range of structural and non-structural measures available to reduce the probability of flooding, the magnitude of the flood itself, or the impact level of the inundation. Some of these are listed in Table 2.2. The list, although not comprehensive, illustrates measures and options for intervention that can be considered in every part of the 'source-pathway-receptor' flood model.

Table 2.2 Flood risk management measures (non-structural measures are shown in italic) (after McBain et al, 2010)

Source control – ***measures that reduce the likelihood of high flows/water levels occurring***	**Pathway modifications** – ***measures that modify or block the pathways taken by floodwater to a site***	**Receptor resilience** – ***measures that reduce the vulnerability of receptors to the impacts of a flood***

• *spatial planning and land use policies*
• sustainable drainage:
  ◦ detention basins
  ◦ filter drains/strips
  ◦ flow control systems
  ◦ infiltration basins/trenches
  ◦ permeable paving
  ◦ retention ponds
  ◦ soakaways
  ◦ swales
  ◦ wetlands
  ◦ green roofs/walls
• oversized pipes/attenuation tanks within the drainage network
• rainwater harvesting
• attenuation reservoirs
• river regulation
  ◦ river restoration and floodplain rehabilitation.
	• ground raising
	• construction of floodwalls and embankments/levees
	• construction of diversion channels or tunnels
	• tidal surge barriers, gates and sluices
	• removal or modification of existing structures
	• demountable flood defences
	• temporary flood defences
	• designing drainage networks for exceedance (eg overland flow routing)
	• managed realignment to make space for water
	• property or asset level flood resistance measures
	• beach replenishment and heightening foreshores.
		• *flood risk identification and mapping*
		• *planning policies and development control*
		• *civil contingency planning*
		• *awareness raising and enhanced preparedness*
		• *flood forecasting and warning*
		• *improved emergency response procedures*
		• *desktop incident management exercises*
		• *field incident exercises*
		• *business continuity management*
		• *risk transfer (eg flood insurance)*
		• *feedback from lessons identified*
		• *property or asset level flood resilience measures.*

2.2.2 Instruments – financial and regulatory

Financial and regulatory instruments are often intended to influence the attitude and/or actions of people who are at risk of flooding. Financial instruments may influence investment in property within the leveed area or may encourage owners to flood-proof their property. Regulatory instruments may also be used to require that:

- responsible authorities issue flood warnings to inhabitants potentially at risk
- individuals and businesses buy flood insurance
- local planners take better account of flood risk in future development
- local authorities prepare flood risk maps and flood evacuation plans.

2.2.2.1 Flood management policy and standards

In some countries safety standards and flood management policy are enshrined in national legislation (Box 2.5). So, for some authorities, there are legal obligations to maintain and operate flood management systems to a certain standard or level of flood risk reduction. Levees are often designed and constructed to meet this minimum standard.

Box 2.5 ***Example of safety standards fixed by legislation in the Netherlands***

The safety standards currently being used in the Netherlands are based on the recommendations of the first delta committee after the storm disaster in 1953. For example, a cost–benefit analysis for Central Holland comparing the cost of dike reinforcement with that of the avoided financial economic damage determined that an economic optimal probability of inundation was 1/125 000 per year. This was translated to a safety standard of 1/10 000 years expressed in relation to the allowable exceedence frequency for the design water level (in combination with the two per cent exceedence wave run-up and a minimum freeboard of 0.5 m). Subsequently, the safety standards of other areas with lower economic values were reduced and the criterion of two per cent wave run-up was changed to a wave overtopping criterion. The resulting safety standards for the primary flood defences area now vary between 1/250 and 1/10 000 per year and are fixed for each dike ring in the Flood Defence Act (1996).

Regulatory instruments, such as land use regulations, can also enforce, allow, limit or prohibit certain development and activities in flood-prone catchments (see Figure 2.10) and coastal areas. Some examples of legislation and policy from different regions and nations are illustrated in Box 2.6.

Figure 2.10 ***Development policy in practice – an elevated construction technique used to reduce the impacts and damage of flooding to a home near the Illinois River, St Louis District, USA (courtesy USACE)***

2.2.2.2 Environmental regulatory considerations

There are often environmental regulatory considerations that can affect the adopted flood risk management measures. This can particularly affect the construction and maintenance of levees. For example, regulations protecting flora and fauna can prevent maintenance activities such as grass cutting during certain times of the year and limit the extent of any construction and/or maintenance work. Tree preservation orders can affect the removal of trees, growing in or adjacent to levees, which may pose a risk to their structural integrity. Such considerations can affect the way in which the required performance objective is to be achieved or maintained.

Box 2.6 Examples of national policy and regulation instruments

National legislation on floodplain management in the USA

The US Congress established the National Flood Insurance Program (NFIP) on 1 August 1968 with the passage of the National Flood Insurance Act of 1968. This was modified by the Flood Disaster Protection Act of 1973, the National Flood Insurance Reform Act of 1994 and the Flood Insurance Reform Act of 2004. The NFIP is administered by the Federal Emergency Management Agency (FEMA). As a participant in the NFIP, a community is responsible for making sure that its floodplain management regulations meet or exceed the minimum requirements of the NFIP. By law, DHS FEMA cannot offer flood insurance in communities that do not have regulations that meet or exceed these minimum requirements. The basis of the community's floodplain management regulations is the flood hazard data provided to the community by FEMA (FEMA, 2011).

National legislation on flood management in France

The Grenelle 2 law transposes the EC Floods Directive (2007) into French Law, which encourages the:

- sharing of a new and homogeneous knowledge of risks through the preliminary assessments of flood risk (EPRI) at the level of each district
- definition of a national strategy for risk management (SNGRI) defining nationally important risk criteria
- identification of priorities for action in each district of the territories
- definition in each district of a plan for flood risk management
- variation and adoption of these plans at the local level by local strategies driven by local actors based on the current tools of risk management: flood risks prevention plans (PPRi) action programmes for flood prevention (PAPI) etc.

Planning policy in England

In England the National Planning Policy Framework (CLG, 2012) sets out national policy on different aspects of land use planning with the aim of protecting the environment while supporting sustainable growth. It includes policies on meeting the challenge of climate change, flooding and coastal change. Other national planning policies for flood risk and water management in England include those set out in the National Flood Risk Management Strategy (EA and Defra, 2011).

National legislation on flood management in Germany

Flood management in Germany is governed by the Federal Flood Protection Act 2005 and the Water Acts of the Federal German States (or Länder). The 16 Länder are responsible for the delivery of flood protection in their respective areas, which is organised differently in each federal state. Technical standards and guidelines are developed by experts in non-government associations, such as DIN, DWA, and BWK. The Standard DIN19712 and the DWA Guideline M 507-1 (2011) are the technical basis for levee operation.

National legislation and practice in the Netherlands

Flood management in the Netherlands is governed by ***Wet op de waterkering*** – the Flood Defence Act (1996). The safety of all primary flood defences along the sea, rivers and lakes must be assessed periodically. Legally, safety assessments must be conducted according to the Directive on Safety Assessment of Primary Flood Defences (Voorschrift Toetsen op Veiligheid) (TAW, 2004). The results of all safety assessments must be presented in a national assessment report to the government. For flood defences that do not meet the standard, remediation plans have to be made and approved by Rijkswaterstaat to obtain national funding. Technical tools such as guidelines and technical reports are also developed by Rijkswaterstaat and validated by the Expertise Network for Flood Protection (ENW).

Note: all European national legislation is consistent with the EC Floods Directive (2007), which has requirements for producing preliminary flood risk assessments, flood hazard maps and flood risk management plans with updates every six years.

2.2.3 Formulating portfolios of measures and instruments

There is no standard recipe of measures and instruments for reducing flood risk. The most cost-effective strategic approach to flood management normally involves the development of programmes or groups of different types of measures and instruments for reducing flood risk (sometimes referred to as portfolios or strategic alternatives). Box 2.7 illustrates a stepped portfolio approach to flood risk reduction adopted in the USA involving a range of measures and instruments.

Box 2.7 An example of a stepped approach to managing flood risk, USA

In the USA, the stepped approach to managing flood risks is increasingly being adopted. Both structural and non-structural solutions that are complementary, interoperable and multi-dimensional can be used to drive down overall risk to acceptable/tolerable levels. The approach recognises the benefits of using watershed or systems approaches and points to solutions that can reduce overall flood risk. However, some level of residual risk will always remain.

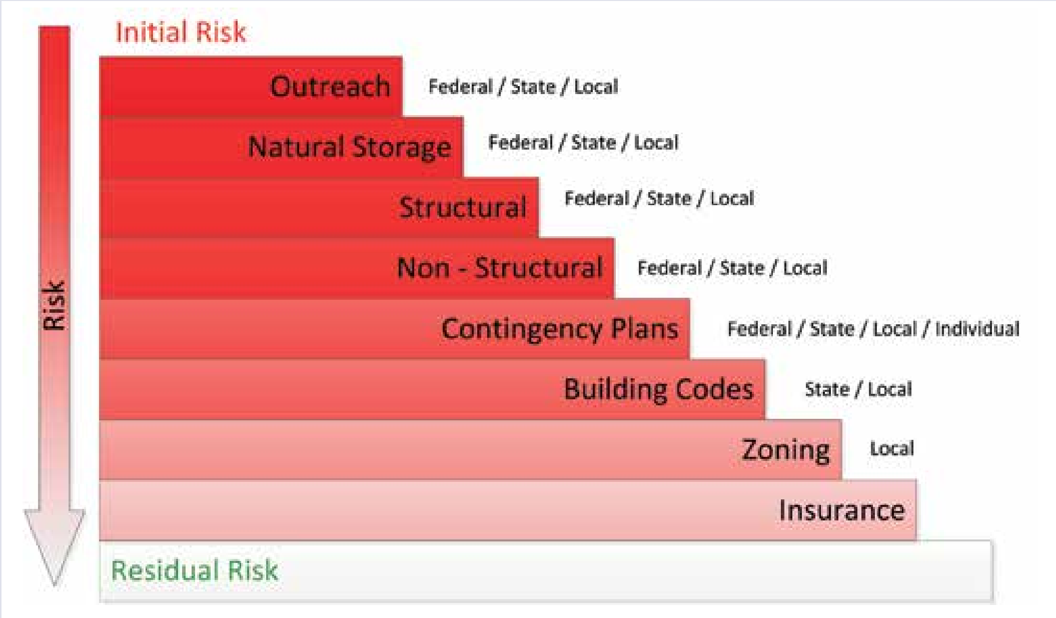


Figure 2.11 An example of a stepped approach to managing flood risk (USACE, 2011)

The choice of particular measures and instruments for a portfolio depends on a range of factors, including:

- the main causes of risk
- national and regional traditions of flood risk management
- the availability of funding
- the likely effectiveness of the measure or instrument in reducing flood risk.

When planning a portfolio of measures it is important to identify what the effects of a measure or instrument will be and how they may interact (including any unintended consequences). Examples of some measures and their impacts are listed in Table 2.3. Each impact must be considered individually and collectively when making decisions on which system best meets the goals set by a community to provide flood risk reduction.

Table 2.3 Impacts of flood damage reduction measures (USACE, 1995)

Measures	Impact of measure		
	Modifies discharge-frequency function	*Modifies stage-discharge function*	*Modifies stage-damage function*
Reservoir	Yes	Maybe, if stream and downstream channel erosion and deposition due to change in discharge occur	Maybe, if increased development in floodplain occurs
Diversion	Yes	Maybe, if channel erosion/deposition due to change in discharge occur	Maybe, if increased development in floodplain occurs
Channel improvement	Maybe, if channel affects timing and storage is altered significantly	Yes	Not likely
Levee or floodwall	Maybe, if floodplain storage is no longer available for flood flow	Not likely	Yes
Flood proofing	Not likely	Not likely	Yes
Relocation	Not likely	Maybe, if flow obstructions are removed	Yes
Flood warning preparedness plans	Not likely	Not likely	Yes
Land-use and construction regulations	Not likely	Maybe, if flow obstructions are removed	Yes
Acquisition	Not likely	Maybe, if flow obstructions are removed	Yes

Impacts on the surrounding environment, during and following construction of levees for example, should also be considered. These may be positive or negative and may include construction noise, damage to or enhancement of natural habitats. In some instances property or habitat displacement may be required and land compulsorily purchased. Some impacts may be quantifiable and can be given a monetary value. However, others may only be described in qualitative terms. Construction impacts are discussed further in Section 10.3.

2.2.4 Options appraisal

The identification of feasible portfolios of measures and instruments, including identification of the impacts of options and comparison of the advantages and drawbacks for each is often called options appraisal. This includes assessing how each measure changes the risk when compared to no intervention at all, often referred to as the do-something and do-nothing options. It can also include comparison of other factors affecting the project. There are various techniques for doing this, including multi-criteria analysis (MCA), cost–benefit analysis (CBA), societal benefit cost analysis (SBCA), life safety assessments and Environmental Impact Assessment (EIA) to name a few. Section 5.2 describes these techniques in more detail. So, the assessment of potential options should encompass consideration of the sustainability principles of social acceptability, and economic and environmental viability as outlined in Section 2.3.2.

In terms of levees this includes evaluating the proposed or existing levee environment, the impact of the environment on the levees and vice versa, and is a topic that is discussed further in Chapter 3. Also, over time there are likely to be changes to the environment, the levee and the leveed area, which will modify the current flood risk and flood defence system, and although they are deemed 'safe' now, this can change (see Figures 2.8 and 2.9). Predicting these future changes is important in order to make better present-day decisions.

Note

It is assumed from this point on in this handbook that the decision has been made (as part of options appraisal) to create or maintain levees as part of the selected response measures.

2.3 LEVEE MANAGEMENT

2.3.1 Performance requirements

In establishing management objectives for levees, it is important to ensure a clear and common understanding of the desired outcome. In situations not covered by legislation or policy, objectives may be set through socio-economic analysis and expert judgement, taking into account societal concerns.

Each component of the levee, each levee segment and the flood risk management system as a whole must perform effectively. Failure of a levee is defined as the inability to achieve a defined performance threshold for a given function, in particular for flood risk reduction. This is discussed in more detail in Section 3.5. Components of a failure process can include slow deterioration over a number of loading cycles and rapid damage during a single event. Predicting and avoiding failure requires consideration of both hydraulic performance and structural (including geotechnical) integrity during critical and fluctuating flood stages. Also, the levee should ideally have a measure of resilience to breach even if overtopped. Potential failure modes for levees are outlined in Section 3.5.

2.3.1.1 Risk-based approach to levee performance

Historically, levees were designed to reduce the risk of flooding up to a particular return period water level – as defined by a historic flood level, in a flood defence policy or strategy, or by cost–benefit analysis. Under this approach, the selected design water level has to be reviewed periodically as circumstances change due to the drivers and responses discussed in the previous section. The approach also only considers single values for all input variables and yields a single value output. Instead, the focus of this handbook is on adopting a risk-based (or risk-informed) approach to the design and management of levees. The levee is considered as part of an overall flood risk management system and with a focus on the consequences of failure on life, property and economic activity downstream. With this intent, the integrity of the levee and the potential flood risk should be considered for a range of operational conditions. For example, the extent of inundation of a leveed area might be investigated for a variety of flood levels and durations (see Box 2.8).

Having stated the general approach, it is important that where prevailing national or even regional standards and approaches exist, that they are applied. For example, in addition to the setting of design flood levels, many countries have specific national guidelines for the hydraulic and geotechnical design of levees and the appropriate factors of safety against failure. The approaches may vary depending on the level of risk associated with the levee (see Section 9.5 for more details). Detailed guidance on the tools for analysis of levee failure is given in Chapter 8 and on the associated design approach in Chapter 9.

In addition to considering performance during the whole life of the levee, temporary conditions during construction should be considered. In the case of levees this is particularly important because in addition to flood risk there is a further risk of failure of earthen structures due to elevated pore pressures generated by the construction process (see Section 9.9). Also, evaluation of likely structural responses during construction should take account of a range of events that might occur, including conditions above and below the selected nominal return period water level (see Box 2.8). Allowing for these conditions, appropriate health and safety protection measures for workers and the public should be maintained both during construction and after completion (see Section 10.1.4 for further details).

2.3.2 Functional objectives

As well as the flood risk management performance objectives described in Section 2.3.1, there are other functional objectives for a flood management system or levee that need to be considered. The *economic viability* of the area being affected by the levee is a critical aspect and is often captured during cost–benefit analysis. However, the *environmental viability* of the area in which the levee is situated (or to be built) should also be considered when planning its construction, design or maintenance. Where possible, options should be selected that, as well as managing flood risk, increase the *social acceptability* and improve the environmental features for the site and the area as a whole. The aim should be to balance and include these three areas (or pillars) of sustainability (Figure 2.12) in the engineering solution, without compromising the primary design standards of the levee. Involving environmental professionals and local communities early in the planning and design process will help to ensure that environmental objectives and societal needs are met as well as the economic aims.

Box 2.8 Making 'encounter probability' easy to understand (after Porter, 2012, and CIRIA; CUR; CETMEF, 2007)

The use of flood risk terms such as 'a 1 in 100 year event' can often lead the public to believe that this means that they are free from risk within their lifetime, which is a common misunderstanding of probability and of risk. A 1:100 year event is one that has a one per cent chance or probability of occurring in any year. The resulting likelihood of exceedance or 'encounter probability' P_e can be calculated using the equation (Kamphius, 2000):

$$P_e = 1 - [1 - (1/T)]_n \quad (2.1)$$

where T is the flood return period (eg a 1 in 100 year event), and n is the lifetime (eg design life of levee, or period of construction) in years. This results in the following percentage encounter probabilities for the given flood events and lifetimes.

Lifetime, n: years	Annual flood return period, T				
	1 in 10	*1 in 50*	*1 in 100*	*1 in 200*	*1 in 1000*
1	10	2	1	<1	<1
2	19	4	2	1	<1
5	41	10	5	3	<1
10	64	18	10	5	1
20	88	33	18	10	2
50	>99	64	39	22	4
100	>99	87	64	39	10

This shows that if people are living in a 1 in 100 year flood area (an area that will only be inundated by events of return periods greater than 1 in 100 years), the probability of being inundated in 100 years is 64 per cent.

This can still be confusing, but further steps can be taken to place the terms into contexts that may be more familiar or easier to understand. For example, considering the same probability, but placing it in the context of an average lifetime of 70 years, then the likelihood of experiencing a flood during that lifetime is 51 per cent – or about a 50/50 chance. Other well-understood time period contexts could be used such as over the life of an average mortgage, or the expected design life of a building.

Placing such terminology into real life contexts does not reduce the quality of the information but does provide a means of communicating a complex concept in a more readily comprehendible way, both for decision makers and the public.

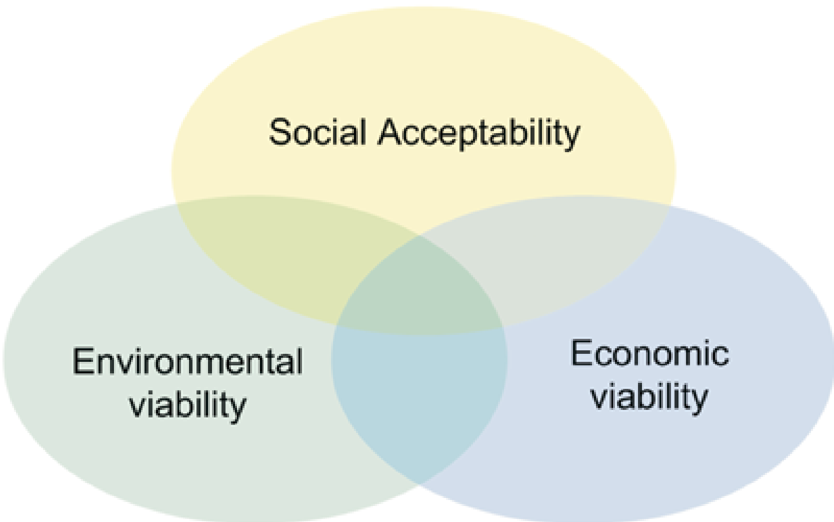


Figure 2.12 Sustainability in levee design, construction and maintenance should consider the balance between social acceptability, environmental viability and economic viability

2.3.2.1 Environmental viability

Levee option selection and management should view environmental aspects as part of the solution not as a problem. Working with the environment, natural history and geomorphology of the watercourse/coast, developing solutions should consider:

- opportunities for enhancement of natural processes that contribute to flood risk reduction while minimising any negative effects on the environment and geomorphology
- opportunities for minimising resources use, which can include:
  - reuse of on-site materials to avoid transport issues associated with importing materials, including treatment of materials where necessary
  - use of less suitable materials in less critical parts of the levee (such as in some berms)
  - use of appropriate waste materials (eg baled tyres for levee fill/stabilisation – see Box 9.55)
- opportunities for environmental enhancements, such as:
  - protection of existing environmental sites
  - potential for habitat creation from on-site borrow pits where material is won for levees (see Box 9.20 for a Natomas levee example)
- the levee in its wider environmental context. Levees can form important linear corridors to link habitats and allow species to migrate along them
- the need for adaptation to future environmental changes.

2.3.2.2 Social acceptability

Levee option selection and management should include consideration of multiple potential social uses and benefits of levees, including:

- political and community acceptance and future community viability
- reduction of risk to the cultural and built heritage in the vicinity of the levee
- recreation and amenity
- quality of the landscape and visual environment.

2.3.2.3 Economic viability

Unless decisions about levee creation and management have already been made at the societal scale (as in the Netherlands), it is normally given that levee schemes have met some test of economic viability over the period of economic appraisal (typically in the range of 30 to 100 years). However, costs are often still a constraint and there is considerable scope for imaginative thinking in developing best options. Sometimes identifying multiple potential functions for levees can attract additional funding from other partners and allow an improved multi-functional concept to be developed. In the UK a formal mechanism was introduced in 2009 to allow and encourage such partnership funding.

2.3.3 The levee management life cycle

Management activities for a levee throughout its serviceable life follow the general approach appropriate to other physical assets shown in Box 2.9. A generic levee management cycle is given in Figure 2.14, which has been constructed to be applicable whatever the national governance arrangements. It should be read in conjunction with Table 2.4, which describes and explains the various stages in the cycle in more detail.

The levee life cycle diagram in Figure 2.14 makes it clear that, in the case of a *new* levee, the normal entry point to the cycle would be from a definition of policy and functional objectives (ie from the top of the diagram). In the case of an *existing* levee, one would enter the levee management life cycle within the routine operational cycle. Changes to existing policy and management objectives or an event that changes the level of flood risk reduction might take the manager outside this loop in order to consider alterations to an existing levee such as improving, rebuilding or decommissioning the levee. However, these activities would not normally be considered as being part of the routine levee management life cycle. In all cases, diversion to the inner emergency management loop is triggered by damage due to a severe flood event or an assessment, which indicates that the levee is not likely to perform as expected.

Box 2.9 *UK generic management life cycle for infrastructure assets*

Physical assets in any industry can be managed using a basic plan-do-check-act (PDCA) cycle (Deming, 2000). Most asset management frameworks are based on this cycle, and Figure 2.13 illustrates one such framework from the UK. It involves a range of assessments and reviews as well as maintenance and other potential interventions.

Figure 2.13 *The asset management system in the UK, plan-do-check-act cycle (from BSI, 2008)*

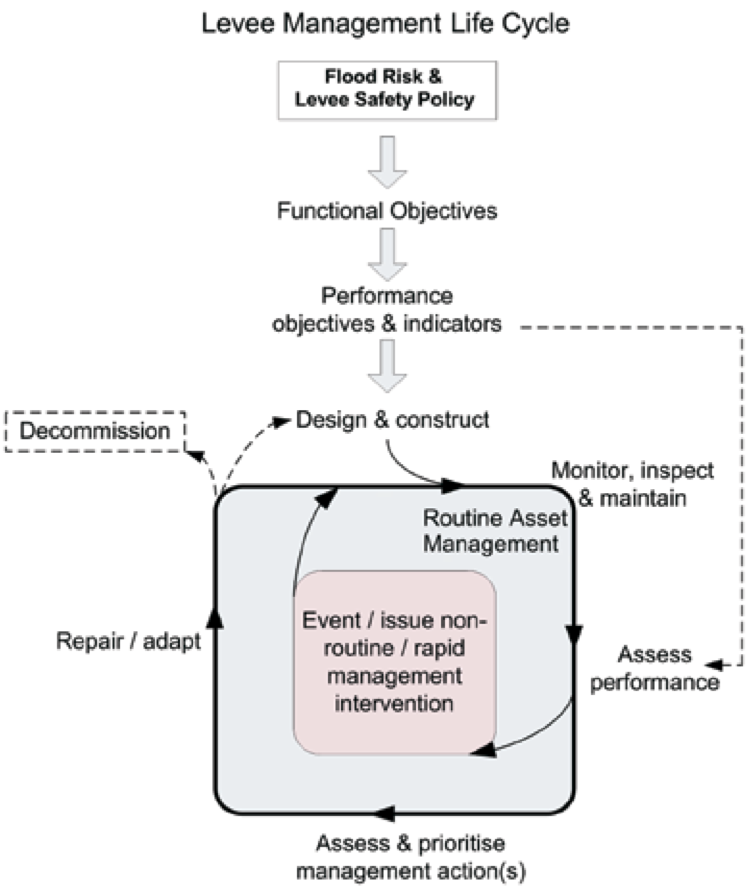


Figure 2.14 *The management life cycle for a levee or flood defence system*

Table 2.4 The principles and main issues associated with the levee management life cycle

Life cycle stage	Principles and issues	
Flood risk policy and levee safety management strategy	Establishing a flood risk management policy and strategy, together with an implementation plan and operational procedures has benefits. It should provide an organisation with a foundation for the processes, tools and performance measures that will enable it to achieve, through a process of continuous improvement, an optimum approach to managing its assets including levees. Such a strategy needs to be owned at the executive level of the responsible authority for the catchment or coastal unit, be evidence-based and be auditable in its application.	
Functional objectives	The primary function of a levee is to reduce the risk of flooding of an area up to a designated water elevation, or to limit the degree or extent of inundation. To design a new levee it is important to first identify the functional role(s) and objectives of the levee. There are often one or more secondary functions that also need to be considered including, for example, environmental, recreational and visual impacts (see Chapter 3 for more information on functional objectives).	
Performance objectives and indicators	To define and assess what level of performance the levee should provide, it is necessary to understand the nature and type of the loads it will have to bear, and what level of risk reduction it should be designed to provide. There are different ways of determining design criteria, setting safety standards and rating and monitoring performance (see Chapter 9 for information on performance objectives and Chapter 5 for information on performance indicators).	
Design	The desired/intended functions of the levee need to be clearly defined. The appropriate levee form will be dependent on these and on the site characteristics. The design phase should include the activities that address the management of the levee throughout its whole life cycle, and the designer needs to consider all associated and interrelated components of a flood risk system (see Chapter 7 for more information on site characterisation and Chapters 8 and 9 for information on design).	
Construction	The levee should be constructed in accordance with the design plans and specifications. Construction should aim to minimise public and environmental impacts, optimise funds and resources, and address any deficiencies (see Chapter 10 for more details on levee construction).	
Routine operational cycle	Following construction, the activities of levee management move into a routine operation cycle as follows:	
	Monitoring, inspection and maintenance	Monitoring by systematic recording over time is important to establish trends in variability of levee data for early identification of possible issues, for example, crest settlement. The level of inspection can vary significantly, but will generally involve the use of indicators of structural condition and notable changes such as an increase in seepage. The process should include performing an evaluation of the levee characteristics/features and comparing them against current standards and guidelines. Maintenance should prolong the life of the levee and promote its performance (see Chapter 4 for further information on maintenance, Chapter 5 for inspections and Chapter 7 for monitoring techniques and instrumentation).
	Performance assessment	Performance assessment can be a qualitative and/or quantitative process of understanding the state, structural integrity, or the performance of a levee to: • confirm adequate performance to requirements • inform the planning of future interventions. See Chapter 5 for further details.
	Assessment and prioritisation of management options	Having identified the requirements for action and intervention, suitable management options need to be assessed and prioritised. There are various ways of assessing the appropriateness and suitability of options and their priority (see Chapters 5 and 9).
Routine operational cycle	Repair/adaptation	Repairs to or adaptation of a levee may need to be undertaken where elements or components have become degraded or been damaged. Sometimes adaptations may be required for other reasons, for example where new or improved access for vehicles is required (see Chapter 4 for information on repair and Chapter 9 for design of adaptation options).
Incident or event	A flood event or other incident (eg vandalism) may give rise to safety concerns with regard to the integrity of a levee. An inner cycle of management may ensue, where rapid management intervention may be required, which includes options such as repair, modification or recovery. Policy and management objectives may affect the decision making process with regard to which options may be selected (see Chapter 6 for more information on emergency management).	
Decommissioning	There may be consequences to the termination of operational activities that need to be considered. So it is important to select appropriate form/approach/measures for decommissioning to mitigate the risk of any identifiable consequences. Public versus privately owned infrastructure may differ in decommissioning procedures and capability.	

2.3.3.1 Approaches to levee asset management

There are various ways in which the cycle of levee management can be implemented but some common features can be identified. An example of a risk-based, tiered approach adopted in the UK is given in Box 2.10. This type of approach entails a risk screening process that indicates for a given levee which tier of investigation and analysis is to be used to support subsequent management decisions and actions. For example, inspections can range from basic visual walkover surveys to advanced geotechnical investigations, and analytical methods can range from a simple rule of thumb to complex modelling. The guiding principle for selecting the approach is to use simple methods where possible, and only use more complex or detailed methods where necessary.

Box 2.11 illustrates how the tiered approach and risk assessment are also integral to activities and aspects of USACE levee life cycle management (Figure 2.16) and how seeking continuous improvement is an essential feature of the levee life cycle (Figure 2.14). The USACE example also illustrates how periodic inspections can instigate assessments of performance and/or risk (see Section 5.3). These then inform the planning of levee management, which combines with the organisational strategy to determine the optimum program of routine operations and maintenance (see Section 4.1.2). The cycle then starts again with the next inspection. In practice, where the need for improvement is self-evident, there can also be shortcuts from inspection to intervention.

The UK example (Box 2.10) also highlights the central role that information management plays in these steps (as discussed in Section 5.6), since each step uses and produces levee asset information.

Box 2.10 Example of a cyclical, tiered approach to flood and coastal defence asset management (from Environment Agency, 2011)

The asset management 'propeller' in Figure 2.15 is a concept that shows how the three tactical elements of asset management, inspection, performance assessment and planning (represented by the wings of the propeller) continuously feed into one another in a cyclical process. Information management is at the centre to allow data transfer. Inspection, performance assessment and planning all involve a tiered approach:

- basic methods are normally adopted for assets at lower risk (shown in the diagram where the wings are wider nearer the base)
- progressively more advanced or specialist methods are needed for the smaller number of higher risk assets (where the wings are narrower near the tip).

The three tiers are not necessarily entirely distinct but are indicative of a progression from simple to more complex methods and tools.

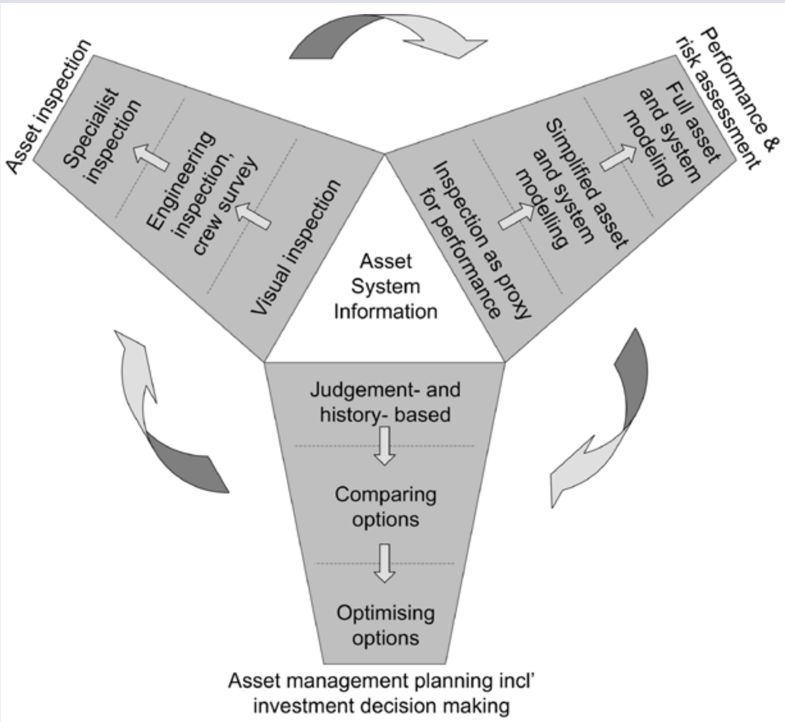


Figure 2.15 Asset performance tools propeller (from Environment Agency, 2011)

Box 2.11 Integrated levee/risk management framework (courtesy USACE)

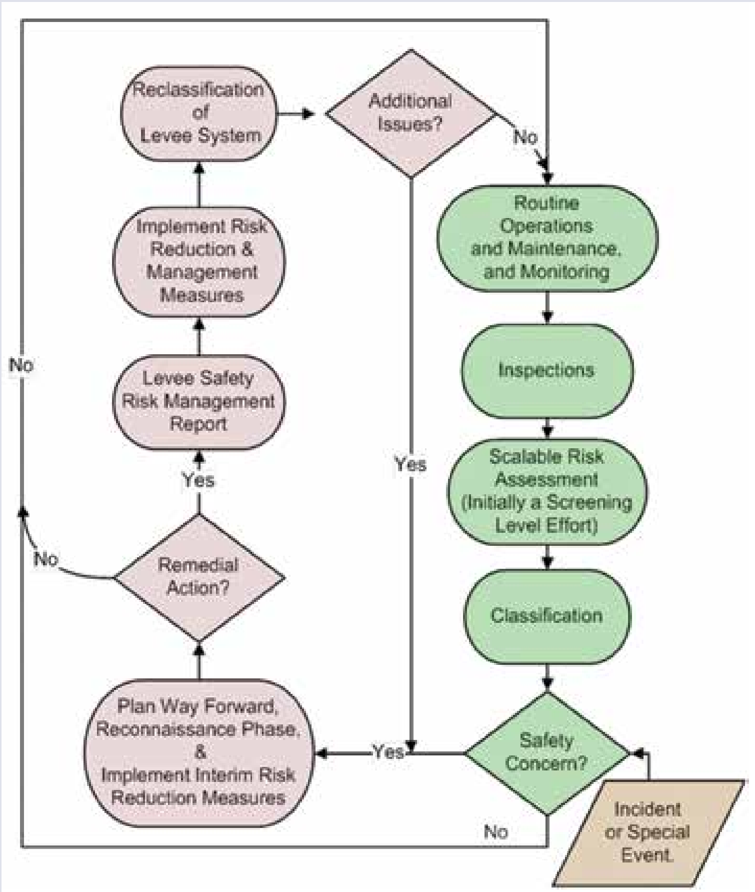


Figure 2.16 Recommended framework for levee safety risk management (courtesy USACE)

Typical routine activities are initiated around the outer loop, including field inspections and screenings. Combining information acquired during the field inspection with historical performance data, projected future performance and consequences to occupants and activities behind the flood defence system, the screening process will result in the determination of a levee safety action classification (LSAC). Assigned LSACs will help to prioritise future funding across the country to address levees with more urgency, based upon performance and consequence, in order to most effectively channel funds to reduce risk to the maximum extent. Only when an issue is identified will the system undergo more extensive studies, such as risk analyses, to evaluate the problem and recommend solutions.

2.3.4 Managing performance and failure of levees

2.3.4.1 Knowledgebase of levee performance

Understanding the intrinsic structural and geotechnical properties of an existing levee is fundamental to understanding its performance and reliability. Levees are generally man-made structures, and range in quality from older 'non-engineered' structures composed of dredged material placed on poor foundation soils to well-constructed engineered embankments built using current standards on prepared and competent foundations. Many older levees have been progressively raised and widened over the years as rural areas behind them became more urbanised. Chapter 3 provides more information on the evolution of levees through time.

Some levees need to control water both dynamically and statically over long periods of time. Others may not come into contact with water until high flood stages occur – in which case they need to maintain integrity over a range of water levels (including being dry over considerable periods of time).

Knowledge about a particular levee and its structural integrity and foundations may vary considerably in its extent and may need to be supplemented by further investigation (see Chapter 7). Relevant information includes:

- construction history
- geology and geomorphology
- soil borings and exploration data

- engineering properties of embankment and foundation conditions
- operation and maintenance records
- frequency of floods and other loading
- repair history
- past performance.

Information on these aspects can be gathered by several means such as:

- condition assessment and inspection
- scientific or routine monitoring
- records of past maintenance and structural failures.

Levee deterioration

Levee condition is also not static. As mentioned in Box 2.4, levees and their components deteriorate over time due to various physical, chemical and biological mechanisms. This deterioration affects their ability to withstand flood loading and provides one of the drivers by which flood risk in the leveed area increases. This topic is discussed in further detail in Chapters 3 and 5. There are several factors that affect the rate of deterioration of a levee, many of which are discussed in more detail in Chapter 7. For example:

- *changes in precipitation patterns* may affect vegetation and levee embankment soil, for example:
  - reductions in soil moisture may cause grass die-off
  - drying of more plastic clay soils may cause shrinkage and cracking (see Section 9.12)
- *sea level rise* can affect ground water level conditions and also the strength properties of subsoils close to the coast and in estuaries
- *the actual history of loadings* (eg from waves and currents)
- *impacts made by humans, animals and vegetation*, which can include:
  - illegal or uncontrolled encroachments that may change the structure geometry, damage (or remove) the cover layer or crest, or penetrate the core or other critical area of the levee. These can arise from vandalism, unapproved installation of utility pipes or cables, and adjacent construction activities
  - inappropriate use (eg vehicular access where not designed for)
  - uncontrolled grazing and animal burrowing
  - poor woody vegetation management practices

  (Chapter 4 describes such effects and impacts in further detail together with potential mitigation measures).
- *subsidence* due to mining or water abstraction, which can cause differential settlements, leading to cracking and mass movements in the levee.

2.3.4.2 Assessments and reviews of levee performance and reliability

As indicated in the levee management life cycle diagram (Figure 2.14), periodic reviews by asset managers are necessary, to check that the levee system as a whole (or individual structures) continues to meet both performance and functional requirements. Assessments and reviews (see Chapter 5) may be also instigated by drivers other than the schedule of a levee management programme. For example, there may be:

- requirements laid down in national legislation dictating frequency of assessments and reviews
- unreliable/uncertain results from previous assessments
- failure of a levee, requiring failure diagnosis or further assessments
- advances in methodologies, analytical and modelling techniques, offering improved results.

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Carrying out an assessment requires the knowledge base information discussed above, together with analysis of the performance and reliability of the asset to determine whether and how likely failure is to occur. The subject of failure and failure modes is described in more detail in Section 3.5.

Hydraulic (non-structural) performance failure

Hydraulic (or non-structural) failure of a levee relates to excessive through-flow, overflow or overtopping of the levee above the amount for which it was designed under that set of conditions (further explanation is provided in Section 3.5.1). Assessments must consider relevant hydrologic and hydraulic conditions of the catchment or coastal area and the influence they may have on water levels, wave conditions etc that may cause overflowing or overtopping. Section 7.3 provides the tools for the assessment of hydrologic and hydraulic conditions and Section 8.2 provides the corresponding tools for assessment of hydraulic responses such as overflow and overtopping.

Structural failure analysis techniques

Analysis of the likelihood of geotechnical or hydraulic failures of levees is a key part of flood risk assessment. The starting point in every case is understanding and assessing the potential mechanisms of failure that might occur with an individual levee segment.

Once these mechanisms are understood, and depending on the goals of the analysis and the level of effort desired, there are a range of approaches available such as:

- index-based methods
- comparatively simple, deterministic methods
- scenario-based, semi-probabilistic methods
- full probabilistic methods using Monte Carlo simulations or fault tree input from experts.

Index-based methods use (Likert) scales to score aspects of levee performance (eg urgency, vulnerability, number of people at risk). From a life cycle management perspective, the classification or score assigned to levee systems guides decisions for prioritisation of rehabilitation/remedial works.

Deterministic methods are based on the concept of calculating failure under design loads, eg wave heights, water levels or discharge rates. Deterministic design methods typically calculate one overall safety factor for one given configuration of loading and strength. One problem with the use of this type of analysis is that it is difficult to account for variance in structural response to loading and does not take into account the differences in uncertainties.

The basis of the **semi-** (or quasi-) **probabilistic** methods is that the parameters used are not known with certainty. In semi-probabilistic design, partial factors of safety are assigned to each of the load and strength parameters to account for uncertainty in their value.

Full probabilistic approaches revolve around determining the likelihood of the levee failing under a given load condition. Reliability assessment approaches can be used, which include the concept of fragility. Typically, this is expressed as a fragility curve relating load to the conditional probability of failure given that load (see Figure 2.17). Combined with descriptors of deterioration, fragility relationships enable performance to be described over time for a specific levee. An alternative to a conditional probability approach is applying a fully probabilistic approach to reliability analysis in which all the uncertainties in the loads and the strength are represented, typically for a representative cross-section of each levee segment.

Overall structural failure analysis

In order to apply any of these methods to assess the likelihood of breach, it is important to understand *how* a levee might fail by a combination of the known mechanisms. Examples of methods that can be used to combine mechanisms include fault tree analysis and event tree analysis, which are discussed in more detail in Section 5.3.

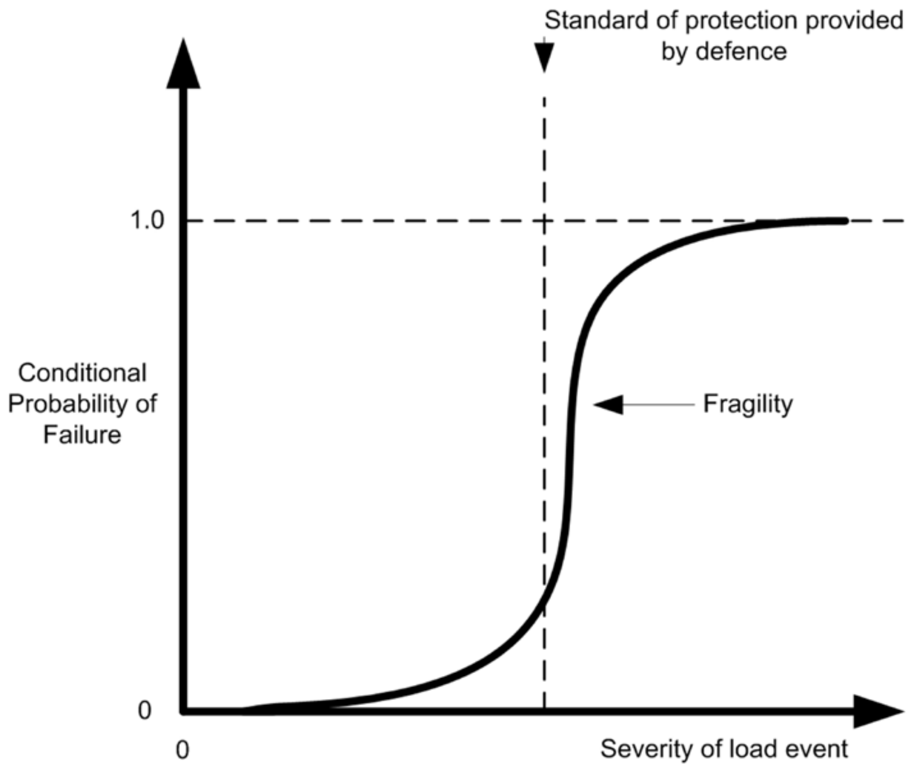

Figure 2.17 Example of a fragility curve (from Bramley et al, 2009)

Also, it is important to know that *length effects* within a flood defence system can influence the probability of failure assigned to an individual levee segment. Length effects are related to the spatial dependence (or independence) between one levee segment and another of the applied loads (eg wave attack) or of levee segment strength (eg geotechnical parameters). They can also be influenced by the total length of the flood defence system. In the case of probabilistic analyses, the effects on the probability of the system failure given a large number of levee segments (the length effects) can be determined by either assuming independence of actions and responses from one levee segment to another or by accounting for the correlations in the load and strength parameters via a probability factor.

2.3.4.3 Measures to reduce the probability of levee failure

Armed with information on levee performance and levee reliability analysis, managers can then explore options for maintenance and/or improvement of the asset to correct any inadequacies. A full management review should consider the 'bigger picture' in which a risk analysis should be conducted to determine level of residual risk given the existing level of service and planned interventions. Such reviews will also consider issues such as:

- future inspection frequencies
- further risk reduction measures
- prioritisation and optimisation of investment.

At the scale of individual levees there are some specific measures that can directly reduce the probability of levee failure (and thereby reduce flood risk) such as:

- reducing or increasing the level of water in river channels in a specific location (whether or not it results in a change to the water level in another area). For example, lowering of water levels could be achieved by dredging or removal of debris/silt
- movement of sediment onto, off or around a beach, or carrying out other works in respect of shoreline improvements
- installation of scour protection to mitigate against erosion by waves and currents

1 2 3 4 5 6 7 8 9 10

- installation of surface protection measures local spillway structures and overflowing weirs (concrete, stone, vegetative) to control overflow and prevent erosion of the landward slopes of levees
- installation of drainage structures (relief wells) to collect and control seepage
- flattening the levee profile including the addition of berms to control a number of forms of potential failure. Flattened slopes can:
  - lower overflowing velocities on the landside slopes of the levee and thereby reduce risk of surface erosion
  - reduce seepage and internal erosion by lowering internal hydraulic gradients
  - increase stability of the levee against mass instability
- planning, erecting, maintaining, altering or removing associated structures such as groynes.

Some of these measures relate more to the management of rivers and coasts, and the detail of their assessment and design is not covered in this handbook, although there is some discussion of these issues in Section 3.4. Those that relate more specifically to the body or foundation of the levee itself are discussed further in Chapter 9.

There are specific analysis requirements associated with assessment and design of measures to reduce levee failure. Hydrologic and hydraulic analyses are mainly undertaken in the earlier phases of the design process to determine the feasibility of different measures. Consideration of geotechnical aspects is also needed during the earlier phases but most detailed geotechnical analyses are carried out in the later stages of design. The degree of analysis in any particular case depends on the complexity of the system and phase of study (see Section 7.1).

2.4 ROLES AND RESPONSIBILITIES IN LEVEE MANAGEMENT

2.4.1 Participants in levee management

There are many participants or 'actors' (individuals and organisations) involved in flood risk and levee management who need to interact together and communicate information so that they can perform their relevant roles and responsibilities efficiently and effectively.

2.4.1.1 Internal and external 'actors'

In Figure 2.18, two main groups of actors (or stakeholders) are shown. The internal actors are typically professionals who specialise in the fields of:

- flood risk reduction policy
- regulatory enforcement
- flood system management
- levee design and construction.

The actors work relatively closely together, or have direct responsibilities to one another.

Interfacing with this group (and surrounding them in Figure 2.18) are the external actors who may have a special, temporary or periodic interest in flood risk/levee management for a variety of reasons. Particular connections can be identified between some groups of internal and external actors. For example, planners often work closely with responsible authorities when proposing planning applications in flood risk areas. Most external actor groups do not communicate on a continual basis about flood risk management with each other (or with the internal actors). However, they may be interested in or use information provided by the internal actors.

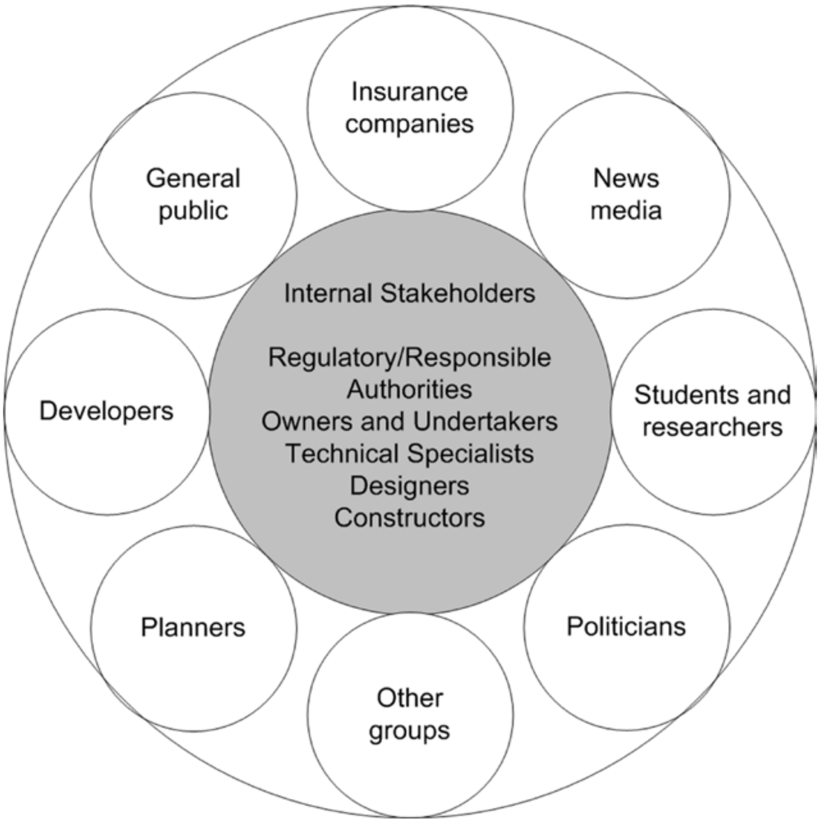

Figure 2.18 ***Internal and external stakeholders***

Another view of the actors involved in flood risk management and levee management is shown in Figure 2.19. This view divides them into three categories of those who are:

- responsible for actively delivering management measures and risk reduction
- affected by the decisions taken
- interested parties.

These categories are not necessarily distinct but can and do overlap to some degree. These interactions serve to increase the complexity of relationships between participants.

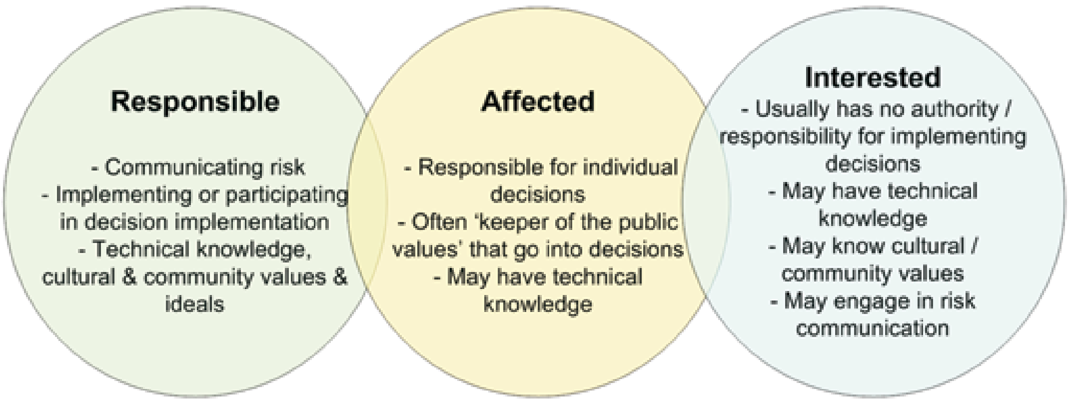

Figure 2.19 ***General stakeholder categories***

Actors can also be categorised in terms of the level of engagement (Figure 2.20) they have with flood risk management generally or levee management in particular.

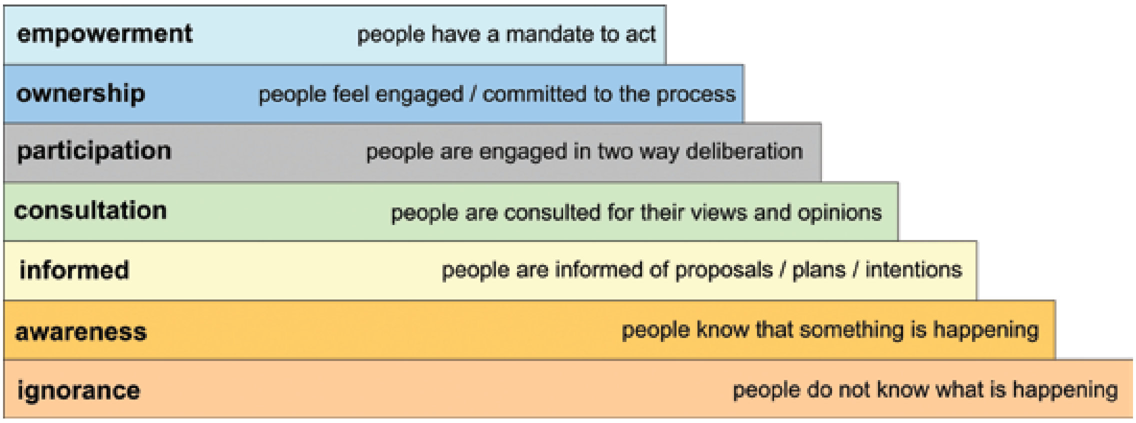

Figure 2.20 ***Levels of stakeholder involvement (from FLOODsite, 2009, after Arnstein, 1969)***

2.4.1.2 Institutional arrangements

Those actors formally engaged in communication about flood risk and levee management in any one area, depend on the situation. Large rivers often have a dedicated authority for their management, but this does not necessarily cover flood risk – which resides in the floodplains (sometimes they are not even responsible for levees). Countries with very large flood-prone areas have dedicated institutions that were formed especially to manage and communicate water and water related problems. In countries with relatively small flood prone areas, it is usually just one task among many others for institutions responsible for the general management of the environment.

In most countries, there is a division of tasks between institutions who are responsible for water management (including flood hazard reduction) and those responsible for spatial planning. This requires *horizontal co-operation* between those institutions. Often there is a division of tasks between national, regional and local authorities. This requires *vertical co-operation*, especially when there is a danger of costs being transferred to another level.

Preventive flood risk management may usually be the responsibility of water managers and spatial planners, but during a flood event it is usually other parties that know what to do and take charge including contingency planners, police, fire brigades and emergency medical responders etc. This also requires *co-operation through time*, as these parties are sometimes not aware of how a particular flood will behave.

As flood risk management measures and policy instruments affect all kinds of property owners, sponsors and other stakeholders, many other authorities and various public, semi-public and private agencies may need to be involved at one time or another. Again this requires much co-operative effort, as well as the establishment of direct lines of responsibility and arrangements for decision making.

2.4.1.3 Working partnerships and public participation

It is generally easier to achieve results working together than alone. Working partnerships and public participation in flood risk management have become more prevalent in many countries in the last few decades as more levees have been built and increased development has taken place in floodplains. Collective partnership approaches to decision making can be encouraged through greater public participation in the risk management process. Investments to reduce flood and coastal risk can deliver a range of other benefits such as increasing tourism and amenities as well as enabling regeneration.

2.4.2 Communication – why and how?

Various factors influence how and why communication occurs, such as:

- roles and responsibilities

- level of dependency/capability
- corporate and public liability
- operating processes and procedures
- level of management integration
- culture/historic communication pathways.

Communication pathways can be complex and any of these can change over time, necessitating subsequent changes in the level, type and degree of communication. However, some form of communication of information from one person or group to another is required for all of the activities and events in the levee management life cycle diagram in Figure 2.14. This communication may be in the form of policy, reporting, warning, debate, advice or guidance, or simply as news of an event or action.

A generic governance hierarchy of internal (or direct) actors involved in flood risk and levee management can be identified. Figure 2.21 shows this hierarchy, which starts with laws and policies in various areas, and disciplines that affect levee management being enforced and upheld by the relevant regulatory or responsible authorities who in turn provide guidance to levee managers, owners and undertakers. Those responsible for managing levees typically then commission designers, contractors and consultants and so on, to implement actions and to ensure compliance with the original laws and policies.

The *flow* of communication in this governance hierarchy is not entirely one-way, especially where clarification or agreement is required between the various parties on aspects devolved down the hierarchy.

2.4.2.1 Communicating risk

It is important to effectively communicate risk and residual risk of flooding with communities that could be directly or indirectly impacted by a flood event. Communication about the nature of risks serves many important purposes and is a crucial part of a flood risk management plan involving levees. Although risk communication does not directly affect the physical performance of a levee system, it is integral to emergency operations and efficient evacuation. Communities need to be provided with accurate information and clear explanations of circumstances and warnings about potential events.

Risk communication should be integral to assessment and management processes, and is not just carried out after decisions have been made (Moser *et al*, 2007). It ensures that the decision makers, other stakeholders and affected parties understand the process of risk assessment.

Risk language and the audience

Risk can often be communicated through the variety of probabilistic risk and performance metrics (see Section 5.2) that are also useful for risk management decision making. However, the use of specialist risk terminology when communicating the level of flood risk offered by any defence structure can often cause problems. The use of flood risk terms such as 'a 100-year event' outside of the flood risk management industry can often lead the public to believe that this means they are free from flood risk within their lifetime. This is a common misunderstanding of probability and of risk (see Box 2.8).

Successful risk communication is difficult to achieve and it will frequently be necessary to engage diverse groups via various media. These audiences may hold different values and have different levels of understanding, and the interpretation of a message can be dependent on a variety of social factors. So, communication objectives should be clearly defined with the goal of engaging stakeholder groups at all points in the risk management process.

As a general rule the messages should be kept simple and straightforward. The general public, decision makers and local authorities may not have the time or the inclination to understand probabilities and statistics when they have many other concerns in daily life. One of the most important points to convey is that (despite the introduction of measures such as levees to reduce it) 'there is still a risk of flooding'.

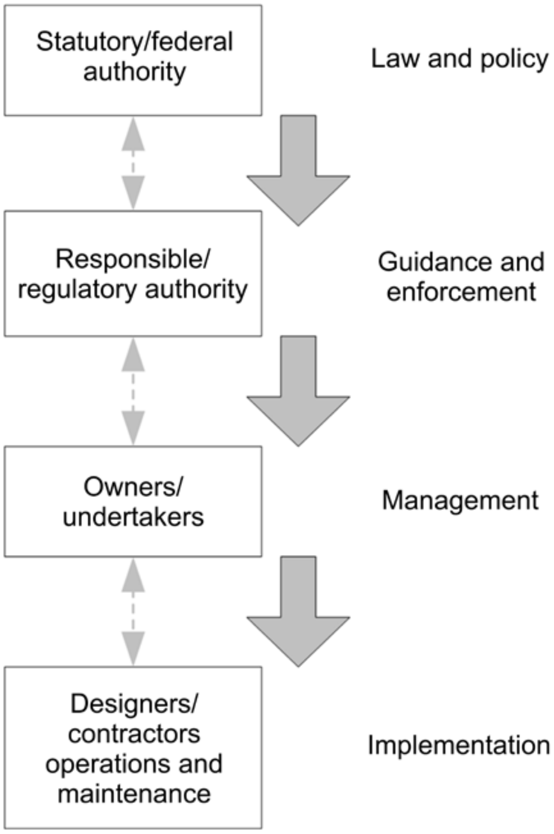

Figure 2.21 A generic levee governance hierarchy

2.4.2.2 Communication planning

A levee safety communication plan is a recognised way of ensuring that the right information is passed to the right people. A levee safety communication plan involves the dissemination of information by the responsible organisation in tandem with the levee owner (sponsor of the levee, local government etc). If possible:

- use appropriate methods and media to convey information in non-technical terms
- embrace and communicate uncertainty
- provide consistent actions, words and messages that are responsive to the concerns and values of others
- anticipate questions from the public and develop responses.

Public understanding of a risk situation is often binary, ie "…am I safe? Yes or no?" The question can only be answered by the individual, but responsible organisations should prepare and provide information to help the public evaluate individual circumstances.

Communication during a flood event

Effective and efficient communication during a flood event is essential to prevent loss of life and avoidable damage to property and levee owners/managers should be involved in this process. Communication methods and arrangements for emergency flood management require careful planning and testing. Chapter 6 gives more information on emergency planning and communication particularly as it relates to levees.

2.4.2.3 Forms of communication

The forms of communication used reflects the variety of the nature and intent of actors and their activities. Means of information transfer include:

- public meeting/forums
- conferences and seminars
- books
- maps
- project/technical reports
- monitoring/sampling records
- design/engineering drawings
- newspapers
- websites (social media)
- public broadcast (TV/radio/webcast/textcast)
- public notice/information boards
- brochures and leaflets.

The nature of communication also varies with its intent. Sometimes the intent is to inform specific people or groups of a specific outcome (eg the results of an analysis of risk for levee system managers). At other times, the focus is on making information generally available to the public, in which case tools such flood zonation maps or factsheets might be used (see Box 2.12).

Such general information can also be provided directly to visitors of levees and passers-by in the form of information boards and notices (Box 2.13).

Access to, and use of, other information types may be tightly controlled by the data owners. For example, monitoring or inspection records may be stored and only communicated to internal actors/stakeholders who need to make use of it.

Box 2.12 An example of a flood mitigation factsheet (Defra, 2012)

Flood and Coastal Erosion Risk Information
Understanding your risk and preparing for flooding

May 2012

This factsheet has been prepared to help you understand what flood and coastal erosion risk information is available to you. It also provides some tips on what you can do to prepare for flooding if you find out your home or business is in an area at risk.

Floods can happen anywhere, at any time. Flooding is caused by a variety of factors including rising ground water levels, sewers overflowing, run-off from heavy rain as well as flooding from rivers and the sea. Even if you live miles away from the sea or a river, there is still a chance that other types of flooding could affect you.

Sources of information on flood and coastal erosion risk

- **River and sea flood maps** The Environment Agency (EA) produces flood maps to set out the risk of flooding from rivers and the sea. They are available through the EA's *What's In Your Backyard?* web pages or from your local EA office.

More information on data management is given in Section 5.6.

Box 2.13 *An example of a public information board in Switzerland*

As well as providing warnings, information signs and boards can prove useful and informative to visitors/users about current issues or, for example, recent developments to levees such as repairs or renovation.

Figure 2.22 ***A public information board highlighting issues to do with burrowing animals on a levee in Switzerland***

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3 Functions, forms and failure of levees

Courtesy B Landreau

CHAPTER 3 CONTENTS

3 FUNCTIONS, FORMS AND FAILURE OF LEVEES

Chapter 3 introduces the form and function of levees and provides an understanding of failure mechanisms. Explanations of these concepts will be useful to all users for both assessment and design.

Key input from other chapters

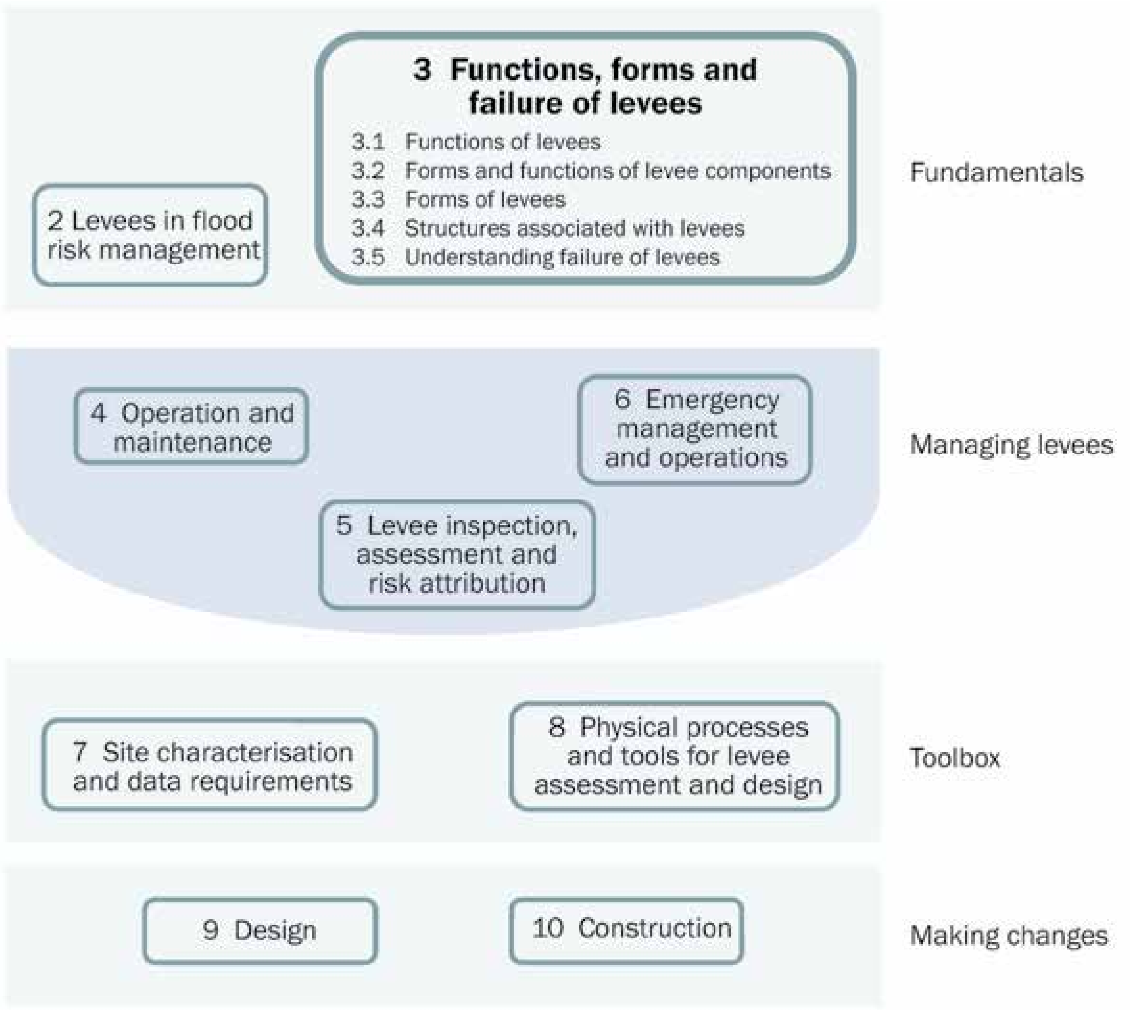
- Chapter 2 ⇨ **flood risk management context**

Key output for other chapters

- **forms, functions and failure mechanisms** ⇨ Chapters 4 to 10

Note: The reader should revisit **Chapters 2** and **3** throughout the levee life cycle for a reminder of important issues.

This flow chart shows where to find information in the chapter and how it relates to other chapters. Use it in combination with the contents page to navigate the manual.



CHAPTER CONTENTS AND TARGET USERS

This chapter is divided into five sections, providing an overview of levee functions, forms and functions of levee components, forms of levees, structures associated with levees, and failures of levees. The primary focus for the chapter is related to fluvial, estuarine and coastal earth structures, however, sand structures as well as canals with banks will also be addressed. Flood defence structures that are not discussed in detail within the handbook are flood and seawalls, dams, groynes, jetties and dredged tailings.

Functions of a levee within a flood defence system

Section 3.1 introduces, defines and describes levee functions within the overall context of a flood defence system. Topographic conditions and environmental considerations that may affect the levee are detailed within this section. Multi-functional roles of levees and the necessary co-ordination of the functions are addressed along with the evolution of the levee.

Main components of a levee, their forms and functions

Section 3.2 presents the various components that may be part of the structure of a levee. It defines each component and explains component functionality within the levee, underlining specific technical issues.

Variations in levee type and form

Section 3.3 presents different types and forms of levees where each is demonstrated with schematics reflecting levee structural components and typical cross-sections. Common weak points within levee systems are identified and historic case studies are presented.

Complementing structures installed in addition to the levee embankment for flood defence

Section 3.4 presents appurtenant structures that are associated with levees. Each structure is defined and described according to its function or effect on the flood defence system. Typical sketches and illustrations of various structures are presented. A distinction is made between structures that are linked and specially designed for flood defence and structures that are not. The importance of transition zones with natural or manmade structures within the line of defence is addressed.

Failure of levees

Section 3.5 discusses failure modes of levees using breach, damage and deterioration categories. Levee performance, deterioration and failure are defined with descriptions of the main elementary mechanisms of failure. Scenarios of failure involving the combination of elementary mechanisms are described through sample diagrams and the kinetics of failure scenarios are described. Varying perspectives of levee failure analysis are also covered.

3.1 FUNCTIONS OF LEVEES

3.1.1 Levees within the overall flood defence system

Levees are often the principal component of a flood defence system, although the system may also include manmade and natural structures complementing one another to reduce the risk of inundation for a designated area (Figure 3.1). Flood defence systems should be designed with features that are appropriate for the specific site characteristics whether fluvial, estuarine or coastal. In addition to the levee, manmade features may include spillways, flood walls, sluices, under-seepage control measures, pumping stations and dams whereas natural structures may be comprised of dunes, cliffs, swamps and wetlands. The intent of using these complementing features is to reduce the likelihood of fatalities, economic loss and environmental damages.

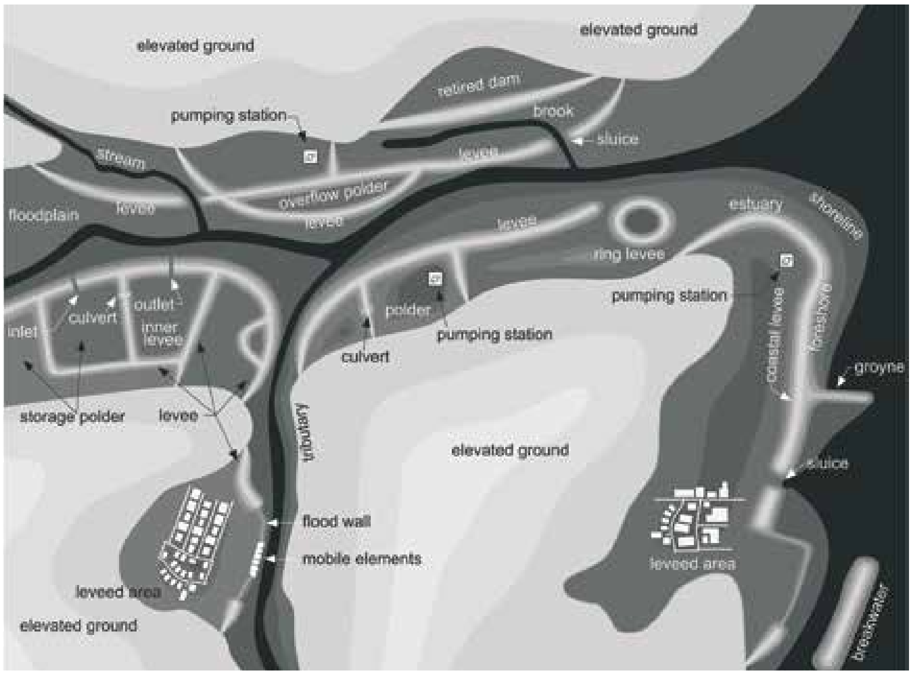


Figure 3.1 Levees within the overall flood defence system (courtesy Reinhard Pohl)

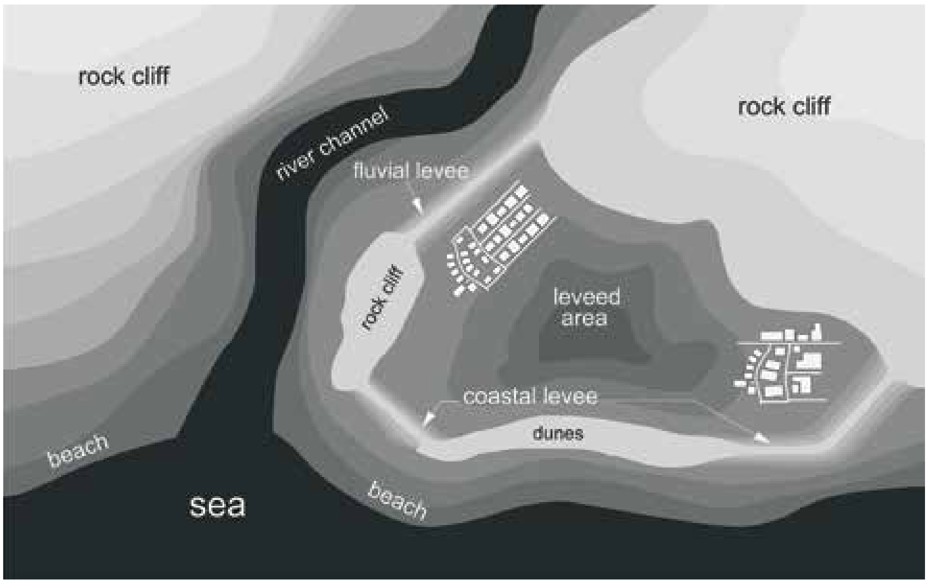


Figure 3.2 Example of defence system including fluvial, coastal and natural structures (adapted by R Tourment)

Levees have three primary hydraulic functions:

- **retain:** to reduce the risk of inundation of an area by temporarily retaining water, keeping it out of the leveed area to a defined water level, and to avoid flooding conditions within this area
- **channel:** to channel floodwater downstream or into a non-leveed area to avoid inundation of the leveed area
- **control release:** to provide a controlled release of water in a designated location that will minimise inundation downstream.

Any levee can have one or all of these functions. However, there is one exception to this list – canals, where the function is to keep water contained within the confines of the canal or the land immediately next to it.

A defence system is designed to reduce the risk of inundation within the leveed area. The height of the levee is typically set by an anticipated water level, often based on historical trends (Section 7.3.3). A defence system achieves containment in a complex array of several flood control features. It generally has to rely on natural structures to ensure that the leveed area is closed in its entirety (Figure 3.2).

In a fluvial environment, the first or main line of defence is usually parallel to the passing floods. The aim of the defence system is to channel water downstream of the river and to reduce the risk of flooding to nearby areas. Levees may be constructed along both banks with one embankment set back from the river channel to incorporate part of the floodplain (Figures 3.3 and 3.4). In this case, during flood conditions, water will flow along both the river channel and the contained section of floodplain. The area between the levee and the river channel (floodplain) is not protected and is subjected to a higher water level during flood conditions. This scenario provides for added storage capacity during high water. Under normal, non-flood conditions, secondary use of this area may be allowed for farming operations, recreation or other approved uses.

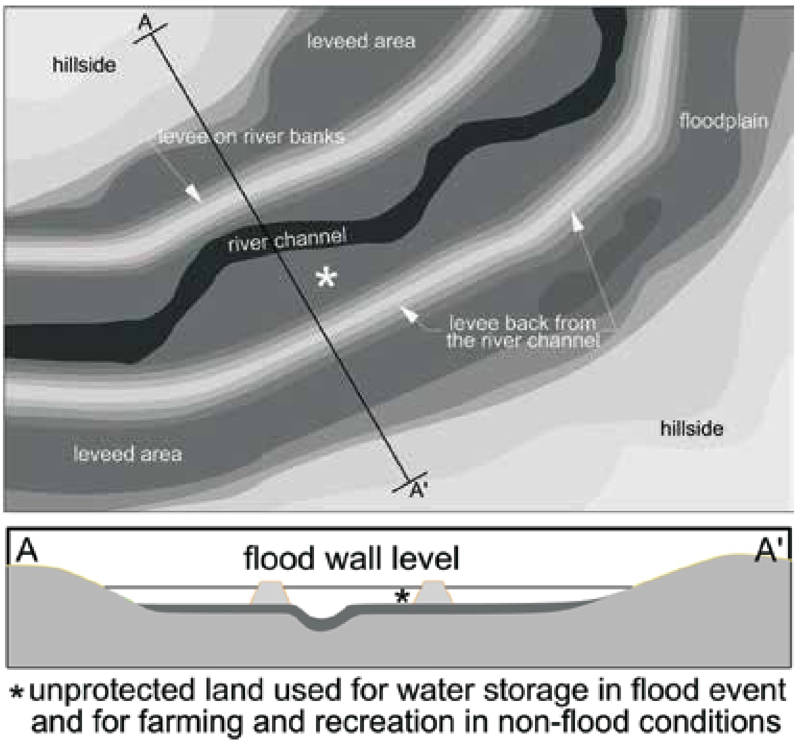


Figure 3.3 *River channel levees (adapted by Y Deniaud)*

In the coastal environment, the defences are usually perpendicular to the incoming flow from the sea. The aim of the defence is to reduce the risk of crossing and overflowing of heavy seas and to moderate wave overtopping. Although it is not possible to channel the flooding sea, water can potentially be stored in closed areas when adequate space is available between the sea and the leveed area.

3.1.1.1 Setback levees along the river

Levees positioned with a buffer zone from the main river channel (ie not along the riverbank as shown in Figure 3.4) are typically referred to as setback levees. Once a setback levee is constructed, its existence may result in higher water surface elevations, larger fetches and higher wave run-up than without levees. However, the water surface elevations are lower when compared to a levee along the riverbank. Setback levees can promote floodplain restoration because their location directly affects the amount and distribution of riparian and aquatic habitats (Konrad *et al*, 2008).

To control inundation within the floodplain or within areas frequented by overtopping of a levee situated closer to the river channel, a setback levee may be constructed. Former flood defence features close to the river may be left in place, but would no longer serve as primary flood defence.

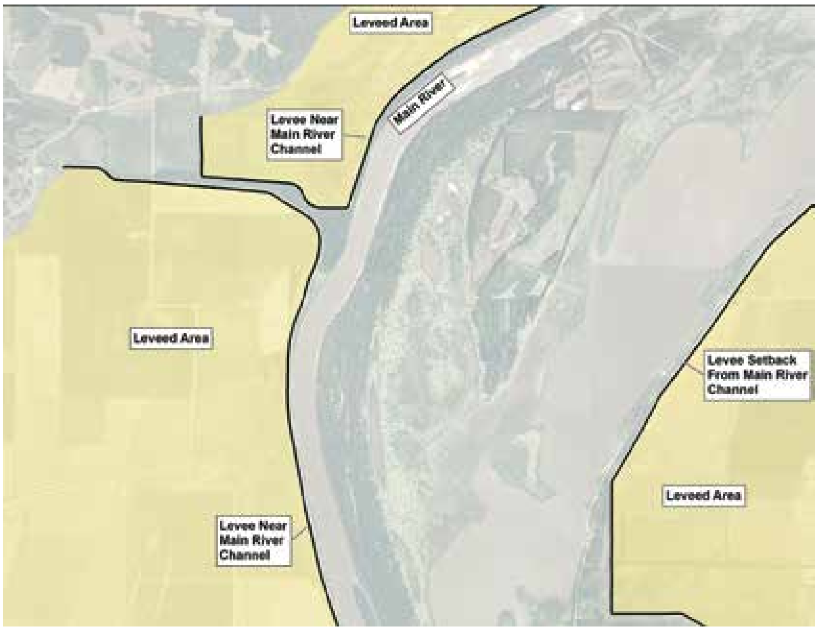


Figure 3.4 Setback levees (adapted by Y Deniaud)

3.1.1.2 Closed protection levees

For closed protection of a specific area, a ring levee may be built around a selected zone (Figures 3.5 and 3.6). Also, levees can abut hills or existing topographic features that are at higher elevations and not subject to erosion (Figure 3.7).

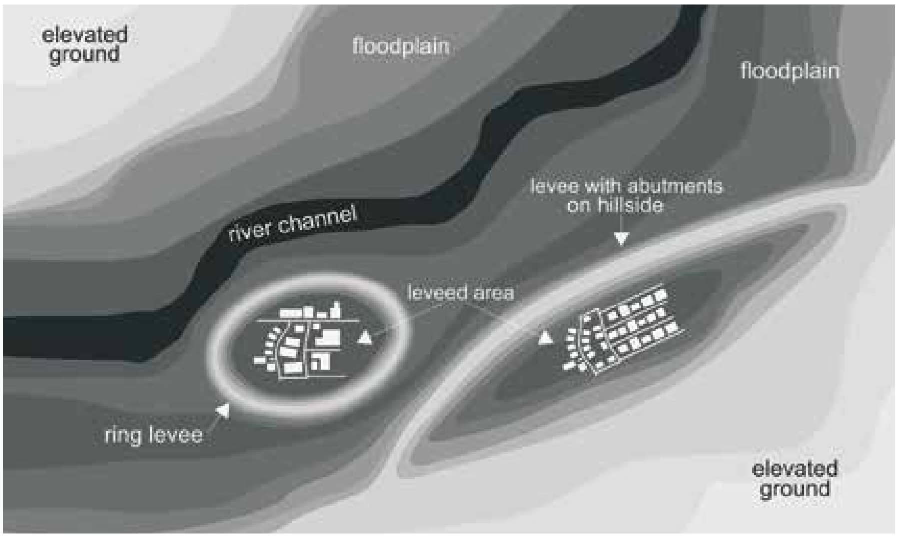


Figure 3.5 Closed protection levees (adapted by Y Deniaud)

Figure 3.6 ***Ring levee, Kaskaskia Island, Illinois, USA (courtesy St Louis District, USACE)***

Figure 3.7 ***Levee abutting a natural topographic feature, southern Illinois, USA***

3.1.1.3 Flood storage

There are two basic methods of storing floodwater that may make use of or affect levees:

- off the river alignment, with a bounded basin next to the river or a tributary emptying into the river
- on the same alignment as the watercourse with a dam constructed across the river.

Offline storage areas are often created through the complementary use of levees, spillways, sluices and other manmade structures, as well as other natural structures and topography. Such storage areas are only used during flood events and are normally empty for long periods of time.

Online reservoirs and lock and dam systems are also used for flood storage, but may be constructed for many or a combination of reasons including flood attenuation, water supply, recreation and power generation. Reservoirs are created by using natural and manmade structures to retain water behind an impermeable barrier of some type. Similar to fluvial levees, reservoir dams perform the function of retaining water. The most distinguishing difference in function is that reservoir dams must resist permanent hydraulic loadings, while fluvial flood embankments are subjected to hydraulic loadings for only a finite duration. The transition from fluvial flood embankment to embankment dam can be gradual.

Water flow in a detention reservoir is either natural or controlled by spillways near the crest of the embankment (Figure 3.8). The reservoir may provide extra upstream storage capacity that may be used to minimise downstream inundation during high water events and reduce loading on levee systems along the channel. These structures are designed to retain water temporarily, and their failure could result in uncontrolled releases of water. Reservoirs may serve multiple functions, including the storage of water during flood seasons. Figure 3.9 demonstrates the controlled release of water from a reservoir at the spillway into a designated floodway.

For data requirements and design considerations relating to internal drainage reservoirs, see Sections 7.3.2.2 and 9.4.3.

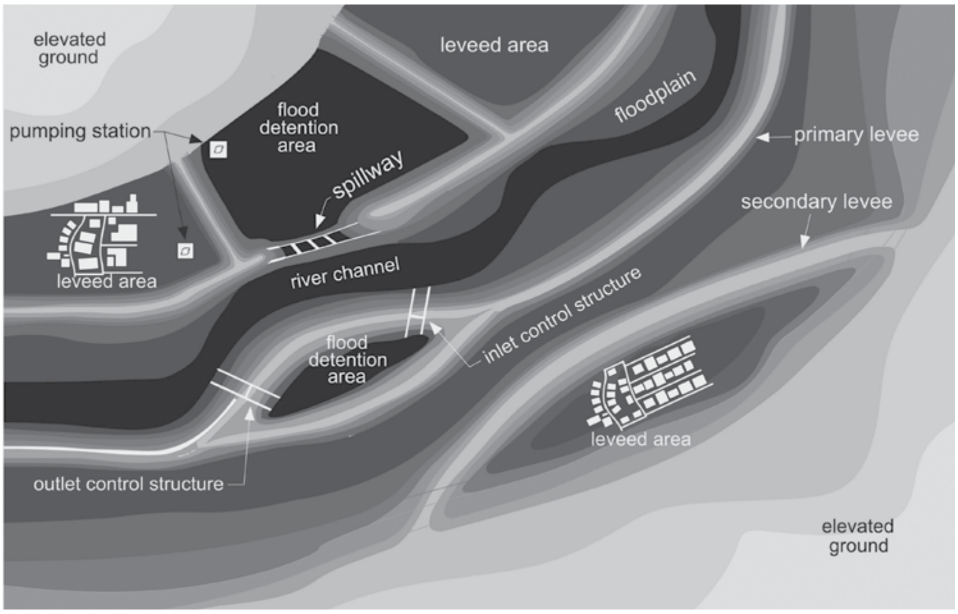


Figure 3.8 Flood detention scheme (adapted by Y Deniaud)

Figure 3.9 Spillway release from a reservoir (adapted by Y Deniaud)

To better regulate depths to help river navigation, a lock and dam may be constructed within the river channel. Typically, a lock and dam is operated as 'run of river', ie inflow equals outflow. Improper operation of a lock and dam can result in adverse consequences to the flood defence system along the river channel where high water loading may be imposed on the embankment for long durations. Figure 3.10 depicts levees that are close to the channel.

Figure 3.10 Mississippi river lock and dam (adapted by Y Deniaud)

3.1.1.4 Coastal flood defence systems

Coastal levees may be associated with other manmade or natural structures such as offshore breakwaters, groynes and dunes. The primary functions of these appurtenant structures are to prevent erosion of coastal levees and maintain the levee integrity (Figure 3.11). Section 3.4 provides further details regarding complementary structures designed for coastal defence.

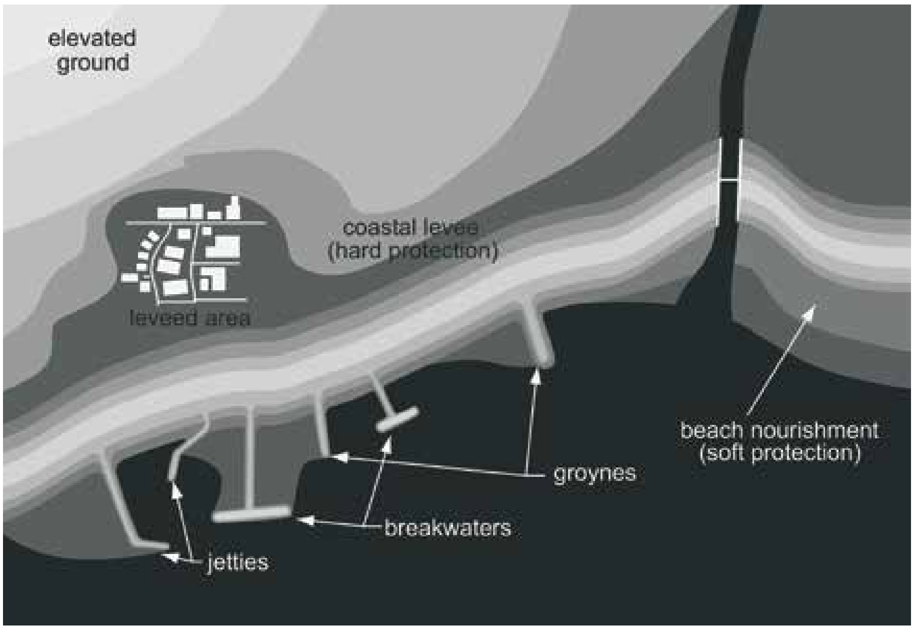


Figure 3.11 Levees in a coastal flood defence system (adapted by R Tourment)

3.1.1.5 Secondary lines of defence

Secondary lines of defence along rivers or coasts are sought by the flood following the primary lines. They might be parallel or perpendicular to the incoming floodwater (Figure 3.12). The goal of the secondary line is to:

- reduce the risk of flooding a more specific and restricted area
- confine flooding in a specific and defined area (flood detention reservoir)
- delay flooding from one sub-area to another.

Diverting water to detention areas may be accomplished by the use of an engineered spillway or a system consisting of primary and secondary levees, where the first line of defence provides resistance to overtopping up to a specific elevation and the second line of defence provides resistance to overtopping at a higher elevation.

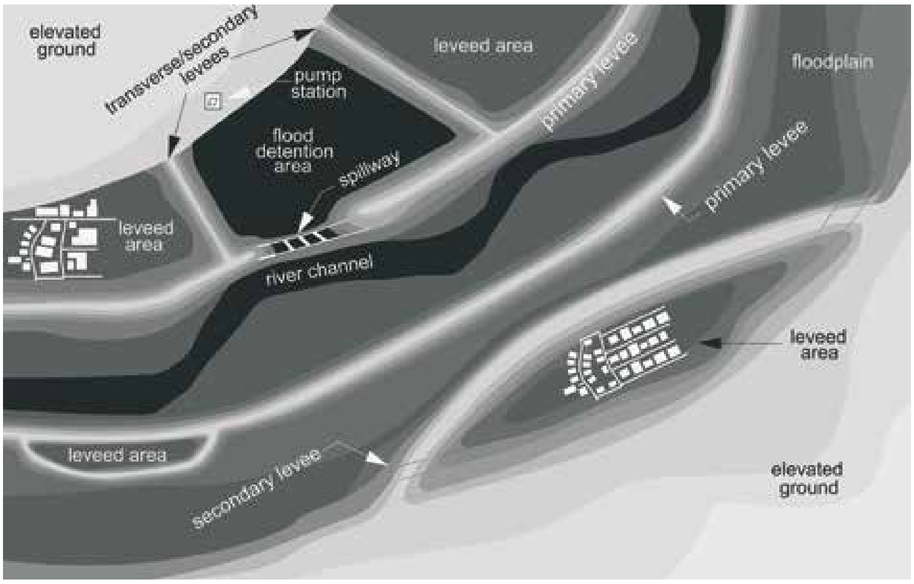


Figure 3.12 Example of primary and secondary levees along a river (adapted by Y Deniaud)

3.1.2 Multi-functional role of levees

In addition to the primary flood risk management intent, levees often serve multi-purpose uses. Secondary uses of the levee are vitally important to those living and working nearby and to those visiting the region. Secondary uses of levees vary in accordance with site characteristics but may include access routes, recreation, farming, utility crossings and both ecological improvements and environmental provisions.

There are concerns with levees that serve secondary uses, including:

- other uses of the levee may become priority over flood defence
- extraneous features may not be designed/constructed with materials appropriate for flood defence
- operation and maintenance of ancillary levee features, serving a secondary use, may not be effective.

3.1.2.1 Access and transportation

Levees are barriers between a river or coast and the population, which result in the need for viable access routes to the water body. There are often features on the waterside of the flood defence system which people have a desire to access for recreational purposes or simply due to aesthetic appeal (Figure 3.13).

Figure 3.13 *Waterside attraction, East St Louis, Illinois, USA (courtesy St Louis District, USACE)*

Access may be accommodated across the embankment or through constructed closure gates (Figure 3.14). Consideration must be given to how people might legitimately access the river or coast that lies beyond the levee without an adverse effect on its performance. Passive design solutions, such as replacing gates with ramps, should then be promoted.

The levee crest may serve as a road for vehicular travel by the general public or operators and maintainers of the levee (Figure 3.15) or as a foot/bicycle path for access to a nearby recreational venue. Integrating these secondary uses into levee planning and design without affecting the integrity of the levee is vital. Access points through the levee are open gaps through the flood defence system and should be properly maintained in preparation for emergency closure. Post-construction monitoring is also critical to ensure that the access route is maintained and does not threaten the primary flood risk management role of the levee. For example, roadway maintenance may involve an asphaltic surface overlay covering the closure structure pockets that are embedded in the pavement, or the installation of safety guardrails across the line of closure. Either of these may prevent the operator from effectively setting and securing closure panels in preparation of high water.

Figure 3.14 *Floodgate access (courtesy Symadrem)*

Figure 3.15 *Danube levee access ramp near Regensburg, Germany (courtesy Reinhard Pohl)*

Road and railway embankments are constructed for transportation rather than flood risk management (Box 3.1). However, in some instances these embankments tie directly into the levee and later serve as part of the line of flood defence. Although the road or rail embankment may have been constructed before the levee, a cost- or time-saving measure may have been employed, allowing the levee alignment to tie directly into the embankment. Perhaps at the time of road/railway construction, material constituents and methods of construction were technologically consistent for transportation and flood control embankments, so tying the alignments together was not a concern. Allowing roadway or railway embankments to serve as a portion of the line of defence increases the level of risk associated with the levee system. However, most railway embankments serving in a secondary flood defence function were constructed over 100 years ago and were built from a wide range of relatively poor quality material. Often there is a lack of documentation related to the original construction and so material constituents are unknown without conducting invasive sub-surface testing. These embankments can suffer serious damage if subjected to high floodwater levels with consequences of settlement or even collapse.

Box 3.1 *Unintentional flood defence systems in the USA*

Within the USA there are many instances in which a constructed railway or highway embankment serves as a segment of the line of flood defence, where the railway/highway embankment ties directly into the levee profile (Figures 3.16 to 3.18). This situation causes concern because typically there is no information available that demonstrates either the material constituents or the methodology of the embankment construction.

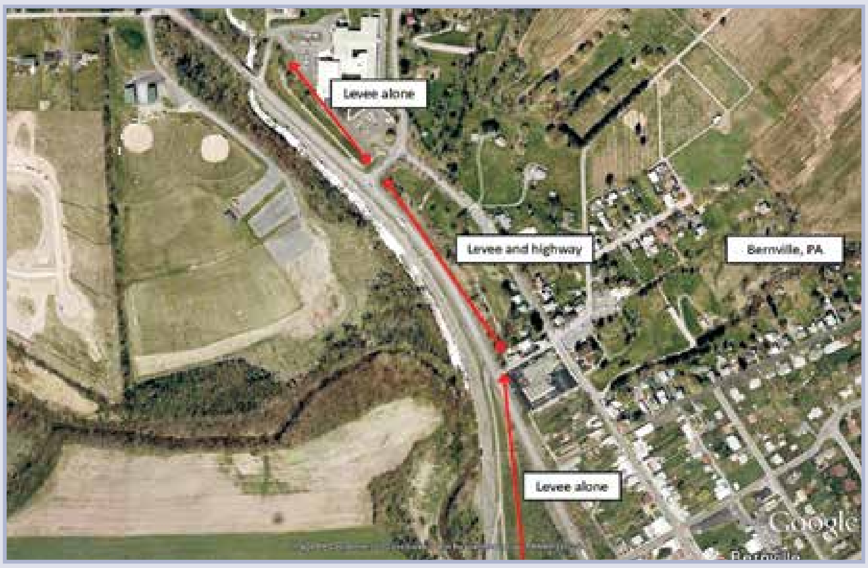


Figure 3.16 *Highway embankment serving as line of defence, Bernville, Pennsylvania, USA (courtesy USACE)*

Box 3.1 Unintentional flood defence systems in the USA (contd)

Figure 3.17 Highway embankment serving as line of defence (courtesy USACE)

Figure 3.18 Railroad embankment serving as line of defence (courtesy USACE)

3.1.2.2 Recreation

Levees may take on a multi-purpose use to accommodate recreational activities such as walking, jogging, bicycling or fishing on the crown or within the cross-section of the levee (Figures 3.19 to 3.21). Embankments provide a higher elevation to view the river or coast, which attracts people. The allowance for such recreation promotes better awareness of the ecology and natural surroundings, which can be educational and informative. Also, levee property that is open to public use provides opportunities for more surveillance of the flood defence system.

Figure 3.19 ***Paved trail on levee crown for pedestrian use, Alton, Illinois, USA***

Figure 3.20 ***Paved/landscaped trails on levee crown, Chesterfield, Missouri, USA***

Careful selection of material properties for the levee crest and recreation features constructed on or near to the levee is important to ensure its integrity. For example, Figure 3.21 displays a privacy fence situated at the toe of the levee that has pin connections and joints for easy removal.

Figure 3.21 ***Aesthetic fencing along levee toe (courtesy Les Perrin)***

Where the levee crest is extremely wide, there may be the opportunity to install park features such as benches and pavilions (Figure 3.22). These features should comprise materials that can be easily removed during high water events and constructed with shallow foundations, not adversely affecting the cross-section of the levee.

Figure 3.22 *Loire River Levee Park, France (courtesy Jean Maurin)*

Provisions to accommodate people may have adverse effects on the levee embankment. For example, recurring recreational vehicle use on the crest of the levee may result in rutting and erosion that creates a preferential route for overtopping and potential damage during a flood event. Also, open access to the public may lead to vandalism.

3.1.2.3 Agriculture

Using levees, including land or waterside berms, for agricultural practices is quite common (Figure 3.23). These practices may consist of grazing animals and, in some cases, the use of no-till crops on berms.

One advantage associated with the allowance for grazing animals within the cross-sectional area of the levee includes the control of vegetation growth (Figures 3.23 and 3.24). Sheep are typically preferred over cattle because sheep tend to graze continuously and cattle over-enrich the soil, which promotes unwanted root formation.

Figure 3.23 *Sheep grazing on levee embankment at the Ley Bay, East Frisia, Germany (courtesy H Schuettrumpf)*

There are disadvantages with grazing animals including excessive surface rutting, destructing surface vegetation, overgrazing and puddling. Puddling is because of hoofed animals congregating in a specific locale for food, shelter or access through confined areas such as gates. When using farm ground within the cross-section of the levee for grazing animals, provisions need to be made to ensure animals have viable paths to flee from rising floodwater to high ground.

Figure 3.24 Controlled animal grazing operation (courtesy Les Harder)

Positive benefits of planting crops on levee berms are profit generation and secondary use of the property. Plants should be limited to no-till crops that are not deep rooted such as soy beans, corn or milo. Row crops that are tilled should not be planted within the immediate area of the levee because ploughed areas may adversely affect levee integrity and areas that are wet or soft will hinder access for levee inspections and flood fighting operations. Farming operations can also result in a loss of berm material over time, compromising the effectiveness of the berm.

Disadvantages of planting crops on levee berms include a lack of visibility of the berm surface during inspection, the possible attraction of burrowing animals and drainage provisions. Fields of crops that are near to the levee may have ditches to help drainage. Ditches that are excavated to excessive depths close to the levee may provide a path for under-seepage. Alternatives for capturing runoff should be considered before constructing landside ditches near the levee toe.

3.1.2.4 Environmental and ecological improvement

Levees constructed for flood defence may also serve other uses that improve the environment or sustainability. Environmental and ecological considerations should be considered at every site where levees exist or are to be built.

Protection of environmentally sensitive sites or inland property

Levees may serve a dual purpose by providing a barrier for a site that has been deemed environmentally sensitive (Figure 3.25) – the site may have historical/archaeological significance or may serve as a habitat for various species of plants and animals. For instance, wetland or saltmarsh habitats provide protection from wave action and rising and falling water levels.

Figure 3.25 ***Wetland near to flood defence system that is deemed environmentally sensitive Alton, Illinois, USA (courtesy St Louis District, USACE)***

In some instances, formerly constructed levees now impound wetland areas that provide habitat, improving the interaction of animals with this newly developed environment (Figure 3.26).

Figure 3.26 ***Wetland near to flood defence system providing protection from wave action, Alton, Illinois, USA (courtesy St Louis District, USACE)***

Often, conflicts of interest arise between embankment maintenance operations to retain levee integrity and provisions for animal habitat. Care must be taken to ensure that animal activities do not adversely affect the primary flood risk management function of the embankment. For example, some animals that inhabit riverine and estuarine environments have a tendency to burrow (Figure 3.27). Animal burrowing results in voids within the embankment that can lead to seepage and piping of embankment materials. The construction of setback levees is ideal to accommodate animal habitat and the riparian corridor. Clear jurisdictional authorities should be established providing reasonable mitigation for ecological areas that are disturbed.

Summer/winter levee systems

A summer/winter or primary/secondary levee configuration allows for the creation of a wetland, for at least part of a calendar year, in the area between two levees (Figure 3.28). The winter levee is intended to retain the peak discharge of the river while the summer levee retains minor floodwaters. This levee system configuration enables the land between the winter and summer levees to be used for agriculture or recreation during the summer months.

Figure 3.27 Opened burrow of musk rat, levee along canal in Friesland (NL) (courtesy Henk Van Hemert)

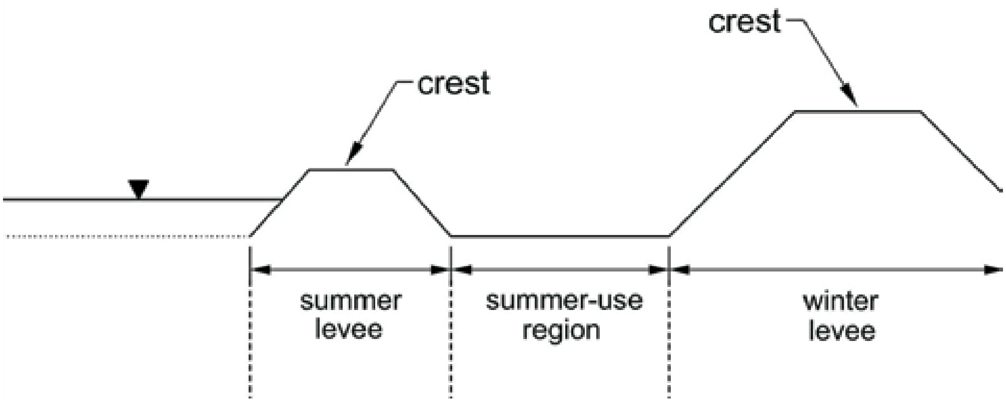


Figure 3.28 Summer/winter levee configuration

Alternatives to improve environmental attributes

There are various engineering options that can be used specifically to improve environmental performance as well as structural performance. One example is longitudinal peaked/filled stone toe protection (LP/FSTP) (Figure 3.29). Longitudinal toe protection can be an attractive alternative to more conventional types of revetment such as rip-rap, articulated concrete mattresses and other forms that do not share the potential for simultaneous ecological enhancement/bank stabilisation. Longitudinal stone toe protection helps with stabilisation while providing cover and habitat for small fish and other organisms.

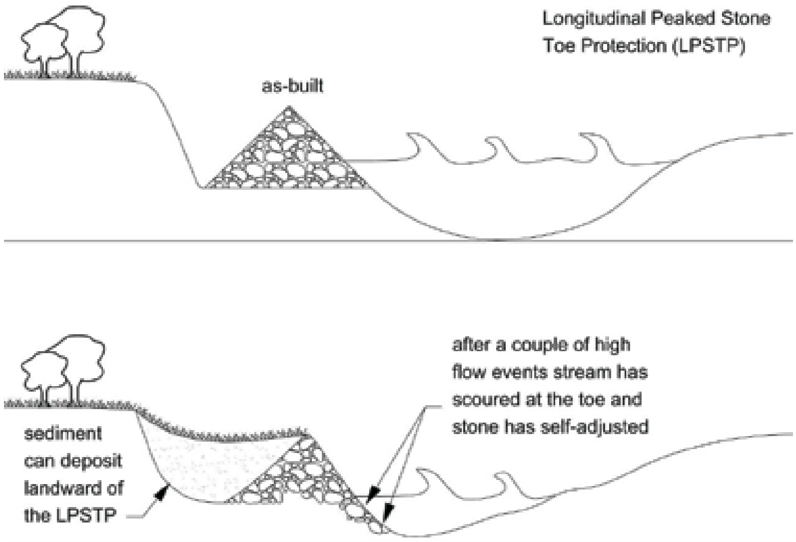


Figure 3.29 ***Longitudinal peaked toe stone protection (courtesy David Derrick, USACE)***

Protected and endangered species

The grass cover of levees can have an ecological value and using seeds of native provenance would be advantageous. Levees can also form linear corridors to link habitats and allow species to migrate along them. See Sections 10.1.3 and 10.3.1.1 for consideration of protected and endangered species during construction-related activities within fluvial and coastal environments and Sections 4.1 and 4.6 for consideration during operating and maintenance.

Coastal vegetation may also provide animal habitats that necessitate human intervention to safeguard these sites. For example, Figure 3.30 depicts a protected bird habitat.

Figure 3.30 ***Protected bird habitat along coastline, Siesta Key, Florida, USA (courtesy J McVicker)***

Noise attenuation

The existence of a levee can assist with noise attenuation by absorption, which provides noise protection to those on the side of the structure. Strategically positioned embankments may help alleviate noise pollution from populated areas near to the river or coastline (ie from ships or barges).

Aesthetic appeal

A well-established, vegetated levee provides aesthetic appeal, in particular where flora helps to integrate the embankment into the local environment, reducing visual effects. Levees also provide green space in urban corridors where the natural attraction of water increases public interest and use.

It is important to ensure that naturally grown or planted flora does not negatively affect the levee. For example, excessive vegetation can affect the embankment and impede levee inspections, whereas limited vegetation may be prone to erosion (Section 4.5 for details on levee vegetation).

3.1.3 Levees in their environments

When dealing with the functionality and evolution of a levee, it is necessary to take into account and to fully understand all aspects of its surrounding environment, and specifically induced loadings, the morphology of the watercourse and coastline, sedimentation, vegetation and climate change.

The form of a levee and its components depend on the environment where the levee is situated (Section 9.2.1). This section describes the relation between some characteristics of the environment of the levee and its form, including specific components that may be required to fulfil its water retaining function.

3.1.3.1 (Fluvial, coastal and estuary induced) hydraulic loading/hydraulic environment

Levees are subject to varying forms of (dynamic) hydraulic loading, which may be derived from:

- the water level (including variations of the high water level causing hydrostatic actions on the levee and influencing the internal hydraulics)
- currents (causing external erosion of the waterside slope and decreasing the stability of the waterside slope through undermining of the foreshore or toe of a levee)
- waves (causing rapid erosion of the waterside slope of a levee, and overtopping due to wave run-up which can lead to external erosion of the crest and landside slope of the levee).

Levees need to withstand all these different hydraulic loads, including the resulting internal hydraulic pressures.

The occurrence of these different loads depends on the hydraulic characteristics of the environment that the levee is situated in. Each hydraulic load affects the required design of the levee. So the form of a levee and its components strongly depends on the acting hydraulic loads, and on the hydraulic environment where the levee is situated. Table 3.1 presents an overview of the hydraulic loads and their importance for levees in different hydraulic environments. For more detailed information related to loads on levees see Sections 7.3 and 7.4.

Note

There are occasions when exceptional and very specific events can occur (such as tsunamis for coastal areas), and that levees to withstand these events may require very specific treatment.

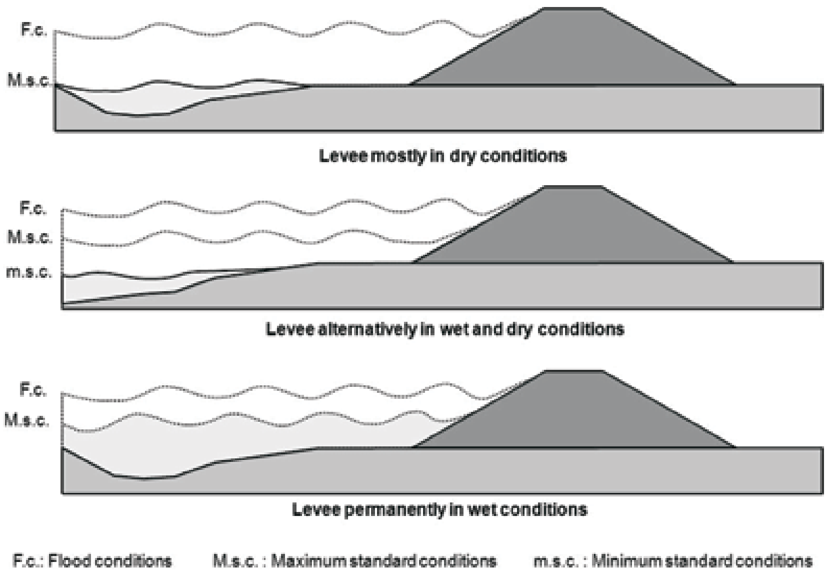


Figure 3.31 Water level: frequency of hydraulic loading conditions on levees

Table 3.1 Hydraulic loads and their relevance in different hydraulic environments

Hydraulic loading characteristic		**Fluvial environment**	**Coastal environment**	**Other environments**			
		River	*(Tidal) sea*	*Estuary*	*Lakes*	*Canals*	*Torrents*
Water level[1, 6]	Flood discharge	✓			✓[3]	✓	✓
	Tide		✓[4]	✓			
	Rapid draw-down	✓	✓	✓	✓		✓
	Storm surge		✓	✓	✓		
Local surcharges	Wave set-up[5]		✓	✓	✓		
	Seiches		✓	✓	✓		
Waves[2]			✓	✓	✓		
Currents		✓		✓			✓

Notes

Only the main influence is indicated in the table.

1 In addition to the water level, the duration of the flood (the hydrograph of the flood level) is also relevant, for example because of the transient response of water pressures on the flood. The duration of a flood stage ranges from hours (coastal levees) to weeks (levees along rivers, especially in the downstream stretches of large catchments).

2 Several other characteristics of waves are relevant for the assessment and design of the levees and its components, such as wave height, including wave steepness and wave period.

3 Lakes along a river or (partially) fed by a river may also be affected by flood discharges.

4 Some seas have minor tides.

5 Wave set-up can cause an increase in water levels within the surf zone due to waves breaking as they travel shoreward. Wave set-up has an extremely local effect on water levels.

6 Depending on their position in the defence system, levees might (Figure 3.31):
- be dry most of the time (but wet during specific flood events), typically for some river levees
- be alternatively wet and dry, typically in estuarine or coastal situations affected by tides
- permanently have water against them, such as canals and perched embanked rivers.

Extra loads imposed on flood defence systems may originate from ice, debris, construction, maintenance, high winds or boat activity causing waves, ship collisions, operational activities, human intervention or extreme natural processes, including from climate change (Sections 7.2, 7.3 and 7.6).

There is uncertainty today regarding the effects of vegetation on the floodplain or beach in front of a flood defence system on the hydraulic loading characteristics. Woody vegetation may provide benefits by attenuating waves, reducing currents and reducing wind speed (reducing wind-driven waves). But, woody vegetation (for river levees) can also lead to an increase of the water level due to increased roughness of the floodplain.

Some hydraulic loads are very dependent on the bathymetrical and topographical conditions in front of the levee. Meandering channels, moving sandbanks and the presence of beaches in front of the levee have a significant effect on the loads derived from currents and waves (Section 3.1.3.2).

3.1.3.2 River, coastal and estuarine morphology

River or coastal morphology is also termed fluvial or coastal geomorphology, respectively, and is used to describe the shape of the river or coastline and how it changes through time. Levees are built along rivers and coastlines, so the features or morphology in these environments and the processes involved in producing or altering the morphology are important when considering the design, construction and maintenance of levees.

This section describes the relation between morphology and morphodynamic processes and levee safety. For a more detailed explanation of river, coastal and estuarine morphology see Sections 7.2 to 7.5.

River morphology

Changes are continuous within a river, as the stream attempts to maintain equilibrium to balance sediments and available energy throughout its length. The longitudinal profile, the sinuosity and the meandering of a river influence the available energy in a stream, which causes sediment movement within and through the system. Manmade changes to the system, including levee construction, can affect both available energy and sediment.

Hydraulic river conditions are primarily influenced by river discharges and bed slope. Other influences include floodplains and embankments, manmade or natural river structures, roughness of the river bed and floodplains, confluences, bifurcations, weirs and spillways (Section 7.3). The morphological behaviour of a river channel is a function of numerous processes and environmental conditions. Some of these complex and dynamic processes and/or conditions are described in Section 7.2.2.

To better understand fluvial morphology researchers and engineers have developed river classification systems (Section 7.3.1). These link observed trends to fluvial and sediment processes contributing to changes within the channel configuration over time.

Coastal morphology

The stability of the coastline is dependent on the physical characteristics of the shore, which is determined by its geology, its geomorphology and the actions of winds, waves, tides and currents. The coastal zone may react differently with varying situations. Changes may occur due to 'normal' coastal processes (accretion, longshore movements, wave-induced erosion, subsidence) or 'extreme' coastal processes (storm surge inundation, storm-induced erosion, wave overtopping, barrier island breaching). Coastal morphology is a function of numerous processes and environmental conditions and controlled by the balance between the aggressiveness of physical processes, the land's resistance and sediment supply. Some of these physical processes and environmental conditions are described in more detail in Section 7.2.2.

Estuarine morphology

Once estuaries are positioned between river basins and the sea, processes influencing morphology and hydraulic behaviour originate from both river and sea. Sediment that enters an estuary may originate from either marine or riverine material, and the presence of tides and/or waves results in a complex pattern of sediment transport within an estuary. A hybrid sediment deposit environment, which is a mixture of fluvial and marine, is called a turbite system.

Effect of morphological processes on levee (safety)

Morphological processes can be critical to the proper functioning of a (static) levee system. The interaction between the levee and morphological processes is complex. The primary morphological processes that may have an effect on a nearby levee include both lateral and vertical movements:

- **lateral movements:** a shift in the position of the channel, the development of meanders, movements of sandbanks, avulsion and stream patterns
- **vertical movements:** degradation and/or aggradation of the floodplain, foreshore or tidal flat and the depth of the river or channel bed by scour and bedform migration.

Both movements may occur slowly over long time periods (several years or decades) or incidentally within a single flood event.

These processes affect the levee in terms of changes of the strength of the levee and by changes of the hydraulic loading characteristics on the levee. The effect of these processes can be either positive (increasing the strength/reducing the hydraulic load) or negative (decreasing the strength/increasing the load).

Table 3.2 presents an overview of the effects of morphological processes on a levee.

Table 3.2 An overview of the effects of morphological processes on a levee

Phenomenon	Strength of the levee	Hydraulic load characteristic on the levee
Erosion Scour of the channel and erosion of the foreshore, beach or floodplain	Decrease of the stability of the waterside slope of the levee and (submersed) slopes of the foreshore, due to the reduced elevation of the surface and/or steepening of the slope. Impermeable layers (contributing to the seepage path) may lose their hydraulic resistance and (eventually) disappear.	Water level: • for river levees an increase of the channel capacity may decrease of the water level during a flood discharge • an increased water depth in front of the levee may decrease wave set-up. Waves[1]: • an increased water depth in front of the levee may increase wave height.
Sedimentation Sedimentation in the channel or accretion of the foreshore, beach and floodplain	Increase of the stability of the waterside slope of the levee and (submersed) slopes of the foreshore, due to reducing the height and/or steepness of the slopes. Reducing water pressures and through- and under-seepage, in the case of an increase of the hydraulic resistance of the zone in front of the levee (foreshore/floodplain).	Water level: • for river levees: a decrease of the channel capacity may cause an increase of the water level during a flood discharge • a reduced water depth in front of the levee may increase wave set-up. Waves[1]: • a reduced water depth in front of the levee may reduce the wave height.

Note

1 If the water is shallow enough to restrict wave height

It is emphasised that, in addition to the issues presented in the table, changes in the flow pattern of currents in rivers and tidal coasts may lead to changes in the flow velocity of the water in front of the levee, affecting the waterside slope (external erosion).

A system-wide morphologic and sediment transport investigation is necessary to assess system response to new levee projects or predict future changes of the morphology for safety assessments of existing levees (Box 3.2), for both short-term (Section 8.3 single event) and long-term (Section 7.3) (project life at a minimum) performance.

There is uncertainty today regarding the effects of vegetation on the morphodynamics in the floodplain or beach in front of a flood defence system, including the banks of a channel in a riverine environment. Woody vegetation may provide benefits of reducing the hydraulic loading characteristics (especially waves and currents), which give a measure of surface erosion protection or even promoting sedimentation. In some areas, scour and wave wash erosion on the waterside of the flood defence system are significant issues. Historic engineering research related to riverbank stabilisation showed no major effect of the existence of trees on the meandering of the river channel. More recent case studies conducted by the US Army Engineering Research and Development Centre, show that waterside slope failure mechanisms of flood defense systems are more appropriately attributed to deep scouring in the river channel, enlarging the scour pool and undermining the upper cohesive bank. Maintenance of vegetation within the levee footprint is discussed in Section 4.5.

Woody vegetation and root systems may provide a measure of erosion protection to riverbanks outside of the levee footprint, delaying river migration. Direct protection of the actual levee is also possible using revetments or hardened surfaces to provide an engineered solution to resist channel migration, as shown in Figure 3.34.

Box 3.2 Movement of deep-water channel Vierhuizergat, the Netherlands

The Waddensea estuary contains several deep-water channels, and among them is the Vierhuizergat (Figure 3.32). The position of these channels migrates by geomorphological processes, and are monitored yearly. This monitoring (Figure 3.33) showed that the Vierhuizergat channel was eroding towards the coastal levee, at an increasing rate. Originally a shallow channel at a distance of 400 m from the levee, between 2011 and 2012, it had deepened by 4 m to NAP −13 m, and had almost reached the levee. At that stage, the channel endangered the safety of the levee, by reduction of the stability of the foreshore and waterside slope; the presence of loose sandy soils and the steep slope of the channel gave a risk of mass movement by liquefaction.

At the end of 2012, emergency measures were taken to prevent further erosion of the channel. The cross-section of the channel was widened on the sea side, and near the levee the channel was filled with sand and protected with rip-rap.

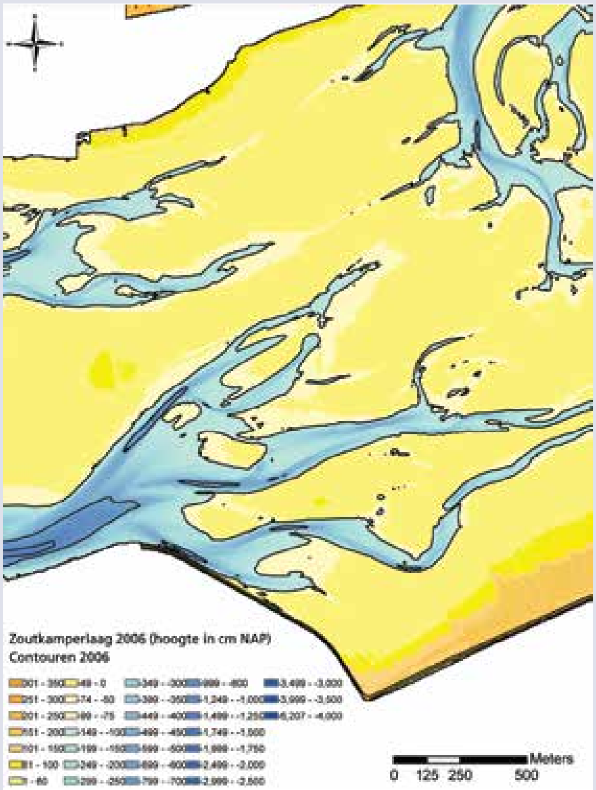


Figure 3.32 Position of the deep water channels (low tide)

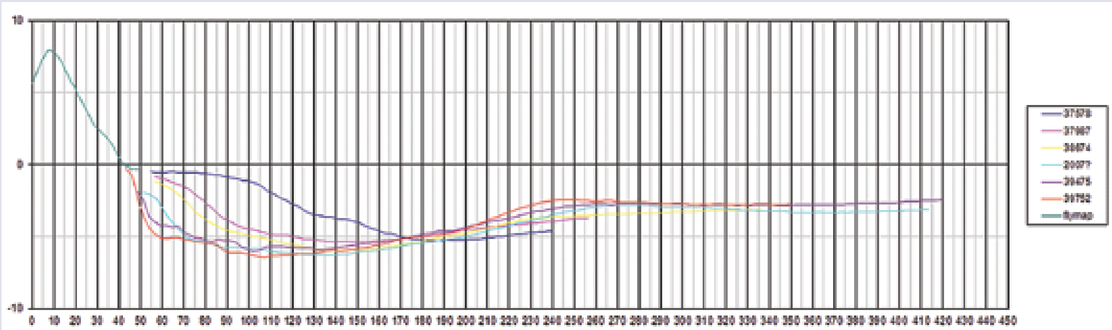


Figure 3.33 Monitoring of the position of the channel (courtesy Waterchap Noorderzijlvest)

Figure 3.34 Articulated mat along creek channel (courtesy St Louis District, USACE)

3.1.3.3 Fluvial and marine vegetation

Definition and general consideration

This section discusses the environment in which the levee functions, in particular the areas near to the levee (Figure 3.35), with emphasis on naturally occurring vegetation. Naturally occurring vegetation depends on climate and soil conditions and may be either beneficial or detrimental to levees or other components of the flood defence system. Vegetation considerations within the levee boundaries are discussed in Section 4.5.

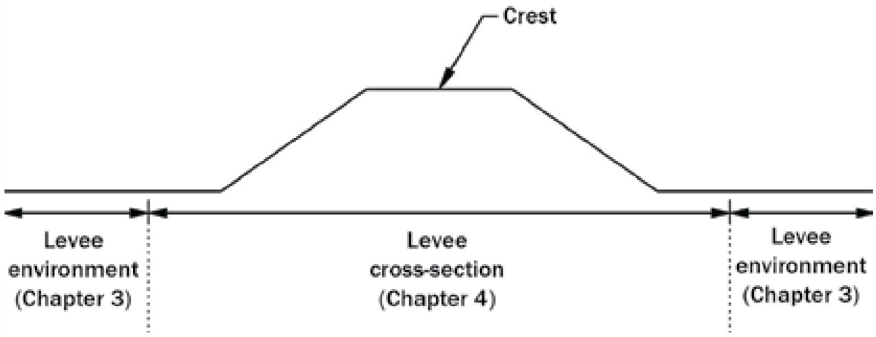


Figure 3.35 Levee vegetation addressed within the handbook

Effects on a defence system

Vegetation that is near to the levee, within the natural environment, may be indirectly beneficial or harmful to the flood defence system. Natural plant growth may help to serve as a buffer between the levee and a watercourse, slowing down water flow during a flood. Conversely, natural plant life may be an attractant to burrowing animals, which could pose threats to the levee integrity.

Fluvial vegetation

Plant life along inland waterways such as rivers, creeks or streams is technically referred to as riparian vegetation and characterised by hydrophilic plants. Riparian vegetation serves as the transition zone between the aquatic and the terrestrial ecosystem and may occur in many forms including grassland, woodland or wetland, and riparian features may also be non-vegetative (eg rip-rap stone or other types of revetment).

Riparian vegetation is often described as being extremely dense, providing habitat for wildlife. It is important in preserving water quality, controlling erosion, supplying shelter and food for many aquatic animals and shade that is an important part of stream temperature regulation. The riparian zones increase biodiversity, and provide wildlife corridors enabling aquatic and riparian organisms to move along river systems avoiding isolated communities (Figure 3.36).

Figure 3.36 Riparian corridor along a riverbank

Recent decades have significantly changed the way that vegetation and the construction of earthworks are managed and regulated around the world, particularly near waterways (Box 3.3).

Box 3.3 ***Environmental laws and the riparian zone in the USA***

In the USA, the passage of environmental laws in the 1960s and 1970s, specifically the Endangered Species Act in 1973, required consideration of impacts to ecosystems and habitat when projects are planned, and mitigation of those impacts when they exist. Due to land development practices, in some areas of western USA, remnant woody vegetation on or near levees provides the last vestige of shaded riverine habitat for endangered fisheries (Figure 3.37).

Figure 3.37 ***Endangered salmon seeking shaded riverine habitat along Butte Creek, California (courtesy California Department of Fish and Game)***

Riparian zones are crucial to ecological and environmental management because of their role in soil conservation and the influences they have on fauna and aquatic ecosystems. They serve as vegetative buffers that prevent sediment from reaching water bodies and trap agricultural chemicals within overland water flow, contributing to improved downstream water quality. Riparian zones maintain stable water temperatures and prevent sedimentation, which are both important for maintaining fish populations. Riparian forests also decrease soil salinity and help lower nitrate contamination in surface runoff from agricultural fields.

Riparian vegetation stabilises stream banks by providing deep root systems that hold the soil in place and by providing a degree of roughness capable of slowing runoff velocities and spreading flow during storm surges. They prevent erosion of stream banks and the production of sediment. Without forest buffers, stream flow scours the stream bed and banks leading to bank erosion and channel straightening. So, straight channels lead to accelerated stream flow velocity and further stream bank erosion. It must also be noted that trees and shrubs along riverbanks may have an adverse effect on bank stabilisation by concentrating flows and causing scour during high water levels and flood events.

Riparian forests have a considerable influence in reducing wind velocity at the soil surface. Many parts of the world use these forests as windbreaks to protect crops, water sources, soil and property. They are important for dune stabilisation as well. Windbreaks reduce wind speeds and prevent wind erosion.

There are some potential disadvantages of riparian forests that must be considered. Plant species inherent within a riparian zone may attract burrowing animals that adversely affect the integrity of the levee. If vegetation is excessive, it may prove a hindrance in monitoring and inspection of the levee.

Marine vegetation

Natural vegetation along the coast, outside the confines of the levee or manmade flood defence structures, may be complementary to the overall flood defence system. The density and type of vegetative cover along the coastline influences land loss by:

- dissipating the wave energy reaching sheltered shores
- encouraging the accumulation of organic and inorganic sediment
- acting as a sediment binder that resists erosion.

Some common coastal vegetation habitats are maritime forests, scrub thickets, grassy upland prairies, freshwater swamps, freshwater marshes, mangrove swamps, saltwater marshes and grassy or forested dunes (Figures 3.38 to 3.40).

Figure 3.38

Coastal vegetation, Siesta Key, Florida, USA (courtesy St Louis District, USACE)

Figure 3.39

Sand dunes and coastal vegetation Siesta Key, Florida, USA (courtesy St Louis District, USACE)

Each type of coastal vegetation has its own unique features that can retard land loss. For example, dense stands of saltmarsh and mangroves trap sediment or offer resistance to waves and currents so that land loss is prevented or mitigated. Dune grasses also help to stabilise blowing sand and can assist in dune enlargement. However, the roots of grasses and trees are generally too shallow to reduce erosion from large storm waves that lower the back-beach and undercut the dunes or uplands.

Figure 3.40 ***Example of coastal vegetation Siesta Key, Florida, USA (courtesy St Louis District, USACE)***

A coastal marsh is a herbaceous (plants lacking woody stems) or grassy plant community along the shoreline that is periodically flooded by salt or brackish water. They occur naturally within the intertidal zone of moderate to low-energy shorelines along tidal rivers and in bays and estuaries. There are two types of coastal saltmarshes, the regularly flooded low marsh, which is considered to be the most valuable and usually the most important for erosion control, and the irregularly flooded high marsh.

Sea grasses are underwater marine vascular plants occurring primarily in the shallow soft-bottom habitats, and frequently form extensive meadows. The plants can generally be characterised as having long, flat, grass-like leaves anchored to the sediment by extensive roots. Sea grasses normally occur in sediments ranging from sand to mud with a depth less than three metres. They have the ability to dissipate wave and current action, decrease sediment transport and protect low-energy shorelines from erosion.

3.1.3.4 Effects of climate change on levees

Considering that levees are built for an established design life, it is important to take into account potential changes in loads due to atmospheric climate change. Climate variability may affect hydraulic loading, eroding of soil with significant precipitation or eroding of soil during drought or high wind conditions. These atmospheric changes over time may affect the structural integrity of the levee. For example, excessive rain events or sea level rise will contribute to the flood defence system experiencing potentially more frequent and higher loadings. In regions that encounter higher wind conditions, the levee may be subjected to more surface erosion. Geographic areas that face arid/drought conditions may result in the levee suffering surficial cracking. Excess vegetation may also become a nuisance and result in costly maintenance for those locations experiencing milder winter seasons.

Possible effects vary around the world, and should be adapted to each area, using local knowledge of historical trends and anticipated future changes. Climate changes may concern, in particular for:

- coastal levees:
  - sea level rise
  - wave heights and directions – nearshore wave heights may be greater and wave transformation patterns may vary in the context of sea level rise and increased storm intensity
  - storm frequency and/or intensity
- fluvial levees:
  - flood frequency or intensity
  - dryness intensity.

Flood defence system appurtenances may also be affected by climate change. For example, with the occurrence of sea level rise, pumping stations may be adversely affected by high salinity concentrations at the pump intakes.

Climate conditions may affect the design of levees or other flood defence system features. With sea level rise along the coast there may be taller, more robust, flood defence systems designed at a higher cost. If historical trends indicate that the region will likely experience flash flooding, design features, such as waterside armouring for scour protection, should account for this probable situation. Intensive dry conditions may also affect impermeable cover of levees. See Chapter 9 for more information related to levee design.

Levee construction is also affected by climate changes. Construction operations for a specific geographic region will be typically scheduled during seasons in which the most effective work can be accomplished. For example, most construction activities are not scheduled during high water seasons unless the work is associated with emergency rehabilitation efforts. See Sections 10.1.2, 10.2.1 and 10.3.1 for more information regarding project constraints and planning during levee construction, especially.

3.1.4 Evolution of a levee function through time

3.1.4.1 Changes over time within a flood defence system

To fully assess flood defence system changes through time, it is important to consider all components of the source-pathway-receptor model as discussed in Section 2.1.1.2.

There are numerous factors that may contribute to changes in loadings from the initiating source. Climate change may affect the magnitude, frequency and duration of weather events that then impose loadings on the flood defence system. Manmade changes within the levee environment may also affect water loading. For example, the construction of reservoirs to retain excess floodwaters, releasing flow during lower river stages, may help to reduce water levels and duration of loading on levees during flood events.

Many levee systems were originally constructed as agricultural levees, built to protect farmland during high water seasons. Since their original construction, a significant number of levees have been raised and widened to provide greater protection (Box 3.4). Construction methods used to raise/widen existing agricultural levees were consistent with the technology at the time of construction. Former agricultural farmland often changed to more urbanised use, necessitating a larger, more significant flood defence system. However, with increased infrastructure comes more people, urban development and/or industry. Urbanisation of land within the floodplain was not typically regulated by stringent laws for development to account for seasonal flooding. In many cases, such unregulated development has left a legacy of flood risk to people and property that today's flood risk managers are still trying to manage. Consequently, more restrictions regarding encroachments and land development near to levee systems are in place today.

Many levees are legacy systems that were built according to local practices and before the advent of modern soil mechanics practices. So their reliability is often uncertain for large flood events. The integrity of these system's is validated through the levee's performance during flood events when defects are visible. Also, the cyclic nature of flooding can have compounding effects that reduce their reliability with time. With technological advancements over time, flood defence system components are improved. For example, levees constructed in the 1950s may have incorporated drainage pipes composed of materials that have a limited useful life, whereas today there are methods such as slip-lining existing pipes to extend the length of their use. Also, pumping station and pipe capacities and efficiencies have increased greatly and features such as gate closures have been improved with a large variety of styles and types from which to choose.

Flood defence systems may be modified in response to damaging flood events. Attitudes regarding funding of capital works and maintenance change with perceived threats. Also, experience with proven resilience of material constituents helps to better plan for future levee repairs and design. Stakeholder dispositions can also change with better understanding and knowledge about the likelihood of flooding and improved methods and technology for weather/event forecasting, estimating damages and capacity for flood warning and evacuation.

Although the primary reason for constructing a levee is to reduce the risk of inundation of an area, over time levees may take on a secondary role as discussed in Section 3.1.3. One of the multi-functional levee purposes discussed in Section 3.1.3 is utility crossings, but the preference is to install necessary utility lines up and over the flood defence system rather than trenching the line through the levee. In recent years, there have been significant advances in methods by which a utility line may be directionally drilled beneath the levee foundation.

Box 3.4 Evolution of Mississippi River levees

Original construction of levees along the Mississippi River dates back to the early 1700s with extensions, connections and levee widening over the course of centuries.

Figure 3.41 Early construction of levees (courtesy USACE)

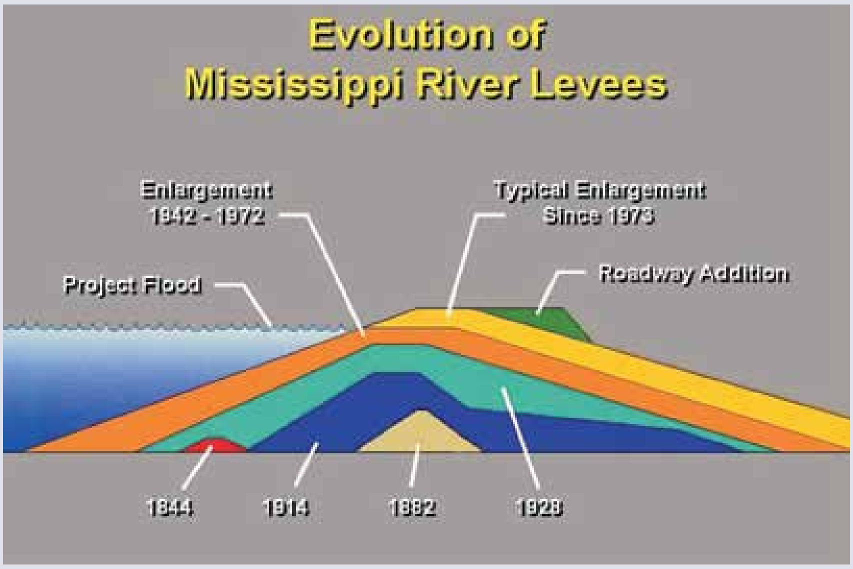


Figure 3.42 Evolution of Mississippi River levees, Sacramento, California, USA (courtesy USACE)

3.1.4.2 Changes in use of nearby property and surrounding conditions

Areas along coasts or river channels was once sparsely inhabited, consisting of natural features or agricultural crops. In densely populated, low-lying countries, people constructed homes and businesses near the river channel or sea and often along the levees for ease of access to the water. This was to help material/product transport by water, to use local sources for suitable foundation materials and to satisfy their desire to live near to the river or sea. As a result, many of these areas have since become urbanised (Figure 3.43). Site characteristics change drastically with urbanisation in which natural vegetation and soil are removed, the land surface is graded, and buildings, impervious pavement and drainage networks are constructed. All of these activities increase stormwater runoff, resulting in the increase of peak discharges, water volumes and frequencies of flooding in nearby rivers and streams (Box 3.5).

Figure 3.43 Urbanisation near to a river channel (courtesy USACE)

In some instances where urbanisation has not already occurred, property near to the water body may be restricted for development to provide a buffer from high water events or to maintain the natural site characteristics and habitats for wildlife. As further discussed in this chapter (see Sections 3.1.1 and 3.1.1.1), the ideal scenario for constructing new flood defence systems along a river channel is to provide adequate space between the channel and the levee to accommodate excess water volumes during flood events.

Box 3.5 Effect of land use on flooding intensity (from Konrad, 2005)

"Land use and other human activities influence the peak discharge of floods by modifying how rainfall and snowmelt are stored on and run off the land surface into streams. In undeveloped areas such as forests and grasslands, rainfall and snowmelt collect and are stored on vegetation, in the soil column, or in surface depressions. When this storage capacity is filled, runoff flows slowly through soil as subsurface flow. In contrast, urban areas, where much of the land surface is covered by roads and buildings, have less capacity to store rainfall and snowmelt. Construction of roads and buildings often involves removing vegetation, soil, and depressions from the land surface. The permeable soil is replaced by impermeable surfaces such as roads, roofs, parking lots, and sidewalks that store little water, reduce infiltration of water into the ground, and accelerate runoff to ditches and streams. Even in suburban areas, where lawns and other permeable landscaping may be common, rainfall and snowmelt can saturate thin soils and produce overland flow, which runs off quickly. Dense networks of ditches and culverts in cities reduce the distance that runoff must travel overland or through subsurface flow paths to reach streams and rivers. Once water enters a drainage network, it flows faster than either overland or subsurface flow."

The relative increase in peak discharge is greater for frequent, small floods than infrequent, large floods (Table 3.3, and Figure 3.44).

Table 3.3 Effects of urban development on flood's peak and peak discharge

Flood frequency	Chance that flood's peak discharge will be exceeded in any year (%)	Increase in flood peak discharge because of urban development (%)
2 year	50	100–600
10 year	10	20–300
100 year	1	10–250

Box 3.5 Effect of land use on flooding intensity (from Konrad, 2005) (contd)

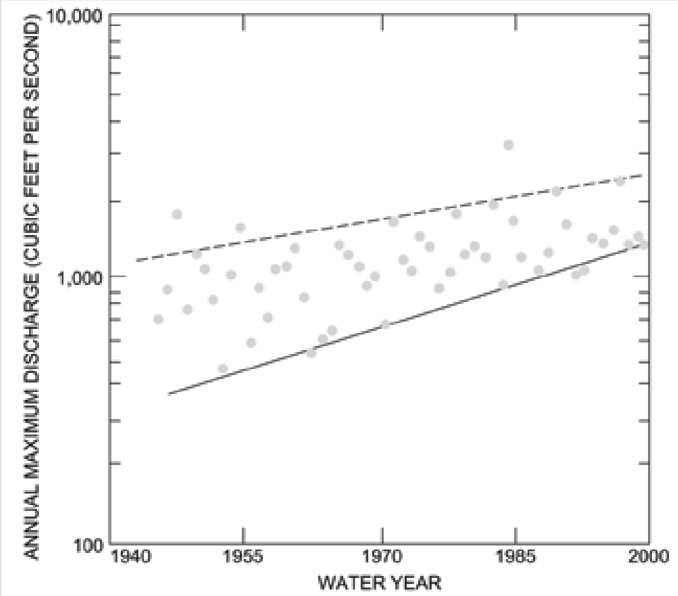


Figure 3.44 Annual maximum discharge per water year for large and small floods

The relative increase in annual maximum discharge in Salt Creek, Illinois (USGS gauging station 05531500) has been greater for small floods (solid line, less than 95 per cent of the annual peaks for the period of record) than for large floods (dashed line, more than 95 per cent of the annual peaks for the period of record).

The effect of urban development in the latter half of the 20th century on small floods is evident in Salt Creek, Illinois. With the exception of an unusually large flood in 1987, large floods have increased by about 100 per cent (from about 1000 to 2000 ft^3/s) while small floods have increased by about 200 per cent (from about 400 to 1200 ft^3/s). However, even a small increase in the peak discharge of a large flood can increase flood damage.

3.1.4.3 Co-ordination of levee functions over time

Management and co-ordination of the various functions of a levee may be administered by numerous stakeholders over the levee life cycle (Box 3.6). For instance, new levees may be financed and constructed by governmental bodies and then handed over to another entity to perform operation and maintenance. Many legacy levee systems were built by private stakeholders (farmers/local property owners) whose younger generations still perform the operation and maintenance today.

Through various phases within the levee life cycle there are numerous stakeholders with specific responsibilities that change depending on the life cycle phase. Before the construction of a levee there will be land-use planners, environmentalists, flood risk managers, surveyors and designers involved. Once the alignment is selected and the property rights are acquired, construction of the levee begins. For a detailed list of stakeholders associated with levee construction see Table 10.4. During the post-construction phase, a responsible agency will be appointed to perform the operations and maintenance of the flood defence system (see Chapter 4). During this phase of the levee life cycle, special inspections and risk assessment studies will be undertaken by a designated stakeholder to identify any necessary remedial action to reduce potential risks regarding levee performance.

Changes to the levee, due to nearby development or other needs, are inevitable. In this case, other stakeholders may become involved in properly designing or constructing levee modifications. For example, it may be necessary to install a utility line through the levee embankment. This action may involve stakeholders associated with the levee, the existing levee owner/manager, operator/maintainer, political jurisdictional authorities, a designer, a constructor and the utility owner. Though these effects are unintentional, they could also affect those living/working in the leveed area. Safeguards, such as permitting for levee alterations, should be in place to prevent improper design/construction methodology from adversely affecting the levee, resulting in damage and contributing to a levee breach.

Should the need arise to flood-fight, numerous stakeholders come together to prevent inundation of the leveed area (for more information on emergency preparedness see Section 6.3).

Finally, decommissioning of a levee system may be necessary. Typically governmental agencies will have a primary role in the decommissioning process.

Box 3.6 Participating stakeholders of a new US federally funded levee system

Table 3.4 lists participating stakeholders and their areas of responsibility during the typical life cycle of a new US federally funded levee system:

Table 3.4 Levee life cycle stakeholders and responsibilities

Life cycle stages	*Federal agency*	*State/local emergency management*	*Other agency stakeholders*	*Political*	*Drainage and levee district*	*General public*	*Constructor*	*Designer*	*Public works utility*
Conceptual layout/planning	R	C	C	C	C	C	–	*	C
Design	O/S	C	C	C	C/S	C	*	*	C
Construction	O	C	C	C	O	C	R	*	C
Operation and and maintenance	O	C	C	C	R	C	*	*	S
Inspections and assessments	R	C	C	C	C	C	–	–	C
Alterations and modifications	O	C	C	C	C	C	*	*	*
Flood-fighting	S	S	C	C	S	S	–	–	C
Decommissioning	R	C	C	C	C	C	–	–	C

Notes

* Depending on the circumstances, the stakeholder may be consulted or involved in life cycle stage.
S = shared responsibility
R = responsible party
O = oversight/approval
C = communication/co-ordination

Some US federal agencies are adopting a partnering approach regarding levee policy by actively engaging non-federal stakeholders during the policy development phase.

3.2 FORMS AND FUNCTIONS OF LEVEE COMPONENTS

3.2.1 Defining components of levees

3.2.1.1 From flood system to levee components

As discussed in Section 3.1, a defence system is divided into distinct parts, some of which may be individual levee segments.

One levee segment is defined by a specific function in the defence system and a defined cross-section. This cross-section reflects an assembly of elementary structures, called components. These components have specific and individual functions to maintain the integrity of the whole levee segment. Each type of levee segment is defined by an association of different components that produce a particular cross-sectional geometry and form of the levee. A variation in function, a change of one component or a difference in general configuration of the cross-section results in a change in levee segment (Figure 3.45).

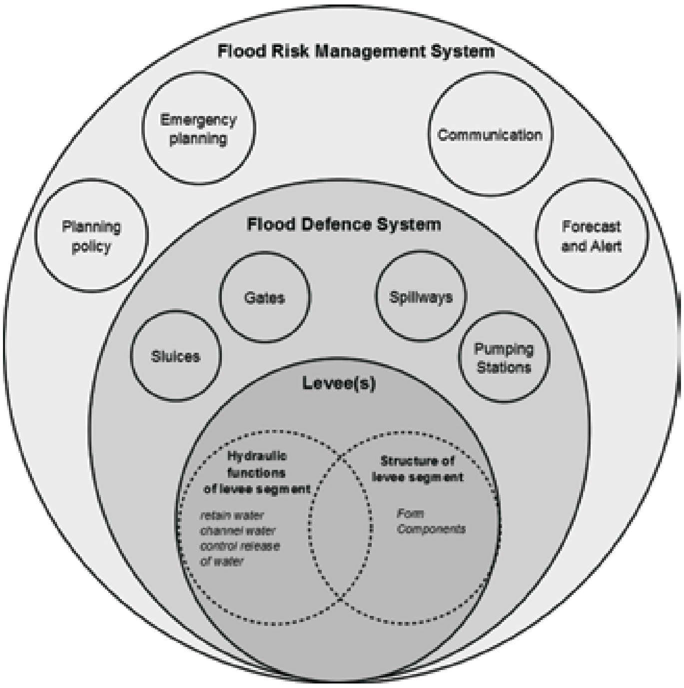


Figure 3.45 From flood defence system to levee cross-section (courtesy Y Deniaud)

Whatever their environment, levee segments are exposed to external loads. To keep their integrity and perform their primary function, segments of levees must be adapted to these load conditions. According to Section 3.5, levees should be adapted to resist:

- external erosion
- internal erosion
- instability.

Each levee segment is generally designed for a particular estimated return period water level and should resist failure up to that height. The overall performance of the system is defined by the lowest height or the most unstable of all the segments. For example, the profile elevation of a stable levee is consistently five metres above mean sea level with the exception of one region where there is a road crossing over the levee resulting in a portion of the flood defence system being only four metres above mean sea level. In this situation, the four metre portion of the levee assumes the overall system level of protection being provided, until remedied.

A direct relationship exists between levee function and performance as related to flood control within its environment. Typically, historical trends and site characteristics prescribe the type of flood defence system for the specific environment, though economic constraints may also play a role. For example, it would not be appropriate to construct waterside armouring on the levee embankment slope for a system that does not, nor is expected to, experience wave action or scouring.

3.2.1.2 Functions of components of levees

To perform its desired function, a segment of levee should be composed of components compatible with the loads engendered by the levee's environment, allowing for water retention up to the design level. This requires:

- protection against surface/external erosion on the waterside, the crest and possibly on the land side
- resistance to internal erosion
- mass stability of the constructed levee, including stability of the foundation.

To achieve these aims, the following individual structural functions must be ensured by the components of a levee segment:

- **external protection:** levees are exposed to external agents (eg waves, current, rain, runoff and animal and human activities). These agents produce mechanical and hydraulic stresses that can lead to external erosion of the levee. To avoid adverse effects on the levee structure, protection must be provided. This protection is a revetment (natural or artificial) that acts to absorb and reduce the effects of the external agents on the core of the levee
- **stability:** a segment of levee must remain stable under normal and adverse conditions. The mass of the levee and the characteristics of the earthfill particles must provide enough strength to be stable in dry conditions and to balance the hydrostatic force due to the water level difference across the levee, during flood events
- **impermeability:** whatever its primary function, a levee will retain an amount of water for a given period. To do so, water passing through the levee must be limited. The aim of the impermeability complex is to control the amount of water flowing through the levee
- **drainage:** interstitial water is a major source of deterioration and damage in an earthfill embankment. An increase in the pore pressure inside or under an earthfill levee can trigger internal erosion or instability of the core of the embankment. To prevent such mechanisms, water pressure must be managed and drawn down by an appropriate drainage system, the aim of which is to extract and evacuate the water outside the levee
- **filtration:** water passing through a levee can induce migration of earthfill particles, especially when the particle size of the earthfill material is heterogeneous, or when different layers with different particle size characteristics are present. This migration alters the required properties of the different layers and can increase their permeability and reduce their strength. The levee is then more fragile and can suffer failure. To avoid such a loss of particles, filtration must be ensured. This can be done at the scale of one layer with a specific graded particle size (self-filtration) or at the scale of several layers or components with specific rules on the particle size for each of these layers or components (Terzaghi rules, see Section 8.5.5.1).

This list mentions generic structural functions and covers the majority of cases. However, it is always possible to consider other functions when further analysis of individual scenarios is undertaken.

In any case, functions of levee components intersect with hydraulic functions and structural design of a levee segment (Figure 3.46).

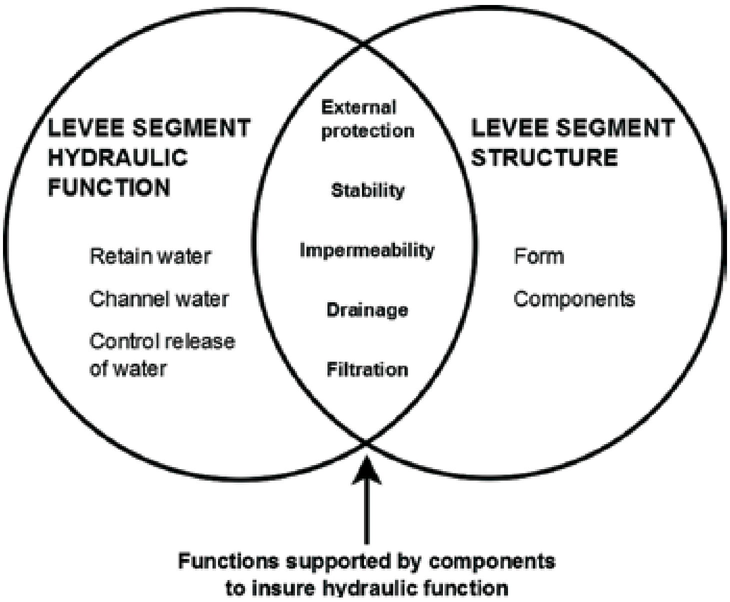


Figure 3.46 Component functions and hydraulic and structural functions of a levee segment (courtesy Y Deniaud)

3.2.2 Main components of levees

Levees are arrangements of components that provide individual functions adapted to loads. Components depicted in Figure 3.47 may be found in a levee, but are not all-inclusive, nor always all necessary. This cross-section and the following information in this section are illustrations of the position of the components and should not be considered for design.

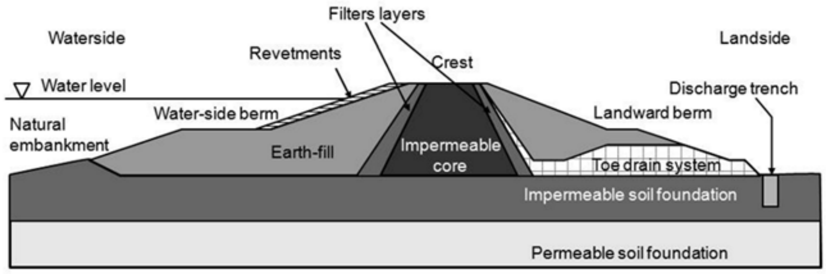


Figure 3.47 Individual components of levee

3.2.2.1 Foundation soils

Description

The foundation soils are the ground situated beneath the levee that interacts with the levee (Figure 3.48). This foundation soil can be rather complex and its characteristics highly variable in terms of strength or permeability. The foundation soils are not strictly a component of the levee but they have to be considered when analysing or designing a levee.

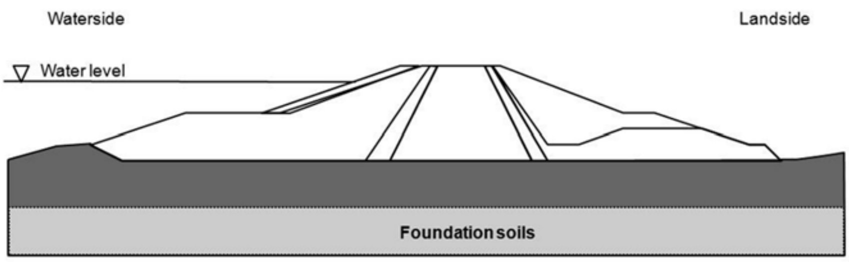


Figure 3.48 Foundation soils

Functions

Foundation soils:

- bear the weight of the levee and must provide a stable support for the levee
- provide impermeability and filtration functions.

Technical issues

Table 3.5 details the main issues and methods associated with this component.

Table 3.5 Technical issues for foundation

Issues	Methods	Chapter ref
Characteristics of the foundation	The geological, geotechnical and hydraulic parameters of soils must be determined through field or laboratory tests and investigations. They are the main input parameters for levee design.	7.7, 7.8, 7.9
Compatibility between components and foundation	The geotechnical and hydraulic calculations must determine the compatibility of the design of the levee with the environmental and mechanical site conditions.	8

3.2.2.2 Earthfill

Description

The earthfill (shaded in Figure 3.49) is the main volumetric component of the levee and is common to all types of levees. Earthfill is usually made of granular or cohesive soil materials (clay, silt, sand and/or gravel). In the majority of cases, earthfill is locally available material. On occasion, there may be a need for better soil material derived from an off-site borrow source. In this case, the earthfill would be hauled to the levee site and then placed.

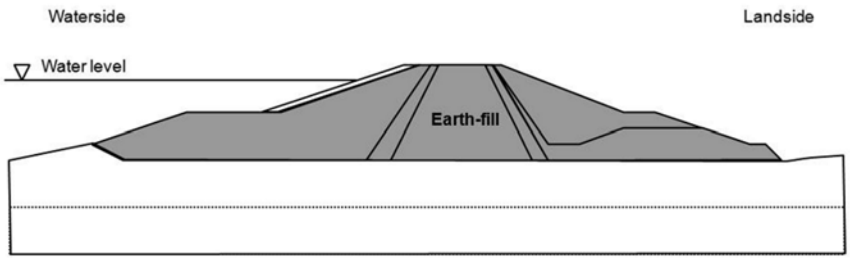


Figure 3.49 Earthfill

Functions

Functions of earthfill include:

- providing mass stability against water pressures
- minimising through-seepage.

Technical issues

Table 3.6 details the main issues and methods associated with this component.

Table 3.6 Technical issues for earthfill

Issues	Methods	Section ref
Appropriateness of material for earthfill	The choice of earthfill material (regarding multi-criteria analysis) will considerably influence the type of levee and the geometry of its cross-section. As the primary function of the earthfill is to provide mass stability against water pressure, typically using locally available material, geotechnical properties of earthfill are not always optimised. For example, earthfill made of sand will succeed in providing mass stability but will have poor characteristics regarding impermeability and erosion. Depending on the material nature, earthfill can be subject to a large range of deterioration and damage mechanisms (external and internal erosion, shallow and deep rotational sliding, cracking and settlement). The quality of the available earthfill material drives the selection of other levee components.	7.7, 7.8, 9.13, 10.4.2
Degree of compaction	Compacted fill: generally made of cohesive material such as clay or silt. The types of compaction, water content control and fill materials govern the steepness of levee slopes from the stability aspect if foundations have adequate strength.	10.4.3.4
	Where foundations are weak and compressible, high quality fill construction is not justified, since these foundations can support only levees with relatively flat slopes. Semi-compacted fill is also used where fine-grained borrow soils are considerably wetter than optimum or in construction of very low levees where other considerations dictate flatter levee slopes than needed for stability.	
	Hydraulic fill: consists mostly of pervious sands or gravels and is more susceptible to soil liquefaction. The high permeability of the earthfill must be compensated by a longer flow path that requires large footprint. Hydraulic fill would also quickly erode upon overtopping or where an impervious covering was penetrated. So it must be associated with an external revetment that provides protection against external erosion.	

Table 3.6 Technical issues for earthfill (contd)

Balancing volume of fill and stability of levee	Levee slopes depend on the stability of the earthfill (material and compaction). The slope design is a compromise between a minimisation of footprint and volumes (steeper slopes) and a maximisation of stability (gentle slopes). The earthfill section will be designed considering the intended levee height, space available, material used and the external solicitation. In most cases, the earthfill is designed to be auto-stable.	5.5, 9.5

3.2.2.3 Impermeable core or mask

Description

Sometimes the earthfill layer can not provide impermeability because of poor hydraulic characteristics (earthfill made of sand or gravel). In those cases, extra components are necessary to ensure impermeability:

- an impermeable core can be added in the central position of the cross-section comprising either impermeable materials (clay) or a bentonite wall (Figure 3.50)
- an earthfill layer with poor hydraulic characteristics can also be covered with an outer layer (revetment or mask) made of clay or other low permeability material (Figure 3.51).

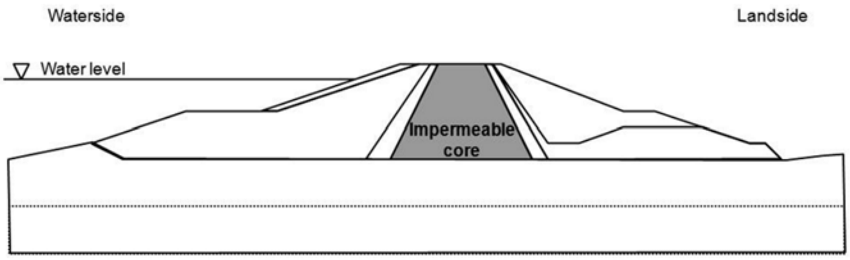


Figure 3.50 Impermeable core

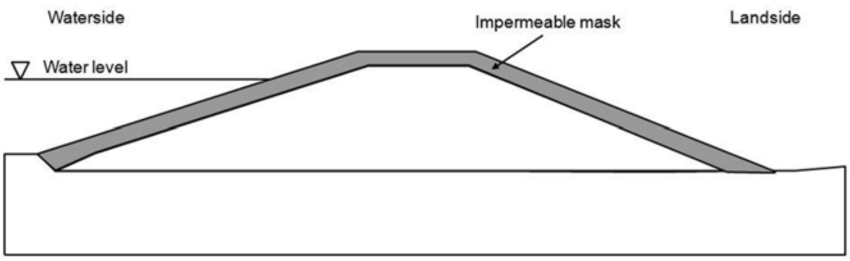


Figure 3.51 Impermeable mask

Functions

Impermeable cores:

- promote water resistance and reduce through-seepage.

Impermeable masks:

- romote water resistance and reduce through-seepage
- promote erosion protection.

Technical issues

Table 3.7 details the main issues and methods associated with this component.

Table 3.7 Technical issues for impermeable core and mask

Issues	Methods	Section ref
Impermeability of the core	The impermeable core height is determined by the height of the levee where water is contained up to this height but not higher. As a general rule, impermeable components must be joined together from the foundation to the crest. As an impermeable structure, the core must be anchored in the foundation soil. However, the core of the levee only needs relative impermeability and not absolute impermeability, as the aim is to retain water temporarily, during a flood event. The thickness of the mask is in close relation to the characteristic and properties of the used material.	9.8, 9.13, 10.4, 10.5
Degree of compaction	Compaction of clay materials should be realised with caution. It may present difficulties and require specific control of moisture. The properties of the core material must be accurately studied.	7.7, 7.8, 10.4.3.4
Effects of wetting and drying	Dry/arid environments or seasonal drought conditions may result in surface cracking of clay materials, which are exposed to such atmospheric phenomena (alternation of dry and wet period). Cracking will occur more frequently in areas where droughts affect the levee. This deterioration mechanism will particularly affect impermeable masks. It will be more difficult to investigate such a mechanism in an impermeable core. Cohesive soils of high plasticity will generally exhibit the greatest shrink-swell potential due to corresponding changes in moisture content; increases in swell potential are generally accompanied by increases in plasticity index.	4.12
Earthfill core interface problems	An impermeable core of clay or silt can be subject to internal erosion, especially contact erosion at the junction between the core and the earthfill. To avoid this problem, care should be given to the compliance with the filter conditions at the interface between earthfill and core. Filter layers might be necessary.	8.5, 9.8
Uplift and sliding of an impermeable mask	An impermeable mask covering permeable earthfill can be affected by uplifting and sliding if pore water pressure does not dissipate quickly enough during and after a flood event. The design and calculations of the thickness and of the properties of the impermeable layer must take into account the situation of rapid draw-down of the water after a flood event.	8.5, 8.6, 9.7

3.2.2.4 Crest

Description

The crest is the relatively flat, top surface or crown of the levee, and it is often a horizontal surface (Figure 3.52). Depending on the levee, the crest can act as spillway (when the secondary function of the levee is to spill water) or just as protection against water infiltration, deterioration caused by access (when the levee is not overtopped by water). Unprotected levee crests should only be adopted where spillway segments are provided as part of the system (Sections 3.1.1, 3.4.1.1 and 9.14).

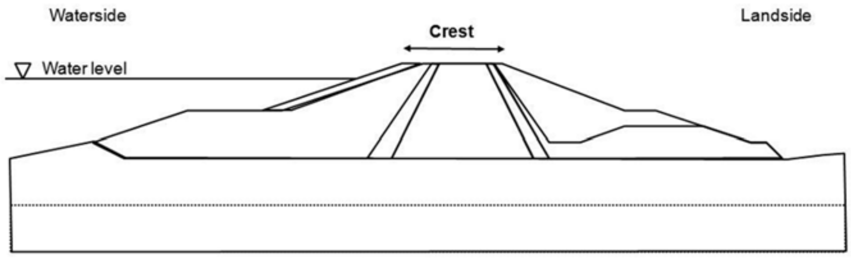


Figure 3.52 *Crest*

Functions

The levee crest is a prime component with multi-functional uses because it:

- provides protection against external agents that may cause erosion (rain, water in case of overtopping flowing)
- provides access (maintenance, roadway, recreational etc).

Technical issues

Table 3.8 details the main issues and methods associated with this component.

Table 3.8 *Technical issues for crest*

Issues	Methods	Section ref
Role of levee crest level in flood defence system	Crest height is sometimes mistakenly associated with level of protection. It must be remembered that levees are not always designed to contain water up to the crest level. A risk-based analysis should be conducted, directly accounting for hydraulic uncertainties and establishing a nominal crest design level. Deterministic analysis using physical properties of the foundation and embankment materials should be used to set the final crest level.	8.2, 9.5
Appropriateness of crest width	The width of the crest largely depends on the constructability and any access requirements. The crest width should be considered as part of the overall geometry in assessments of overflowing/overtopping etc.	8.2, 9.5
Surface protection (see Section 3.2.2.5)	The crest is typically covered with grass, asphalt or gravel to protect against erosion and rutting. In a case of a levee that is designed to be submersible and to act as a spillway, the design of the cover needs specific attention.	9.6, 10.4, 10.5

3.2.2.5 Revetments

Description

Revetments act at the interface between the external environment and the levee, on both the waterside and landside slopes (Figure 3.53).

Sometimes termed ‘revetment’ or ‘armouring’, waterside slope protection is often constructed as added assurance for levee stability to resist erosive properties.

Revetments typically consists of grass, rip-rap, asphalt, geotextiles or cellular confinement systems, but there are numerous other materials that may be used, as detailed in Section 4.13.

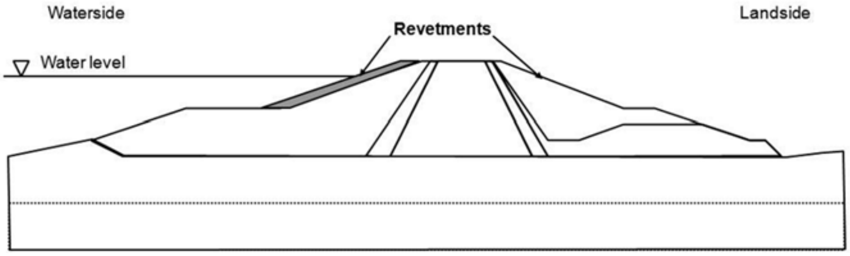


Figure 3.53 *Revetments*

Functions

Revetments provide the following functions:

- on the waterside:
  - to protect the levee from external erosion caused by currents and wave action. In coastal environments, roughness elements or slopes partly covered by rock are often used to increase the surface roughness and to reduce wave run-up heights and wave overtopping rates
- on the landside (revetment on the crest is implicitly understood as part of the crest):
  - to protect against erosion from surface runoff and other non-hydraulic agents
  - (eventually) to protect against erosion due to overtopping and overflowing, especially for coastal levee or for submersible levee

Technical issues

Table 3.9 details the main issues and methods associated with this component.

Table 3.9 *Technical issues for revetments*

Issues	Methods	Section ref
Appropriateness of type of revetments	Specific guidelines and disciplines are relevant for each technique or material used. The following examples and recommendations are not all inclusive. Grass cover must be adapted to the environment of the levee. Grass revetment follows shear stress analysis principles and botanical considerations. Revetments constructed of rocks are designed according to stability (CIRIA; CUR; CETMEF, 2007).	8.2, 8.4, 9.6, 10.5.3, 4.13, 4.5
Appropriateness of materials	The materials selected for armouring a levee should be defined and selected consistently with the anticipated environment loading (CIRIA; CUR; CETMEF, 2007).	7.6.1, 7.9, 9.6, 10.4
Weakness of transitions	The transition between the waterside revetments and the embankment and from an embankment with revetment and an embankment without revetment are weak points that need specific attention. They are subject to differential behaviours that may affect the integrity of the levee.	4.16, 9.6, 9.11
Aesthetic and environmental considerations	The revetment is a visible component of the levee, so aesthetic aspects have to be considered. Secondary objectives of plantings on levee embankments and near to flood walls are to harmonise with the surrounding natural and human environment, improve structures, control dust and erosion, separate activities, provide privacy or screen out undesirable features, provide incidental habitat for wildlife and create a pleasant environment for recreation. Although aesthetics are important, levee integrity and the ability to inspect the levee should be the governing factors when it comes to landscaping. Aesthetics should be considered in the design of levees from the perspective of protecting the environment and blending the embankment with the surrounding environment. When possible, the project should appear to be a natural extension of the local topography.	9.6

Table 3.9 Technical issues for revetments (contd)

Loss of protection due to revetment deterioration	Assessment of the appearance and observable features on revetments are visible indicators of the state of the levee. Revetments are subject to deterioration caused by external agents (currents, waves, human access, burrowing, settlement, uplift).	4.5, 4.6, 4.9, 4.13, 4.16
Effect of crest height on overtopping of coastal levees	For visual or cost reasons, some coastal levees have a lower crest, which allows greater overtopping. In this scenario, the landside slope must be protected against erosion caused by overtopping flows.	8.2, 8.4, 9.6
External erosion of spillway sections	In the case of a levee that acts as a spillway, the design of the landside revetment needs specific attention, to resist external erosion, infiltration and sliding. A hydraulic jump that may be damaging must be anticipated when designing the landside toe.	8.2, 8.4, 9.6

3.2.2.6 Berms

Description

A berm is typically constructed as an extension of the levee on one side of the flood defence system (Figure 3.54). A constructed berm is typically composed of earthfill materials or rock.

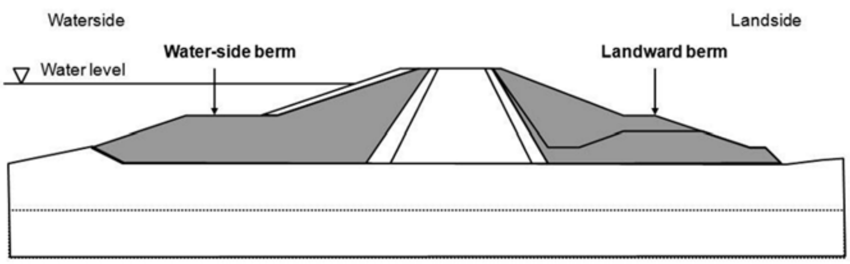


Figure 3.54 Berms

Functions

Berms help to:

- stabilise the levee by increasing the weight of the toe and/or flattening the side slopes
- stabilise the levee by increasing the seepage length under or through the levee
- reduce wave run-up and overtopping discharge reducing loads on the levee (Pullen and Allsop, 2007 and TAW, 2002).

Berms can consist of several types, including drained, undrained and permeable. Berms may also be built for levee inspection and maintenance purposes. A sea-side berm for levee inspection is typical for coastal levees constructed in the Netherlands. This berm is able to withstand the forces of traffic, but should have no effect on the waterside stability. It should also have a positive effect on the wave run-up, if it is designed correctly.

Technical issues

Table 3.10 details the main issues and methods associated with this component.

Table 3.10 Technical issues for berms

Issues	Methods	Section ref
Appropriateness of berm type and geometry for controlling seepage and internal erosion	Drained berms allow seepage to normally occur, but a designed filter layer (sand/gravel or geotextile) is constructed over all or part of the ground beneath the seepage berm. This filter layer is used to ensure that finer particles are not piped from the foundation layer. A drainage collection layer (gravel/drain rock) overlies the filter layer. Sometimes, pipes are constructed within the drainage collection layer to help collect and discharge seepage. Soil is placed over the filter/drainage layers to provide protection and to help hold the materials in place. An undrained berm does not include a drainage layer. It is generally constructed with soil with the same or lower permeability of the surface soils near the levee toe, effectively providing additional weight that counteracts the upward force of under-seepage pressures. Undrained berms also effectively lengthen the seepage path beneath the levee such that hydrostatic uplift pressures are reduced as seepage exits further from the levee. Permeable berms are generally composed of materials more permeable than the surface layer near the levee toe. Seepage can exit the berm surface, but the gradient condition is generally reduced due to the increased vertical length of the seepage path. Seepage berms are generally sloped to provide surface drainage away from the levee. Sometimes surface/subsurface drainage collection features are located at the toe of the seepage berm.	8.3, 9.7, 9.8, 10.4
Appropriateness of berm type and geometry for levee stability	Drained stability berms have a designed filter layer (sand/gravel or geotextile), which is used to ensure that finer particles are not piping through the levee embankment. Sometimes piping is constructed within the drainage layer to help collect and discharge the collected seepage. The filter/drainage layer is placed on the landside slope of the levee and near the levee toe. Soil is placed over the filter/drainage layer to provide protection and to help hold the materials in place. An undrained berm does not include a drainage layer. It is generally used as a buttress to provide extra stability for the landside slope.	8.6, 9.9, 10.4
Role of berm in controlling wave run-up and overtopping	In coastal environment, berms may be used to reduce the wave run-up and overtopping. Design of such structures needs specific attention and good knowledge of hydraulic conditions through specific investigations or analysis.	7.4, 8.2, 9.12
Appropriateness of materials	Appropriate material must be used with suitable compaction. Properties of the used soil materials must be accurately studied.	7.9, 9.7

3.2.2.7 Filter layers

Description

Filter layers are zones of relatively pervious material within the levee cross-section between two layers or between earthfill and drains (Figure 3.55). The composition of a filter layer may include geotextile fabric or pervious granular graded materials.

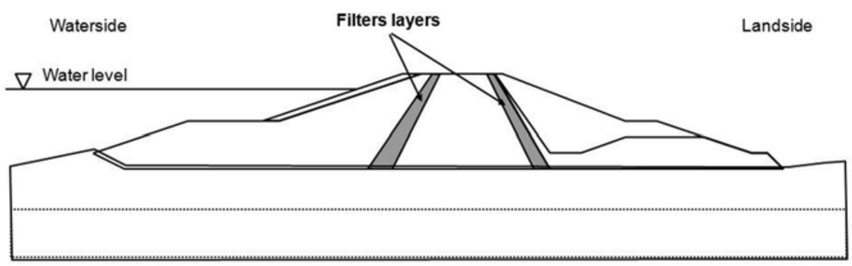


Figure 3.55 Filter layers

Functions

Filter layers help to promote filtration, by preventing soil from migrating especially from the impervious core. Weighted filters are used primarily to allow seepage to occur but to ensure, finer particles are not removed (piped) from either the levee embankment or the foundation layer.

Technical issues

Table 3.11 details the main issues and methods associated with this component.

Table 3.11 Technical issues for filter layers

Issues	Methods	Section ref
Appropriateness of materials	Filter layers generally consist of sand/gravel or geotextile. The properties of the filter materials must meet criteria based on materials close by. Selection of filter layer composition is important for long-term resiliency and minimal future maintenance. Designer needs to ensure that proposed materials complement one another, working in harmony for desired function.	8.5, 9.13
Maintaining filter integrity during construction	Implementation and maintenance of filter layer should be made with caution to ensure integrity of the material composition needed for the filter function.	4.10, 9.13, 10.5

3.2.2.8 Drainage and seepage system

Description

A drain system might be carried out at the landside toe of the levee (toe drain system) or behind an impermeable core to collect the seepage through the levee embankment or in the foundation soil at the levee toe and/or to reduce internal pore pressure inside the levee (Figure 3.56). Seepage collection trenches might also be used to collect seepage at or near the landside levee toe. To help discharge of the drained water, pipes might be installed in the drain.

Caution

There is an important distinction between toe drains and relief wells. Extreme care should be taken when designing, installing and operating drainage collection systems at the landside toe of a levee to ensure that the drain has the capacity to carry the full discharge from the foundation area. Care should also be taken to ensure that drains do not penetrate into the permeable layer underneath the levee.

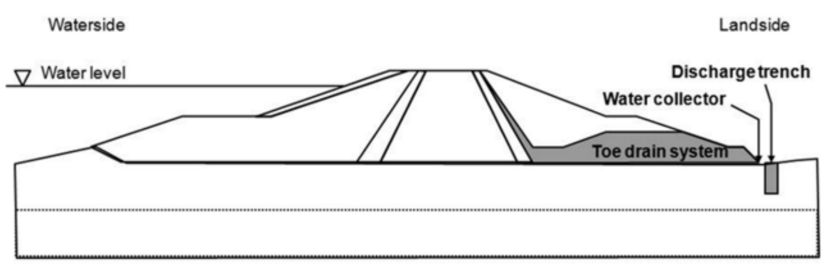


Figure 3.56 Drainage and seepage discharge system

Functions

A drain system helps to:

- drain water to control embankment through-seepage flows
- promote levee stability.

Technical issues

Table 3.12 details the main issues and methods associated with this component.

Table 3.12 Technical issues for drainage and seepage discharge system

Issues	Methods	Section ref
Disposal of collected water	The use of drainage and seepage discharge system increases the amount of seep water that must be addressed at the ground surface. So, a means of collecting and disposing of the discharged water needs to be provided and maintained properly to ensure its capacity over time.	7.9.9, 9.4.3, 9.7, 9.7.3
Facilities for maintenance	If pipes are installed in the drain, the design should include points of entry to the system for maintenance.	9.15.4
Malfunctions	Drainage and seepage discharge systems may malfunction or gradually lose efficiency with time for a variety of reasons including clogging, vandalism, breakage, excessive deformation due to ground movements, corrosion or erosion. The reduction of specific capacity with time can result from mechanical, chemical or biological processes.	4.10
Biochemical clogging of pipes	If pipes are used, their design should preferably aim for a position permanently below the ground or surface water table, to prevent chemical clogging by intermittent aerobe and anaerobe conditions.	9.15.4

3.2.2.9 Seepage relief trenches and relief well system

Description

When an impermeable layer of soil is underlain by a permeable layer, seepage relief trenches or pressure relief wells might be used to reduce the hydrostatic pressure, to increase levee stability and to prevent piping or sand boils (Figure 3.57).

Relief well systems are positioned in zones behind the levee, and are generally used where the permeable strata underlying a levee are too deep to be penetrated by cut-off walls or toe drains, or where space for landside berms is limited.

Seepage relief trenches are installed at various depths and locations. A filter layer consisting of either sand/gravel or geotextile is used. Pipes are sometimes installed within the relief trenches to collect the seepage and help discharge.

Caution

There is an important distinction between toe drains and relief wells. Extreme care should be adopted when designing, installing and operating relief trenches or wells, to ensure that the well has the capacity to carry the full discharge from the permeable layer and that this discharge can be drained away properly. Care should also be taken to ensure that wells cannot lead to the unfiltered exit of groundwater.

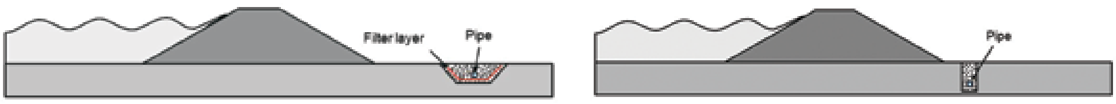


Figure 3.57 Relief trench and relief well system

Functions

To provide controlled outlets for seepage that would otherwise emerge uncontrolled landward of the levee, in order to:

- reduce uplift pressures
- promote levee stability
- prevent piping/sand boils.

Technical issues

Table 3.13 details the main issues and methods associated with this component.

Table 3.13 Technical issues for drainage and seepage discharge system

Issues	Methods	Section ref
Accommodating flows without excessive head loss	The design of a relief wells aims to accommodate the maximum design flow without excessive head loss. The design includes the depth and diameter of the riser pipe and the spacing of the pipes.	9.7, 9.7.3
Preventing in-wash of foundation materials	Aside from the hydraulic capacity, the design of the screen and surrounding filter material should focus on prevention of in-wash of foundation materials into the well. Well screen sizing/design for each relief well must be based on the grain-size distribution of the aquifer materials specific to the proposed location where the relief well will be constructed.	9.7.3
Disposal of collected water	The use of relief trenches or wells increases the amount of seep water that must be dealt with at the ground surface during well flood situations and any other event in which the river level is higher than the elevation of the surface at the landside. So, a means of collecting and disposing of the discharged water needs to be provided.	7.9.9, 9.4.3, 9.7.3
Capacity	Relief wells should be installed to adequately penetrate permeable strata and be spaced sufficiently close to intercept enough seepage to reduce hydrostatic pressures acting beyond and between the wells. Soil investigations and characterisation are important input for design.	7.7
Facilities for maintenance	The design of seepage relief trenches and wells should consider inspection holes, manholes or other points of entry to the system, for maintenance and inspection.	9.7, 9.7.3
Malfunctions	Over time, relief wells tend to experience a loss in efficiency for a variety of reasons to include clogging of the well screening, bacterial growth or carbonate incrustation. Periodic maintenance is required to maintain system efficiency. Damage to relief wells or relief trenches, and inadequate performance of wells or trenches and their associated collection and discharge systems must be corrected promptly. Any condition that restricts flow in or from relief wells or trenches, or that permits piping of foundation soils into relief wells or trenches and/or associated collector/transport/discharge systems results in potentially unstable and hazardous conditions.	4.10
Attract burrowing animals	If trenches will be permanently filled with surface water they may form a habitat for some burrowing animals, such as muskrats, thus attracting these animals to the levee.	4.6, 9.12.3

3.2.2.10 Cut-offs and seepage barriers

Description

A cut-off wall or material zone may be installed at the junction between the impervious part of the levee and the impervious soil foundation (Figure 3.58). The cut-off may consist of excavated trenches back-filled with compacted clay, slurry trenches, steel sheet piling, vinyl sheet piling or bentonite mats.

In some case, seepage barriers might be adopted throughout the whole levee embankment and permeable foundation strata (Figure 3.59).

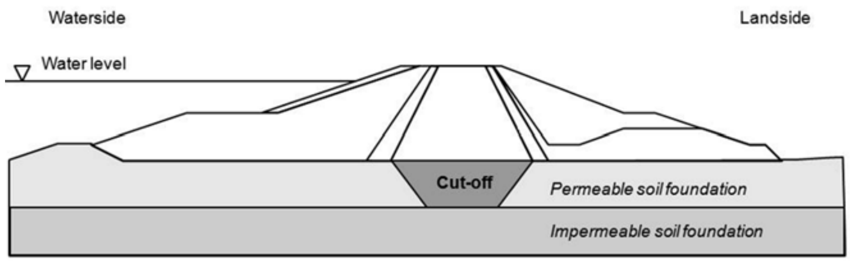


Figure 3.58 *Cut-off*

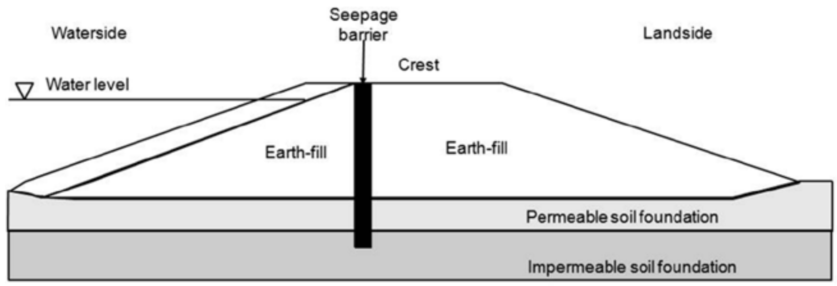


Figure 3.59 *Seepage barrier*

Functions

Cut-offs and seepage barriers help to:

- reduce seepage through permeable foundation strata and levee embankment.

Note

Unlike for dams, impermeability is not necessarily a goal for a levee. Consequently, the seepage barrier does not necessarily has to cut through all of the permeable foundation to reach the impermeable foundation.

Technical issues

Table 3.14 details the main issues and methods associated with this component.

Table 3.14 *Technical issues for cut-off and seepage barriers*

Issues	Methods	Section ref
Appropriateness of cut-off technique	The choice of the technique to be adopted, and its geometry, will depend on ground conditions, the thickness and properties of the permeable layer and the depth and properties of the underlying impermeable layer.	7.7
Appropriateness of materials	The materials used for cut-off or seepage barrier must have adapted properties. Particular care must be given to installation techniques.	9.7.3, 10.4.2
Ensuring continuity of cut-off with levee	The anchorage and the sealing of the cut-off or seepage barrier are critical, as is the continuity with the other components of the levee (earthfill, core or mask) that act on permeability of the structure. For this reason, bentonite mats or sheet pilings will be used as waterside mask extension in the foundation. When an inner core provides the impermeability, the foundation extension will be beneath the core.	9.7.3, 10.4.3.5

3.2.2.11 Walls

Description

Walls are structural elements that may be included in a levee (Figure 3.60). A wall may be added on both sides, more often on the waterside, on the levee and inside the levee. The structure may be gabion walls, concrete structures and/or steel sheet piling.

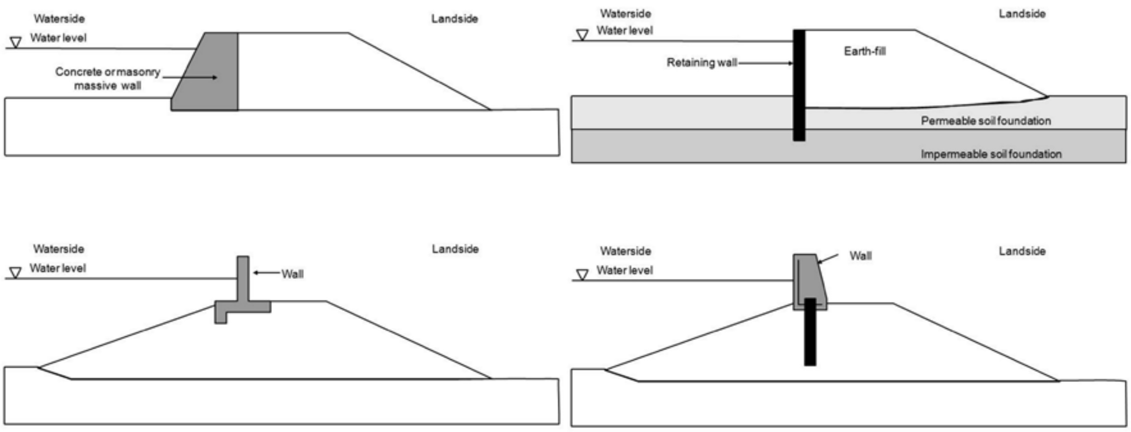


Figure 3.60 Types of walls

Functions

Walls can help to:

- retain part of the levee
- provide part of the impermeability of the levee
- protect one side of the levee
- raise the levee.

Technical issues

Table 3.15 details the main issues and methods associated with this component.

Table 3.15 Technical issues for walls

Issues	Methods	Section ref
Interaction between wall foundation and levee	The foundation of a wall on the top or on the waterside of a levee is critical. Accurate studies of the interactions of the wall and the levee must be conducted. These require knowledge of the geologic and hydraulic conditions and accurate investigations of the properties of the soil and of the levee materials.	7.7, 7.8, 8.9
Stability of levee taking account of wall	The composite structure of the levee needs a detailed stability analysis, taking in to account all the interactions between the wall and the earthfill embankment.	8.9, 9.15
Effect of wall construction on levee	The construction of a wall crest must be adapted so as not to degrade the material properties of the underlying levee.	10.5.5

3.2.3 Association and functions of components

Regarding the combination of components that define the geometry of a levee segment, functionality is most often accomplished by several components working collectively rather than by one single component. This configuration gives a better guarantee of the integrity of the levee segment.

However, the choice of the components of a specific levee is a part of the complex design process (Chapter 9). Because of this complex process of design, the projected levee is defined by specifications, drawings and reports that define a certain form, resulting from a specific combination of components.

Table 3.16 summarises the main relations between functions and components of levee.

Table 3.16 ***Components and functions***

Structural components of levees	**Existence within levee**	**Functions of components**				
		External protection	***Stability***	***Impermeability***	***Drainage***	***Filtration***
Foundation soil	Always present		Always	Sometimes		Sometimes
Earthfill	Always present		Always	Sometimes	Sometimes	Sometimes
Impermeable core	Sometimes present			Always		
Impermeable mask	Sometimes present	Sometimes	Sometimes	Always		Sometimes
Crest	Always present	Always			Sometimes	
Waterside revetment	Sometimes present	Always	Sometimes	Sometimes		Sometimes
Landside revetment	Sometimes present	Always	Sometimes		Sometimes	Sometimes
Waterside berm	Sometimes present	Always	Always	Sometimes		Sometimes
Landside berm	Sometimes present		Always		Sometimes	Sometimes
Filter layers	Sometimes present				Sometimes	Always
Drainage system	Sometimes present				Always	
Relief wells	Sometimes present		Always			
Cut-offs and seepage barriers	Sometimes present			Always		
Walls	Sometimes present	Sometimes	Sometimes	Sometimes		Sometimes

Key

Always	Always play a role regarding the function
Sometimes	Sometimes play a role regarding the function
	Not applicable

3.3 FORMS OF LEVEES

Numerous cross-sectional variations of levees exist, each with their own primary objective of reducing flood risk within the leveed area. Considering the ease and cost of construction and anticipated loads, there are different types of levees. The simplest type of levee is the homogeneous earthfill levee. Levees may be earth filled with a zoned composition. The levee may also be composite with superstructures, waterside and/or internal structures included within the overall system. This classification is not the only one, and many real cases are very complex. Non-traditional levee cross-sections also exist and often historical levees constructed of various material constituents fall within this category because they were built with local materials available at the time of construction.

The adopted type of levee is a consequence of functionality and performance under different load situations but also feasibility, and other economic, environmental constraints (also Section 9.2).

Levee geometry is dependent on the levee type, loads, height, available land, earthfill material and foundation conditions. The availability of land plays a role in determining the steepness of the slopes. Agricultural levees tend to have flatter slopes and are un-compacted or semi-compacted. Urban levees tend to have steeper slopes with controlled compaction. Fully compacted levees generally enable the use of steeper slopes than those of levees constructed by semi-compacted or hydraulic means. A 1V on 2H slope is generally accepted as the steepest slope that can easily be constructed while ensuring stability of any rip-rap layers. A 1V on 3H slope is the steepest slope that can be conveniently traversed with conventional mowing equipment and walked on during inspections. For sand levees, a 1V on 5H landside slope is considered flat enough to prevent damage from seepage exiting on the landside slope. Riverside slopes flatter than those required for stability may have to be specified to provide protection from damage by wave action.

Space limitations in urban areas often dictate minimum levee sections requiring select material and proper compaction to obtain a stable section. Where the footprint of a proposed flood defence system is significantly limited by nearby property or other existing features, particularly relevant to highly urbanised settings, flood walls may be a more attractive and appropriate alternative to even a levee constructed with controlled compaction and a relatively narrow levee footprint. Floodwalls also offer unique aesthetics that can improve the appearance of a defence system.

Levee composition is often dependent upon locally available material. Suitable soils can often be excavated from nearby or stockpiled from the excavation of appurtenant structures. In general, sand levees tend to be designed with flatter slopes than clay levees, to address stability and seepage issues. For levees of significant height or when there is concern about the adequacy of available embankment materials or foundation conditions, embankment design requires detailed analysis. Low levees and levees constructed of suitable material resting on proven foundations may not require extensive stability analysis. For these cases, practical considerations such as type and ease of construction, maintenance, seepage and slope protection criteria control the selection of levee slopes.

3.3.1 Earthfill levees

Earthfill levees are only made of soil material (granular or cohesive) that provides every individual function. Material used can vary from the finest clay to silt, sand, gravel and rocks. Depending on the available material, earthfill may be homogeneous or zoned with a specialisation of some material as individual components.

3.3.1.1 Homogeneous levees

Definition and general considerations

Homogeneous levees are composed of uniform soils obtained from site excavation or a borrow source. Levees are usually composed of low permeability materials such as clay or silt depending on the fill source that is available. Care should be taken as to the nature of the soils that are used for constructing levees, as some fine grained soils or highly organic soils are not suitable for this purpose. However, in some cases even these soils may be considered for portions of levees. Ease and cost (influenced by accessibility and proximity) are often controlling factors in selecting borrow areas. However, the availability of better borrow materials involving somewhat longer haul distances may sometimes lead to the rejection of poorer but more readily available borrow (USACE, 2000).

Homogeneous levees can only be made of cohesive and relatively impermeable soil (clay or silt). They are mainly found in river and estuarine environments. Sand levees, more familiar in coastal environments, need at least to be protected from erosion (internal and external) as sand is not cohesive. When impermeability is provided by an additional core or mask, the levee is no longer considered homogeneous.

Geometry of the cross-section is dependent on the environment (marine or fluvial), the earthfill material (sandy levees are larger than levees made of impermeable material) and the foundation soil characteristics.

Main components

Earthfill provides both stability and impermeability because of the material constituents selected for construction. Components of a homogeneous levee may also include toe drains, cut-offs, relief wells, landside seepage berms and slope protection. The inclusion of these components is dependent on the foundation properties, the type of material used for levee fill and the external loads.

Sea levees are usually built as a mound of fine materials such as sand and clay, with a gentle seaward slope to reduce wave run-up and the erodible effect of waves. The surface of a sea levee is typically covered with grass, asphalt, stones or concrete slabs. River levees allow steeper slopes.

Technical issues

Permeable soils such as sand are more susceptible to erosion than clay and may require some form of slope protection. Clay levees may experience settlement due to loading. To reduce under-seepage, a cut-off wall may be built as a seepage barrier, or a berm may be constructed to increase the flow paths. Where impermeable soil overlies a permeable layer, relief wells or toe drains may be needed to reduce high exit gradients.

As the function and the structural integrity are based on material homogeneity, these aspects must be controlled during construction. Particle soil distribution should allow self-filtration. Drainage must be anticipated and designed to avoid internal erosion and stability problems. Heterogeneity in a levee segment (material propriety variations) may lead to preferential flow path by seepage and degradation. Compaction of material is also an important aspect of construction and must be controlled. Water content during construction has to be considered, especially when local material is used (excavation of the river bed).

Coastal levees may be susceptible to a high-energy environment. Such cases require a stronger form of slope protection (stones, artificial block, gabions etc). Figure 3.70 gives an example of these structures. The geometry of the levee (especially waterside slope) is influenced by protection elements' stability. More details about use of rocks for slope protection purpose may be found in CIRIA; CUR; CETMEF (2007).

Typical sketches

Figures 3.61 to 3.70 show typical sketches of homogenous levee types, and are representative of both fluvial and coastal cases. Figure 3.70 is more specific to coastal environments.

Note

The typical sketches shown throughout this chapter are illustrations and not recommended or fully designed cross-sections. Layer thickness, slopes and dimensions must not be considered as realistic examples. Many other associations of components may be found in practice (whether they are on homogeneous, impermeable or permeable soil foundation, and whether or not they include protection, toe drain or berms).

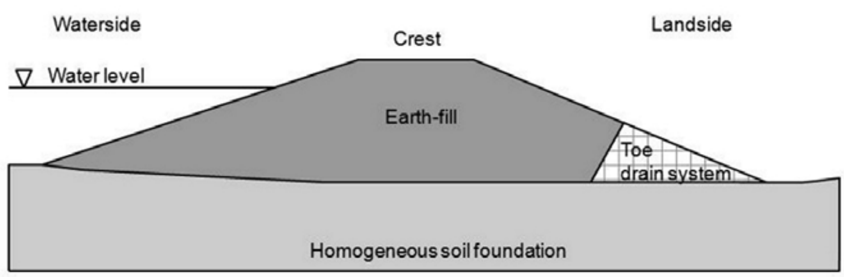


Figure 3.61 Homogeneous levee on homogeneous soil foundation

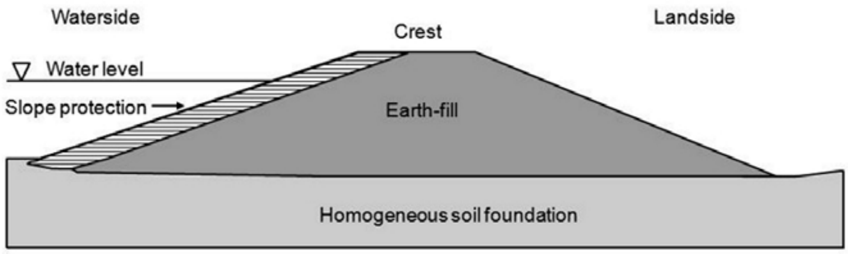


Figure 3.62 ***Homogeneous levee with slope protection on homogeneous soil foundation***

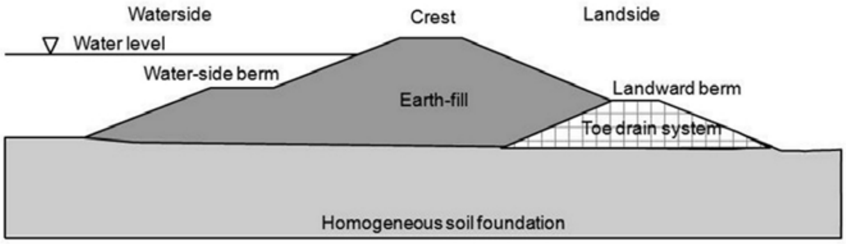


Figure 3.63 ***Homogeneous levee with berms on homogenous soil foundation***

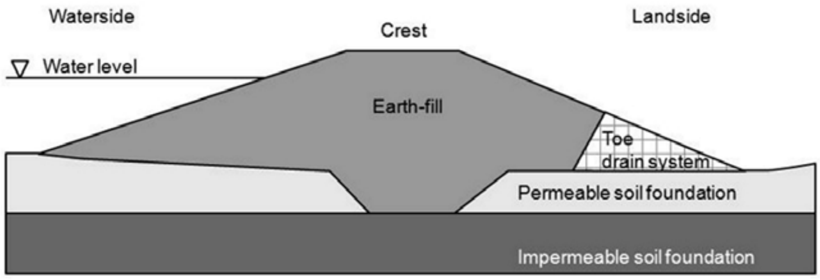


Figure 3.64 ***Homogeneous levee on thin permeable soil foundation***

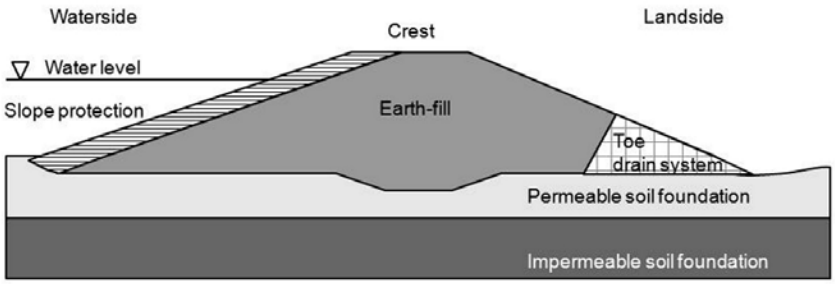


Figure 3.65 ***Homogeneous levee with slope protection on thin permeable soil foundation***

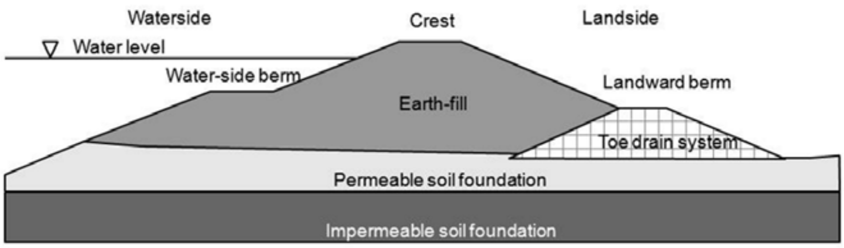


Figure 3.66 ***Homogeneous levee with berms on thin permeable soil foundation***

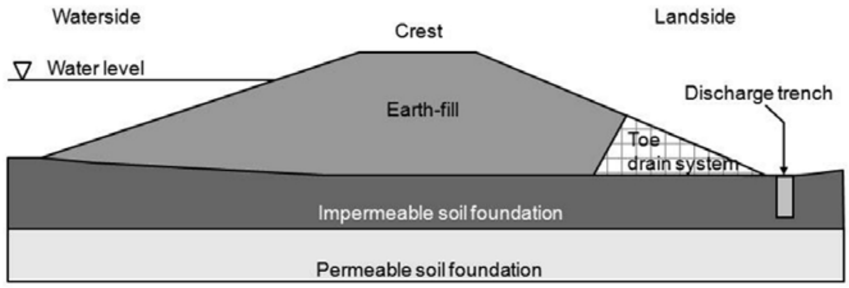


Figure 3.67 *Homogeneous levee on thin impermeable soil foundation*

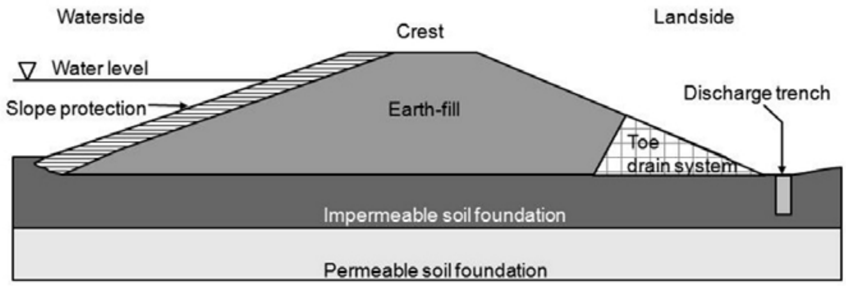


Figure 3.68 *Homogeneous levee with slope protection on thin impermeable soil foundation*

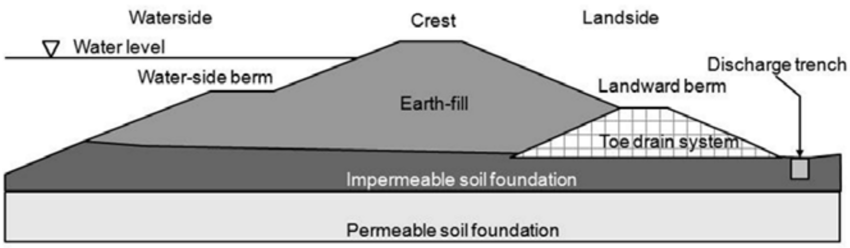


Figure 3.69 *Homogeneous levee with berms on thin impermeable soil foundation*

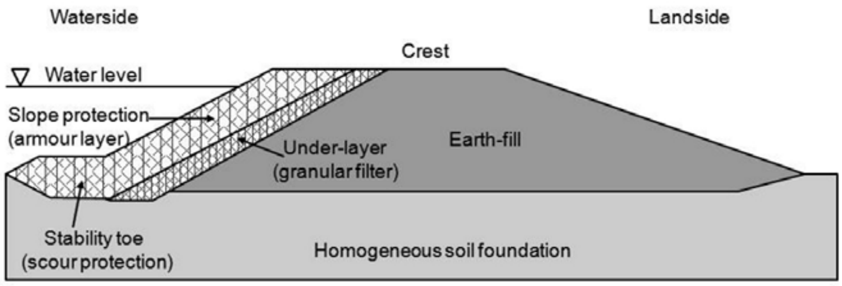


Figure 3.70 *Homogeneous levee with rock slope protection (coastal levee)*

3.3.1.2 Zoned levees

Definition and general considerations

Zoned levees consist of a combination of permeable and relatively impermeable material making up the levee cross-section. Zoned levees are typically constructed when one type of material is scarce or there is an abundance of another type of material that needs to be used. In cases where the site material used as earthfill is too permeable (gravel, sand etc) the cost of transporting impermeable material imposes the reduction of those volumes.

There are two different types of zoned levees, which are levees with an impermeable core and levees with an impermeable mask (ie clay layer on the waterside of the levee). The first ones are conceptually similar to earthfill zoned dam and can retain water for relatively long periods. The second ones are very commonly used for coastal levees where local material (sand) has no ability to fulfil the impermeability function and is susceptible to erosion. The latter is generally more economical than a central impervious core and, in most cases, is entirely adequate.

Another intermediate type of zoned levee consists of impermeable material placed on the waterside and permeable material on the landside. As a general rule levees are constructed as homogeneous sections because zoning is not usually necessary or practicable. However, where materials of varying permeabilities are encountered in borrow areas, the more impervious materials should be placed toward the riverside of the embankment and the more pervious material toward the landside.

A levee may originally be zoned, but in some cases, a homogeneous levee can be transformed into a zoned levee to reduce the through-seepage by adding a bentonite core.

Main components

Zoned levees include a component that provides impermeability (central impervious core, thick layer of impervious material covering the waterside slope) and the earthfill that provides mass stability (and eventually drainage and filtration). The impermeability component (mask or core) is eventually separated from the permeable material (earthfill or permeable soil foundation) by a filter of geotextile or graded material.

Where required to improve under-seepage conditions, landside berms should be constructed of the most pervious material available and riverside berms of the most impervious materials.

Coastal levees made of a homogeneous earthfill covered by a more or less tight cover of clay may be classified in both homogeneous or zoned levee. As mass stability and impermeability are provided by two different components, the earthfill and the external layer, it can be considered as zoned levee. Figure 3.74 presents one typical case of a levee that may be found on the Dutch or German coast.

Technical issues

Zoned levees on permeable soils may require seepage control to prevent excessive uplift pressures and piping through the foundation. The methods for control of under-seepage can include horizontal drains, cut-offs (compacted backfill trenches, slurry walls, sheet piles and concrete walls), upstream impervious blankets, downstream seepage berms, toe drains and relief wells. The integrity of the impermeability component (core or mask) is important and must be protected and confined by filter layers to prevent migration of fines.

In the coastal environment, filter layers ensure the transition between two adjacent layers of material. Graded material is often used to prevent the migration of fines and to ensure a mechanical transition.

When impermeability is provided by an external layer of clay, levees may be susceptible to cracking and other deterioration processes due to external agents. As the external impermeable layer plays a major role in levee integrity and function, it must be well protected from external deterioration. The external layer weight must compensate for the uplifting pressure caused by the presence of water in the earthfill.

A good practice should be to make the levee more permeable from the waterside to the landside. This practice can reduce the levee section under uplift and lower the phreatic line inside the levee (by landside drainage).

Typical sketches

Typical sketches of zoned levee types are presented in Figures 3.71 to 3.75.

Note

These sketches are illustrations and not recommended or fully designed cross-sections. Layer thickness, slopes and dimensions must not be considered as realistic examples. Many other associations of components may be found in reality.

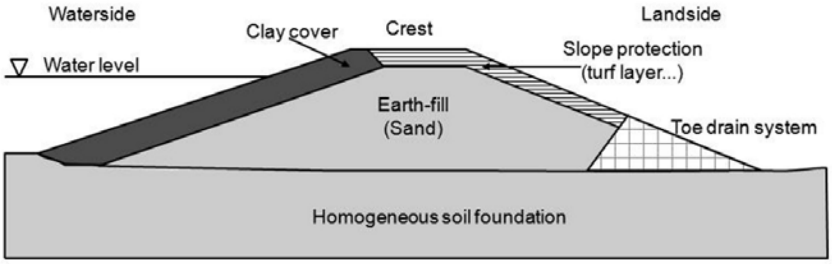


Figure 3.71 *Zoned levee (with impermeable mask) on homogeneous soil foundation*

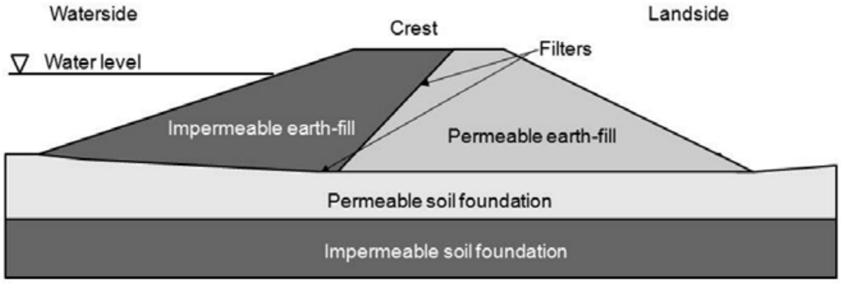


Figure 3.72 *Zoned levee on thin permeable soil foundation*

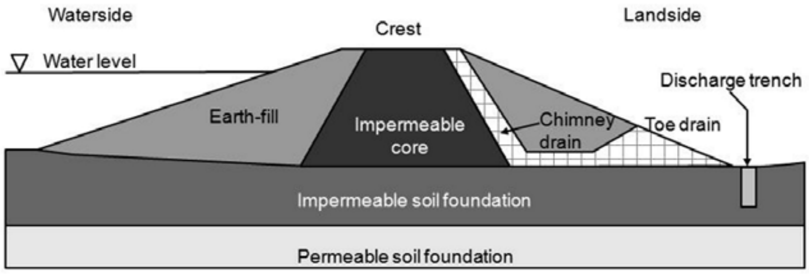


Figure 3.73 *Zoned levee with impermeable core on thin permeable soil foundation*

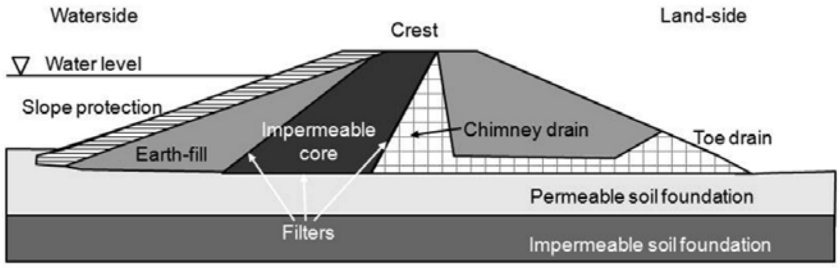


Figure 3.74 *Zoned levee with impermeable core and slope protection on thin permeable soil foundation*

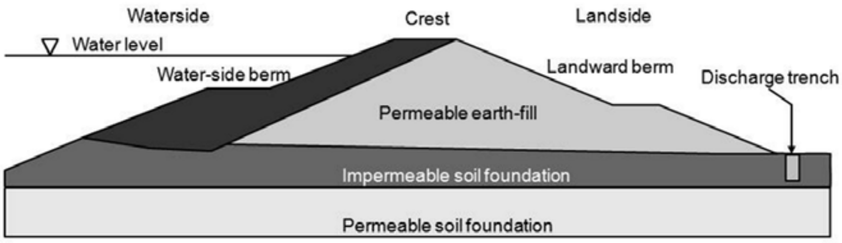


Figure 3.75 *Zoned levee with berms on thin impermeable soil*

3.3.2 Composite levees

Composite levees are made not only of soil material but also of structural components that have different reactions to external stresses or loads. These structural elements are hard engineering such as concrete wall, steel sheet piles and 'softer' components such as geotextiles.

3.3.2.1 Levees including superstructures

Definition and general considerations

Levees that include walls, sheet piling or other structures often have limited extra rights of way. They are found when other land reclamation is too expensive, or the foundation conditions will not permit an increase in the levee section. In some cases, a vertical or very steep wall is placed on the top of a slope to reduce wave overtopping. Vertical walls on top of the slope are often adopted if the available place for an extension of the base of the structure is restricted.

Main components

A levee including a superstructure has two main components, a basement component made of soil material and a structural component that sometimes support some additional earthfill material.

An I-type flood wall is a vertical wall partially embedded in the levee crown or foundation. The stability of such walls depends on the development of passive resistance from the soil. I-type flood walls rarely exceed two metres above the ground surface for stability, and they may consist of a row of sheet piling with a concrete cap.

An inverted flood wall is a reinforced concrete wall whose members act as wide cantilever beams in resisting hydrostatic pressures acting against the wall. The inverted flood wall is used to make flood wall levee enlargements when walls higher than two metres are required.

Superstructures may be massive walls (concrete, masonry).

Technical issues

The flood wall requires adequate stability to resist all forces that may affect it. An I-type flood wall is considered stable if sufficient passive earth resistance can be developed to yield an ample factor of safety against overturning. The depth of penetration of the I-type flood wall should provide adequate seepage control. The penetration depth of the I-type flood wall required for stability should be checked to meet seepage requirements. For all these reasons, the I-type flood wall should be anchored into the foundation soil.

Distinction should be made between flood walls to prevent overtopping or flood walls to prevent overflow (ie footing of flood wall above or below still water level). The latter type should be discouraged and if it cannot be avoided, detailed design of the foundation is needed. Because the wall is constructed on top of an earth structure, passive force is limited.

Typical sketches

Figures 3.76 to 3.82 are typical sketches of composite levee types including superstructures.

Note

These sketches are illustrations and not recommended or fully designed cross-sections. Layer thickness, slopes and dimensions must not be considered as realistic examples. Many other associations of components may be found in reality.

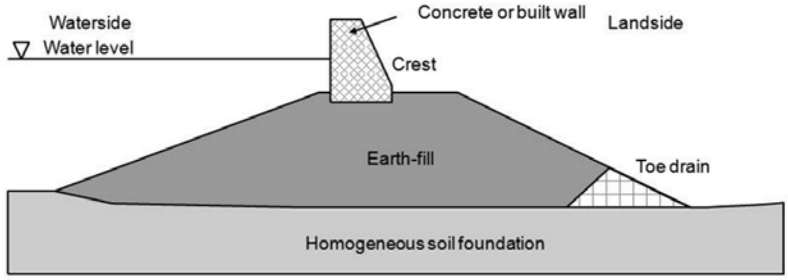


Figure 3.76 Massive built or concrete wall on crest of levee on homogeneous soil foundation

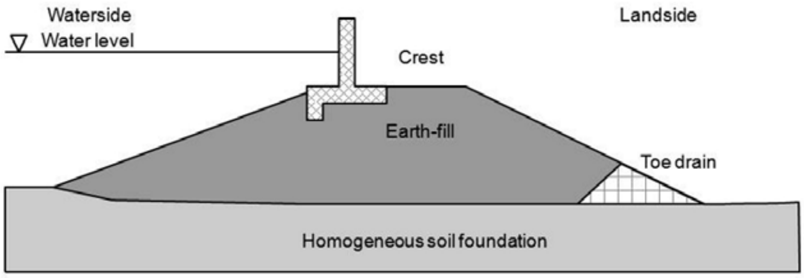


Figure 3.77 Concrete inverted T flood wall on crest of levee on homogeneous soil foundation

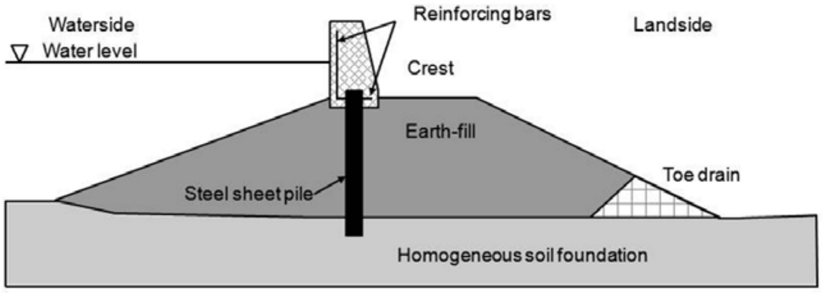


Figure 3.78 I-type flood wall on crest of levee on homogeneous soil foundation

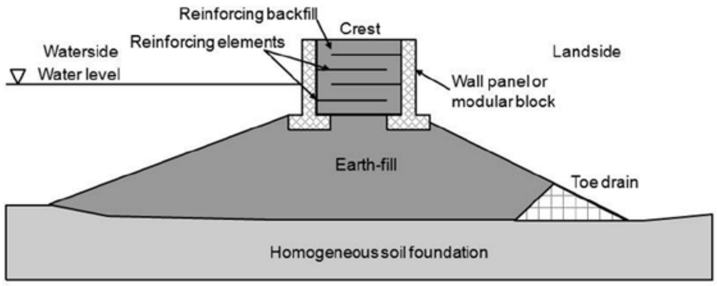


Figure 3.79 Levee crest raising with mechanically stabilised earth

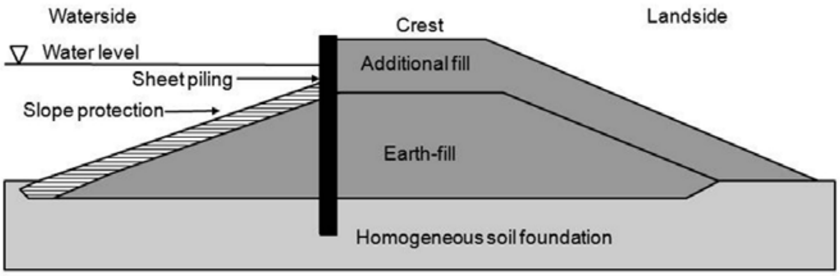


Figure 3.80 Levee crest raising with sheet piling

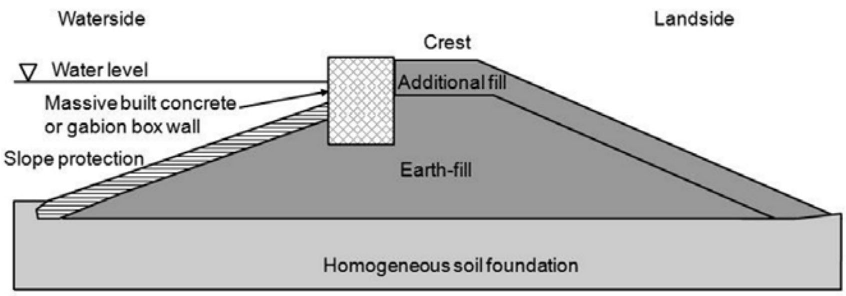


Figure 3.81 ***Levee crest raising with massive built or concrete wall***

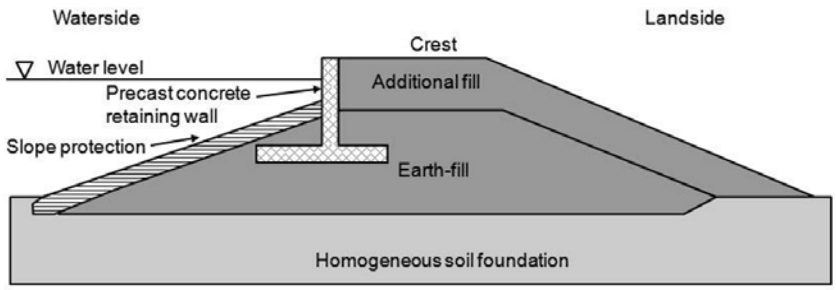


Figure 3.82 ***Levee crest raising with pre-cast concrete retaining wall***

Figure 3.83 ***Levee crest raising with concrete wall (courtesy T Mallet)***

3.3.2.2 Levees including structures on the waterside

Definition and general considerations

In most cases, the levee slope is restrained by the angle of repose of the earthfill. The levee slope cannot physically exceed a certain value, so the base width of the levee may be very large. When available space to build a levee is restricted, retaining walls are used to reduce the footprint. The stability is not provided by an auto-stable massive structure, but a retaining structure is used to hold back the earthfill. Retaining walls are multi-functional components of a levee as they also provide:

- protection for the earthfill
- impermeability (in some cases)
- filtration, which prevents the migration of fine fractions of the earthfill.

Main components

A composite levee including a retaining structure comprises the components of an earthfill levee and the retaining structure. Different retaining structure can be classified as follows:

- gravity walls (masonry stone walls, massive concrete walls, gabions etc)
- piling walls (steel, vinyl or wood planks sheet piling etc)
- diaphragm walls
- anchored walls
- cantilevered walls (T or L walls).

Technical issues

As the retaining wall is in contact with external agents (air and water), it is subject to corrosion. Water variations in the retained earthfill affect its stability and the wall must have a seepage control system to reduce hydrostatic pressure gradients. Solids transport (toe erosion) may also affect the stability of these structures. In the coastal environment, toe erosion may be more prominent because of wave reflection on the wall, inducing wave scouring, especially for vertical walls.

Typical sketches

Figures 3.84 to 3.86 are typical sketches of composite levee types including superstructures.

Note

These sketches are illustrations and not recommended or fully designed cross-sections. Layer thickness, slopes and dimensions must not be considered as realistic examples. Many other associations of components may be found in reality.

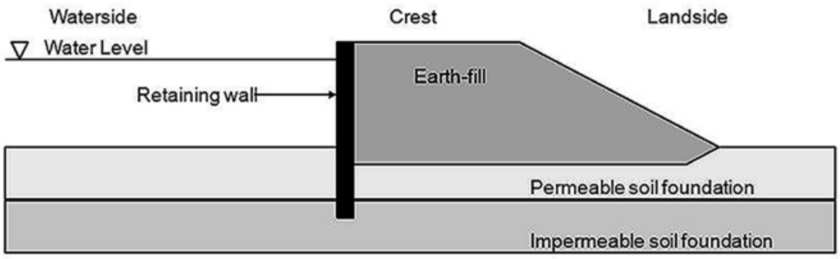


Figure 3.84 **Composite levee with retaining wall on thin permeable soil foundation**

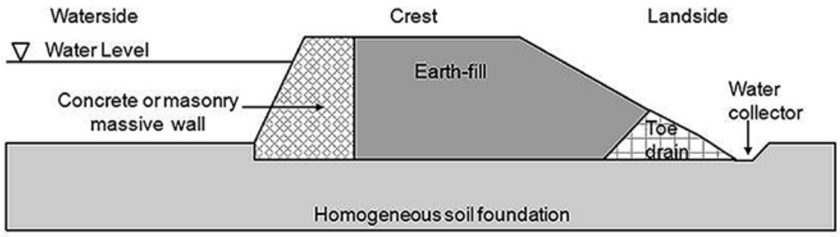


Figure 3.85 **Composite levee with massive retaining wall on homogeneous soil foundation**

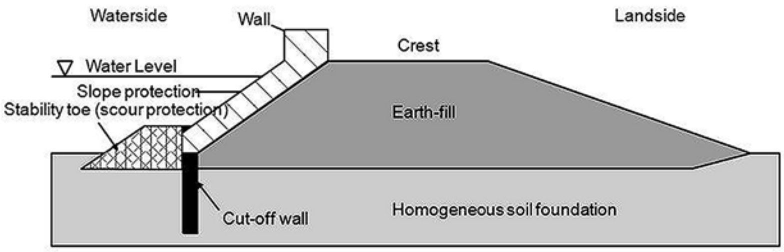


Figure 3.86 **Composite levee with slope protection, cut-off, stability toe**

3.3.2.3 Levees including structures inside

Definition and general considerations

In some cases, when levee permeability is affected, operation on the sealing component of the levee is undertaken. This operation consists of impermeable wall implantation.

Seepage barriers are features constructed into the levee embankment and/or foundation. Their function is to retard or prevent seepage across the feature.

Main components

Levees can include seepage cut-off walls of varying widths constructed with combinations of soil, cement, lime, bentonite or other admixtures. In some cases, the seepage barrier may consist of steel or vinyl sheet piles or sheet membranes consisting of polyvinyl chloride (PVC) or other synthetic compositions.

Technical issues

Special caution must be taken when the wall is added to an existing levee (as repair work). Destabilisation caused by work vibration and loads need to be anticipated.

Seepage barriers need to be carefully designed to take into account the flow of water in the entire system affected by that water path. Preventing the flow of water in one area has the potential to cause water-related issues in another area (Box 3.7).

Box 3.7 Adverse hydraulic effect of sheet piling along a river – a case from Ireland

In the town of Clonmel, located in Tipperary County, Ireland, a line of sheet piles was installed along the river to accommodate the installation of a combined sewer line. The area historically had a 1 in 5 chance of flooding in any given year, but following the installation of the piles, property owners noted that the flood levels were higher on the landside of the piles than they were on the river side. The floodplain on the bank of the river at this location is narrow and the land rises steeply. It was concluded that the flow path of groundwater to the river had been lengthened by the piles, and when the river rose, the extra head needed caused the water table to surface. Once the source of the issue was identified, locals were able to work together with water resource engineers to resolve the issue.

Typical sketches

Figure 3.87 is a typical sketch of composite levee types including superstructures.

Note

This sketch is an illustration and not a recommended or fully designed cross-section. Layer thickness, slopes and dimensions must not be considered as a realistic example. Many other associations of components may be found in reality.

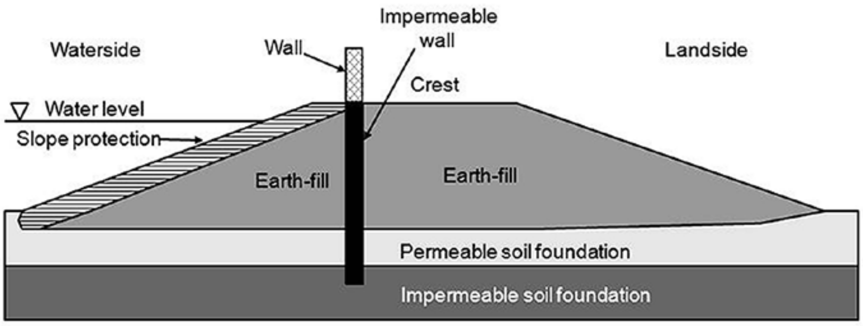


Figure 3.87 Composite structure of levee with impermeable wall

Figure 3.88 is an illustration of a levee being reinforced by sheet piles.

Figure 3.88 Composite structure of levee including sheet piles (courtesy T Mallet)

3.3.2.4 Levees including alternative constructions

Definition and general considerations

Levees built with alternative construction methods or materials often comprise an earthen embankment associated with non-hardened structure. They are characterised by the use of proprietary systems, the use of a particular fill or other materials that are susceptible to giving an advantage in the design solution. Box 3.8 presents discussions about some alternative levee construction experiments in the UK.

Technical issues

Although these techniques are recent, experiences are limited. Engineering materials including geotextiles are sometimes subject to vandalism, which can jeopardise the integrity of the levee. As well as vandalism, the durability of geotextile, especially to UV, is also a weak point. When used in construction projects of this type, tyres or tyre-derived material should always be covered with soil to prevent UV degradation of the rubber. Visual aesthetics of using a geotextile is often a negative element.

Box 3.8 *Experiments from the UK (Morris et al, 2007)*

Proprietary reinforced earth/geotextile systems

Recent years have seen the increasing use of alternative systems such as gabion walls, reno mattresses, geotextiles and reinforced earth for levee raising, steepening and surface protection. These systems offer various advantages such as the increase of external erosion resistance. One particular advantage of some proprietary systems is that embankment side slopes can be steepened thus reducing land take. However, they may require special techniques for installation and can have high whole-life costs. Geogrids used for horizontal reinforcement can also act as drainage paths through an embankment, so care is needed if such options are considered.

Individual tyres as soil reinforcement

Tyres can be linked together by strapping to form a grid of rubber, and within soil fill these form a reinforced earth system. Embankments of some height have been constructed in this way, for example as described with an embankment in Brazil (Sayao *et al*, 2002). Using tyres as soil reinforcement is more common in lower risk land-forming and road construction.

There are some examples of embankment-type structures that have been built without even linking the individual tyres together. In all these uses, careful and well-informed evaluation is needed.

Use of tyre bales

Tyre bales have a low density of the order 600 kg/m^3 to 650 kg/m^3, which means that they are able to significantly reduce ground pressures while having a reasonably high inter-bale shear coefficient ($\mu = 0.7$). However, they do have a high porosity (about 50 per cent) and a permeability equivalent to that of gravel. So where the bales form the core of the flood embankment, there needs to be some impermeable surrounding to the bales to keep leakage down to an acceptable value. The use of tyre bales has been investigated at present in a Defra/Environment Agency funded R&D project (Simm *et al*, 2005). As part of this project, a major pilot was conducted on the River Witham near Lincoln where some 4000 bales (400 000 tyres) were successfully used by the Environment Agency in stabilising a 1 km length of flood embankment. The main attraction for using tyre bales at this site relates to the low-strength peat foundation. Because of the relative weakness of the peat base, a shallower bank was needed to stabilise the embankment and prevent the weight of conventional clay fill from causing a slip(s). To re-profile the bank to 1:4 would have meant moving a power line and soke dike. Such an exercise would have been very disruptive to the local environment, time consuming and expensive.

By using tyre bales as fill, the embankment shoulder could be steepened. Less material was used because the tyre bales were much less dense than clay. The stability issues arising from re-profiling the embankment to 1:4 were also overcome. Environmental monitoring at the site has not revealed any detrimental effects on water quality, flora or fauna since construction.

Lightweight fill

The use of lightweight fill has also been tested in the UK. None of these materials are without problems; for example, concerns are often raised about PFA's chemical characteristics and the very lightness of polystyrene makes it difficult to hold down when submerged. Similarly, the impermeability of artificial materials is often questionable. For these reasons, it is recommended that these materials are only used following evaluation with carefully planned trials.

Typical sketch

Figure 3.89 is a typical sketch of composite levee types including alternative.

Note

This sketch is an illustration and not a recommended or fully-designed cross-section. Layer thickness, slopes and dimensions must not be considered as realistic examples. Many other associations of components may be found in reality.

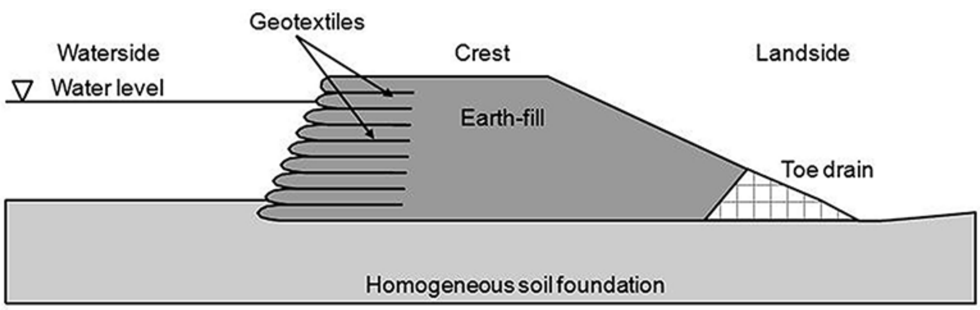


Figure 3.89 ***Composite levee with geotextile retaining wall on homogeneous soil foundation***

3.3.3 Historical levees

Levees through time

Historical levees consist of multiple layers of soil constituents constructed over the course of many years. The resulting embankment is a peculiarly complex structure derived from the results of national, regional and local history. Legacy levee systems are those that were built by our ancestors and they have continually been modified and expanded over the years through multi-stage construction with varying materials.

Levees may be composed of a variety of materials. Typically the initial construction material used to build the first line of defence was locally available soil that could readily be transported and placed. Through experience and with technological advancements, more recent levee enlargements have likely been constructed of more impermeable, highly stable soils.

The following issues are commonplace with historical levee systems:

- inaccurate historical documentation regarding construction methods and material constituents used to build the levee
- embankment core and foundation materials lacking, due to improper placement or permeable characteristics
- discontinuity in composition along the longitudinal section
- bearing capacity failure
- complexities of the structures with different stages of construction
- missing archived documents for levee knowledge
- material evolution through time.

Case studies

Boxes 3.9 to 3.11 are examples of historical levees.

Box 3.9 ***Historically grown levee at the River Odra, Germany***

The historically grown levee at the River Odra was removed at the Oderbruch area and rebuilt further from the river to get wider floodplains. The soil material was partly reused for the construction of the new levee. During the removal the internal levee structure was investigated.

The total levee height is about 6 m, and the levee has been heightened in several steps. The levee dates back to the 15th century. Around 1717, the embankment was raised to about 12 ft (≈ 3.75 m). Following the catastrophic flood of 1832 the final raise towards the present height was made (Figures 3.90 and 3.91).

A refurbishment with adaption to the actual technical standards was made using the experiences of the 1997 flood.

Figure 3.90

Old levee cross-section at km 3 (courtesy F Krueger, Frankfort on Odra)

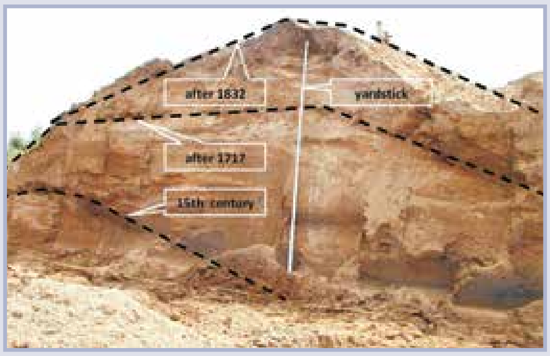


Figure 3.91

Old levee cross-section at km 3.2 (courtesy F Krueger, Frankfort on Odra)

Box 3.10 Historical levee in the Netherlands

In the low-lying parts of north-western Europe levees were constructed to protect farmland. These levees were originally rather low, and frequently overtopped. Along the rivers the farmland behind the levees often consists of soft soil. This meant that the levees were also used as the main roads in the area and consequently farms and villages were constructed on and along the levee. The main levees were constructed further inland, so levee height was the most relevant point (Vierling, 1579).

The consequence of this history is that improvements of levees in these situations became rather difficult. Following a flood, it was decided to increase the height of a levee, and this often led to a steeper levee being constructed. Widening the levee to increase the geotechnical stability can be problematic. River levee can be recognised as levees with a steep outer slope and a gentle inner slope.

This problem occurred less with sea levees. But with sea levees, historically, the main problem was to cope with wave attack. To decrease this effect usually a mild outer slope was used. So a sea levee can be recognised as a levee with a gentle outer slope and a steep inner slope. Where no space was available (eg around cities) there was a need for revetments. In the first instance these revetments were constructed as palisades with seaweed (Figure 3.92).

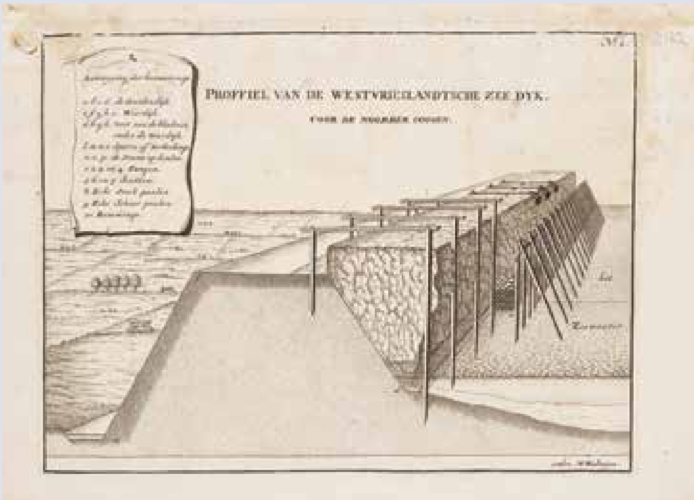
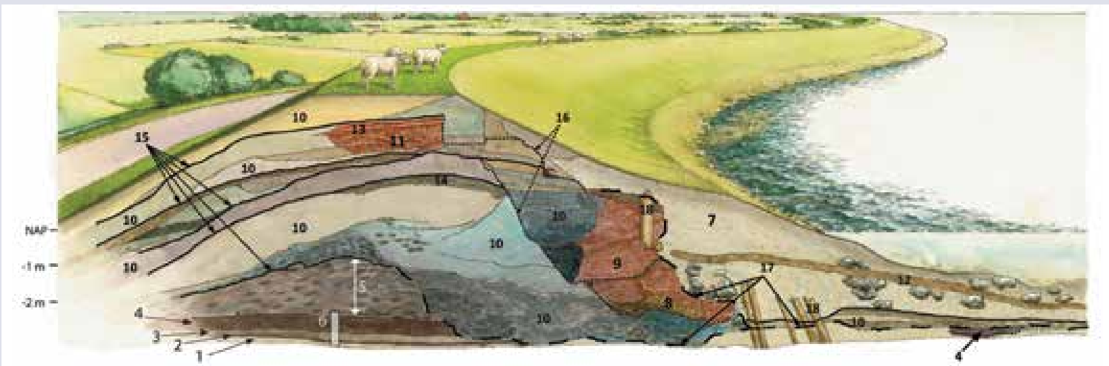


Figure 3.92 Sea levee near Medemblik, the Netherlands, from around 1600 (drawing from Medemblik Municipality – original copper engraving from Maurits Walraven)

Because of shipping connections with the Far East, the worm (*Teredo navalis*, shipworm), was introduced in the north-western Europe. This worm eats wood in salty waters and destroys all revetments. Within a few decades, all palisades and seaweed protection had to be replaced by stone revetments. Because of the lack of quarries in this area, all stones had to be imported quite long distances, making stone revetments very costly. So designs were made to minimise the amount of stone in a revetment. This resulted in placed-block revetments, because rip-rap revetment required much more stone. Although blocks for placing a revetment, and also placement of the revetment, are expensive, this was cheaper than importing a larger quantity of rip-rap for a rip-rap revetment.



Notes

1 Subsoil of sandy clay.
2 Formal Bronze Age surface with charcoal particles.
3 Deposit formed in shallow freshwater (gyttja).
4 Peat deposit.
5 Peat lumps (oldest dike 13th century).
6 Location of samples.
7 Shells.
8 Reed and twigs.
9 Sea weed.
10 Clay, sandy clay and sand layers.
11 Rubble.
12 Willow mattress.
13 Bricks.
14 Strongly rooted layer.
15 Old surface.
16 Extension of digging in older strata.
17 Extension of erosion by the sea.
18 Palisade debris.

Figure 3.93 Cross-section of a sea levee near Enkhuizen, Netherlands (courtesy B Brobbel, after Van Geel et al, 1983)

Cross-sections of old dikes show their history of improvement, for example, the cross-section of the sea levee near Enkhuizen, Netherlands. The oldest part (near 4 in Figure 3.93) was constructed between 1170 and 1319.

Box 3.11 *Historic levees on the Loire River, France*

Levees on the Loire (France) are earthen structures built over time to protect the Loire floodplains from flood damage (Figures 3.94 and 3.95).

Figure 3.94

Loire levee, Sandillon (courtesy DREAL Centre)

Figure 3.95

Loire levee, Melleray (courtesy DREAL Centre)

Current flood defence systems are the legacy of successive construction stages:

- **Middle Ages:** even if houses were not built in the floodplain during the Middle Ages, lords who wanted to extend their cultivated areas erected, with the support of farmers, a mix of earth and plants, strengthened by wooden piles, to prevent field erosion caused by minor flood. These structures were mainly designed to slow down the flow and were notably inefficient in preventing flooding.
- **15th to 18th centuries:** with the rise of river navigation, many structures were built to concentrate the flow of water in the minor bed, especially close to the ports. As these structures were unable to protect increasing stakes and land use in the floodplains from floods, the monarchy decided to erect unified earthen flood defence systems all along the River Loire, joining the navigation structures. The Loire levees have been alternately raised and widened with local materials after each major flood. The levees' slope was progressively equipped with a stone pavement covered by topsoil to prevent damage from erosion and the first spillways were experimented with. New regulations forbade any construction at less than 20 m from the levee toe.
- **19th century:** three floods in 1846, 1856 and 1866 caused major damage to the levees (more than 200 breaches). To prevent such damage, shoulder curbs (approx 50 cm high) were built on top of the levees, but these shoulder curbs were never very efficient. A programme of spillways was partially carried out after these floods.
- **20th to 21st centuries:** many crossing pipes were built in the levees during the 20th century. To prevent seepage, the Loire levees have been widened and, where practical, a filter drain has been installed at the levee toe on the landside. Where widening is not possible, impervious screens are built on the levee body. Rip-raps are used to protect the levee toe on the riverside from external erosion.

These different stages of construction have been a very long process, with geographical disparities. Construction and maintenance stopped for long periods (for example, during the French Revolution) and major floods were usually followed by major reinforcement programmes. Many breaches were quickly restored with different types of materials that are still in place in levees.

The Loire levees are composite structures that cannot be seen because of a single and homogeneous design process (Figure 3.96).

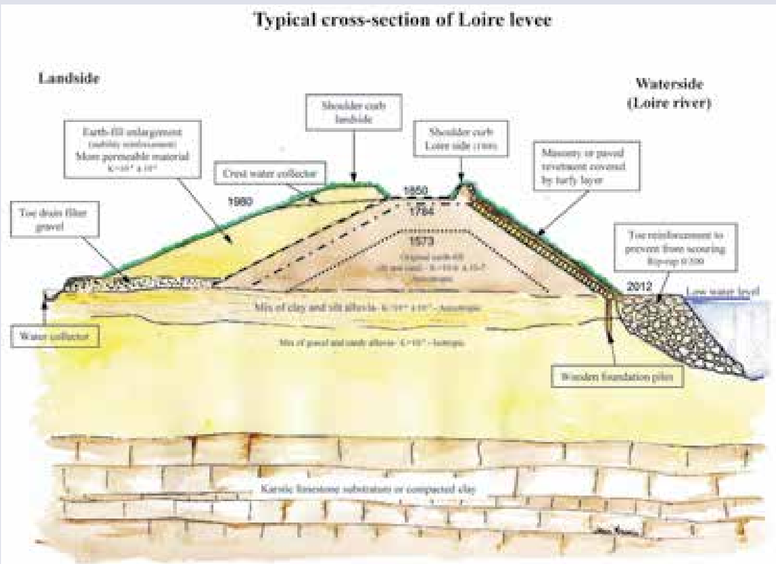


Figure 3.96 ***Typical cross-section of a Loire levee today (courtesy J Maurin)***

3.4 STRUCTURES ASSOCIATED WITH LEVEES

There are numerous structures associated with levees that are built to complement the levee (Figure 3.1). Each structure has its own primary function. Some are designed and constructed specifically to reduce the risk of inundation during flood events. Other structures are installed to regulate interior water levels under daily conditions, and some are constructed for other purposes. Regardless of the functionality of the appurtenant feature, connection points of these structures, whether through or over levees, are critical to the integrity of the flood defence system.

The leveed area often requires provisions for drainage of interior water, resulting from seepage through the levees or storm runoff from local uncontrolled inflows that drain into the channels within the leveed area. Interior flood control is typically addressed through the use of pumping stations, tide or flap gates, or temporary storage of water in low-lying areas or channels that are not subject to flood damage. These features, along with others installed for flood defence, are described in Section 3.4.1.

When considering each discontinuity created by a structure placed beneath, through or over the levee, it is important to ensure that the standard of protection and service offered by the structure is consistent with that offered by the levee. A flood defence system can be compromised by a single weak point within that system.

Riverine structures associated with levees are constructed for one or more of the following objectives:

1. Temporarily retain water to prevent inundation of a designated area.
2. Allow for a controlled release of water.
3. Provide access through the flood defence system.
4. Convey water from the landside to the waterside of the flood defence system.
5. Allow for utility services to pass over, through or beneath the flood defence system.
6. Measure levee characteristics and integrity.
7. Allow drainage of water from the leveed area.

Coastal levee structures are designed to manage the risk of damage to inland areas from the effects of coastal storms occurring over large bodies of water. The intent of coastal levee structures are to:

- **reduce damage from waves:** coastal storms can produce large waves that can significantly damage onshore infrastructure and flood risk management structures
- **reduce damage from storm surge:** often coastal storms present a danger to the onshore areas from flooding caused by the storm surge; these effects are reduced by the construction of levees and flood walls that serve as surge barriers
- **reduce shoreline erosion:** maintaining a shoreline is important for several reasons; infrastructure near the shore depends on a stable shoreline for economic and structural stability purposes; wetland and forested areas, which can reduce the onshore effects of winds and surges from offshore storms, depend on stable shorelines for their existence.

Structures associated with levees within an estuarine environment may be built to address a combination of riverine and coastal flood defence objectives as previously discussed.

For many of the structures associated with levees, there are some common issues about the interface between levees and structures that are described in Table 3.17.

Table 3.17 Technical issues for the interface between structures and levees

Issues	Methods	Section ref
External erosion at the surface of a transition area that may be caused by contact erosion occurring inside or by an external source, such as turbulence caused by a difference in roughness coefficient across a transition between two materials.	Use of appropriate filter materials at the interface. Adequate slope protection to prevent scour.	8.4, 9.1.3
Internal erosion at a soil/structure transition • backward erosion • concentrated leak erosion • suffusion • contact erosion.	Expansive (non-shrink) grout mixtures may be advantageous to reduce the risk of piping along soil structure interfaces.	8.5, 9.1.3
Settlement under or near the structure, causing a preferred path for water flow.	Conservatively design foundation support systems for structures bearing in soft or compressible soils, and a combination of foundation support systems with pre-consolidated foundation support soils (achieved by wick drains, pre-loading or use of settlement plates).	4.9, 7.7, 7.8, 8.7, 9.1.3
Sliding (induced shear) due to the presence of a structure.	Constructed components may be keyed in for extra stability; appropriate material use for embankment and adequate compaction near the transition point.	8.6, 9.1.3
Failure, instability or collapse of the structure.	Continual maintenance and periodic inspection of the structural integrity. Store information in a database with transition structure details including type, material constituents, ownership, historical loading, maintenance record and performance.	4.2
Owner/maintainer of structure may not be the levee maintainer.	An O&M manual with specific requirements should be endorsed by all operators/maintainers.	4.1.4, 4.3

3.4.1 Structures contributing to flood defence

Features described within this section are structures built for flood defence during high water and storm surges, and for the daily management of interior water levels. Many of these structures may be used in conjunction with other features within fluvial, estuarine or coastal environments. For data requirements and design detailed considerations relating to flood protection structures, see Section 9.11. The sequence in which the structures are considered in this section are:

- **structures or natural features that retain water:** dunes, floodwalls
- **structures that retain water but need to be closed during flood:** gateways, discharge pipes, sluices surge barrier
- **structures that retain water in conjunction with the levee or natural ground:** seawalls, bulkheads, revetments
- **structures that assist the levee by reducing the hydraulic actions:** protective beaches, jetties, groynes
- **structures that contribute in other ways to flood defence:** air vents, trash racks, pumping stations.

3.4.1.1 Spillways and floodways

Definition

A spillway is a structure that is designed to provide a controlled release of water from one area to another, either over the structure or through it (Figure 3.97 and 3.98). It can be designed to divert water from the river or restore water to the river. Most often, spillways release floods to prevent overflow or damage to the dam or levee. Except during high water events, water would not normally flow over the spillway. If the flow rate can be controlled by mechanical means, such as gates, it is called a controlled spillway. However, if the geometry of the spillway is the only control, it is referred to as an uncontrolled spillway.

Figure 3.97 Spillway on the Vidourle River (courtesy G Degoutte, Irstea)

A secondary spillway function may be to pond tailwater on the landside of a flood defence system. This can reduce the applied differential head on the levee by providing an extra landside surcharge that will counteract applied uplift pressures.

A spillway may be constructed as a low crest embankment, which is protected against surface erosion from high velocity water flow. The surface protection is usually concrete, built stone or rocks. Connecting a spillway section to the levee embankment is accomplished with ramps or vertical walls.

Figure 3.98 Spillway on the Loire River (courtesy Y Deniaud, CETMEF)

Low-head, gated spillways typically have crest elevations set near the riverbed elevation to maximise capacity. Riverbed elevations generally vary across the proposed spillway section, and bed elevations in alluvial rivers vary with discharges, so it is necessary to fully understand the characteristics during flood conditions to accurately select the optimum crest elevation.

The length of spillways designed as overflow embankments is often determined by selecting the combination and number of gates, length of overflow section, flow easement and levee raising that has the lowest total cost. Overflow sections with significant head differentials will require properly shaped crests, energy dissipation structures, and downstream channel protection.

A floodway refers to reserved property set aside to divert floodwaters from a river channel or other watercourse. Use of a floodway induces draw-down of the watercourse and alleviates pressure on levee systems near to the river channel.

Functions within the defence system

Spillways and floodways may work singly or in tandem with one another as the spillway provides the location for water conveyance to the designated floodway.

Two main types of spillways can be identified, depending on the objective of the downstream area:

1. Safety spillways are structures that discharge in a leveed area. Their objective is to protect the levee from failure under excessive water levels.
2. Diversion spillways are structures that discharge in a flood detention area, a secondary river channel or a designated floodway. Their objective is to divert part of the flood from the main river channel to reduce the downstream water level.

Appurtenant components of the spillway include the spillway crest, dissipation structures and channel protection, all of which are designed to minimise landside scour and erosion due to operation of spillways.

Issues

Table 3.18 highlights the main issues for spillways and floodways.

Table 3.18 Technical issues for spillways and floodways

Issues	Methods	Section ref
Scour, resulting from high velocities during peak discharges, at waterside and landside toe	Placement of conventional revetment materials, placement of pre-cast articulated concrete mattresses (ACM), peaked stone toe protection, turf establishment for erosion control, inspection and continual maintenance.	4.13, 9.14
Crest stability	Turf establishment and revetment construction to minimise erosion, flattening of spillway embankment slopes, design/construction of stability berms, inspections and continual maintenance.	4.11, 4.13, 9.14
Resiliency of material components	Appropriate selection of minimum particle size for stone revetments, appropriate selection of correctly sized ACM components, use of concrete structural components in design of spillway, inspections and maintenance.	4.13, 9.14, 10.5
Uncontrolled development	Regulate development within a designated floodway to ensure there are no increases in upstream flood elevations and to safeguard against unnecessary inundation of existing infrastructure.	2.2
Human error in operating the floodway	Develop written manuals for operating procedures to assist the owner's decision of when and how to operate the flood defence system to ensure safe operation and adequate protection of the public, property and environment.	4.2
Uncontrolled spillways which cannot be 'controlled' in the event of a flood – they are either active or inactive depending on the water level	Consider designing a controlled spillway, one with flood gates or other appurtenances to temporarily retain water below the threshold that it has been designed for.	9.14

Typical sketches

Figure 3.99 demonstrates a floodway that is a lateral extension or widening of the river channel where the 'flood fringe' limits extend beyond the floodway.

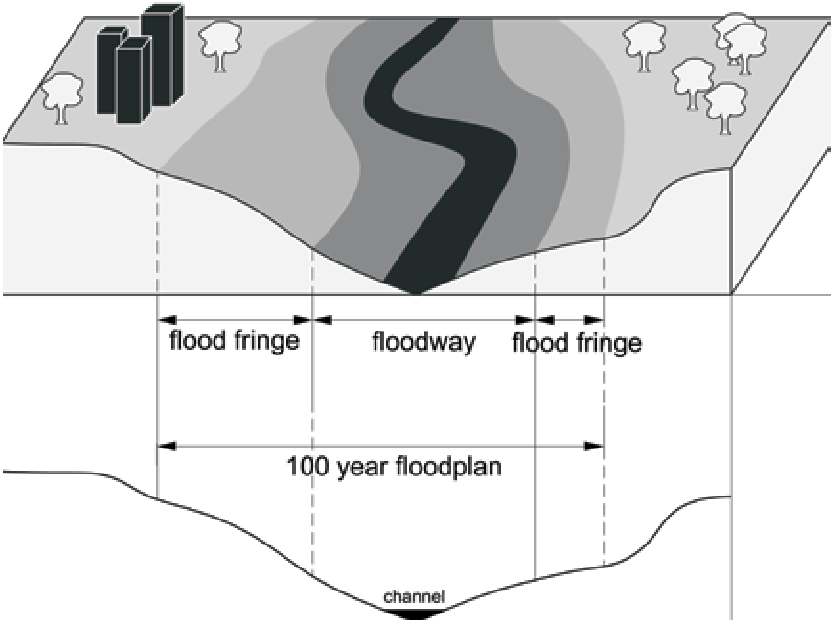


Figure 3.99 ***General depiction of a floodway within a floodplain***

A floodway can either be situated along the alignment of the watercourse or as a separate path (Figure 3.100).

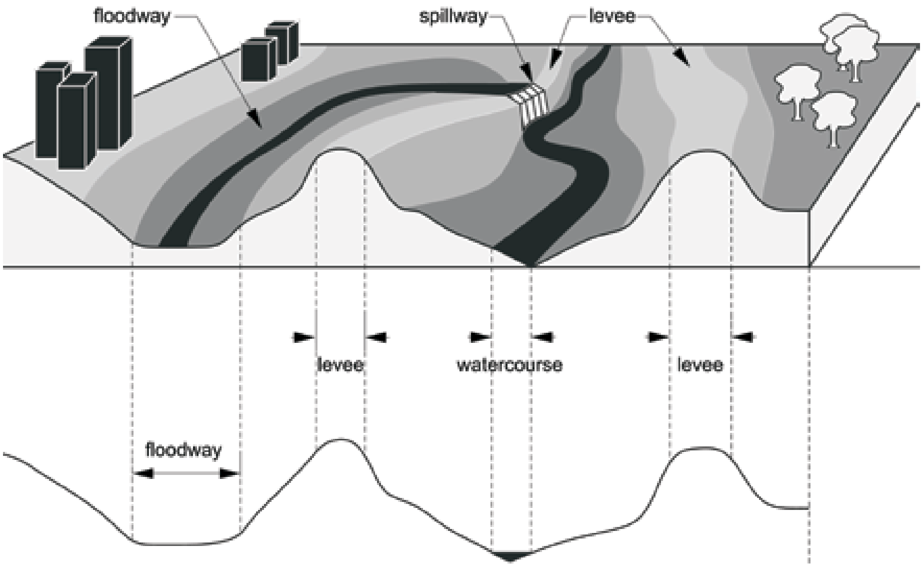


Figure 3.100 ***Spillway and floodway configuration***

Box 3.12 gives an example of river management using spillways.

Box 3.12 *Flood water diversion in lower Mississippi River*

Figure 3.101 shows combined spillways and floodways along with other flood control features for the lower Mississippi River.

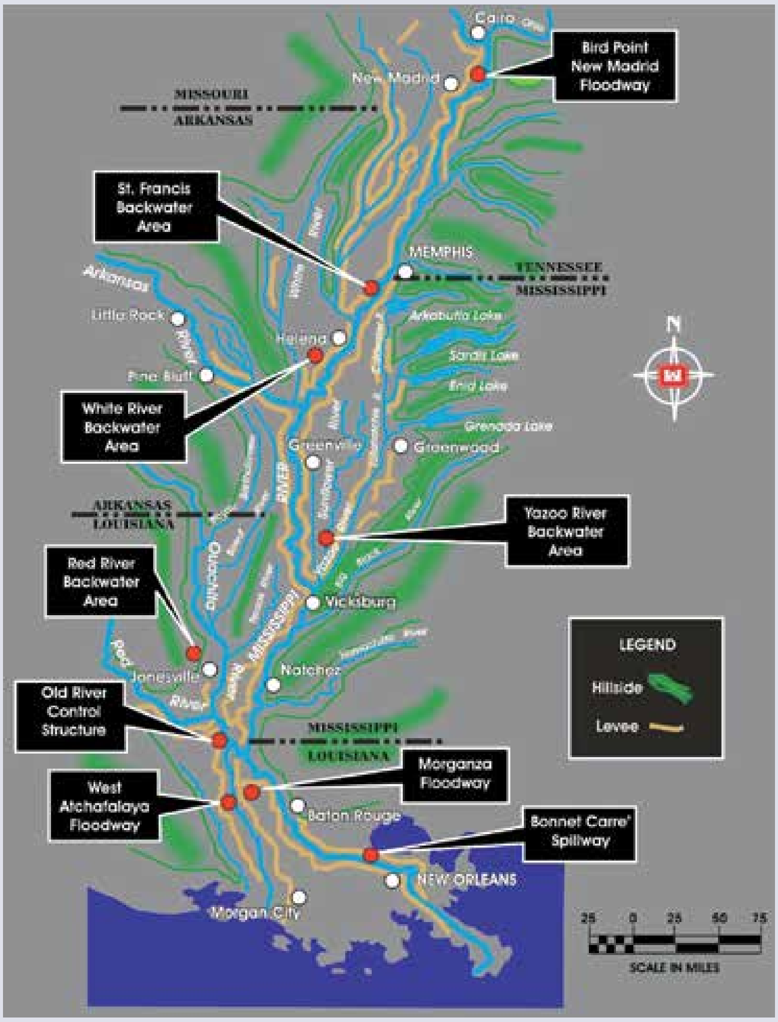


Figure 3.101 *Spillways and floodways in lower Mississippi River (courtesy Mississippi Valley Division, USACE)*

3.4.1.2 Flood walls

Definition

A flood wall is a vertical barrier designed to temporarily contain a river or other body of water in which the water may fluctuate, rising significantly during seasonal or extreme weather events. Flood walls are mainly installed in locations where space is scarce, such as urban environments or where the construction of a levee is not feasible. Flood walls, along with levees and other interior drainage features including pumping stations, should work as a system to maintain the interior water surface below the damage elevation.

In addition to levees, flood walls are the most common structures used to manage flood risk from offshore storms. In many cases they are used to reduce the risk of flooding from rivers and storm surges that may occur near the mouth of a major river. Flood walls are designed for a river flow or storm surge of a specified return period. The ultimate design is based on the most severe condition. Temporary/removable flood walls are also occasionally used to reduce the risk of inundation to property. Figures 3.102 to 3.104 show three styles of flood walls, two of which have architectural detailing that improve aesthetic appeal of the structure.

Figure 3.102

Painted concrete flood wall improvements, Cape Girardeau, Missouri, USA (courtesy USACE)

Figure 3.103

Flood wall with aesthetic, St Louis, Missouri, USA (courtesy USACE)

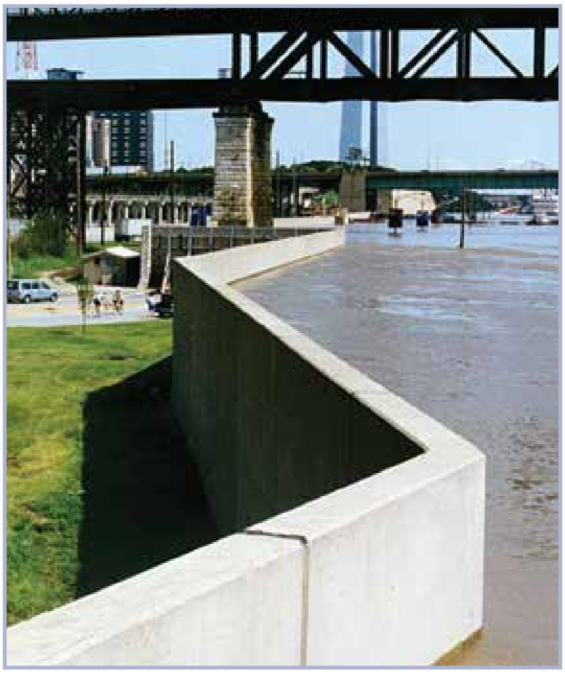


Figure 3.104 ***Concrete flood wall, St Louis, Missouri, USA (courtesy St Louis District, USACE)***

The effects of a storm larger than the designed flood wall should be considered. In the event that the flood wall is overtopped, the structure should be built with resiliency to prevent entire collapse or catastrophic failure.

Some flood walls are intended to function as retaining walls in the absence of a hydraulic loading. However, when hydraulic loads are applied to either side of the retaining wall, its function then changes to that of a flood wall. A flood wall is a type of retaining wall subject to water force on one side that is usually greater than any resisting earth force on the opposite side. Typical retaining walls are intended to retain earthen backfill material and designed for the appropriate lateral earth pressure applied on only one side of the wall. In comparison, flood walls are designed to withstand hydraulic loads applied on either or both the waterside and landside in addition to lateral earth pressure. The flood loading (surge tide, river flood etc) may be from the same or the opposite direction as the higher earth elevation.

Flood walls constructed from prefabricated concrete materials are designed in conjunction with flood gates to provide access to/from the waterside and landside. Walls that are cast-in-place concrete structures are either gravity or cantilever design. The gravity design relies on the mass of concrete to provide stability with little or no internal steel reinforcement, whereas cantilever designed wall components are relatively thin and are reinforced. Other design systems and materials are possible, including mortared stone or brick. Walls may be founded on rock, soil or piles, often with sheeting driven below the wall and embedded into the footing. This method of wall construction is to prevent or minimise under-seepage and through-seepage due to the presence of relatively pervious materials near the ground surface within the fine-grained blanket soils (ie sand lenses, localised deposits of pervious fill materials). A toe under-drain system is often constructed on the landward side of the wall to intercept potential seepage flows and to prevent undermining of the wall by loss of soil materials.

The most common types of flood walls include cantilever T-type and I-type walls. The cantilevered T-type has a 'T' configuration in which the cross bar of the T serves as a base, and the stem serves as the water barrier. When founded on earth, a vertical base key is sometimes used to increase resistance to horizontal movement. If the wall is founded on rock, a key is usually not provided. Where required, the wall can be supported on piles. A sheet pile cut-off can be included to control under-seepage or

1 2 3 4 5 6 7 8 9 10

provide scour protection for the foundation. T-type walls may be provided with a horizontal or sloped base. Cantilever I-type flood walls consist of driven sheet piles capped by a concrete wall. I-walls are most often used in connection with a levee and T-wall junctions for protection in narrow restricted areas where the wall height is not over 2.5 m to 3 m, depending on soil properties and geometry.

Another type of flood wall includes a braced sheet pile flood wall consisting of a row of vertical pre-stressed concrete sheet piles, backed by batter piles connected to the sheet piles by a cast-in-place horizontal concrete beam with shear connectors as required to resist the vertical component of load in the batter pile. This type of wall is commonly used along the coast because it is ideal for wet areas because no excavation or dewatering is required to construct the wall. The disadvantage is that it is more indeterminate than other wall types.

Other less commonly used types of flood walls are buttress, counterfort, gravity, cellular and cellular sheet pile.

Function within the defence system

Flood walls are often designed for urban areas that have narrow footprints in which available land is limited and will not accommodate the construction of a levee. The intent of the flood wall is to temporarily contain the river or other body of water and protect nearby property from inundation.

Issues

Table 3.19 highlights the main issues for flood walls.

Table 3.19 Technical issues for flood walls

Issues	Methods	Section ref
Scour holes along the base of the flood wall	A designer should be consulted. For scour holes, consider performing soundings or a bathymetric survey if scour appears to be affecting a large area. Repair with compacted soil and/or rip-rap. Earthfill should be reseeded.	4.17
Displacement and deterioration of wall joints	Refill and reseal joint. If a flood wall monolith moves laterally or vertically more than about 0.025 m relative to the nearby monolith, the water-stop has probably failed and an external water-stop should be designed for its replacement.	4.17
Settlement of wall monoliths	A designer should be consulted.	4.17
Managing connections/ transitions with closure structures	Inspect joint materials at seals and replace degrading joint materials. Inspect steel corrosion and sanding/epoxy coating or replace corroded steel components. Inspect for spalling/cracking of concrete components and grouting/caulking of observed potential seepage conveyances and possibly replace distressed components.	4.14, 4.17
Managing transitions from flood wall to levee embankment	Inspect landside ground surface for indications of seepage/sand boils. Inspect soil–structure interfaces for formation of voids/shrinkage. Use non-shrink grout mixtures as an engineered fill transition zone at the soil–structure interface to minimise piping/shrinkage at interfaces and possibly form seepage conveyances.	4.16, 4.17, 9.11, 9.15
Surface deterioration – cracking and/or spalling	Determine the cause of cracking/spalling and design a repair system to treat the cause. Seal cracks to prevent further deterioration from water intrusion. Coat the horizontal surfaces of flood walls with a soluble reactive silicate concrete treatment.	4.17

Typical sketches

Figures 3.105 to 3.107 are typical sketches of flood walls.

Note

These sketches are illustrations and not recommended or fully designed cross-sections. Layer thickness, slopes and dimensions must not be considered as realistic examples. Many other associations of components may be found in reality.

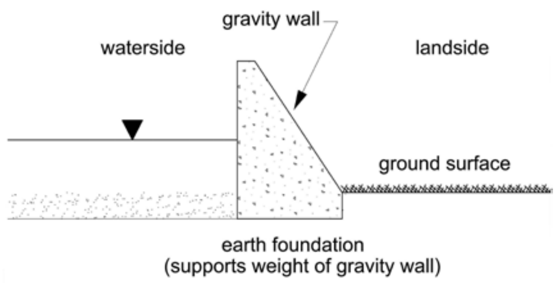


Figure 3.105 Gravity flood wall

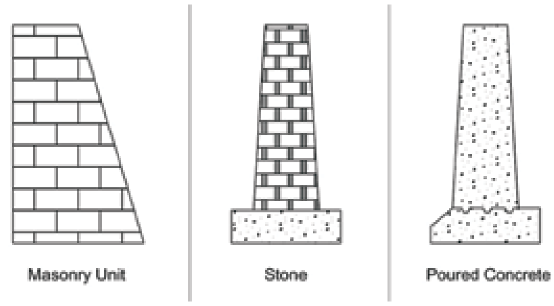


Figure 3.106 Gravity wall types

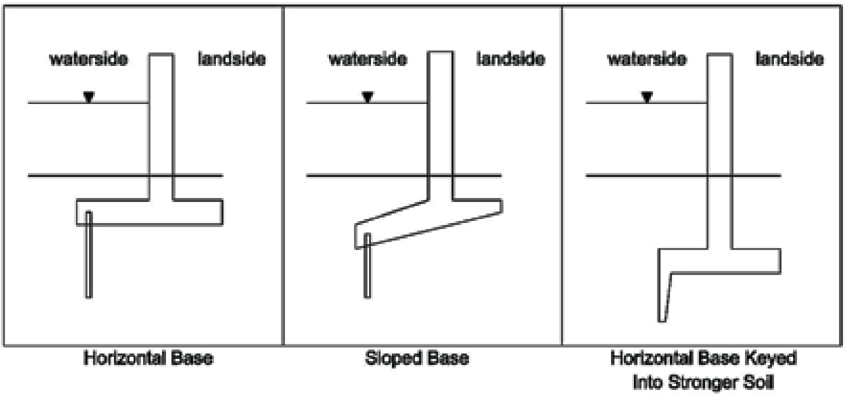


Figure 3.107 T-type cantilever walls

3.4.1.3 Dunes

Definition

Dunes are naturally occurring, non-rigid shore protection features created through the combined action of sand, wind and vegetation. Dunes often develop with an initial obstruction on the beach that lowers wind velocity, causing sand grains to deposit and accumulate. Often, sea levees are constructed in locations where dunes originally stood, using the original material composition. Many coastal defence systems use configurations with both levees and seawalls in heavily populated areas and dunes in more sparsely populated regions.

Function within the defence system

Dunes serve as levees to reduce the risk of inundation to coastal property during heavy wave action and storm surges. However, unlike levees, dunes are normally designed as sacrificial features of beach fills and are expected to erode in response to high waves and water levels. This is an important functional and performance difference between dunes and levees. Dunes also provide a reservoir of sand to nourish eroding beaches.

Issues

Table 3.20 highlights the main issues for dunes.

Table 3.20 Technical issues for dunes

Issues	Methods
Severe storm energy through wave action and high water levels may destroy dunes	Certain types of vegetation can be established to help retain dune sands, also detached breakwaters can be used to help re-establish dune sands lost during high water/storm events.
Elimination of beach grasses may destroy dunes	Periodic inspection and re-establishment of dune vegetation can help to minimise degradation of dunes.
Need to manage erosion of dune system	Periodic inspections of dunes to identify erosion/vegetation loss with a remediation program can help to extend the lifespan of dunes.

Typical photo

Figure 3.108 shows a typical dune environment.

Figure 3.108 Beach dune, Siesta Key Beach, Florida, USA (courtesy St Louis District, USACE)

3.4.1.4 Gateway 'closure' structures

Definition

Closure structures are barriers constructed within the levee embankment or flood wall designed to temporarily contain a river or other body of water and to provide access during non-flood conditions. There are three main types of closure structures:

- **gated closures:** consist of either swing, hinged gates (single leaf, double leaf or multiple leaf) or rolling or trolley gates
- **assembled closures:** consist of:
  - assembled trusses with purlins supporting sheeting panels
  - pinned frames with purlins supporting sheeting panels
  - stop logs (with and without intermediate posts)
  - panels supported by purlins without intermediate supports
  - single piece bulkheads for pedestrian openings
- **sandbag closures:** consist of filling sacks with sand and placing them like bricks on top of one another to create a physical barrier.

The closure may be permanently in place or movable. Movable or assembled, the structures are normally left in the open position and must be closed before a flood event, providing a watertight seal. An impervious membrane of appropriate thickness may be used in conjunction with a temporary sandbag or stone closure to reduce seepage through the temporary structure.

Movable gates are often fabricated from rolled steel shapes and plates. Flood gates may be operated manually, mechanically or automatically where some slide vertically in slots and others are hinged along one side. Others have hinges on both sides of the opening, and then seal in the middle.

A stop log is constructed by having logs inserted on top of one another. Most stop logs are too large to be installed without some type of heavy equipment. Cranes, forklifts or trucks will be needed for installation of the stop logs. The availability of equipment should be taken into consideration during the development of operation and maintenance budgets and activities. Chapter 4 provides more information related to operations and maintenance of closure structures.

Function within the defence system

Closure structures are designed and constructed to temporarily contain a river or other body of water and to provide access during non-flood conditions.

Issues

Table 3.21 gives the main issues for closure structures.

Table 3.21 Technical issues for closure structures

Issues	Methods	Section ref
Managing transitions between flood gate and flood wall embankment	Inspect landside ground surface for indications of seepage/sand boils. Inspect soil structure interfaces for formation of voids/shrinkage. Use of non-shrink grout mixtures as an engineered fill transition zone at the soil structure interface to minimise risk of piping/shrinkage/ possible seepage conveyances.	4.14
Corrosion or damage of hinges	Properly store and maintain a protective coating system, such as paint, to prevent corrosion damage.	4.14
Deterioration or damage of seals or anchors	Repair by grouting/caulking damaged joint materials at seals, and possibly replace badly damaged joint materials.	4.14
Differential settlement/ movement/change in location of sill	Repair settlement by pressure grouting below sill, or fabricate new stop logs or other components to provide a functional and sealable closure.	4.14
Improper storage for removable gates and missing components	Store in well-organised bins, replace damaged parts, maintain a supply of spare parts, take regular inventories and maintain a list of all parts.	4.2, 4.14
Vandalism or damage	Consider installing bollards or other protective barriers to prevent vandalism or damage.	4.14
Stop logs not marked properly and do not fit together tightly (or fit at all) during an emergency	Ensure that components are properly marked and stored to allow proper sequence of installation, use weld beads to permanently inscribe part designations (beads remain if paint fails), and provide clear, simple checklists for order of assembly and materials.	4.2, 4.14
Obstruction by debris collecting in or near the gates	Regularly remove debris.	4.14
Flood gate hinges and seals rusted or improperly lubricated	Store in well-organised bins and lubricate small parts, including pins, bolts, nuts and washers.	4.14
Failure of humans to install/ operate the closure	Design swing-gate/sliding-gate closure structures where possible to stop need for human operation. Adequately train O&M staff where structures requiring human operation must be constructed.	4.2
Effective installation/operation requiring adequate flood warning	Adequately train O&M staff, and provide an O&M plan, indicating the flood stages at which closures must be erected, where the need for erection has to be specified for each structure closure stage.	4.2
Increased operation and maintenance costs	Strategically place closures to minimise the number installed along the entire system, for short- and long-term cost savings.	4.2

Note

These sketches are illustrations and not recommended or fully designed cross-sections. Layer thickness, slopes and dimensions must not be considered as realistic examples. Many other associations of components may be found in reality.

Typical sketches and photos

Figures 3.109 to 3.111 present typical sketches and pictures of closure systems.

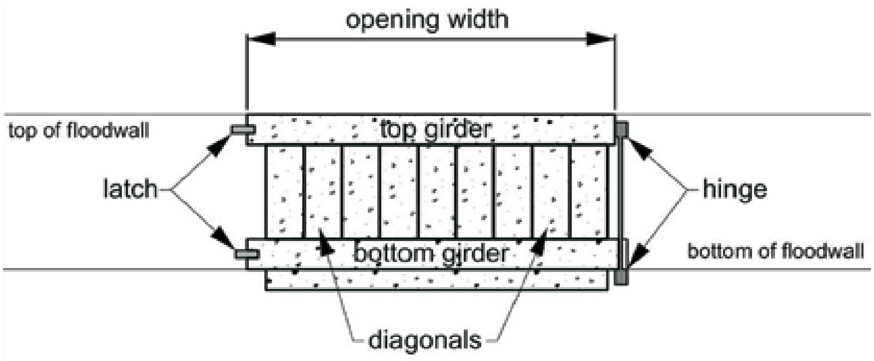


Figure 3.109 Closure structure

Figure 3.110 Slide closure (courtesy USACE)

Figure 3.111 Swing closure, St Louis, Missouri, USA (courtesy St Louis District, USACE)

3.4.1.5 Discharge pipes

Definition

The most common structures associated with levees are discharge pipes and their associated inlet/outlet structures (Figure 3.112a and b). A gravity pipe or drainage culvert is a structure used to channel water, typically beneath a road, railway or earthen embankment. They can be installed within the foundation or body of the levee embankment, are non-pressurised lines and typically use materials such as brick, vitrified clay, cast iron, corrugated metal, PVC, reinforced concrete or cast-in-place concrete. Drainage culverts can also be constructed as hybrid structures, consisting of more than one material property (eg corrugated metal pipe (CMP) with a concrete collar around the ends). Gravity pipes can be manufactured in a variety of shapes to include round, elliptical, flat-bottomed, pear-shaped and box. The relationship between flow rate and water level within the conduit will vary, directly based on the pipe's cross-section. The cross-section selected is based on drainage needs at different waterside and landside water elevations. Sizes of pipes also vary, depending on the site characteristics and hydraulic requirements. In some cases gravity flow is carried through levees by box culverts, which are typically constructed of cast-in-place concrete or pre-cast sections joined together.

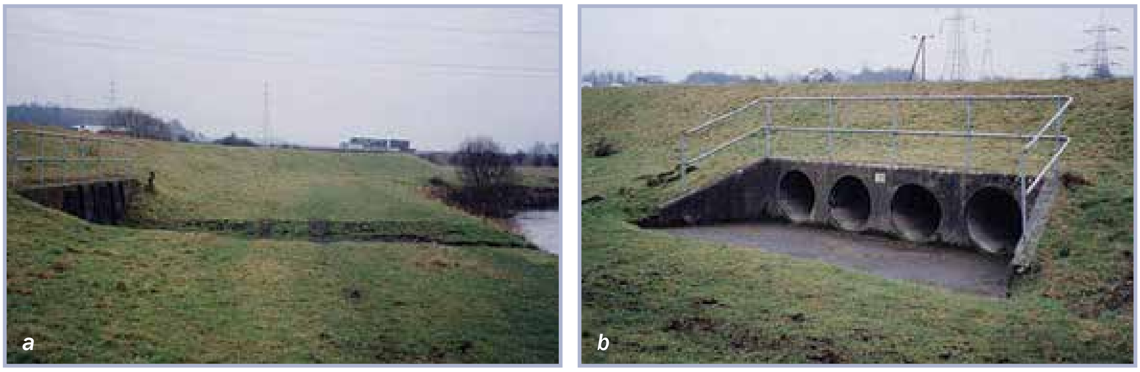


Figure 3.112 Inward (a) and outward (b) face of outfall structure (courtesy Defra)

Pumping station discharge lines typically convey stormwater that is pumped from the landside to the waterside of the levee to prevent interior flooding and potential damage to structures. Two types of systems are used, over the levee either with or without siphon recovery and through the line of protection. Alternative studies of different types of discharge may be required to select the one that is best when considering the layout of the station, site requirements and the choice of pumping equipment.

Pumping station discharge pipes are low-pressure lines typically made of steel or ductile iron and there are high-pressure pumping station intake pipes often made of pre-cast concrete sections. The discharge pipes may vary in size from 20 cm (8 inches) to 305 cm (120 inches) or larger. Pump discharge pipes are almost always circular. Backflow prevention should be accomplished by two means when the discharge is through the protection. First, discharge lines through the levee should terminate with a flap gate to prevent backflow. Secondly, in addition to the flap gate, provisions should be made for emergency shutoff valves, emergency gates or individual stop log slots to place bulkheads in case of flap gate failure.

Function within the defence system

Gravity and pumping station discharge pipes are important to the performance of the flood defence system because they ensure the controlled passage of water through a levee. Gravity lines provide a means for evacuating rainfall collected within the leveed area by passing it through the levee system to the riverside, except when the river is in a flood condition and gravity lines are closed to prevent backflow. Pumping station discharge lines are used during high river levels when the gravity lines are closed to overcome waterside hydraulic heads. This is when landside water levels are not sufficient to help flow through gravity drains by overcoming the hydraulic heads applied at the waterside. These discharge lines convey water from pumps to the receiving body of water, typically a river for inland defence systems.

Issues

Table 3.22 gives the main issues for discharge pipes.

Table 3.22 Technical issues for discharge pipes

Issues	Methods	Section ref
Seepage/leakage and piping along the soil/concrete structure interface or into the pipe, which can create preferential seepage routes leading to subsidence or collapse of the surrounding embankment	Install a clay or concrete surround to pipe culverts over their entire length with clay or concrete keys constructed within the embankment and its foundation.	4.15, 5.4.2, 9.15.4
Inadequate erosion protection to the base slab and wing walls from floodwater or overflow discharges, leading to undercutting and settlement or collapse of the surrounding embankment	Concrete keys or cut-offs provided at the outer edge of outfall base slab and wing walls.	4.15, 5.4.2, 9.15.4

Table 3.22 Technical issues for discharge pipes (contd)

Inlet/outlet structures provide focal points for recreational users leading to surface erosion of the embankment/structure interface	Construction of barricades/rails/fences to keep recreational users out of restricted areas, co-ordination with facility security staff to prevent recreational users from entering these areas.	5.4.2
Poorly designed inlet/outlet structures cause interference with flood flows leading to increased turbulence and erosive forces at the embankment/structure interface	Use of high-strength non-shrink/expansive grout mixtures at soil structure interfaces to prevent piping/erosion due to turbulence, also use of revetments/ACMs to reduce wave velocity and minimise surficial erosion due to wave action.	4.14, 9.15.4
Inadequate compaction of embankment near to the culvert and/or pipe collars may cause subsidence and an entry path for water	Cementitious flowable fill or non-shrink cementitious bentonite grout mixtures may be used because these materials can be placed without the need to compact, unlike earthen backfill material; take extra care during construction of the surrounding embankment material to ensure that specified fill and compaction requirements are achieved.	4.14, 9.15.4
Deterioration of the culvert, possibly resulting in eventual collapse of the pipe and a breach in the levee (Figure 3.113)	A realistic projection of the pipe's design life and future plan for replacement or rehabilitation.	4.15, 5.4.2

Figure 3.113 Culvert failure in Missouri (courtesy USACE)

Typical sketches and photos

Figure 3.114 shows pumping station discharge pipes that penetrate the levee. A culvert passes through the base of the levee for use during low river stages. During flood events, the sluice gate is closed and the pressure discharge pipes passing up and over the levee are used.

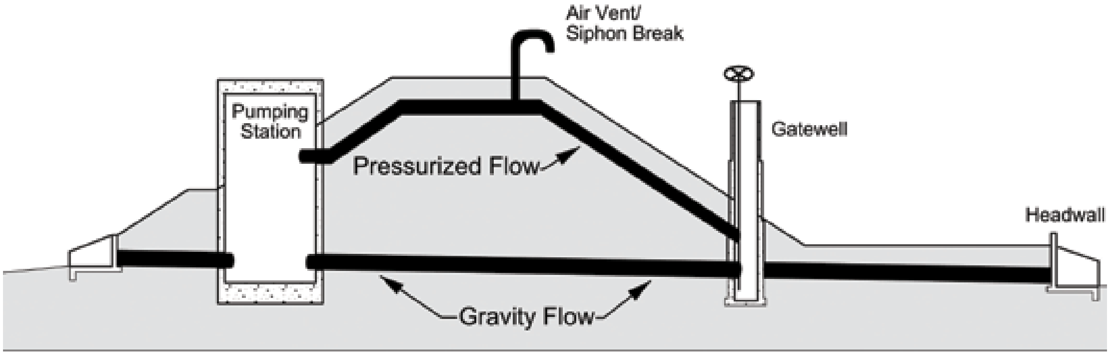


Figure 3.114 Pumping station pipe configuration

Figure 3.115 shows a release of water from a pumping station discharge pipe, part of a multiple discharge pipe configuration. Note that flap gates on non-producing pipes are shut to prevent backflow.

Figure 3.115 Release from pumping station discharge pipe (courtesy Paducah Kentucky Levee System, Louisville District, USACE)

Figure 3.116 is a typical sketch of a gravity pipe through a levee.

Note

This sketch is an illustration and not a recommended or fully designed cross-section. Layer thickness, slopes and dimensions must not be considered as realistic examples. Many other associations of components may be found in reality.

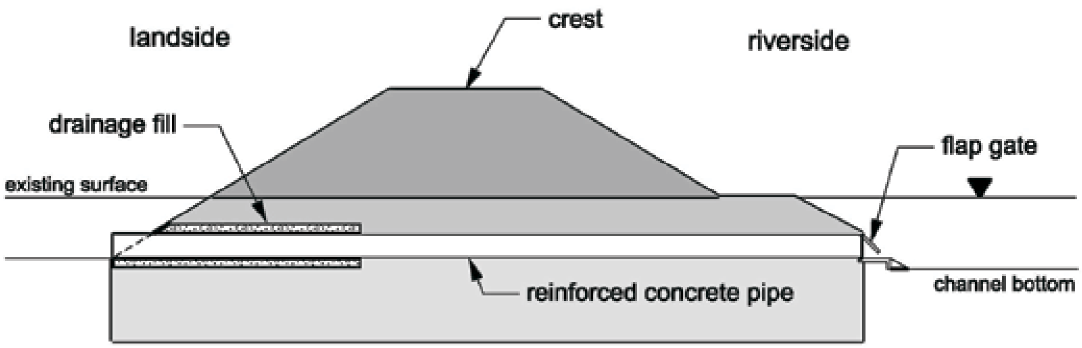


Figure 3.116 Gravity pipe through a levee

3.4.1.6 Gatewells, sluice/slide gates and tide/flap gates ‘check valves’

Definition

A gatewell is a reinforced concrete structure constructed within a levee embankment, which houses either a sluice gate or flap flood gate for closure of a gravity drainage pipe.

A sluice/slide gate refers to a structure that allows water to flow beneath it, consisting of a gate that slides on a fixed frame, used to prevent backflow into the leveed area by closing off a pipe/conduit (Figure 3.117). When the sluice gate is fully lowered, water may spill over, in which case the gate then operates as a weir. It allows interior drainage to flow from the leveed area out through the flood defence and keeps floodwaters from inundating the leveed area when water levels are high. The operating device for the sluice gate is generally installed on the top surface of the gatewell and drives the sluice gate up or down. These components may be controlled by either manual or automatic control systems, and the driving mechanism (actuator) may be strictly mechanically powered, electro-mechanically powered or hydraulically powered. A gatewell generally includes a ladder on the outside for access and a ladder on the inside to allow for inspection of the sluice gate. Sluice/slide gates can be square, circular or rectangular.

A top-hinged or side gate ‘check valve’ is a structure built into the flood defence system that lets the drainage water flow out into the water body at low tide and prevents the water from coming back in at high tide. Levees constructed to keep tidal waters from flowing back into the local stormwater drainage system may have pipes penetrating through the levee with tidal flap valves on the waterside of each pipe to prevent backflow. Commercially produced flap gates/valves are typically cast iron, stainless steel, composite (glass fibre encapsulating mild steel), wooden or neoprene rubber.

Figure 3.118 reflects a flap gate mounted to a headwall at the outlet of a gravity or pumping station discharge pipe. The flap gate is chained in the open position to accommodate a low river stage. Staff must be prepared to close this gate as floodwaters approach (also Section 4.2.1).

Figure 3.117

Slide gate (courtesy USACE)

Figure 3.118

Flap gate (courtesy USACE)

Function within the defence system

A tide gate or flap gate is constructed to keep water fluctuations from flowing through a flood defence, keeping water out of the leveed area by closing when the water pressure differential is high enough to hold the gate closed.

A sluice/slide gate is used to prevent backflow into the leveed area by closing off a pipe/conduit. The slide gate shown in Figure 3.121 is situated on the waterside of the levee. The pipes are submerged and the gates are in the open position. Crews must ensure that the gates are not blocked by debris when they are shut to prevent flow.

Figure 3.119 is a gatewell structure on the waterside of the levee that contains a sluice gate (Figure 3.121). The top of the gatewell is at the same elevation as the top of the levee to provide access at all river stages.

Figure 3.119

Slide gate mounted to headwall (courtesy USACE)

Figure 3.120

Gatewell structure (courtesy USACE)

Figure 3.121 Sluice gate within a gatewell structure (courtesy Covington Kentucky Levee System, Louisville District, USACE)

Issues

Table 3.23 gives the main issues for gatewells, sluice/slide gates and tide/flap gates.

Table 3.23 Technical issues for gatewells and gates

Issues	Methods	Section ref
Failure of manual operation of gate settings allowing floodwaters to reverse flow and inundate leveed areas	A qualified engineer to develop an operation and maintenance plan, including specified gate settings for sluice/slide gates as a function of flood stage.	4.2
Deterioration of structural components/tracks for slide gates	Periodic inspections to ensure that tracks are not corroding/deteriorating, replacement of tracks when excessive deterioration is observed.	4.2

Table 3.23 Technical issues for gatewells and gates (contd)

Accumulation of debris blocking the gate outlet points	Periodic inspections to ensure debris has not collected against gate, debris removal when necessary.	4.2
Increased O&M costs	Municipal tax increases by local sponsor, automation of gate components and use of remote instrumentation/monitoring methods where possible (powered sluice/slide gates operated manually or automatically, remote gate level sensors, remotely operated video cameras for monitoring debris collection at flap gates).	4.2
For coastal check valves there may be challenges with the fluctuating tide where the tide may rise and fall several feet each day. At high tide the entire outfall side of the check valve structure may be under water, allowing only a short time period during tide to perform maintenance	Adequately trained O&M staff. Training should include identification of flood side coastal levee maintenance issues, pre-specified plans for remediation of varying maintenance issues that may be encountered in the field, and safety training to ensure O&M staff are aware of and prepared for potential safety risks due to rapidly rising water levels.	4.2

Typical sketches

Figure 3.122 shows typical sketches of sluice gates.

Note

These sketches are illustrations and not recommended or fully designed cross-sections. Layer thickness, slopes and dimensions must not be considered as realistic examples. Many other associations of components may be found in reality.

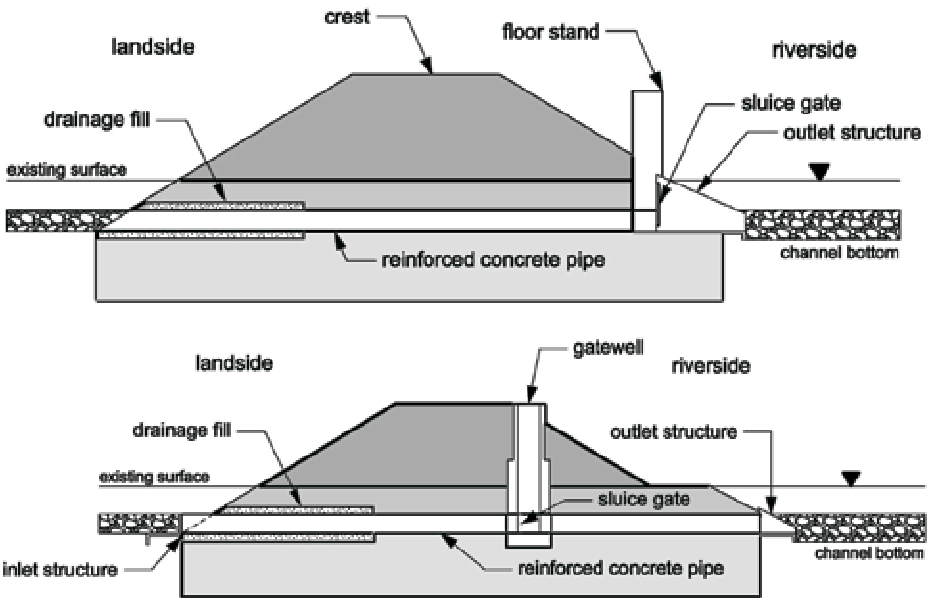


Figure 3.122 Sluice gate configurations

3.4.1.7 Surge barrier

Definition

These structures are similar to levees, except their function is to prevent a storm surge from entering the onshore area and to keep the storm surge out of the continental waterway. The surge barrier may need to include a structure that remains open during the time that a storm does not present a danger, but can be closed to prevent the surge from entering. As with levees and flood walls, consideration should be given to the effects of a storm larger than the one that the design is based upon, such as wave and surge overtopping.

Function within the defence system

Surge barriers provide protection against storm surges of high magnitude with short duration. Alternatively, they may function as a controlled spillway if components include mechanical sluice gates or other types of controlled gates structures. Surge barriers also serve as a wave attenuation barrier for coastal applications.

Issues

Table 3.24 highlights the main issues for a surge barrier.

Table 3.24 Technical issues for a surge barrier

Issues	Methods	Section ref
High initial cost of construction	Efficient design can reduce overall initial construction costs, and use of recycled concrete as engineered fill/large stone revetments can also help to reduce initial cost of construction.	10.1.2.4
Reliance on human operation for effective functionality	Adequately trained field staff should be able to recognise signs of degradation during periodic inspections to ensure that erosive damages do not compromise stability of structures.	4.2
Continual maintenance and inspection	Field staff should be adequately trained in recognising issues that may compromise stability of structures, as well as remedial measures for minimising scour and surficial erosion of jetties/groynes.	4.2, 4.3

Typical photo

In Figure 3.123, note the surge barrier tying to the flood wall in the foreground and the opening for navigation under construction in the foreground.

Figure 3.123 Surge barrier (courtesy R Robertson, USACE)

3.4.1.8 Seawalls, bulkheads and revetments

Definition

Seawalls, bulkheads and revetments protect land and structures from wave damage by preventing soil from sliding and being washed away, providing a barrier between land and water. These structures are composed of durable materials and may be vertical walls with sloping armoured features, or designed to turn back waves. They are most appropriate in locations where a gently sloping shoreline is not vital.

There are three basic design types of seawalls or bulkheads:

- thin, interlocking sheet piles driven deeply into the ground
- individual piles used to support an above-ground structure
- a massive gravity construction resting on the shore bottom or embedded slightly in it, supported by its own weight rather than by piling.

An apron of stones or alternative material that is heavy may be piled at the base of the wall to absorb wave energy and protect the underlying earth from eroding. Bulkheads and seawalls may be constructed of various materials including steel, timber or concrete piling, gabions or rubble-mound structures.

Bulkheads and seawalls are similar in design with slightly different purposes. Bulkheads are primarily soil-retaining structures designed to resist only minimal to moderate wave attack. Conversely, seawalls are principally structures designed to resist severe wave attack, but also may retain some soil to assist in resisting wave forces. Note that the sloping rock face leading up to the concrete wall in Figure 3.127a and the curved face of the wall in Figure 3.127b are designed to turn back waves.

Function within the defence system

Bulkheads act as retaining walls, keeping the earthen material behind them from collapse, and seawalls are primarily intended to resist minimal to moderate wave action. A constructed revetment armours the existing embankment by dissipating the energy of waves. Each of these structures helps to prevent erosion of property on the landside.

Issues

Table 3.25 gives the main issues for seawalls, bulkheads and revetments.

Table 3.25 *Technical issues for seawalls, bulkheads and revetments*

Issues	Methods	Section ref
Waterside beach material erosion may accelerate	Periodic inspection to identify excessive scour at waterside toe will help to minimise the costs of O&M repairs, also armouring of waterside toe similar to Figure 3.131.	4.13, 8.2.4
Overtopping of the bulkhead or seawall may result in landside erosion	Use landside resiliency and/or construction of a feature with freeboard.	7.4.1
Groundwater penetrations through the soil may induce pressure on the wall	Free-draining granular backfill materials placed against the face of the wall, possibly combined with a mechanical drainage system, can help to minimise active pressures against the face of a seawall.	9.7
Water flow around the sides of the structure may cause erosive damage	Use of properly designed multi-stage graded filters constructed at backfill material interfaces as well as armouring/revetments can help to resist erosive damage at edges of the structures.	9.6

Typical sketches and photos

Figures 3.124 to 3.127 present typical sketches and pictures of seawalls and bulkheads.

Note

These sketches are illustrations and not recommended or fully designed cross-sections. Layer thickness, slopes and dimensions must not be considered as realistic examples. Many other associations of components may be found in reality.

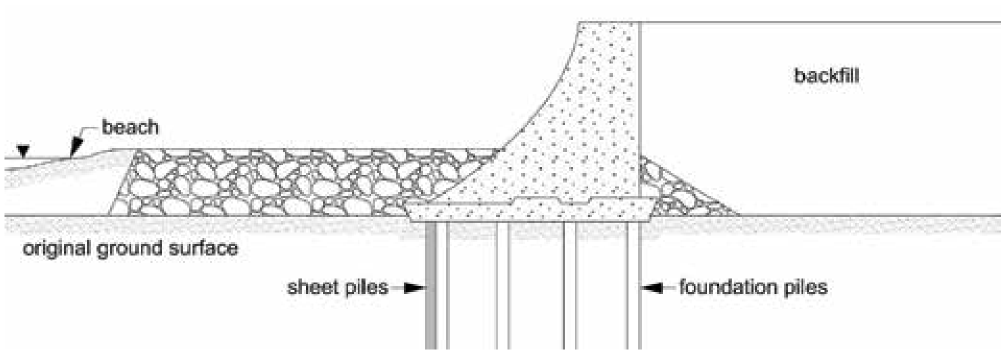


Figure 3.124   Seawall configuration

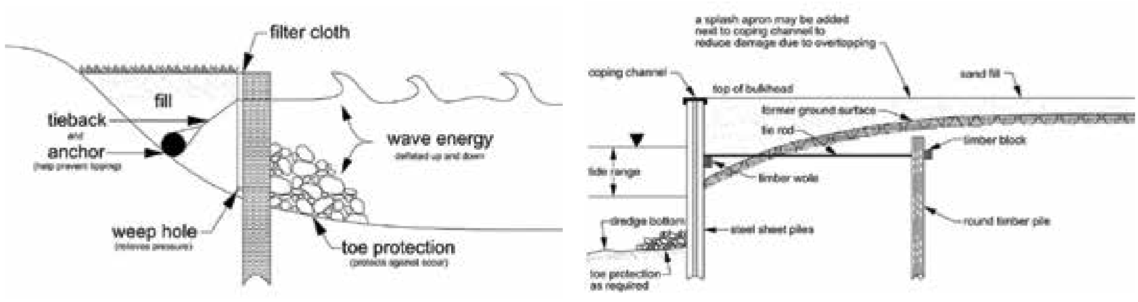


Figure 3.125   Bulkhead configurations

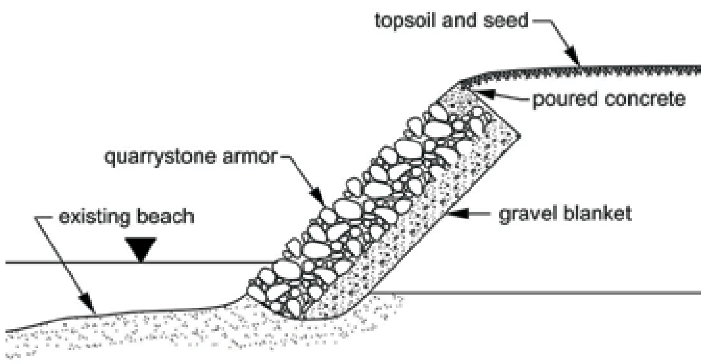


Figure 3.126   Stone revetment

Figure 3.127   Seawalls (courtesy R Robertson, USACE)

3.4.1.9 Protective beaches

Definition

Protective beaches make up the shoreline, which is the interface between the land and the sea. The shoreline is where the tides, winds and waves attack the land and where the land responds with various measures to dissipate the sea's energy.

The sediment on typical beaches ranges from fine sands to cobbles, with the size and character related to forces that the beach is exposed to and the material composition along the coastline. A significant portion of the beach sediment originates many miles inland where the weathering of mountains produces small rock fragments that are reduced to sand and gravel. When the sand and gravel reaches the coastal area, it is moved along the shore by waves and currents. Beach material may also be derived from erosion of nearby coastal beaches and dunes or by onshore movement of sediment from deeper water. In some regions, a sizeable fraction of beach material may be composed of marine shell fragments, coral reef fragments, cobbles or volcanic materials. Beaches may also be managed through nourishment; they are not always entirely natural in formation. The cross-section may be singularly or intermittently supplemented with extra sediment dredged from offshore sources (of similar grade) and the profile graded by mechanical equipment, which is often in conjunction with the addition of beach control structures (Sections 3.4.1.10 and 3.4.1.11). Such re-nourishment is often applied to areas of coastline that are important for tourism, critical for protection of economically viable areas or crucial to guard against the flooding of developed land due to storm surges.

Function within the defence system

Sloping beach sands serve as the outer line of defence in absorbing most wave energy, comprising a natural system of shore protection for coastal lowlands and development.

Issues

Table 3.26 gives the main issues for protective beaches.

Table 3.26 Technical issues for protective beaches

Issues	Methods	Section ref
Severe storms or long-term trends in erosion may affect beach volumes.	A plan can be instituted to set aside funds for a beach restoration programme.	8.2.4
Placement operations of beach sands may damage environmentally sensitive habitats.	Use of low ground pressure grading equipment can reduce disturbance of beach soils. Use of excavation/filling methods other than dredging will minimise waterborne sediments in environmentally sensitive habitats.	2.3

Typical sketch

Figure 3.128 shows the different environment and the characteristics of a beach profile.

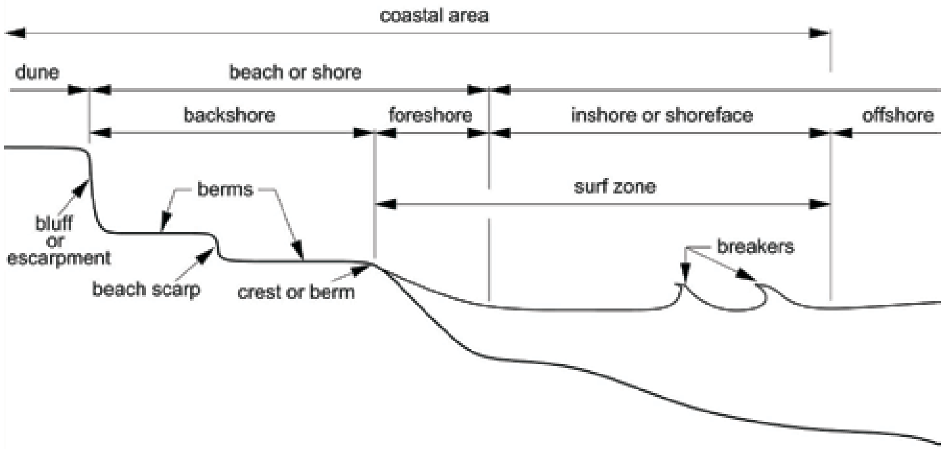


Figure 3.128 Beach profile

3.4.1.10 Jetties and detached breakwaters

Definition

Jetties are structures built at the mouths of rivers, estuaries or coastal inlets to stabilise the position and prevent or reduce shoaling of entrance channels. Jetties also help to protect the entrance channel from severe wave action and cross-currents.

Detached breakwaters are used as shore and coastal protection measures as depicted in Figure 3.1. Their primary function is to protect the harbour, water basin, or shoreline from destructive wave forces. These structures are usually composed of large rocks, or manufactured pieces, located at a prescribed distance offshore to reduce wave heights. They also promote deposition, which maintains a buffer of wetlands and forested areas, further reducing wave heights and other storm effects.

There are three types of breakwaters:

- **offshore:** typically constructed to provide shelter for an offshore ship wharf, and to minimise sedimentation and effects to the coastline. It is often placed outside the surf-zone to limit the effects on coastal morphology
- **coastal:** situated closer to the shoreline and helps to trap sand, which provides coastal erosion protection
- **beach:** located directly off the shoreline and helps to trap sand on the foreshore.

Function within the defence system

Detached breakwaters provide partial wave attenuation and assist with modifying littoral transport to improve shore restoration and coastal protection.

Issues

Table 3.27 gives the main issues for jetties and detached breakwaters.

Table 3.27 Technical issues for jetties and detached breakwaters

Issues	Methods	Section ref
Breakwaters only provide partial wave attenuation, and residual wave energy must still be addressed.	Erosion control measures can be strategically located between detached breakwaters to minimise erosive damage due to wave action and undertows. Use continuing periodic inspection and maintenance.	7.4.1
Wave overtopping of submerged or low breakwaters may cause more currents to be generated.	Landside toe of low breakwaters can be armoured with revetments or ACMs to limit erosive damages due to extra currents. Use continuing periodic inspection and maintenance.	7.4.1
Foreshore wave set-up is less within the leveed area than outside, generating local currents that may develop eddies from both sides.	Jetties and breakwaters can be armoured with revetments or ACMs to limit erosive damages due to eddy currents. Use continuing periodic inspection and maintenance of shoreline to identify scour areas and use remediation to minimise further erosive damages.	7.4.1
When longshore currents are disrupted due to breakwater, erosion near the head of the breakwater may develop.	Armour the waterside toe of that breakwater, and use continuing periodic inspection and maintenance of breakwaters to identify scour areas and remediation, and to minimise further erosive damages.	7.4.1

Typical sketch and photo

Figure 3.129 presents alternative positioning for breakwaters.

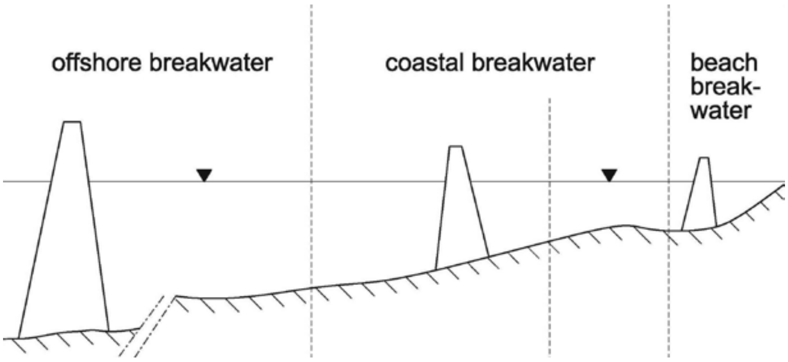


Figure 3.129 Alternative breakwaters

In Figure 3.130, note the deposition that has occurred shoreward of each breakwater. This deposition increases the stability of the land and the forested area, which serves to reduce the effect of storms as they approach the land.

Figure 3.130 Detached breakwaters in Les Saintes-Maries-de-la-Mer, Mediterranean sea, France (courtesy Symadrem)

3.4.1.11 Groynes

Definition

These structures are usually composed of large rocks or manufactured pieces with one end on shore and the other end extending offshore for a prescribed distance. This interrupts the littoral drift along the shoreline and promotes deposition, maintaining a stable shore. Though they may be constructed singly, typically groynes are built in a series along the shoreline for protection.

A wide variety of materials may be used in the construction of groynes. Impermeable groynes can be constructed of stone, asphalt or concrete, timber or steel sheet piles, or a combination of these. Permeable groynes may be built with sandbags, large stones or earthen material.

Function within the defence system

Groynes help to alter the longshore movement of sand by accumulating sand particles on the shore or retarding sand losses.

Issues

Table 3.28 gives the main issues for groynes.

Table 3.28 Technical issues for groynes

Issues	Methods	Section ref
Construction alters the equilibrium of physical processes.	Through comprehensive, holistic planning and strategic placement of groynes, adverse effects to physical processes can be minimised.	7.4.1
Groynes cause a change in beach profiles and the shoreline.	Perform comprehensive modelling beforehand to study likely effects of groynes.	7.4.1
Groynes interfere with on/offshore transport processes by displacing the position of longshore currents and rip currents.	Conduct full studies of indirect effects of groyne placement before construction.	7.4.1

Typical sketches and photos

Figures 3.131 and 3.132 are typical sketches of groynes.

Note

These sketches are illustrations and not recommended or fully designed cross-sections. Layer thickness, slopes and dimensions must not be considered as realistic examples. Many other associations of components may be found in reality.

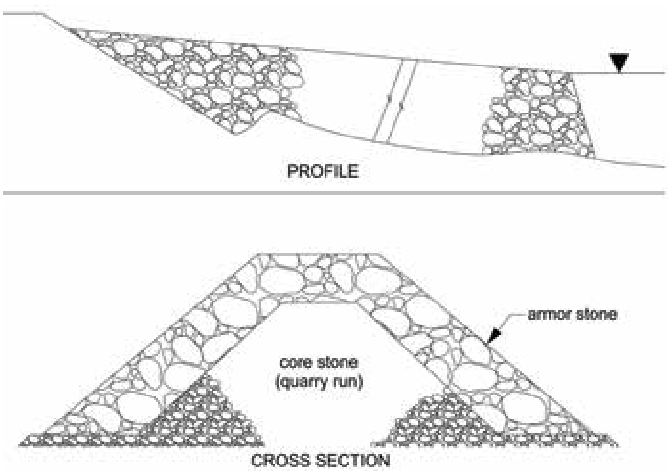


Figure 3.131 Groyne profile and cross-section

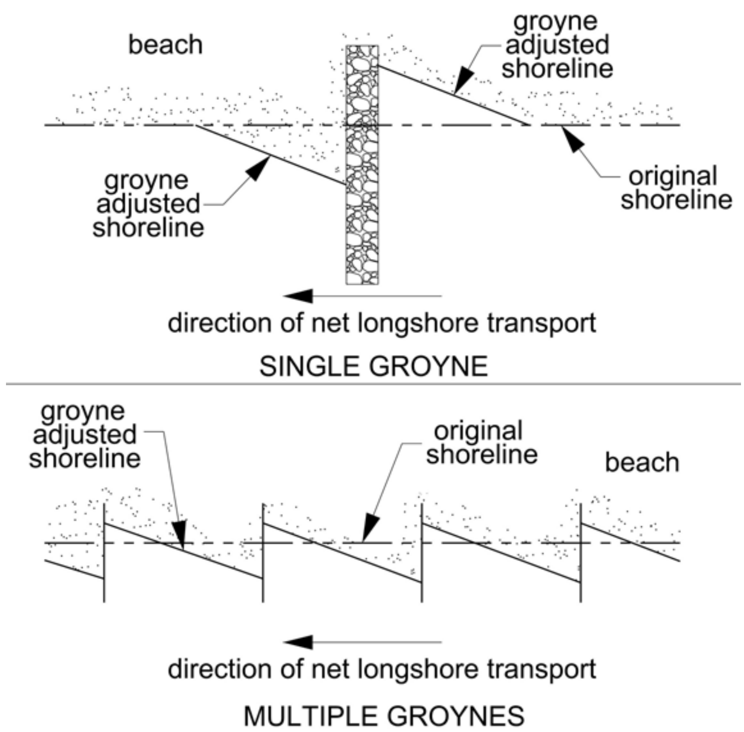


Figure 3.132 Single/multiple groyne(s)

Within the groyne image (Figure 3.133), notice that the deposition is in the down-drift direction. This deposition increases the stability of the land, reduces the effects of waves and provides recreational areas for the urban region.

Figure 3.133 Groynes – large-scale view (courtesy Symadrem)

Note

In Figure 3.134, which is a close-up view of a groyne, the deposition again is in the down-drift direction.

Figure 3.134 Local effect of a groyne (courtesy Symadrem)

Notice the shape and configuration of the individual pieces composing the coastal protection structure in Figure 3.135. These shapes, and numerous others, interlock with each other and provide a very stable structure. However, they are subject to breakage, which affects their interlocking capabilities.

Figure 3.135 Coastal structure (courtesy Jamie McVicker)

3.4.1.12 Air vents/air relief valves/siphon breakers

Definition

Air relief valves are normally installed in the top of each pump discharge pipe, on or near the riverside shoulder of the levee crest. Their function is to admit air and relieve the vacuum in the pipes to prevent the backflow of water when the pumps are stopped and the ends of the discharge pipes are completely submerged. So, the air relief valves are a siphon breaker to admit air into the pipe to prevent the backflow of water. These valves are automatic, relying on the pressure of the pumped water for closing. Also, some pump discharge pipes have simple air relief vents to perform the same function. Each discharge pipe has a separate relief valve (Figure 3.136).

Function within the defence system

Air relief valves complement discharge pipes, helping to prevent the backflow of water into the leveed area.

Issues

Table 3.29 highlights the main issues for air vents, air relief valves and siphon breakers.

Table 3.29 Technical issues for air vents, air relief valves and siphon breakers

Issues	Methods	Section ref
Continued maintenance of air vents is necessary to maintain proper functionality.	Down-hole inspection of air vent lines with down-hole cameras can be used to ensure that debris is not limiting the air flow and that the vent pipe is not deteriorating.	4.15

Typical photo

Figure 3.136 shows numerous air relief valves on the levee crest.

Figure 3.136 Air relief valves on levee crest (courtesy Paducah Kentucky Levee System, Louisville District, USACE)

3.4.1.13 Trash racks/screens

Definition and general consideration

A trash screen is typically a steel grate constructed at the upstream end of a pipe, to try to prevent material/debris from entering and potentially causing a blockage. The screen material is generally a mild structural steel that should have a protective coating system. Occasionally, stainless steel screens are also used.

A trash rack is a similar grated structure, installed at the entrance to a pumping station at the intake pipes, and is intended to stop debris from entering the pumping station and causing damage.

Function within the defence system

Trash racks and screens help to avoid foreign debris entering the drainage or pumping system and causing blockage.

Issues

Table 3.30 gives the main issues for trash racks and screens.

Table 3.30 Technical issues for trash racks and screens

Issues	Methods	Section ref
Continual maintenance is required to maintain functionality.	O&M staff should perform remote video camera monitoring to identify excessive debris collection. Use periodic maintenance to remove accumulated debris.	4.15
Excessive debris can cause structural damage.	Use periodic inspection and maintenance of trash racks/screens to ensure that accumulated debris is removed frequently enough. Weirs/breakwaters are constructed surrounding racks/screens to stop very large debris before it can damage structural components.	4.15

Typical photos

Figures 3.137 and 3.138 show typical trash screens and trash racks.

Figure 3.137

Trash screen on upstream pipe end (courtesy USACE)

Figure 3.138

Pumping station trash rack (courtesy USACE)

3.4.1.14 Pumping stations

Definition

A pumping station is a facility that houses pumps and other equipment for pumping fluids from one place to another (Figure 3.139 and 3.140). Pumping stations are used for a variety of infrastructure systems, such as the supply of water to canals, the drainage of low-lying land and the removal of sewage to processing sites. They vary in size, depending on the amount of expected flows from interior drainage channels and surface areas that drain to the pump.

Small pumping stations can be very simple, and just have an electric pump that will discharge small volumes where the leakage or runoff is a concern but does not accumulate quickly. Large pumps will handle drainage and seepage from a network of drainage channels that can be very high volume.

Figure 3.139

Pumping station (courtesy St Louis District, USACE)

Figure 3.140

Interior of a pumping station (courtesy St Louis District, USACE)

Function within the defence system

To effectively assist the drainage of floodwater within the leveed area behind a flood defence system, pumping stations are constructed and put into operation. Pumping stations may be used in conjunction with other features designed to remove runoff from low lying areas, for instance drainage ditches. When the land is below sea level it is necessary to pump excess water from the region into a desired location, which in some cases may be a detention basin, river channel or sea.

Pumping stations may be used to remove water from interior storm runoff, seepage water from the river or sea that has found its way into the low-lying area because of leaks within the flood defence system or flooding.

Issues

Table 3.31 highlights the main issues for pumping stations.

Table 3.31 *Technical issues for pumping stations*

Issues	Methods	Section ref
Mechanical/electrical systems – safety hazards	Design of electrical systems to account for components placed underwater, all shock hazards clearly labelled, all moving mechanical parts enclosed.	
Excessive pump line pressures and temperatures	Hydraulic pump lines possibly installed in conduit as a secondary safety measure, periodic inspection of hydraulic lines for deterioration, use of three- or four-way hydraulic spool valves in hydraulic circuits.	
Pumping capacity	Instrumentation of pump flow rates to identify any changes in pumping capacity, periodic inspection of water intake lines with down-hole cameras.	
Back-up diesel operability/ lubrication	Period inspection of back-up diesel motor, timely remediation of observed issues or replacement of motor.	
Building structure deterioration	Settlement monitoring and periodic inspection of structural components.	
Building structure – flood-proofing to minimise damages	Periodic inspection of outer/inner walls for indications of water damage or siltation through joints/cracks, sealing of deteriorated structural components with grout/caulk.	2.2
Intake pump operation, maintenance and inspection	O&M staff adequately trained and familiar with pump operating parameters, staff inspecting intake during low water periods by down-hole camera inspection, any debris/siltation in intakes removed quickly.	4.1.4
Debris/silt accumulation within pipes	Down-hole camera inspections completed periodically, and debris/silt accumulation cleaned out where observed.	4.15
Discharge point erosion	Placement of revetment stone or pre-cast articulated concrete mattresses at discharge point, construction of ramp weirs or other kinetic energy dissipation components at discharge point.	4.15

Typical sketch

Figure 3.141 is a schematic of a pumping station.

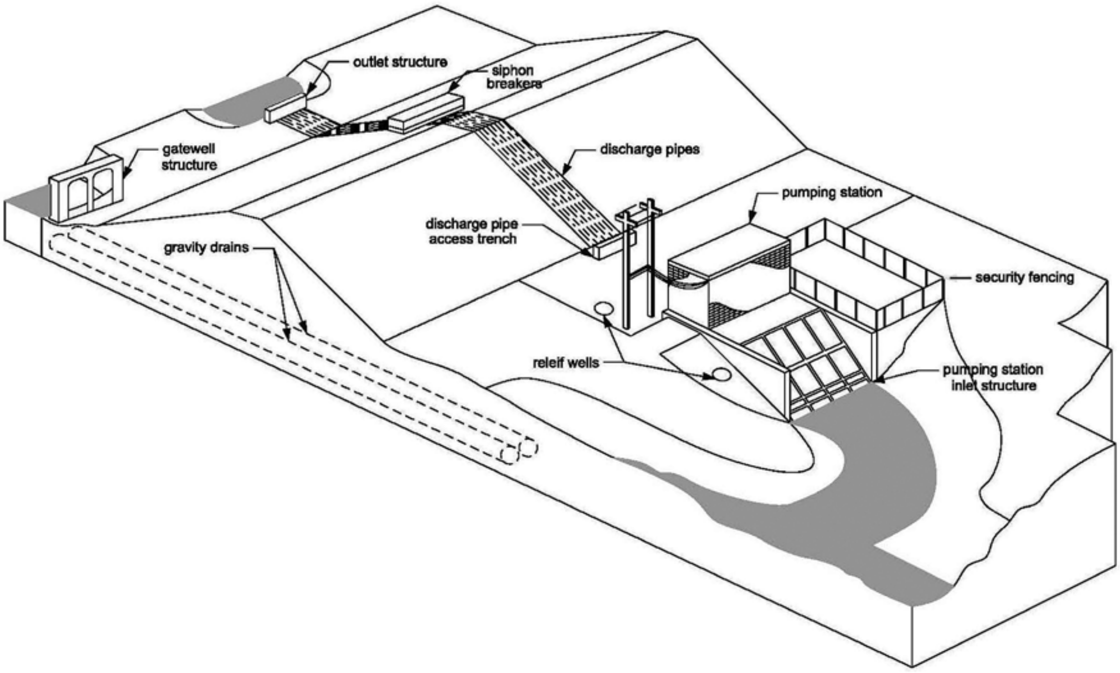


Figure 3.141 Pumping station schematic

3.4.2 Structures encroaching into levees

3.4.2.1 Penetrating structures

Definition and general consideration

This section focuses on pipes, conduits and lines that are **not** required for the proper operation of the flood defence system. It describes different types, uses and potential issues where each is either permitted or non-permitted (encroachments) by the levee sponsor or stakeholder for use by a third-party entity (ie owner/local entity/utility company) wishing to modify the levee by passing a pipe or line under, over or through it (Figure 3.142). The deterioration, improper installation and malfunction of these third-party pipes and lines can create potential consequences to the levee system.

Third-party pipes are predominately high-pressure or non-pressure pipes that are primarily used to convey water, natural gas, hazardous chemicals or sanitary sewage. Third-party lines are usually utility lines that are primarily used for the transmission of electricity or internet or phone service (often via fibre optics).

Utility pipes may be installed up and over a levee. However, any excavation must be backfilled with appropriate low-permeability material, and compaction under the pipe is critical. Lean concrete is often used for this purpose.

Figure 3.142 Utility line installation (courtesy Lawrenceburg Indiana Levee System, Louisville District, USACE)

Reasons for third-party pipes and lines passing through or over a levee embankment or flood wall

There are many reasons why a third-party entity might want to pass a pipe or line under, over or through a flood risk management structure (FRMS). For example, a park development on the riverside of the levee system may need water service and electrical power to support bathrooms or fire hydrants. The only route for an overland petroleum line to the neighbouring community may be through the flood defence system from the landward side and under the nearby river or stream.

Material constituents

Pipelines are typically made of steel, ductile iron, cast iron or plastic. The selection of material type (ie corrugated metal, concrete, cast iron, steel, clay) is largely dependent upon the substance it is to carry, its performance under the given loading, including expected deflections or settlement, and economy. Although economy must certainly be considered, the main factor must be safety.

Sizes and configurations

Pipe diameters range in size from a few inches to several feet. Electrical lines may be direct buried or protected by an encasing conduit made of PVC or polyethylene pipe. Fibre optic, internet cables and phone lines are generally sheathed within a protective outer pipe made of PVC or polyethylene.

Configuring combinations of pipes, casings and sheathes, and determining the location of manholes, hand-holes and pull boxes for the various utilities can be complex. For this reason, all configurations require that a plan be developed for flood-proofing the interface between the carrier pipe and the casing, and for the annular space within a sheath around fibre optic, cable television, internet or phone lines. Also, if a manhole, pull box or hand-hole is located on the flood side of the FRMS, floodwater under pressure can easily enter these boxes and structures. For rigid pipes within protective outer casings, commercially produced annular seals are available that can effectively seal the annular space.

Loadings and strength

Earth and live loads acting on a pipe should be considered where live loads may be imposed from equipment during construction and the loads from traffic and maintenance equipment after the levee is completed. Typically, pipe manufacturers have recommended procedures for accounting for such live loads.

Required strengths for standard commercially available pipe should be determined by the methods recommended by the respective pipe manufacturers. The design life of a pipe is the length of time it will be in service without requiring repairs. The term does not imply that the pipe will fail at the end of that time. Normally, a design life of 50 years can be economically justified.

Connections

Leakage from, or infiltration into, any pipe crossing over, through or beneath a levee must be prevented. So the pipe joints, as well as the actual pipe, must be watertight. For pipes located within or beneath the embankment, the expected settlement and outward movement of the soil mass must be considered. Where considerable settlement is likely to occur, the pipe should be cambered. Generally, flexible corrugated metal pipes are preferable for gravity lines where considerable settlement is expected. Corrugated metal pipe sections should be joined by exterior coupling bands with a gasket to assure watertightness. Where a concrete pipe is required, and considerable settlement is anticipated, a pressure-type joint with concrete alignment collars should be used. The collars must be designed either to resist or accommodate differential movement without losing watertight integrity. Flowable fill or non-shrink grout mixtures placed as backfill against the collars can prevent damage due to live loads applied by the compacting effort of construction equipment if cohesive or granular soils are alternatively placed as backfill. Where settlement is not significant, pressure type joints capable of accommodating minor differential movement are sufficient. Cast iron and steel pipes should be fitted with flexible bolted joints. Steel pipe sections may be welded together to form a continuous conduit. All pressure pipes should be tested at the maximum anticipated pressure before they are covered and put into use.

During design, the potential for electrochemical or chemical reactions between the substratum materials or groundwater and construction materials should be determined. If it is determined that there will be a reaction, then the pipe and/or pipe couplings should be protected. The protective measures to be taken may include the use of cathodic protection, coating of the pipe or use of a corrosion-resistant pipe material.

Effects on a defence system or levee

Serious damage to levees can be caused by inadequately designed, maintained or constructed lines beneath or within levees. Each crossing should be evaluated for its potential damage that would negatively affect the integrity of the flood defence system and could ultimately lead to failure. The methods of installation should be understood by the designer to anticipate problems that may occur. Some of the principal inadequacies that are to be avoided or corrected are:

- pipes having inadequate strength to withstand loads of overlying fill or stresses applied by traffic
- pipe joints unable to accommodate movements resulting from foundation or fill settlement
- unsuitable backfill materials or inadequately compacted backfill
- high pressures from directional drilling that could result in hydro-fracturing the surrounding materials
- materials that have unstable behaviours being conveyed through the pipe (gas and other explosive substances).

When deciding if an existing pipe can remain in place under a new levee or must be rerouted over the levee, or if a new pipe should be laid through or over the levee, it is important to consider:

- the height of the levee
- the duration and frequency of high water stages against the levee
- the susceptibility to piping and settlement of levee and foundation soils
- the type of pipeline (low- or high-pressure line, or gravity drainage line)
- the structural adequacy of existing pipe and pipe joints, and the adequacy of the backfill compaction

- the feasibility of providing closure in the event of ruptured pressure lines, or in the event of failure of flap valves in gravity lines during high water
- the ease and frequency of required maintenance
- the cost of acceptable alternative systems
- possible consequences of piping or failure of the pipe
- previous experience with the owner in constructing and maintaining pipelines
- regional frost depths and cover thickness required over the pipe to safeguard from the effects of freezing.

Pipelines crossing through or under levees

It is preferable for all pipes to cross over a levee rather than penetrate the embankment or foundation materials. This is particularly true for pipes carrying gas or fluid under pressure. Before consideration is given to allowing a pressure pipe (and possibly other types of pipe) to extend through or beneath the levee, the pipe owner should provide an engineering study to support the request. The owner, regardless of the type of pipe, should show adequate capability to properly construct and maintain the pipe. Future maintenance of the pipe by the owner should be carefully evaluated. It may be necessary to form an agreement to the effect that should repairs to a pipe in the levee become necessary, the pipe will be abandoned, sealed and relocated over the levee.

In recent years, there have been significant advancements in methods by which a utility line may be directionally drilled beneath the levee foundation (Figures 3.143 and 3.144). To successfully accomplish a directional drilling operation the contractor must follow specific guidelines for installation procedures, equipment and materials.

Figure 3.143

Directional drilling operation (courtesy USACE)

Figure 3.144

Horizontal directional drilling rig (courtesy USACE)

Pipelines crossing over levees

Pipes must be properly designed and constructed to prevent:

- flotation if submerged
- scouring or erosion of the embankment slopes from leakage or currents
- damage from debris carried by currents etc.

In some areas, climatic conditions will require special design features.

Issues

Table 3.32 gives the main issues for penetrating structures.

Table 3.32 Technical issues for penetrating structures

Issues	Methods	Section ref
Subsidence due to pipe collapse or improper compaction	If pipe collapse has caused subsidence, the entire pipe or the entire damaged portion of the pipe should be removed and replaced. If improper compaction has caused settlement near to the pipe, removal, moisture conditioning of soils, and re-compaction of backfill soils should be performed.	4.9, 4.10, 4.15
Risks arising from abandonment procedures (ie grouting vs. complete removal and re-compaction of earthen embankment)	Clean and grout conduits proposed for abandonment in-place with a non-shrink (expansive) grout mixture, such as a cement-bentonite mixture, ensuring that vent pipes are placed such that entrapped air in the conduit can escape during grouting procedure.	4.15
During high water, seepage tending to concentrate along the outer surface of pipes resulting in piping of fill or foundation material	Use of non-shrink grout mixtures or flowable fill materials as backfill of annular space around conduit can minimise piping at soil-structure interfaces.	4.15
High water results in uplift pressures that cause buoyancy of structures	Inspection during high water events will help to identify this issue at an early stage.	4.15
Seepage of material from broken pipe within the embankment	Periodic field inspections will assist in the identification of trouble areas due to seepage.	4.15
Seepage of material from broken pipe crossing over the levee causing erosion	Early identification of leakage is critical for assessing the severity of the situation and carrying out a plan of remedial action.	4.15
Open pipe joints contributing to the loss of fill or foundation material	Field identification of subsidence during inspections is critical for reporting to establish a plan of action.	4.15

Typical sketch

Figure 3.145 is an example of a horizontal drilling path under a levee.

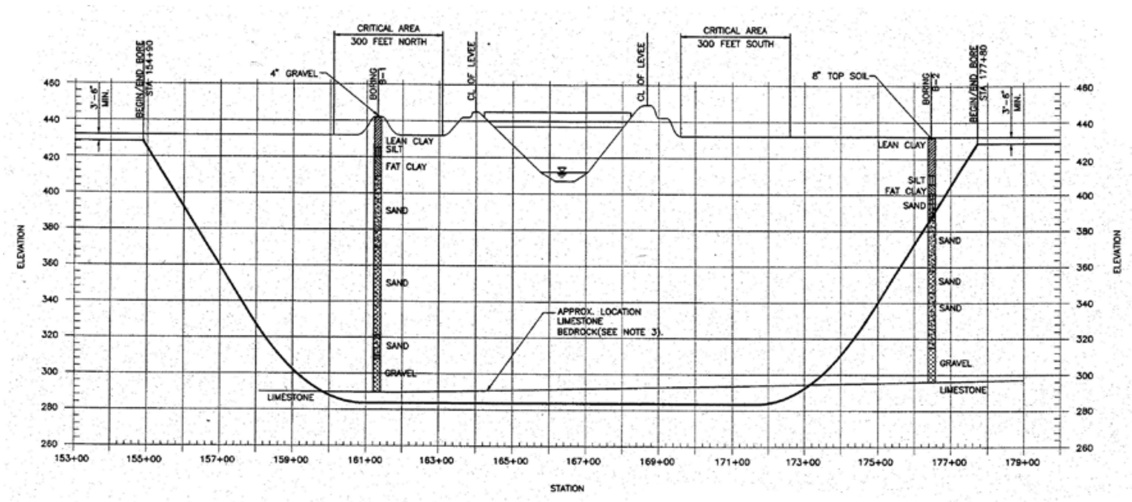


Figure 3.145 Horizontal directional drilling path

3.4.2.2 Buildings

Definition

Buildings sometimes form part of the defence line. This situation typically occurs with legacy levee systems constructed with limited funds, and inappropriate development can induce excess loads and/or affect the integrity of the levee. There are significant considerations with foundation materials and methods of construction to take into account with structures such as buildings serving as the flood defence.

Effects on a defence system or levee

A building, serving as a portion of the defence system, may impose issues related to the bearing capacity in saturated soils, the bearing pressure in relation to resistance to applied uplift pressures and the buoyancy of the structure as a single unit.

Also, there are complications related to flood-fighting around buildings (Chapter 6).

Issues

Table 3.33 lists the main issues for buildings.

Table 3.33 *Technical issues for buildings*

Issues	Methods	Section ref
Increased risk of seepage at soil–structure interface	Use of non-shrink grout mixtures or flowable fill materials as backfill of space between building and levee can minimise piping at soil–structure interfaces.	4.4
Complicated access for operation, maintenance, inspection and flood-fighting	Periodic inspections should be made to ensure that the integrity of the building has not been compromised, and records of any deficiencies noted, whether remediated or not, should be made available to flood fight teams.	4.2, 4.3
Foundation of the structure providing seepage paths	Construction of below-grade cut-off walls can help to minimise through-seepage in upper soil strata, including but not limited to sheet-pile cut-offs and impervious trench cut-offs.	9.7

Typical photo

Figure 3.146 shows a house situated in the levee slope.

Figure 3.146 *Residence situated in levee slope, River Loire, France (courtesy DREAL Centre)*

3.4.3 Transition zones

A flood risk reduction system may include levees and other structures such as walls, gates, sluices, dams or other innovative construction in which the geometric configuration of the continuous line of defence differs. Transitioning between different geometric configurations or material compositions within the line of defence creates a critical junction that is typically more vulnerable than a section of levee that has a consistent cross-section. Past experience shows that these transitional areas are at high risk for issues occurring because there is a discontinuity in shape and/or material constituents. This discontinuity may create a focal point for the concentration of flows, specifically at the joint or directly before or after the transition point. These localised areas are subject to erosion during high water events.

Numerous variations of transition types exist, such as:

- between the soil embankment and surface revetment
- between the levee and a flood defence structure constructed on the levee
- between the levee and a linear structure (eg pipes and culverts)
- between the levee and a non-linear structure (eg buildings, trees or bridge piers on the waterside slope).

Each variation influences the nature of problems that may be encountered. Figure 3.147 shows various transition types, their inherent problems and proposed solutions (Tourment *et al*, 2012). Some of these issues have already been considered in previous paragraphs. This section focuses on transitions between two levee segments (sections of a levee with differing exterior geometry or sections of a levee with differing exterior or interior material composition).

From a geometrical perspective, structure transition may be viewed as an external transition (ie a change in the geometry of the cross-section or form of the surface of a levee) or an internal transition (ie acting on the interface between different components within the cross-section of the levee). Partially embedded structures such as buildings have both an internal contact surface and an outside line of limit.

This section focuses on external transitions between two levee segments (sections of a levee with differing exterior geometry or sections of a levee with differing exterior or interior material composition). Issues related to internal transitions between the levee and structures on the levee or structures embedded inside the levee are discussed in Sections 3.4.1 and 3.4.2.

3.4.3.1 Transitions with other structures

Transitioning between different geometric configurations or material compositions within the line of defence creates a critical junction that is typically more vulnerable than a section of levee that has a consistent cross-section. This discontinuity may create a focal point for the concentration of flows, specifically at the joint, or directly before or after the transition point. Experience indicates these transitional areas are at high risk for issues to occur because there is a discontinuity in shape and/or material constituents. So, these localised areas are subjected to erosion during flood events.

Some examples of these types of transitions are presented in Figures 3.148 to 3.152.

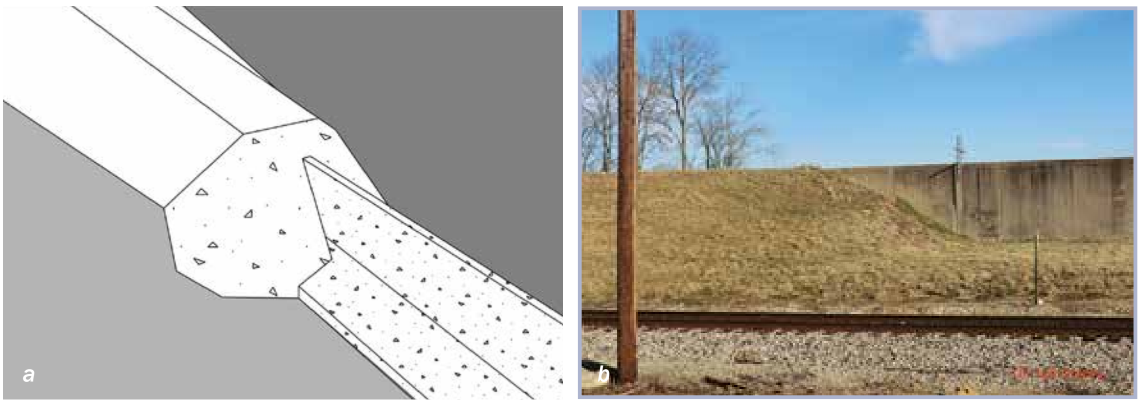


Figure 3.148 Schematic (a) and associated photo (b) of transition in terms of the geometry of the levee: change in cross-section between a levee and a flood wall, Cape Girardeau, Missouri, USA

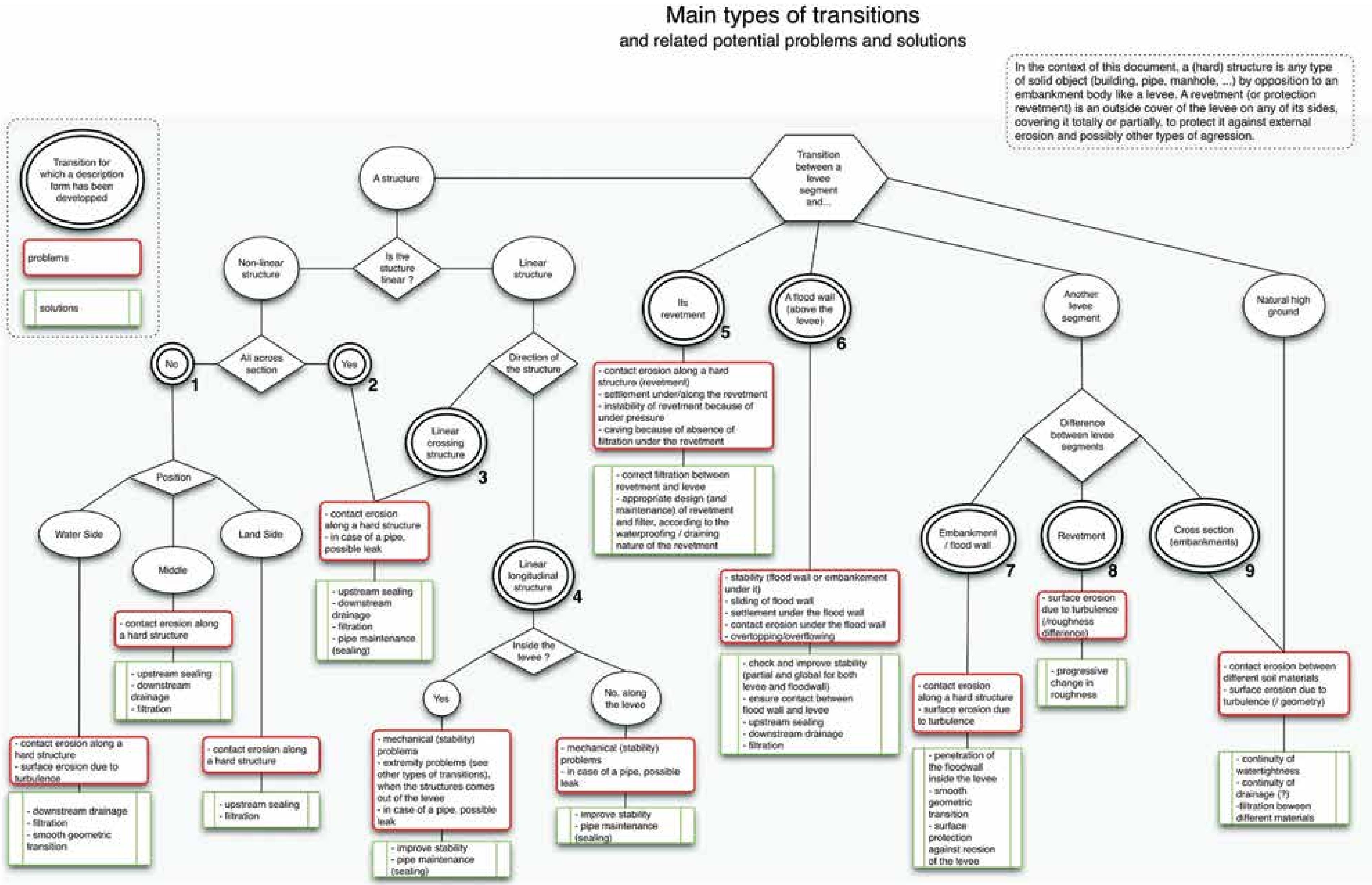


Figure 3.147 Primary transition types (R Tourment, Irstea)

Figure 3.149 Transition in terms of the geometry of the levee: change in cross-section between a levee and a flood wall, New Orleans, Louisiana, USA (courtesy USACE)

Figure 3.150 Transition in terms of the geometry: transition from levee in to hillside, Columbia, Illinois, USA (courtesy St Louis District, USACE)

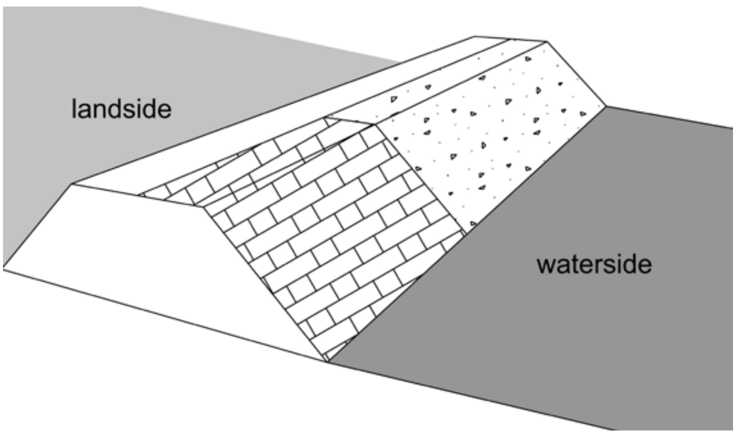


Figure 3.151 Transitions in terms of the surface of the levee: different surface revetments

Figure 3.152 Transition between gabion basket and stone revetments, Rhone River, France (courtesy Jean Maurin)

A transition may cause adverse effects on the flood defence system. A common issue with transitions of this type is erosion that occurs in four phases: initiation, continuation, progression and breach.

Issues

Table 3.34 gives the main issues for transitions due to differing material composition.

Table 3.34 Technical issues for transitions due to differing material composition

Issues	Methods	Section ref
Different material constituents where one segment possesses better resiliency than the other*	Waterside stability berm may provide extra impervious material at a natural transition.	4.16, 9.11
Varying geometric cross-section from levee to natural ground	Design and construct a progressive change, not an abrupt change in cross-sectional geometry.	4.16, 9.11, 10.5.5
Greater risk of seepage, which can occur at the interface between natural high ground and a levee and between soil and impermeable structures on the levee (flood walls)	Adequate subsurface exploration can be performed to determine the natural geology and associated seepage risks at interfaces between natural and manmade structures. Seepage measures such as drains, relief wells and cut-offs can help to reduce these risks.	7.5, 9.11

Note

* The nature of the embankment soil determines the levee's vulnerability to erosion. Two primary classes of soil are cohesive soils and granular, non-cohesive soils.

Typical photo

Figure 3.153 illustrates erosion at the transition of the cross-section of the flood defence: a transition from a (coastal) levee to a dune.

Figure 3.153 Transition erosion dune north of coastal levee Hondsbossche-en Pettemerzeewering, the Netherlands (courtesy H Van Hemert)

3.5 UNDERSTANDING FAILURE OF LEVEES

3.5.1 Defining the failure of a levee

3.5.1.1 What is a levee failure?

Definitions and concepts

A failure is defined as the inability to achieve a defined performance threshold (response to a given loading) (Morris, 2008) or performance indicator, for a given function. Failure is a state.

A levee system is generally composed of several elements that include both levee segments and other structures (walls, spillways, gates etc). At the system scale, the main function is flood risk reduction. So failure can be defined as the unintentional inundation of the leveed area. For the system to fail there can be either hydraulic or structural failure of any of its segments:

- **hydraulic failure (non-structural)** occurs if water ingress into the leveed area (by through-flow, overflow or overtopping of the levee) occurs before the planned protection level is reached and without prior damage to the system element
- **structural failure** occurs by a breach in the levee system that results from damages affecting at least one system segment.

Structural failure can induce hydraulic failure and vice versa (Figure 3.154).

For example, hydraulic failure can lead to structural failure if a levee segment is overtopped in an unplanned situation, leading to major erosion on the landward side of the levee and then to breach.

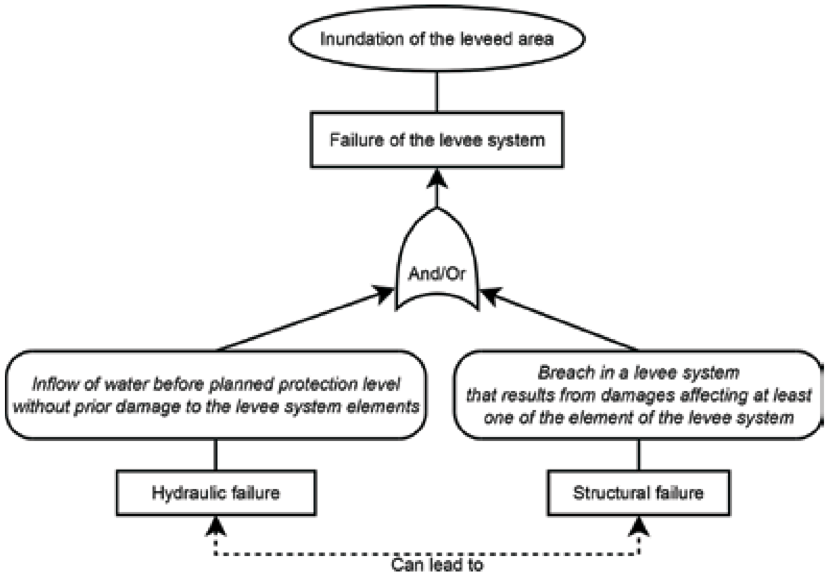


Figure 3.154 Levee system failure (courtesy R Tourment, Irstea and Y Deniaud, CETMEF)

The structural failure of a levee segment occurs when the weakness of one or more components of the levee reaches an incompatible ultimate state with its performance threshold. This weakness induces a failure of the component that is no longer able to achieve its function to ensure the integrity of the levee. This structural failure of a segment of the levee can result in failure of the hydraulic function of this segment of levee and in failure of the levee system through the appearance of a breach.

At the scale of a specific levee segment, hydraulic failure occurs when the levee segment is no longer able to achieve, at the defined or assigned level of design, its appropriate hydraulic function within the flood defence system. Hydraulic failures can result from either (Figure 3.155):

- an error in the design or construction process of the levee (resulting in through-flow, overflow or overtopping of the levee)
- modifications within the environment of the levee system (for instance a raise of the river bed or of the foreland, or a settlement under the levee, leading to an overflow for a lower return period than the design level)
- an operational failure (for example a gate that is not closed during an event, by human failure or poor maintenance, resulting in the impossibility of closing it)
- a breach resulting from a structural failure scenario.

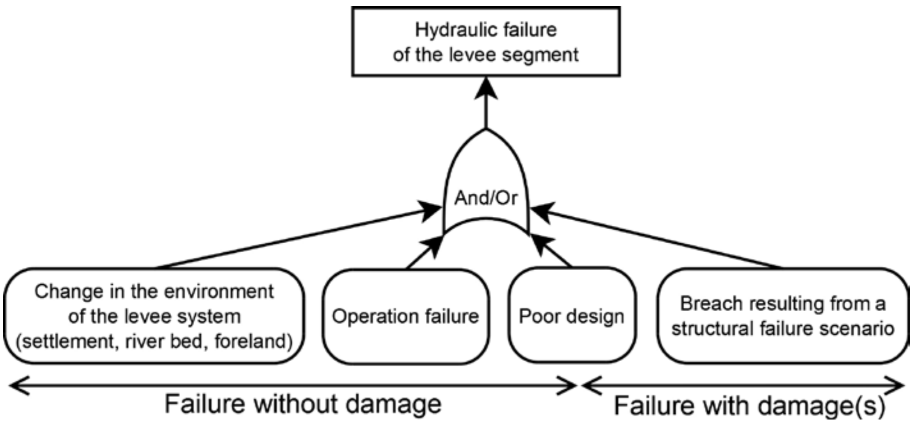


Figure 3.155 Possible sources of failure of a levee segment (courtesy R Tourment, Irstea)

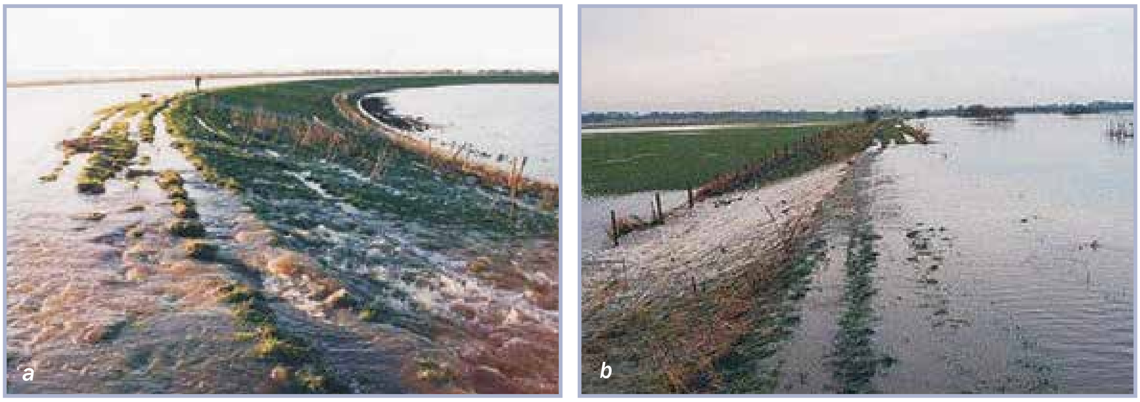


Figure 3.156 Localised overtopping (a) and overflowing (b) (courtesy Defra)

> **Note**
>
> **Overflowing of a levee (Figure 3.156) that has been designed to be resistant to overtopping/overflow, should not be considered as a failure if it occurs at the designed level.**

Deterioration and damage

Severe deterioration such as erosion, scour and slippage, can commonly affect a levee. Even if this deterioration does not result in a breach, the levee is still weakened. Damage resulting from deterioration processes is related to failure of structural components of a levee. As some structural functions of levee components can no longer be relied upon, the levee is no longer able to withstand another flood event, at its defined level of protection, without a major risk of breach. Immediate repair or emergency actions are required.

Damages affecting the waterside slope or toe of a levee can dramatically lower its assumed level of protection in case of a flood event (Figure 3.157).

Figure 3.157 Wave damage and rotational sliding (courtesy R Tourment, Irstea)

The presence of a permeable layer, or the existence of cracks or fine fissures, within the fill or the foundation soils, can trigger piping and seepage through a levee. In the long term, as deterioration evolves, the impermeability of the levee is no longer guaranteed. Piping and seepage can induce excessive internal erosion. This can lead to a washout of fine soil, a void and a settlement of the crest or the onset of piping as the flows increase with time. So the performance of the levee is weakened, and an uncontrolled major release of water can occur in the leveed area during a flood event (Figure 3.158).

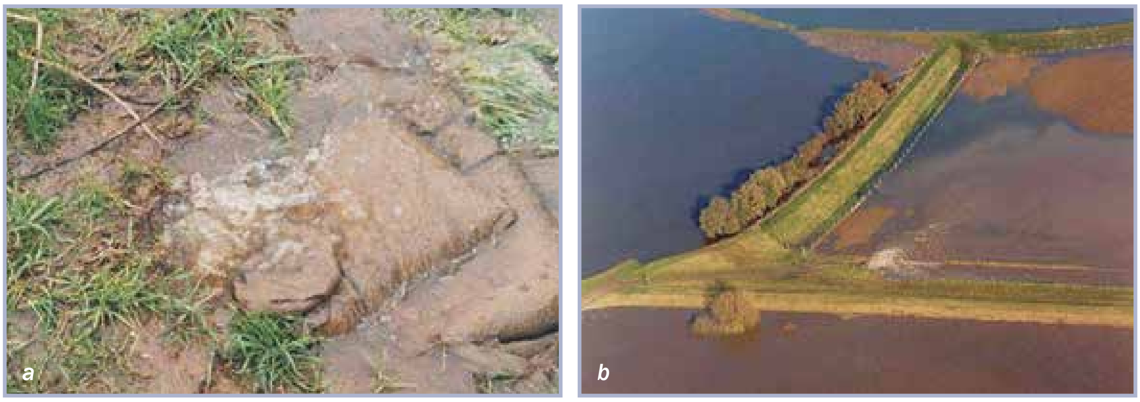


Figure 3.158 Seepage (a) and major release of water in leveed area (b) (courtesy Defra)

Erosion of the toe of the levee can be caused by wave action, propeller wash from vessels using the watercourse or high velocity flow on the outside of a meander. It reduces the width of the levee and can initiate slides and even a levee breach during a flood event (Figure 3.159).

Figure 3.159 Erosion by river flow and instability due to undercutting toe (courtesy J-Y Hardy, DDT37 and E Durand, CETE NC)

Breaches

Breaches are final final phases of erosion or any other deterioration or damage mechanism following a gross enlargement of piping, a slope instability or an overtopping due to settlement of the crest or due to formation of a sinkhole from a pipe in the embankment. A breach is a catastrophic collapse that results in significant loss of crest or the creation of a significant hole through the levee, causing a substantial uncontrolled loss of water. In these scenarios, the failure of one or more components of the levee leads to a sudden state of failure for the hydraulic function of the levee.

In Box 3.13, the presence of very weak surface layers of peats and clays in the foundation result in a horizontal block movement of the embankment when the undrained shear strength of the surface layer is insufficient to resist the hydraulic forces created by the floodwaters acting on the embankment.

Box 3.13 Example of instability caused by peats in levee

Levees are considered vulnerable to drought if the levee or its subsoil contains peat or highly organic clay in a zone/level below which the groundwater table may fall during drought. During long-term periods of drought, the peat in levees tends to dehydrate. Peat levees occur in the Netherlands and, for example, also in Ireland, the UK, France, the USA and Italy.

Dehydration of peat in levees causes shrinkage and a further loss of weight (to about 1.5 to 5 kN/m^3) causes reduced stability. This may ultimately result in breaching due to horizontal sliding (the levee being horizontally pushed aside).

An example of failure due to sliding is Wilnis (Figure 3.160), the Netherlands in 2003.

Figure 3.160 Translational sliding in Wilnis Levee (courtesy STOWA)

Major deterioration in the drainage, impermeability or stability functions of the levee's components can weaken the levees and lead to a sudden uncontrolled breach and to a major release of water. It is sometimes difficult to fully understand the mechanism of the breach after the event due to the effect of the water flows (Figure 3.161).

Figure 3.161 Breaches in Scott County, USA (courtesy USACE)

However, breach is not always a failure. Some levees in Germany are designed to be breached by explosion in the event of a flood while some levees on the Mississippi were recently breached by design. In the Mississippi River case, floodwater was diverted into designated floodways to relieve pressure on the downstream flood defence system and to reduce water levels downstream in the main river channel.

Links between forms, functions and failures

The performance of levees, and of the failures of levees, are related to their function in the flood defence system (Section 3.1).

Mechanisms of deterioration and damage are linked to the nature and structure of the components of each particular levee.

Failures of levees are directly related to forms and functions of levees, as depicted in Figure 3.162.

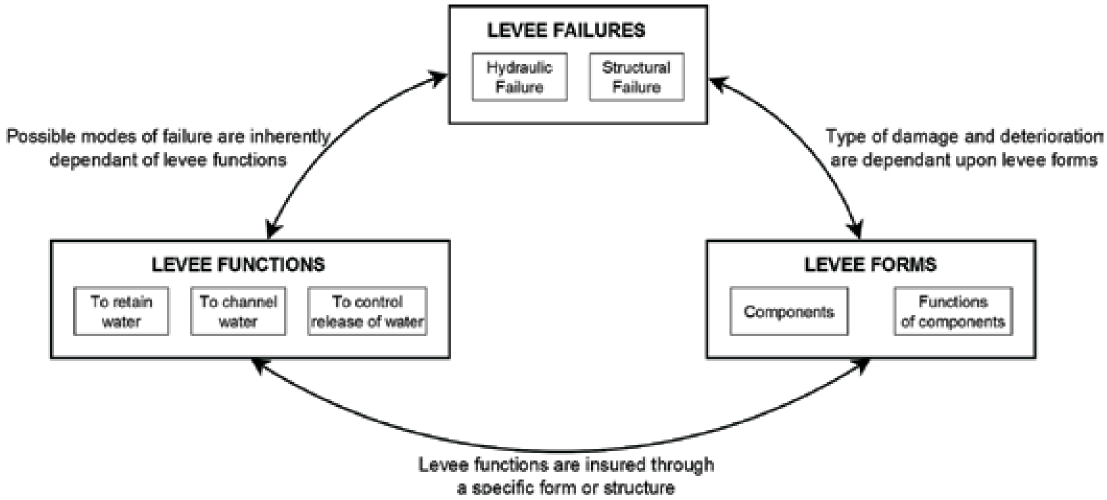


Figure 3.162 Relationship between functions, forms and failures of levees (courtesy Y Deniaud, CETMEF)

3.5.1.2 Understanding the process of failure

Mechanisms and failures

A structural failure scenario leads to a breach and consists of a process (traditionally called 'failure mode') which involves both physical and functional phenomena affecting the levee segment (Figure 3.163):

- in the physical domain, the components of a levee segment undergo mechanisms leading to deterioration and damage
- in the functional domain, the functions of a levee component can degrade to failure.

The relations between components and functions are multiple and not univocal.

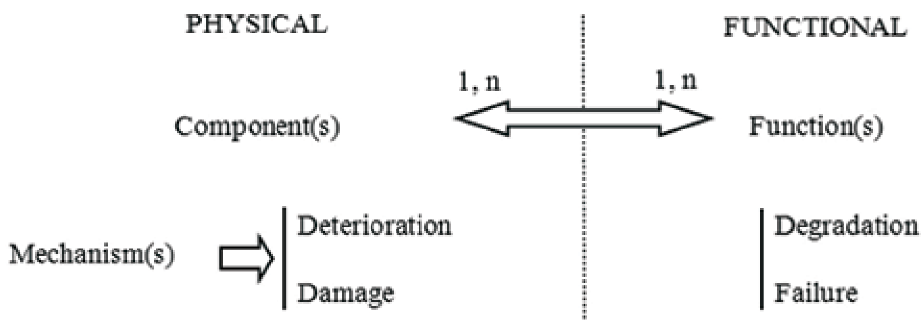


Figure 3.163 Physical and functional domains and vocabulary in the analysis of levee failure (courtesy R Tourment, Irstea)

A mechanism is a time-dependent process that affects a levee component and involves a decline in the state of its structural properties. A mechanism can result in the deterioration and damage of one or more physical levee components.

Deterioration is triggered by initiators and aggravated by factors that act on the levee components. Initiators and factors are physical or chemical agents such as wind, currents, wave, tides, temperature, vegetation, animal and human activities, loading and unloading.

The effects of deterioration are revealed by observable features or symptoms, such as cracks, fissures, depressions, bank caving, steep slopes, slides, slumps and lack of vegetation. However, there is no direct relationship between the symptoms and the mechanisms. One symptom can be linked to several mechanisms and one mechanism can lead to several observable features or symptoms.

Severe deterioration results in damages to the structural components of the levee that require immediate repair or emergency action. Deterioration and damages could also lead to a levee breach, which is a low crest situation or a significant hole. In this case, a breach is an ultimate degradation of the hydraulic function of a levee or a sudden levee failure, resulting from a quick collapse of the levee structure (Box 3.14).

A function assumed by a levee can be degraded or failed, depending of the threshold retained for the failure (Box 3.14). A levee can be in a state of failure before being completely destroyed.

Scenarios or chains of events

The combination of events affecting a particular levee depends on its form, components, associated structures and the loads and their evolution over time.

The appearance of a mechanism can be activated by a phenomenon initiator such as:

- an overload
- a high water level or a rapid change in water level
- seepage
- an abnormally raised flow of water.

Component deterioration or damage results in the degradation or failure of one or more functions, associated to the component(s). Degradation or failure of a function can then initiate or aggravate mechanisms, creating new elementary chains of events (Figure 3.166).

Scenarios leading to a structural failure of a levee can be complex. One mechanism of deterioration and damage acting on one component of a levee can trigger another mechanism in a 'domino effect' (Figure 3.166). Two different mechanisms can also develop in parallel.

Box 3.14 *Thresholds and kinematics of mechanisms and failure*

In the structural domains, the threshold between deterioration and damage (deterioration threshold) may be referred to a service limit state (SLS), and the threshold between damage and break (damage threshold) to an ultimate limit state (ULS).

Kinematic of mechanisms and the resulting consequences on the physical state of a component can be progressive (deterioration) or brutal (break) (Figure 3.164). Breaching of a levee during a flood event is a typically fast process of failure, although it is generally preceded by a phase of initiation, the duration of which can be important.

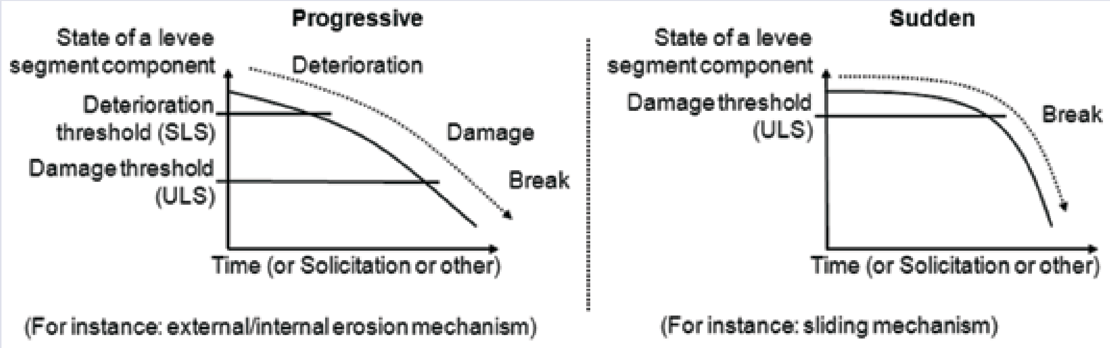


Figure 3.164 *Kinematics of a mechanism (courtesy R Tourment, Irstea and Y Deniaud, CETMEF)*

In the functional domain, the deterioration of one physical component can degrade the function that is supported by this component. The kinematics of the degradation can be illustrated by three main modes (Figure 3.165):

- **progressive:** from a good performance to a degraded performance and a failed function
- **sudden:** from a good performance to a failed function
- **binary:** the function is effective or failed.

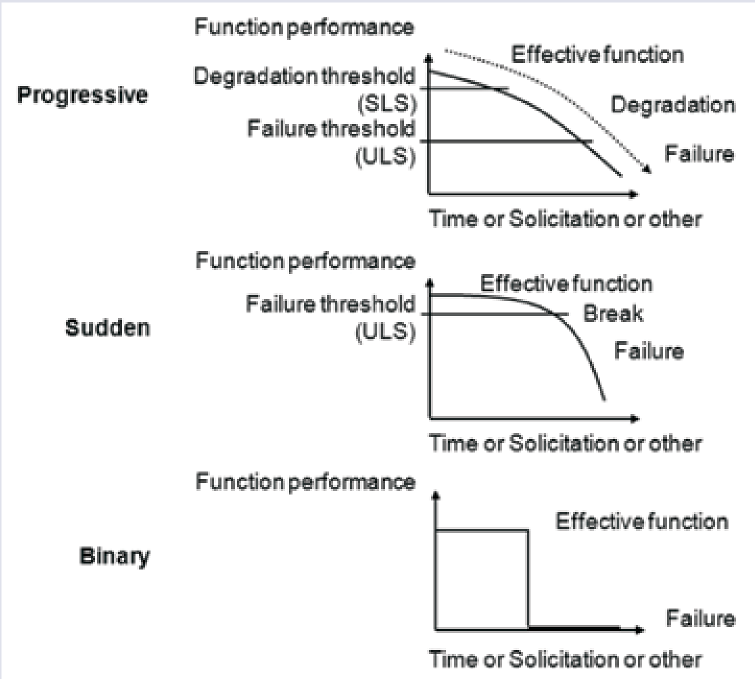


Figure 3.165 *Kinematics of functional degradation (courtesy R Tourment, Irstea and Y Deniaud, CETMEF)*

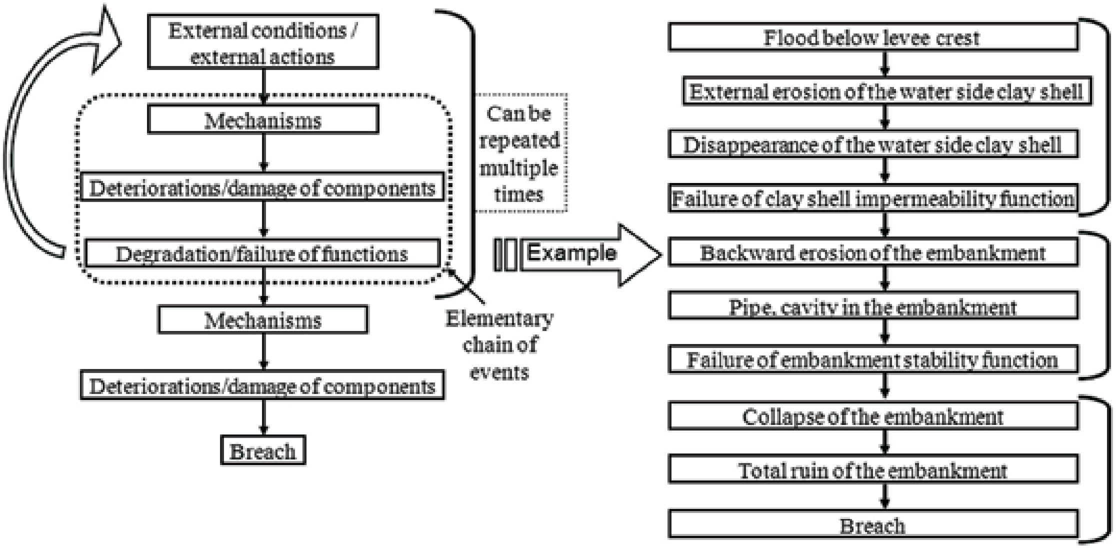


Figure 3.166 Scenario or chain of events leading to a structural levee failure and breach (courtesy R Tourment, Irstea)

3.5.2 Main processes of deterioration, damage and breach

3.5.2.1 External erosion

External erosion definition

External erosion is the wearing of a surface (bank, streambed, embankment or other surface) by floods, waves, wind or any other natural process (FEMA, 2004). External erosion is initiated by hydrodynamic forces acting on soil particles at the surface of a levee. It occurs when the surface material of the levee is not sufficiently resistant to the aggressions of the environment, that is, when the shear stress induced by flows exceeds the critical value associated with the nature of the materials of the levee. This situation can arise over time because of the aging of surface materials, but it can also be due to an increasing effect of the environment on the levee (eg during floods).

The result of external erosion is a reduction of the constitutive materials of the levee that can affect its thickness and density. So the affected area is weaker and more likely to collapse during extreme events.

Material that is exposed by erosion deteriorating the levee surface is not usually designed to resist environmental aggressions. As it is now directly in contact with the source of aggression, the result is an acceleration of the process.

Factors contributing to external erosion

The main causes of erosion are the movement of water directly against the surface of a levee or near to woody vegetation along the bank of the water body. Currents, waves and tides are the main initiators and aggravating factors of external erosion. However, wind, woody vegetation and animal and human activities can play a major role in surface erosion as they may displace soil particles. In this way, some sources also refer to the degradation of the levee by vehicular traffic as erosion.

Mechanisms

In this mode of deterioration, mechanisms such as scouring, global or local slope instability, toe instability, desiccation, structural cracking, shock or vibration wear away embankment materials, which may lead to undermining the structure of the levee.

Wind- or tide-generated waves can cause erosion damage of the slope protection and structural damage to seawalls.

Soil washed away by river currents or wave action results in over-steepened slopes. This may be worse in areas with obstructions on the waterside such as trees, bridge piers, walls and pipes. Hydraulic conditions that put unexpected stresses on the levee can also encourage erosion. For example, eddies formed at the toe of the levee can put more stress on the levee bank.

External erosion of a riverbank is primarily caused by the action of waves, currents or turbulence within the channel (Figure 3.167), for example, wave action, propeller wash from vessels or high velocity flow on the outside of a meander. Flows along the levee generate surface erosion, where the shear stress induced by flows exceeds the tolerance of the levee material constituents. Fill material is washed from the face of the riverbank or waterside slope of the levee, leading to loss of the levee section and undermining.

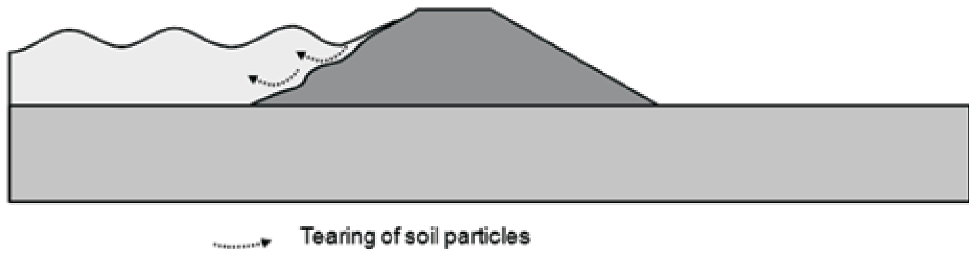


Figure 3.167 External erosion due to currents or turbulence within the channel (courtesy Y Deniaud, CETMEF)

River morphology and its evolution have a major influence on erosion due to lateral movement (meandering) of the entire river channel, or a deeper flow channel within the main river channel. This process can undermine the levee, leading to deep or shallow slope instability (bank caving) as well as direct erosion (Figure 3.168).

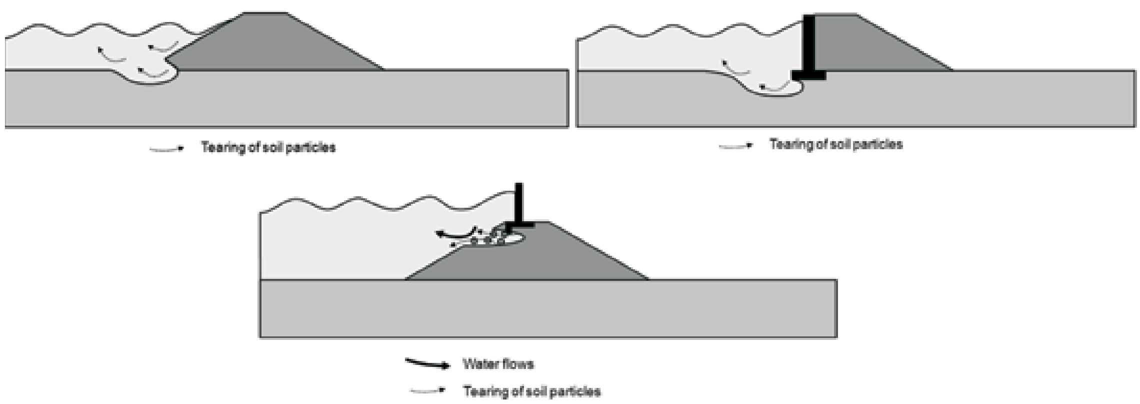


Figure 3.168 External erosion of the toe and foundation of a levee (bank caving) (courtesy Y Deniaud, CETMEF)

Animals that regularly inhabit earthen levees create burrows or tracks that affect the structure. Burrows within the levee can cause weakness, increase hydraulic gradients and in extreme case form holes through the levee. Tracks or scrapes on the surface can encourage infiltration of rainwater and/or runoff that induces erosion.

Woody vegetation growing on the levee may damage or penetrate the protection component. Roots of trees or other deep-rooted plants growing on the levee can penetrate the core and cause flow concentration if the crest is overtopped. Falling trees can also cause damage and roots and stumps may leave voids within the embankment.

Structural issues are shocks, vibrations or collisions that affect the integrity of the levee. They could be linked to human activity (eg vandalism, construction, barge/ship/vehicular collision or circulation) or to solid transport by flows (ice, debris etc).

Unexpected overtopping (Figure 3.169) and overflowing (Figure 3.170) of a levee can induce major damages linked to surface erosion. The shocks and velocity of flows induce a wearing away of the material of the levee. Scour and erosion can develop on the crest or the landward side of the levee when the water passes (Figure 3.171).

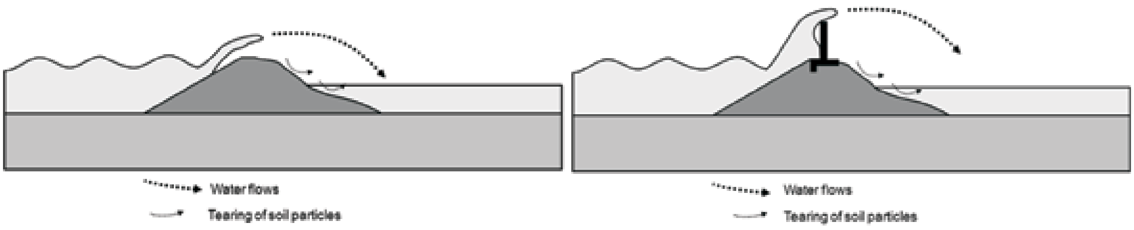


***Figure 3.169** External erosion of the landward side of a levee due to overtopping (courtesy Y Deniaud, CETMEF)*

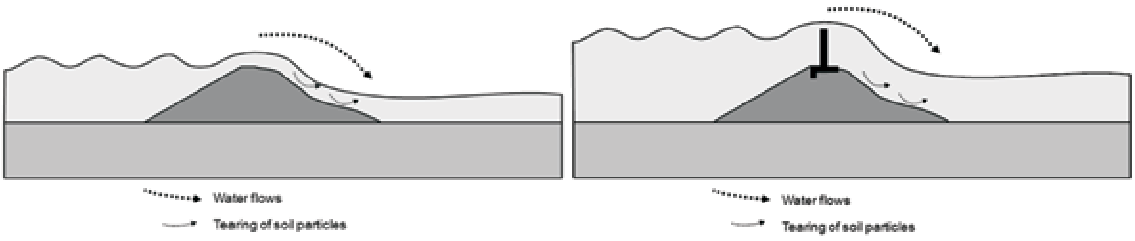


***Figure 3.170** External erosion of the landward side of a levee due to overflowing (courtesy Y Deniaud, CETMEF)*

***Figure 3.171** External erosion of the landward side (courtesy G Degoutte, Irstea)*

3.5.2.2 Internal erosion

Internal erosion definition

Internal erosion is initiated by hydrodynamic forces acting on soil particles inside or through the body of a levee. Internal erosion occurs when soil particles within a levee or its foundation are carried downstream by seepage flow (Bonelli *et al*, 2012). In this process, migration of material particles induced by pore pressure and flow of interstitial water forms channels through the levee or within the foundation soils. These pipes undermine the structure of the levee and lead to a failure and a breach when the seepage is uncontrolled and a steady passage of water through, underneath or around a levee occurs.

Factors contributing to internal erosion

The main factor for the development of internal erosion is seepage. Seepage through an embankment is generally caused by the presence of permeable layers or lenses within the fill or by the existence of cracks or fine fissures. But some other factors can also trigger or aggravate internal erosion through the creation of pipes in the levee, such as:

- animal activity with the uncontrolled development of burrows
- vegetation with the uncontrolled development of roots
- human activity with pipes or other structures penetrating through a levee.

Seepage through or under an embankment constitutes a failure of a levee to perform its main function of water retention. However, especially in its early stages, the volume of water lost is often relatively small, and a small amount of seepage is acceptable. If it is left untreated, finer particles of soil will be washed out of the embankment or its foundation by water flow. As the soil becomes more permeable, the flow rates increase and, as a result, more particles of soil are eroded.

Seepage will also increase the likelihood of slip failure because of changes to the soil–water regime within the embankment, causing a weakening of the fill materials or increasing the uplift pressures beneath an embankment toe. High pore water pressure can lead to increased seepage, hydraulic fracture or instability, especially when they are higher than allowed for in the design. Uplift pressure in foundation soils can generate major instability. Uplift pressures on joints can lead to instability or cracking of the protection component of a levee.

Mechanisms

Different mechanisms of internal erosion have been described and categorised (Bonelli *et al*, 2012):

- with backward erosion, or piping, erosion starts at the exit point and a backward erosion develops a continuous passage when the seepage gradient exceeds the 'flotation gradient' of the soil (Figure 3.172)

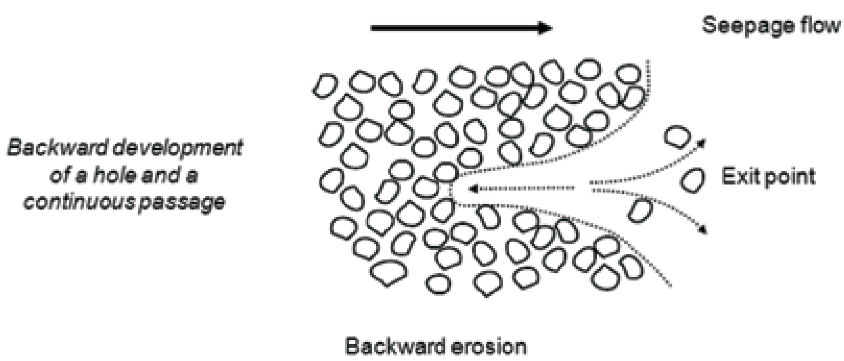


Figure 3.172 Principles of backward erosion

- with concentrated erosion, erosion occurs along the sides of an open crack or in the interconnecting voids in a continuous permeable zone, where the shear stress induced by flows, exceeds the critical value (Figure 3.173). At low flows, leakage can occur with no erosion. Concentrated erosion can also take place alongside structures such as pipes, culverts or spillway walls

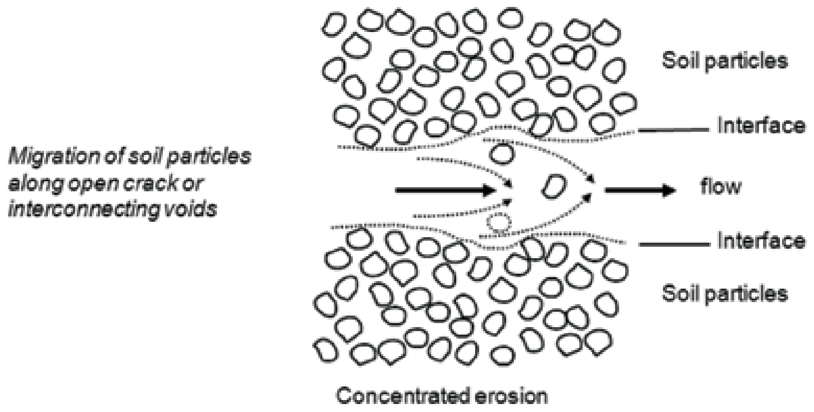


Figure 3.173 Principles of concentrated erosion

- contact erosion develops at the horizontal boundary of a fine soil overlying a coarse soil where the fine soil is washed into the coarse soil by horizontal flow (Figure 3.174). Contact erosion may also occur at the horizontal boundary of a fine soil when it overlays a fractured foundation and is washed into rock fissures by horizontal flow

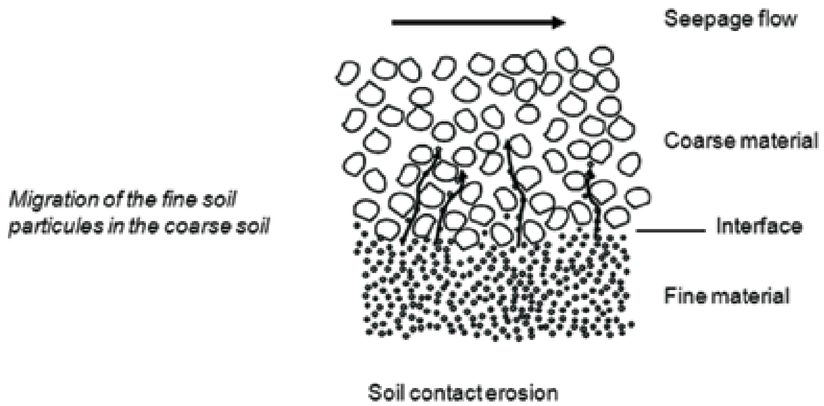


Figure 3.174 Principles of contact erosion

- suffusion is mass erosion in soils that are internally unstable. Small particles of soils transported by seepage flow between the larger soil particles (Figure 3.175).

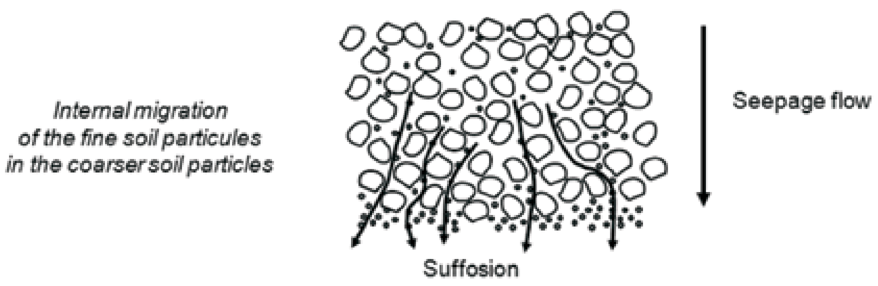


Figure 3.175 Principles of suffusion

Internal erosion can also develop at the boundary between the soil and the appurtenant structures. Internal erosion of a levee can be represented by four phases (Bonelli *et al*, 2012):

- initiation of erosion
- continuation of erosion where the relationship of the particle size distribution between the base (core) material and the filter controls whether or not erosion will continue
- progression to form or sustain a pipe and/or to increase seepage and pore pressures in the downstream part of the embankment or its foundation
- breach resulting in uncontrolled release of the water.

Some typical sketches for internal erosion affecting different types of levee are presented in Figures 3.176 to 3.180.

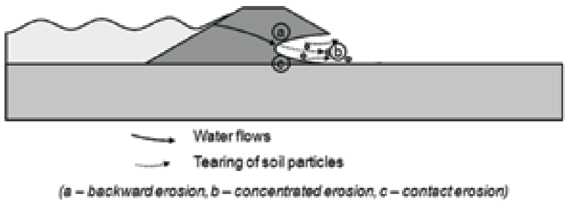


Figure 3.176

Internal erosion of the body of a levee (courtesy Y Deniaud, CETMEF)

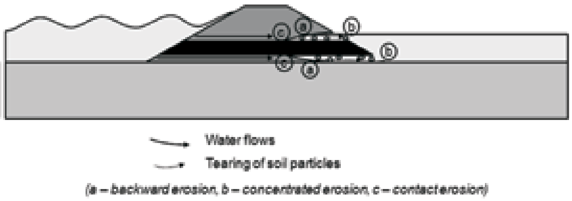


Figure 3.177

Internal erosion along a penetrating structure (courtesy Y Deniaud, CETMEF)

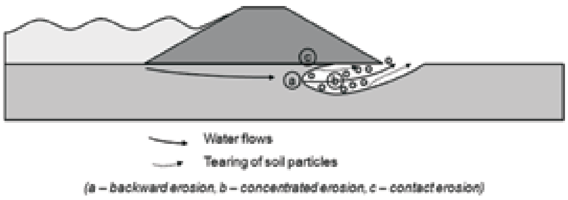


Figure 3.178

Internal erosion of the foundation soils of a levee (courtesy Y Deniaud, CETMEF)

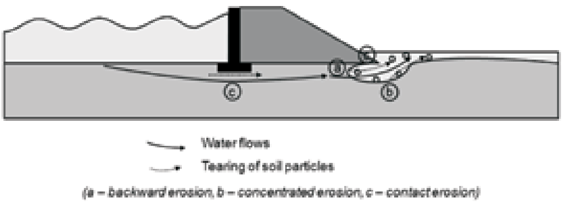


Figure 3.179

Internal erosion of a levee under a waterside wall (composite structure of levee) (courtesy Y Deniaud, CETMEF)

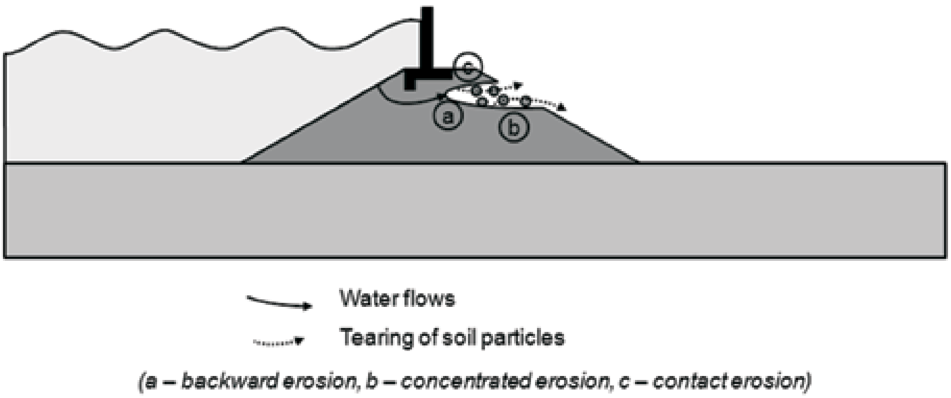


Figure 3.180 Internal erosion of foundation soils under a top wall (composite structure of levee) (courtesy Y Deniaud, CETMEF)

3.5.2.3 Instability

Instability definition

Instability occurs when the active strengths of soil particle movement exceed the resistant strengths. Excess loading on a levee, or weak physical properties of the levee materials or the foundation soils, generate sliding along a shear surface within the levee embankment and/or foundation soils that damage the levee. These processes are related to mechanisms such as rotational or translational sliding, tilting, settlement or liquefaction.

Factors contributing to instability

Factors contributing to the instability of levees may include the following:

- **weight (loading/unloading):** a primary factor of instability as it is the main cause of soil particle movement. Instability of a levee slope can be triggered by an inappropriate or unintentional load on the crest or an inappropriate or unintentional unloading at the toe of the embankment
- **water pressure:** the levee soil may become saturated during high water and be unable to drain as fast as the water recedes. This causes increased weight (driving force) in the levee embankment and ultimately slope failure. This condition is worse in poorly compacted soils with lower density/strength and more voids which water can fill.
  However, water pressure in the levee fill or in the foundation soil decreases the shear strength and the resistance to levee sliding.
- **decline of material properties:** a decline in the properties of the fill body or of the foundation soil can reduce the shear strength and the resistance to the levee sliding
- **human activity:** construction activity near or on the levee may cut away the supporting toe material, steepen slopes, create erosion gullies and remove erosion-resisting vegetation. This condition could be for a variety of reasons, including ditch installation or cleaning near the toe, road construction or new pipe installation. Pile driving and other vibratory activity and deep excavations near levees are of particular engineering concern in areas prone to liquefaction and under-seepage
- **animal activity:** the development of burrows in the body of a levee can reduce the mechanical properties of the fill materials, and induce piping that leads to eventual levee collapse
- **woody vegetation:** the development of roots and their decomposition after the disappearance of the vegetation may result in instabilities. Woody vegetation may also affect levee stability should a tree be blown over by high wind, displacing a mass of the levee embankment
- **impacts:** shocks, vibrations and collisions linked to human activity or to solid transportation by flows can trigger movements of soil particles and instability. It can reduce the properties of the levee slope protection or initiate liquefaction of the body or of the foundation soil of the levee
- **seismic activity:** lateral and vertical seismic forces may cause slope instability. Seismically induced liquefaction can result in levee or foundation soil failure or both. These effects are severe and should be investigated immediately to detect areas with saturated soft/loose soils and over-steepened slopes
- **erosion:** erosion can initiate instability by generating an unloading through toe erosion and bank caving, at the base of a levee slope.

Mechanisms

There are various mechanisms of instability linked to the particular geometric configuration of each levee section that may involve different types of components. However, the main individual mechanisms that must be considered are as follows:

- **shallow sliding and creeping:** the tendency for slumping is highly dependent on the side slopes of the embankments. When a clayey material is compacted to form a levee, its initial shear strength will depend on the characteristics of the materials constituent soil particles, the soil's moisture content and the degree of compaction. However, over time, the soil will weather and potentially soften from the surface down. This effect will be aggravated by seasonal variations; the embankment will dry and possibly crack over the summer months and these cracks will then form a pathway for water in the autumn or winter (by infiltration of rain or seepage of floodwater). For steep-sided embankments, this softening process will reduce the factor of safety against shallow slip failure, potentially to a point where surface slumping occurs (Figure 3.181).

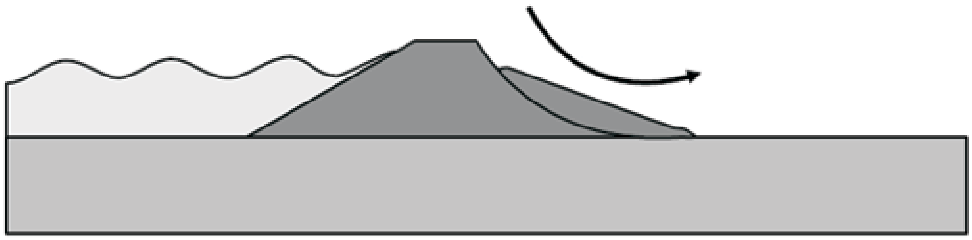


Figure 3.181 Shallow rotational sliding of the landside of a levee (courtesy Y Deniaud, CETMEF)

The tendency for shallow slips to form can be exacerbated on the river face by high river levels over a period of time being followed by a rapid draw-down of floodwater level. In a cracked, distorted and furrowed state, the side slopes of the embankment will be more vulnerable to erosion, particularly from wave action and overtopping (Figure 3.182).

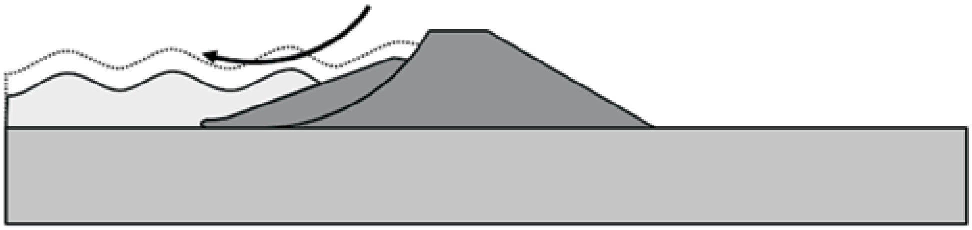


Figure 3.182 Shallow rotational sliding of the waterside of a levee during rapid draw-down (courtesy Y Deniaud, CETMEF)

Levee embankment soil may also 'creep' or move away laterally from the crown. This occurs primarily in clay or peat soil types and is worse when soil is poorly compacted, is saturated or both.

Shallow sliding can be triggered or it can also affect a superstructure such as a wall that is built on the crest of a levee (Figure 3.183).

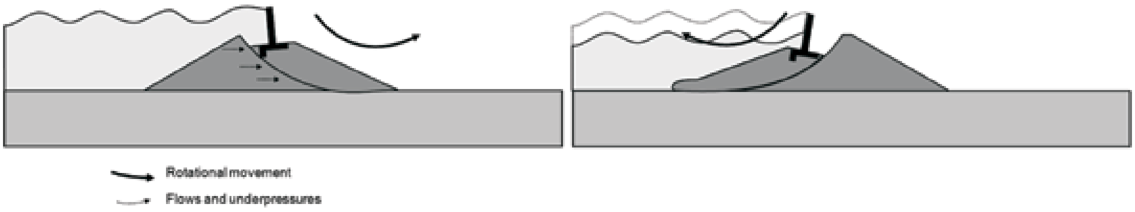


Figure 3.183 Shallow rotational sliding affecting a wall at levee crest (courtesy Y Deniaud, CETMEF)

- **deep rotational sliding:** deep rotational failures will tend to be initiated by changes to an existing situation. Examples of causes include a new embankment being built or an existing embankment being raised, a high load being applied to the crest, an unusually high retained water level or the excavation of a ditch at the toe of the embankment. Changing the size and condition (ie water level) of a soak or drainage ditch is a common cause of problems, particularly with soft clays and silts. One particular form of deep rotational failure, which is often referred to as a 'blowout' failure, can be triggered by high groundwater pressures acting in a permeable layer beneath the embankment. Deep rotational failure will usually appear as cracking and downward displacement of part of the crest, bulging of the embankment slope, particularly the base, and heave of the ground in front of the toe (Figures 3.184 and 3.185). It results in a softening of the embankment fill and a weakening of the foundation soils. If it does not trigger a breach, this may quickly follow unless immediate repairs are carried out

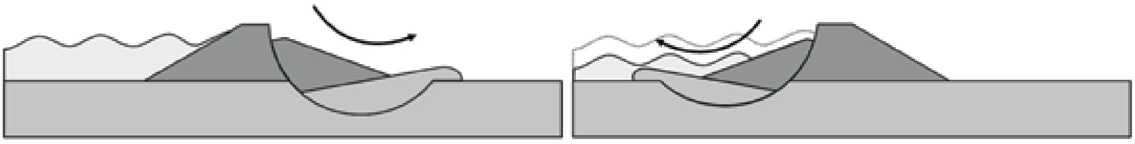


Figure 3.184 Deep rotational sliding of a levee (courtesy Y Deniaud, CETMEF)

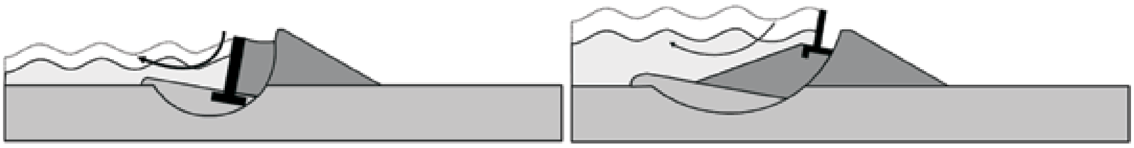


Figure 3.185 Deep rotational sliding affecting a waterside wall of a levee (courtesy Y Deniaud, CETMEF)

- **translational sliding:** the presence of very weak surface layers of peats and clays, in the foundation or in the body of a levee, can result in a horizontal block movement of an embankment. This is when the undrained shear strength of the surface layer is insufficient to resist the hydraulic forces created by the floodwaters acting on the embankment (Box 3.13 and Figure 3.186). The mechanism requires high river levels and low shear strengths of materials beneath the embankment

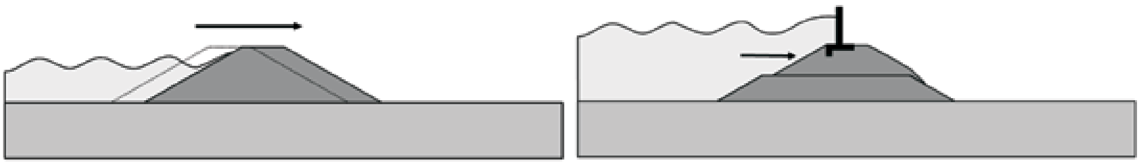


Figure 3.186 Translational sliding of a levee on a soft soil foundation or in the levee body (courtesy Y Deniaud, CETMEF)

- **consolidation/settlement/tilting:** almost by definition, most flood embankments have been constructed on floodplains. Many will have been built on foundations that contain layers of soft clay or peat. It is a characteristic of these materials that they undergo relatively large time-dependent settlements as they consolidate under an imposed load. This is particularly the case for larger embankments of over two metres high (Figure 3.187).

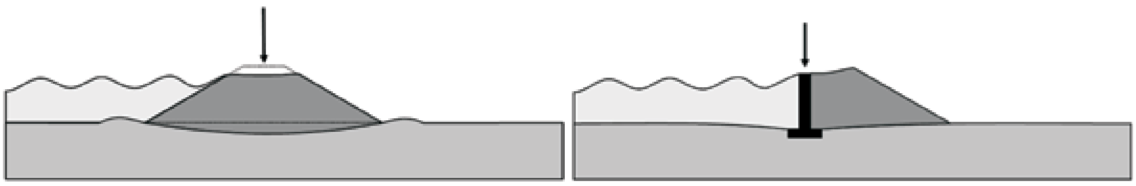


Figure 3.187 Settlement of a levee on a soft soil foundation (courtesy Y Deniaud, CETMEF)

For historical levees that can be many centuries old, much of this settlement will already have occurred and possibly been obscured by later filling. In contrast, newer levees constructed to full height in one lift may be prone to large ongoing settlements of perhaps hundreds of millimetres. In addition, the process of embankment raising will often trigger further settlement, especially where fill material is placed over the side slopes and toe of an embankment. The problem caused by settlement is that the embankment may not achieve its primary purpose of providing an impermeable barrier to a required level, causing a reduction in the standard of protection. One issue of particular concern is the time dependency of the consolidation process: an embankment that meets its height requirements one year will not necessarily meet those objectives in later years. Another problem caused by settlement is distortion-induced cracking of the potentially brittle fill material. This will make the embankment more permeable as well as being more prone to damage and possibly breach because of overtopping (which will be more likely because of the settlement).

Settlement can also affect the actual levee. Changes in water level, seasonal swelling and shrinkage or overloading can deform the embankment (Figure 3.188). All these phenomena can lead to cracking and the development of seepage paths in the different components of the levee.

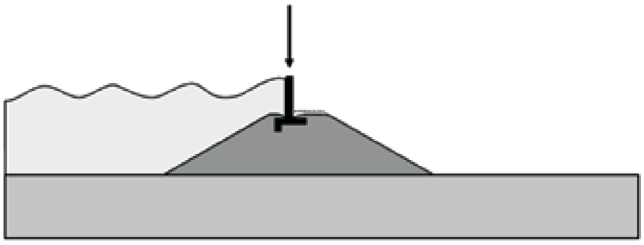


Figure 3.188 Settlement inside a levee due to overloading by a wall on the crest (courtesy Y Deniaud, CETMEF)

Differential settlement in the foundation soils or in the body of levee can induce a tilting of the walls (edge wall or top wall) that are part of the composite structure of a levee (Figure 3.189).

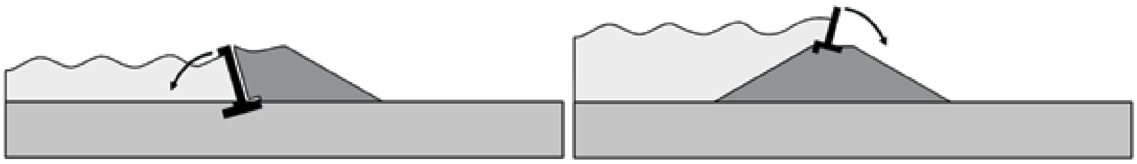


Figure 3.189 Tilting of walls in composite levee due to differential settlements (courtesy Y Deniaud, CETMEF)

- **liquefaction and uplift:** an increase in the pore pressure can trigger a liquefaction of the soil and the appearance of sand boils at or beyond the levee toe (Figure 3.190). The sudden decrease of the shear strength properties of the fill or of the foundation soil allow the development of sudden instability, such as the collapse or sliding of the levee toe. This type of mechanism is deeply linked to vibrations and seismic activity that may affect a levee

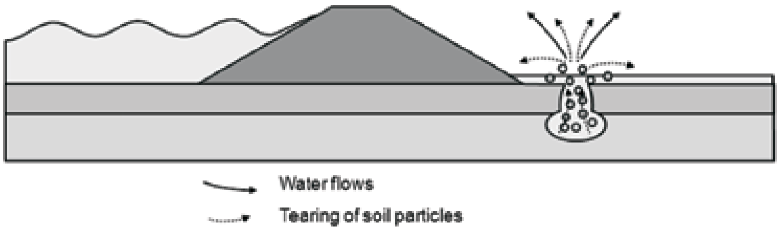


Figure 3.190 Soil boil (courtesy Y Deniaud, CETMEF)

An increase in the pore pressure of a permeable layer underlying a thin impermeable layer can also trigger an uplift of the covering layer and a deep sliding of the landside of the levee (Figure 3.191).

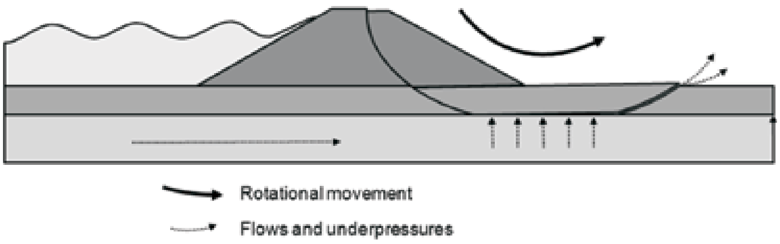


Figure 3.191 Uplift (courtesy Y Deniaud, CETMEF)

- **bearing capacity:** when the weight of the levee and the loads that it supports exceeds the bearing capacity of the foundation soils, levee collapse may occur (Figure 3.192). This can be avoided by good design, construction and a limited and controlled overload during stages of human activity on the levee

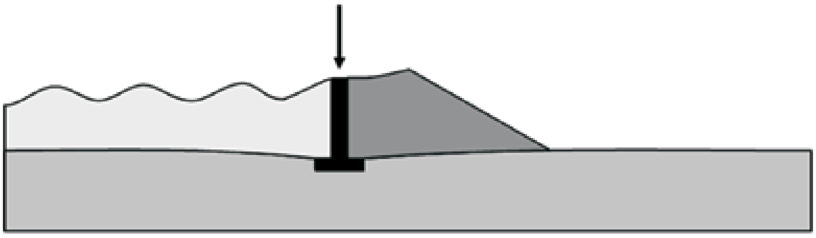


Figure 3.192 Settlement of an edge wall in a composite levee due to low bearing capacity

- **instabilities during construction:** almost all mechanisms that are described here may arise during the construction stage. These instabilities are mainly triggered by overloading in relation to the construction process, the unexpected weak mechanical properties of the foundation soils or an unexpected flood event.

3.5.2.4 Some statistics about levee failure mechanisms

An extensive database on the historical levee failure mechanisms is not available. However, some analysis of historical flooding events and of historical levee failure mechanisms at regional scale have been performed. This is to identify and classify the weight of the different mechanisms of failure for the levee. Boxes 3.15 to 3.17 present the results of some of these studies.

Box 3.15 Levee failure statistics on the flood event of August 2002 in Saxony rivers, Germany

In August 2002 due to extreme floods in many Saxony rivers, more than 100 levee breaches were reported. Following a post-event survey, a levee breach database was established containing all the information available for each breach. Statistical analysis could be conducted for 84 records as sufficient data could be gathered for these cases. This included evaluations in many respects, for example breach geometry, pre-event state (geometric, biological, soil-mechanical conditions) of the breached levee section, hydraulic conditions and time and direction of collapse. Although it is believed that levee failure is usually because of various factors and processes (process chains) an analysis regarding the dominant cause of failure was also conducted. A reliable determination of the main failure cause and mode requires the availability of direct observations and/or measurements, so only four general failure categories were defined:

- external erosion
- internal erosion
- stability failure
- subsoil failure.

By means of a decision tree analysis, taking into account, for example, the breach shape, hydraulic conditions and direct observations, the main failure causes of breaches were classified as follow:

- 70.2 per cent (59 cases) to external erosion (mainly due to overtopping)
- 16.7 per cent (14 cases) to stability failure (slope failure)
- 9.5 per cent (8 cases) to subsoil failure (hydraulic uplift etc)
- 3.6 per cent (3 cases) to internal erosion (eg piping).

It is believed that the ‘external erosion’ classes contain cases that actually belong to one of the other categories but could not be identified due to lack of information. More detailed descriptions regarding the levee failure survey and the conducted univariate and multivariate statistical analysis is given in Heyer *et al* (2010) and Horlacher *et al* (2007).

Box 3.16 ***Levee failure statistics in France (from ERINOH database 2006–2012)***

A database has been developed in France at a national research project called ERINOH (2006–22012). This database lists the incidents and failures, mainly caused by internal and external erosions (overtopping) of embankment dams or dikes (navigation or hydroelectric canals and levees). Regarding levees (dikes for protection against floods), the database gathers 120 sheets, each sheet containing 70 incident fields, grouped into six sections:

- identification of the levee and the type of incident
- geometry of the structure where the incident occurred
- materials of the levee body
- materials of the foundation
- description of the river
- description of the incident and the breach.

Most of the collected data relates to historical breaches on Loire river levees during the three main 19th century floods, and breaches on the Rhone River and tributaries during the 1993, 1994, 2002 and 2003 floods.

The main features of the levees were as follows during the breach:

- height of 1.5–6 m, usually between 3 m and 4.5 m
- relatively steep slopes: 1 < H/V < 2
- systematically sandy clay semi-homogeneous fill, without filter or drain
- ancient works, built in stages over the centuries.

Of the 120 records, internal erosion is identified as an initiating mechanism in 19 cases (16 per cent), the location of pipes or burrows are explicitly mentioned in 11 cases. Overtopping is identified as the initiating mechanism in 50 cases and strongly suspected in 51 cases where the mechanism has been (tentatively) identified as indeterminate, which is 84 per cent in total.

The width of breach opening caused by internal erosion is an average of 21 m (3 m to 65 m). That caused by overtopping is an average of 190 m and can reach noticeable values, up to 740 m for the largest opening. The size of the breach opening appear weakly related to the distance between the levee and the riverbed, but the most important breach openings appear where levees are located near the riverbed. There is no clear correlation between the breach opening and the height of the levee. The type of material in the levee as well does not appear to be a decisive factor but the quality of this data is sometimes questionable. Further research should be carried out to find a possible relationship between the breach dimensions and the flood hydrograph.

Finally, another point concerns the erosion of the foundation. Information is available in 62 sheets and describes pits of several hectares of erosion up to 600 m long and 650 m wide (the depth is rarely filled in). The pit erosion is often described as having a form of a glove, erosion developing along preferential paths, and the presence of palaeo-channels is often mentioned.

More information can be found in Bonnelli *et al* (2012).

Box 3.17 ***Levee failure statistics in England and Wales, 2007 flood event***

For the three most significant flood events since 2007 in England and Wales, the Environment Agency commissioned an analysis of flood defence performance, with a particular focus on failure and with the aim of learning lessons for asset management. These analyses were carried out for the 2007 summer floods, the 2009 Cumbria floods and the 2010 Cornwall floods. Of these, levees only played a significant role in the 2007 event.

In the 2007 event, which is by far the largest flood in England since 1953, about 1000 km of levees in England and Wales were tested (ie they had water against them) and about 500 km were overtopped. There were initial reports of about 20 asset failures, but the analysis concluded that there had in fact been only four breaches. Each of them occurred in small levees (a large majority in the Anglian and Midlands areas most affected by the event). At least three of them, possibly all four, breached before overtopping occurred. The analysis looked at all available data and carried out some limited geotechnical research. It was concluded that all breaches occurred due to a combination of factors, including seepage, slope instability and subsoil instability. An important conclusion was that all of them were caused by local irregularities, ie the levees around them seemed to have similar geometry, geotechnical characteristics, condition and loading, but all four breaches were of limited width – less than 10 m. These local irregularities ranged from animal burrows to local variations in subsoil (eg crossing old river meanders showing up on satellite imagery) to vegetation.

The analysis is described in Royal Haskoning (2008) and its findings and statistics are quoted in the high profile Pitt Review into the floods, as evidence of the Environment Agency's performance in managing the levees (Pitt, 2008).

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4 Operation and maintenance (O&M)

Courtsey Jean Maurin, DREAL Centre

CHAPTER 4 CONTENTS

4 OPERATION AND MAINTENANCE (O&M)

Chapter 4 discusses the operation and maintenance of levees. It examines the challenges a maintainer may face and suggests preventive measures and repair techniques. Key inputs from other chapters:

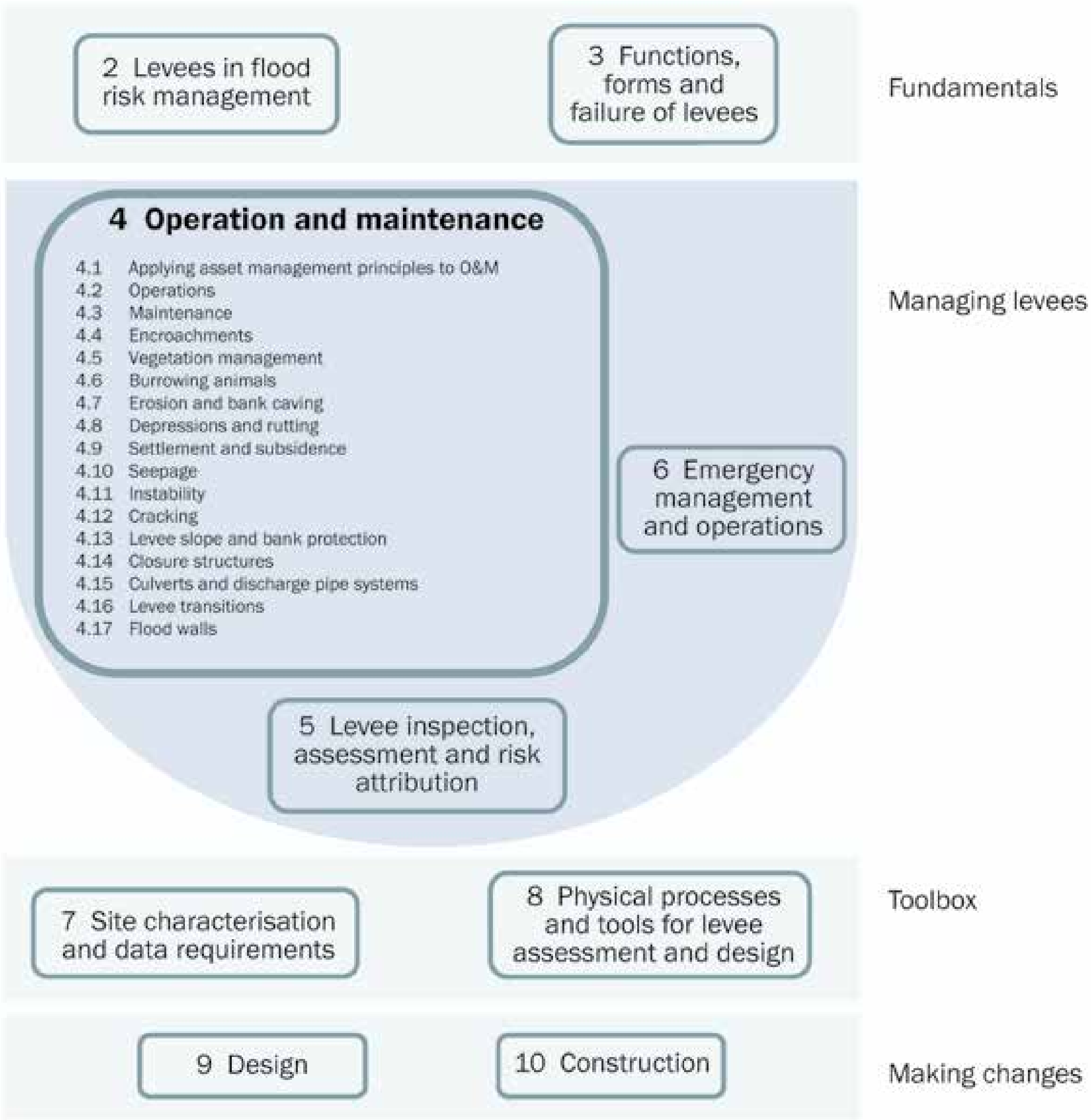
- Chapter 3 ⇨ **forms, functions and failure mechanisms of levees**
- Chapter 5 ⇨ **visual inspection methods to identify and assess levee issues**
- Chapter 9 ⇨ **insights into maintaining the levee's intended design**

Key outputs to other chapters

- **routine operations** ⇨ Chapters 5 and 9
- **maintenance procedures** ⇨ Chapters 5, 6 and 9

Note: The reader should revisit **Chapters 2 and 3** throughout the levee life cycle for a reminder of important issues.

This flow chart shows where to find information in the chapter and how it relates to other chapters. Use it in combination with the contents page to navigate the manual.



CHAPTER CONTENTS AND TARGET USERS

This chapter's 17 sections provide an overview of asset management principles as they apply to operation activities and maintenance activities.

Applying asset management principles to O&M

Section 4.1 introduces O&M activities in relation to the levee management life cycle. It focuses on the organisation of O&M, importance of an O&M manual, and activities and practices of asset management. The section on general approaches to O&M addresses the use of risk-based and sustainable issues, operating for the long-term, and managing and organising data produced and used during O&M.

Operations

Section 4.2 presents information related to the three key functions of a levee system, which are operating to keep water out of the leveed area, operating to get water out of the leveed area, and operating to keep the levee standing.

Maintenance

Sections 4.3 to 4.17 discuss maintenance of earthen levees and associated structures. Maintenance on earthen levees addresses activities to prevent damage and progression to failure, while maintenance on associated structures highlights key activities to address routine maintenance.

4.1 APPLYING ASSET MANAGEMENT PRINCIPLES TO O&M

4.1.1 Introduction

The O&M function comprises a host of management and care techniques that help levees and their associated features meet specific performance objectives. These objectives are either part of the levee's original design (eg flood risk reduction) or are added later by the levee owners or managers (eg recreation, ecosystem support). This chapter focuses on maintenance techniques to help levees perform their risk reduction function. It includes preventive maintenance measures, repairs that can be made within the O&M function, and guidance for determining when repairs are beyond O&M.

4.1.2 Levee management life cycle

The routine asset management portion of the levee management life cycle (discussed in Section 2.3.3 and shown in Figure 4.1) outlines O&M's typical functions. These include:

- monitoring, inspecting (discussed generally in this chapter and in greater detail in Chapter 5) and maintaining
- assessing performance (see Section 5.1.1 and 5.2)
- assessing and prioritising management action(s) (see Section 2.3.4)
- repairing and adapting (see Sections 4.3 to 4.17).

Decomissioning is not part of the O&M function, but it is important to consider how the levee will be dealt with when it has exceeded its design life or is no longer serving its intended purpose. Being clear about which levees need to be operated, maintained, inspected and monitored is important to ensure resources are not spent on levees that are no longer needed.

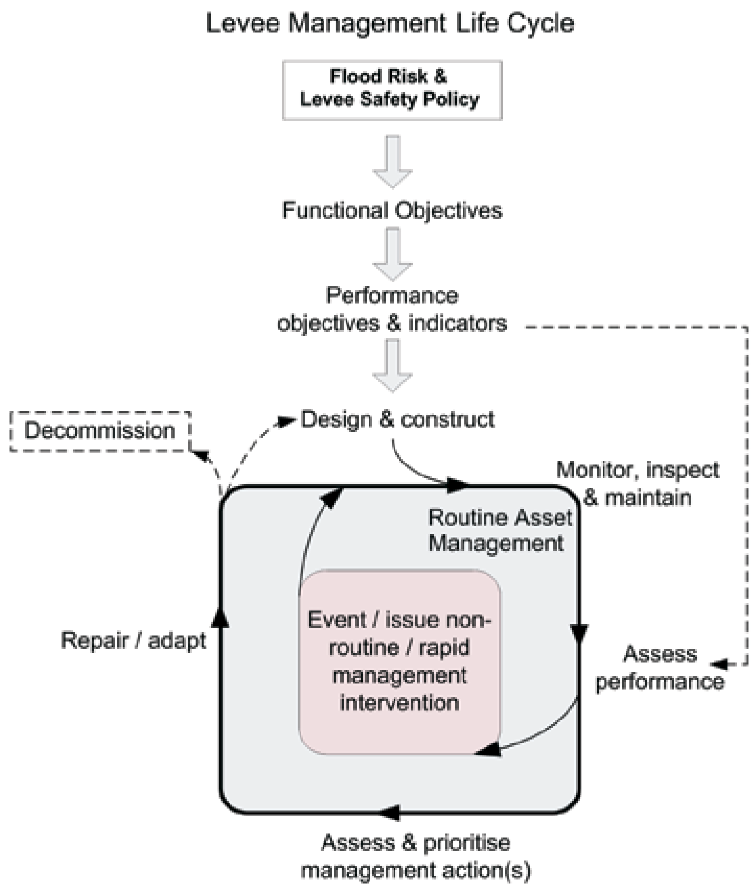


Figure 4.1 Levee life cycle diagram

Levees are continuously challenged by flowing water, precipitation, wind, waves, vehicular traffic, animal traffic and vandalism – as well as by changes in vegetation and in the needs of the people in the leveed areas. So, over time, levee materials may degrade or shift, mechanical parts may wear out, and new features may be added to the levee. O&M is the function that takes action to observe, assess, stop, repair and/or accommodate these changes.

In this chapter:

- **Section 4.1** explains how to apply the asset management principles (introduced in Chapter 2) to levee operations and maintenance
- **Section 4.2** discusses levee operations
- **Sections 4.3 to 4.17** offer recommendations for resolving levee maintenance issues.

4.1.3 Organisation of O&M

To support a levee's O&M needs throughout its life cycle, the O&M function is typically divided into three roles, each requiring different skill sets. These roles are:

- planning (including performance assessment, Section 5.2)
- O&M (including the actual repair work on the ground)
- inspection.

How these roles are organised varies depending on the size and nature of the managing organisation. For example, in small organisations the roles may be combined within one unit. In larger organisations they may be in separate units. And in some cases, the operational role may even be contracted out to private parties. Whatever the method of organisation, the highest priorities are that:

- there is good communication between the roles
- the roles and responsibilities be clearly defined and effectively carried out
- the planning role specifies the performance standards for the operational role, to ensure that the O&M activities meet specified objectives.

Box 4.1 illustrates several ways that the O&M roles have been organised.

n addition to the work done by government agencies and levee owners, members of the community protected by the levee can also help flag levee anomalies. Ways of involving them include:

- having a general phone number for local residents to call to report concerns about changes in the levee condition, such as a badger hole or a slough or slide. Some countries have built-in reporting systems in addition to a general telephone number to call, where stakeholders report issues or concerns to the town counsellor. The town counsellor can then escalate the issues to make sure that the necessary parties are informed
- offering special training to selected individuals in the community who have some understanding of or an interest in the levee (eg training in how to watch for current issues of concern on the levee).

Financial resources are a frequent constraint for levee maintainers. However, funds can often be stretched by taking maximum advantage of the interest and ability of those protected by the levee. This practice can also raise awareness about the presence and function of the levee.

Box 4.1 Examples of the division of organisational roles

In England and Wales: for assets managed by the Environment Agency, the asset performance team creates the standards for the work (the required conditions of the asset) in its system asset management plan, and the operations field team determines the most efficient way to achieve the required standards (they also perform the work). When they are in the same geographic area, both teams report to the same senior manager, the operations manager, to whom a catchment engineer (a chartered civil engineer) also reports. The catchment engineer is the focus for engineering expertise.

The majority of levees are managed by the Environment Agency and other risk management authorities, rather than by private landowners. The Environment Agency has the in-house capabilities to plan and perform needed work, while other authorities need to contract these functions out. Private owners, who generally do not have the technical ability to manage their levees, use contractors to do the work (though the Environment Agency's asset performance teams may provide advice).

In France: levee owners are responsible for the maintenance of their levees, though they have the option to carry out the O&M themselves or to subcontract it. Depending on the size of the owner's organisation and its level of expertise, sometimes a consortium is formed to manage and maintain the levees. French levee owners are also required to submit information to a state agency that oversees hydraulic structures. This information is used to demonstrate whether O&M practices are being carried out in compliance with regulations and if they are sufficient to ensure the reliability of the flood defence system.

In Germany: the 16 federal states (including 13 areal states and three free cities – Berlin, Hamburg, Bremen) are responsible for flood protection. In some states, eg Bavaria, the authorities organise the flood protection and the O&M of flood protection facilities. In those states, the federal state is responsible for the coast and first order rivers, and the communities are responsible for the second order rivers and their levees. Other states may use companies (eg Sachsen, Sachsen-Anhalt) to handle their flood protection, or they may use 'levee associations', which include persons, companies and other agencies from flood-prone areas (eg North-Rhine Westphalia) with an interest in the levee performing as designed (ie stakeholders). These associations have to finance and organise flood protection efforts as a statutory corporation under public law.

In the USA: on levees owned by the national government, local cost-sharing partners ensure that O&M is performed. The government conducts levee inspections to verify that the O&M functions are done in accordance with the terms in the initial assurance agreement with the cost-sharing partner and with current engineering criteria. The local cost-sharing partner may do the O&M work themselves, or they may delegate these tasks to another agency. For example, within the jurisdiction of the US Army Corps of Engineers' Sacramento District in California there are about 2763 km of national government levees. For 90 per cent of them, the Central Valley Flood Protection Board (a State of California agency) ensures that O&M occurs as outlined in the O&M manuals. The O&M of most of those levees has been delegated to approximately 60 different local maintaining agencies, such as cities and counties, which are responsible for both the planning and operational roles and are subject to inspections from both state and national government representatives.

In the Netherlands: the regional water boards are the levee managers. They are government agencies with their own governing bodies and financing structures, and are solely concerned with water management. They are responsible for the operation, maintenance and inspection of the levees, even when they outsource levee work to private companies. In the Netherlands, each province sets up, creates rules for, supervises and discontinues the water boards as necessary. From a hierarchical point of view, these boards have the same status as the municipalities (see Figure 4.2). Water boards do not necessarily own the levees, but per the keur (the document containing all rules the water board can apply for protecting the levees) and legger (the levee profiles) they can enforce requirements for maintenance and require the owner of the levee to ensure that it is maintained as prescribed.

To help maintain the turf (sod cover) on the levee, the water boards will sometimes rent the levee to farmers whose lands it borders, and allow them to graze their sheep on it.

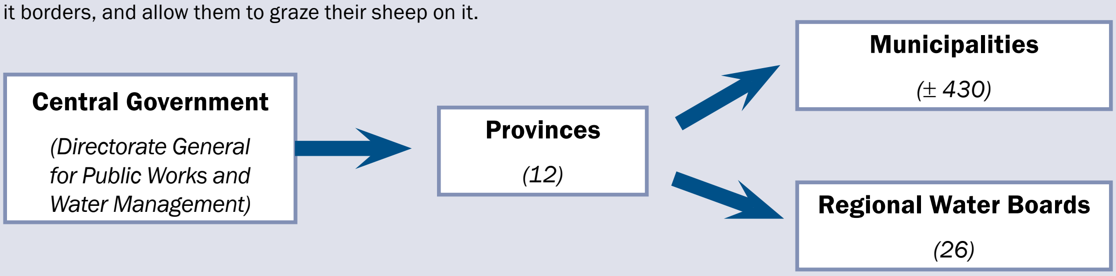


Figure 4.2 Water board hierarchy

4.1.4 Importance of an O&M manual

The O&M manual describes the specific tasks a maintainer should perform to ensure the reliability and durability of a levee system, and the methods and resources that should be used to perform them successfully. It forms the foundation of the levee's quality assurance management plan. Ideally, the O&M manual's guidance and instructions take into account the levee system and its environment, the system's intended level of reliability, the resources available to the levee's maintenance program, and the need for the manual to serve multiple purposes. These purposes include:

- functioning as a key document for asset management and organisational strategy
- showing third parties that maintenance is being performed in compliance with legal requirements.

Where an existing levee does not have an O&M manual, it is considered good practice to create one.

Note

Under some circumstances, a set of authoritative instructions may perform the same function as an operations and maintenance manual.

Benefits of an O&M manual

In addition to providing guidance to maintenance staff on how to perform their tasks, the O&M manual may also:

- reduce the time lost to ineffective and inefficient practices
- provide information for decision makers about the link between resources allocated to maintenance and performance of the levee
- allow problems to be identified and resolved in an open manner, thereby encouraging continuous improvement and transparency.

Suggested contents of an O&M manual

To provide these benefits, the O&M manual should contain the following information:

- the precise location of the levee, including regional (secondary) levees, if any
- a verbal description and map of the extent of the levee system
- a set of as-built drawings that have been updated to indicate on-site modifications or observations noted in the field during construction (such as the actual relationship of the levee toe to any encroachments – including trees, pipes, buildings or fences – or to any easements)
- references to engineering standards (by not duplicating the standard in the manual, it will not be necessary to update the manual with every update to the standard)
- roles, contact information and a list of the responsibilities of the stakeholders
- legal requirements concerning maintenance
- procedures to follow if a detected problem is beyond O&M
- manufacturers' specifications for equipment and structures, and a list of authorised products (eg seeding mixtures, concrete and rip-rap types) and their specifications (may be included as an addendum to the manual)
- specific tasks to perform to ensure adequate maintenance (see Box 4.2)
- appropriate interventions (eg repairing a breach may not be appropriate if the levee is planned for decommissioning in the near future)
- residual risks that continue to need to be managed
- environmental considerations that affect O&M practices and timing, such as restrictions to accommodate nesting bird habitat or bat habitat.

A risk register, where risks associated with operating and maintaining the levee are listed, should also be included in the O&M manual. The register may include:

- potential hazards associated with contaminated material that may be in the vicinity of the levee
- risks associated with working on a platform
- requirements that workers only work from the levee crest due to the steepness of the levee slope
- risks to the project (risk of failure)
- risks to individuals performing construction or O&M tasks (and how to minimise those risks)
- risks to the environment (from the presence of the levee and mitigating measures that may be needed)
- risks to the environment if the levee fails.

Those who perform O&M activities should strive to minimise these risks while managing efficiently and in alignment with budgetary and environmental constraints.

Box 4.2 Tasks the O&M manual should include

The O&M manual should define the tasks associated with:

- vegetation management (see Section 4.5)
- flood risk mitigation structures, including stop logs (see Sections 4.14, 4.15 and 4.17)
- encroachments (see Section 4.4)
- non-flood risk mitigation equipment such as fences, stairs, scales
- daily and routine inspections and flood patrols (see Section 5.3 and 6.4.1.1)
- what to do in case of an emergency (see Section 6.2.3 for information on a flood response plan and Section 6.2.2 for information on an emergency response plan)
- auxiliary structures that are needed to ensure the integrity of the levee system (eg closure structures, pipes, pump stations)
- all other required maintenance (including interior drainage).

It should also include the following information about each task:

- where the task should be performed (the portion of levee or the structures involved in the task – with their location and method of access, if needed)
- when and how often it should be done (including conditions if maintenance is condition-based)
- applicable engineering standards and as-built drawings
- equipment/staff required, including required training
- design details and plans for the structures
- risks associated with O&M, such as safety associated with mowing steep slopes (see Sections 9.14, 9.5, 9.6, and 10.1.4.2)
- a detailed step-by-step description of the task
- practices that should be avoided
- measures to be taken to limit safety risks to workers and third parties
- measures to be taken to limit environmental or social impacts (including historical or recreational impacts– if any)
- efficiency ratio of the task
- how the task should be tracked and documented.

If the intended reliability or the maintenance requirements differ from one portion of the levee to another, specify the nature and frequency of the tasks for each portion. If a task is subcontracted, the manual's description of the tasks can be used as the contractor's specification.

The O&M manual and emergency events

The O&M manual should include or reference the tasks related to emergency events (eg floods) as well as non-emergency events (eg suggesting that low waters during spring tides are good times to inspect the toes of coastal levees). Its description of emergency activities should be in compliance with emergency action plans and should also explain (or reference other documents that explain) the effect emergencies can have on available resources. For example, emergency events can:

- occur outside of working hours and last more than a work shift
- often require repeated tasks, such as levee patrols
- strain the availability of limited, trained staff.

Maintainers should make sure that sufficient trained staff are ready to perform emergency tasks. Also, they should take into account that staff may be mobilised on other missions, such as supporting evacuations, or they may not be able to get to the levee because access is blocked.

Emergency measures identified in the manual should be regularly rehearsed and tested to be sure that materials that need to be on hand for emergencies are present and available. Completing trial assemblies of closure structures is recommended where possible.

See Chapter 6 for more information on emergency preparedness and management.

Compiling and updating the O&M manual

A good time to define a levee's O&M procedures and document them in an O&M manual is during the levee's design phase (see Chapter 9). The first version of the manual should be issued after the levee is constructed. Updates are generally made:

- after the first maintenance cycle is completed, to take into account unanticipated issues
- when there are changes in regulations, technologies or funding
- if a problem unexpectedly requires a levee designer's expertise or special techniques.

The manual might be available in paper or electronic form (PDF) or as a password-protected website accessible only by authorised people via PC or smartphone.

O&M manuals should be evaluated regularly for relevance and to determine if updates or new standards are needed in light of changes in legislative or local policies. Updates should be done in accordance with appropriate local and national laws and regulations.

To ensure the manual's completeness, the authoring team should have expertise in the hydraulics of flood risk mitigation structures, maintenance, design, environmental issues, health and safety, finance and law.

4.1.5 Activities and practices of asset management

Asset management is defined as the systematic and co-ordinated activities and practices through which an organisation optimally manages its asset's condition, performance, risks and expenditures over the life cycle of the asset for the purpose of achieving its organisational strategic plan. O&M is a critical part of asset management. Chapter 2 describes the general principles of asset management and how these apply to levees.

The most critical systematic and co-ordinated activities and practices to apply to O&M are the following:

1 **Define O&M objectives during the design and construction phase:**

For new levees

- identify routine O&M activities during design and construction
- identify any specific O&M requirements established during construction (see Section 9.16)
- record identified activities and requirements in the O&M manual to ensure that the levee manager understands the designer's considerations and intentions.

For existing levees

- if sufficient structural information is not available, it should first be collected to establish the O&M requirements.

2 **Define functional objectives, performance objectives, and performance indicators:**

- functional objectives should include flood risk management, but may also include recreation or the environment

- functional objectives help asset managers prioritise investments, as they take into account the purpose of the structure and the risk associated with it
- functional and performance objectives should inform O&M (as they do design) as part of asset management planning
- performance indicators translate objectives into specific levee features. They may be used to define targets that ensure O&M's focus on achieving the objectives.

3 **Determine when and how decommissioning may occur:**

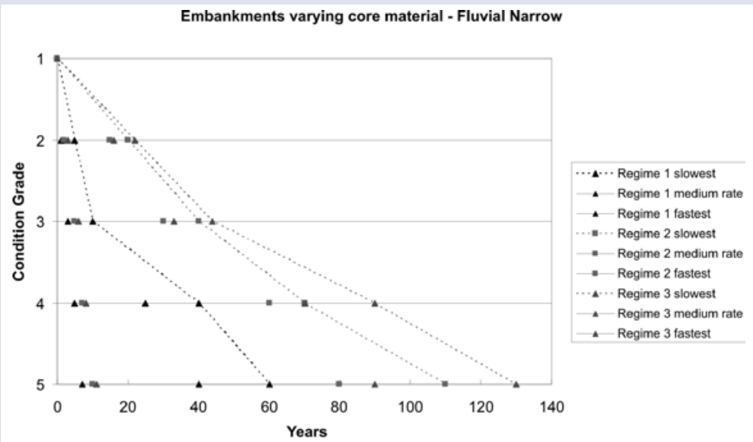
- decommissioning results from an asset management decision that weighs the benefits of continued maintenance and its associated costs (based on a performance/risk assessment) against policy or management objectives. This overview of benefits and costs helps the asset manager decide whether continued O&M is viable and affordable. Decommissioning of levees is discussed in more detail in Chapter 10.

Box 4.3 illustrates how the Environment Agency helps asset managers assess the deterioration of typical flood and coastal defence assets.

Box 4.3 Guidance on how to assess the deterioration of flood and coastal defences and how to assess their residual lives

The Environment Agency for England and Wales has developed practical guidance to help asset managers assess the deterioration of typical flood and coastal defence assets as well as their residual lives. The guidance helps asset managers in all flood and coastal risk management authorities assess the residual risk of assets under different conditions and maintenance regimes.

An asset's rate of deterioration depends on the asset type, its environment and the load it experiences. To continue to provide protection to people and properties from flooding and erosion, asset managers need to understand the likely deterioration rates across their asset stock. This allows them to gauge when appropriate interventions are most effective for maintenance, repair and replacement. Flood and coastal risk management (FCRM) assets within England and Wales have an estimated replacement value of about £34bn.



The guidance is based on a wide range of evidence, including historic data, targeted monitoring and expert input from practitioners. It also has benefits for other applications, such as assessing changes to a wider stock of assets as part of a strategic assessment of future investments and funding needs.

As deterioration can vary from asset to asset, it is essential to use engineering judgment and practical experience, along with this guidance, to apply and adapt the deterioration curves appropriately. Figure 4.3 is a sample deterioration curve for an embankment.

Figure 4.3 Example of deterioration curve (from Environment Agency, 2013)

4.1.6 General approaches to O&M

The following sections explain the benefits of operating and maintaining a levee by:

- using a risk-based approach to O&M
- using a sustainable approach to O&M
- operating for the long-term
- managing and organising the data produced and used during O&M.

These are complementary approaches to O&M. It is recommended that all of these approaches be considered for incorporation in a comprehensive O&M plan.

1 2 3 4 5 6 7 8 9 10

4.1.6.1 Using a risk-based approach to O&M

The risk-based approach takes into account the flood risk associated with a levee system when prioritising levels of O&M. This approach helps to optimise the benefits of O&M (in economic, environmental and social terms) against its costs (see Box 4.4 for example).

Challenges associated with the implementation of risk-based approaches include:

- the cultural change an organisation may need to go through to overcome O&M practices that are based on habit and tradition
- the difficulty of quantifying risk and the effect of O&M (ie its benefits) on risk. Methods are only starting to become available (as of 2012) for practical use (see Chapter 5)
- the political controversy associated with protecting rural areas to different condition grades than urban areas.

Box 4.4 Implementing a risk-based approach to levee inspection in England and Wales

> The Environment Agency in England and Wales uses a risk-based approach to determine the frequency of levee inspections and the minimum condition grade that a levee should be maintained to. When determining what standard to operate and maintain to, some areas (such as the UK) balance the risk associated with a failure on that levee against the cost of maintaining the levee to a certain condition grade. When examining the risk, the consequences of a breach are considered. To determine the potential extent of these consequences, factors considered include the height of the levee, type of land being protected (urban, agricultural or rural), the value of the property behind the levee, and the type and frequency of loading (eg by infrequent hurricanes, occasional coastal storms, regular tidal fluxes, riverine high water events). This approach has helped to reduce costs and to focus on funding areas where the consequences of levee failure are the greatest.

4.1.6.2 Using a sustainable approach to O&M

O&M practices are considered sustainable when they help the present generation meet its needs without compromising the ability of future generations to meet theirs. Sustainable O&M practices ensure that flood risk mitigation systems can be maintained well into the future, so that subsequent generations do not become dependent on levees that are not able to perform. Sustainable practices do this by balancing long-term feasibility and technical, economic, environmental and social considerations with the system's flood risk mitigation requirements.

Long-term feasibility: planning O&M practices for the long-term, rather than only for the present, can encourage the development of methods that are appropriate for each phase of a levee's life. These methods can help a maintainer take care of the entire extent of the levee now and into the future. It can also ensure maintenance of the:

- designed levee cross-section
- designed levee height
- location of the levee (morphology impacts).

Long-term feasibility is discussed further in Section 4.1.6.3.

Technical considerations: O&M practices need to be technically sound. Unsound practices may compromise a levee's ability to perform during a flood event.

Economic considerations: O&M practices need to be economically viable (benefit/cost ratio) and affordable. Limited funding usually means compromising in one area to spend money on another area. Efforts should be made to ensure that O&M practices make the best use of the money that is available.

Environmental considerations: while the primary purpose of most levees is to protect the life or property of people, levee O&M may also have an impact on animals, plants and other forms of life in the vicinity of the levee. To avoid negative environmental impacts, when operating and maintaining a levee, the maintainer should follow all local and national environmental laws and regulations that apply (see Section 2.2.2.2).

Social considerations: because levees are for people, they protect them (to some known degree), but they may also be used and even enjoyed by them. Multi-functional uses may influence O&M. The health and safety of those maintaining or using the levees should always be kept in mind.

4.1.6.3 Preventing long-term negative impacts on the levee

This section elaborates on three aspects of operating a levee for the long-term:

- maintaining the entire levee cross-section as designed
- effectively managing encroachments (also called non-water retaining objects)
- understanding the impacts of channel and foreshore morphology on the levee (such as activities in the river, in the channel or on the beach).

Maintaining the entire levee cross-section as designed: the role of as-built drawings

As-built drawings help maintainers understand what needs to be maintained by showing what the entire levee cross-section is and whether what is currently in the field matches what was designed. Maintaining the entire levee cross-section can help avoid issues such as the one described in Box 4.5. When as-built drawings are not available, a levee designer may be able to help identify and define the levee cross-section (see Chapter 9).

Box 4.5 The importance of knowing the levee features

> During a levee inspection in its Sacramento District, California, the US Army Corps of Engineers discovered that a farmer had dug an irrigation ditch into the toe of a seepage berm. The filter fabric, which was meant to keep material from moving through the levee, had been ripped and was visible. The effect of this change was to reduce the resistance of the levee. Had there been a high water event while the levee was in that condition, the performance of the levee would have been compromised.

Box 4.6 describes the system currently in use in the Netherlands to ensure that the entire cross-section of the levee is maintained. It is a system that gives levee managers optimal control over changes to the levee's zones.

Box 4.6 ***An example of good practice: how the Netherlands ensures that the entire cross-section is maintained***

In the Netherlands, the water boards are the levee managers and are responsible for the operation, maintenance and inspection of the levees. Without the permission of the regional water board, it is generally prohibited to build, excavate, plant greenery or encroach the levee with an object within the levee's protection zone. This is stated in the water board's ordinance (the keur or byelaws). When reviewing permission requests, the water board determines whether the work could negatively affect the stability or height of the levee or embankment.

The type of permission required depends on the type of activity and the zone for which the activity is proposed. Every water board has a register, or legger, which defines and maps the measurements and locations of the levees. Levees are placed in the register of municipalities so that anyone who is involved in an activity, such as the sale of real estate or construction within the zone of influence, is aware that, by law, they must request a permit for work they plan to perform. This is particularly important for home buyers. The listing in the register makes them aware that upon purchase of their home there will be construction restrictions on their property. For example, a swimming pool cannot be built in the zone of influence.

The levee owner is responsible for ensuring that the entire cross-section, including related structures such as dams in front of the levee, are maintained, undamaged, during routine maintenance. The owner does this by inspecting the integrity of the entire protection zone as indicated in the register. The register also identifies the maintenance duties of the levee owner (ideally the water board). When the levee is owned by an individual or a company, they are responsible for maintenance. The protection zones are (see Figure 4.4):

- **core zone:** influence zone of levee failure mechanisms
- **protection zone:** the zones on each side of the core zone where restrictions and regulations to protect the levee are applied
- **outer protection zone:** the zones on each side of the protection zone where less strict restrictions and regulations to protect the levee are applied
- **minimal profile:** the minimal profile (levee prism) of a levee, where activities and encroachments or other non-water retaining objects are prohibited (eg trees, utility lines, pipes, boat docks, stairs, homes, swimming pools, power poles, roads, irrigation ditches and railways)
- ‘**profile of free space**’: some water boards specify a zone for possible future levee enlargements. Though not officially defined in the keur or legger, this zone ideally should be kept free of objects.

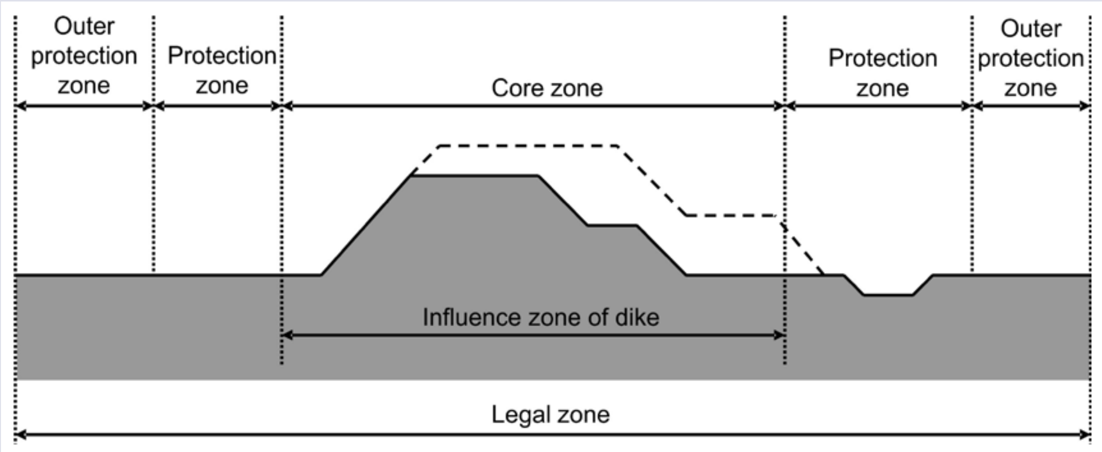


Figure 4.4 ***Cross-sectional diagram showing the zones that comprise the legal zone in the Netherlands***

Effectively managing encroachments

One of the maintainer's most important tasks is to keep tight control over the levee by not allowing any construction or installation of third party objects over, under or through the levee (known as encroachments or non-water retaining objects) without their permission. The permitting of (ie the issuing of permits for) third party activities in and around the levees is an effective tool for reducing the risk of failure due to encroachment (see Section 4.4). Having a comprehensive system for tracking these encroachments is also important, as illustrated in Box 4.7.

Box 4.7 ***The magnitude of encroachment permit challenges, California, USA***

The Sacramento District of the US Army Corps of Engineers (with nearly 3000 km of levees) has over 18 000 permit applications on file. In addition, there are many unauthorised encroachments that were installed without permits. Unauthorised encroachments can impede inspection, access, and in some cases even threaten the integrity of the levees.

However, unauthorised encroachments are only part of the district's permitting challenges. For over 10 years in the 1990s and early 2000s, the levee log (the document that records the field verification of the encroachment permits) was discontinued. Without this information, it is difficult to know what is in some levees. Not knowing can interfere with flood-fighting, impede the decision making process for levee improvements and affect cost estimates for levee inspections that are based on the number of encroachments anticipated.

Starting in 2008 the state (California) and national governments placed a higher priority on documenting encroachments and, through much co-ordination, implemented an effective system to document existing permits and ensure the documenting of future permits. It will take many years of research to replace the documentation that was not maintained.

Understanding the impacts of channel and foreshore morphology on the levee

Maintaining the entire levee cross-section and managing encroachments are important to a long-term operation plan, but maintainers should also understand the effect changes in the channel can have on their levees. For example, blockages in the stream (fallen trees, debris) can increase water levels (and the risk of overtopping) and changes in sediment transport regimes can increase local scour (and the likelihood of destabilising a levee slope). See Box 4.8.

Box 4.8 ***Channel morphology impacts on levees in California's Central Valley, USA***

In the early 1900s, enormous quantities of hydraulic mining debris were deposited in the rivers and streams of the Central Valley in California. With the cessation of hydraulic mining and the building of dams for debris control, flood control and water supply, the sediment transport regime of these rivers changed drastically, and the rivers caused even more erosion. This has resulted in widespread incision/undercutting of all rivers and streams in the Central Valley and problems with levee integrity, toe erosion, and levee slope instability.

Changes in the channel affect each levee system differently depending on the design of the system (its materials, encroachments, revetments) and conditions, such as the width and depth of the channel, the proximity of the levee to the bank, and whether the waterside is armoured or not. For these reasons, it is difficult to offer specific guidance for any one situation without first evaluating all of the contributing factors. A good approach for the maintainer is to recognise when there may be a threat to the design or functionaliy of the levee system and correct each threat on a case by case basis. O&M staff frequently need the help of a professional levee designer or engineer to make the necessary repairs, especially when the problem or its repair may have environmental consequences.

4.1.6.4 Managing and organising data produced and used during O&M

Managing data and keeping organised records are vital to an effective O&M function. Data should be gathered to support the objectives of asset management. Data and records related to the levee and its environment should be gathered because of the key role they play in levee assessments. The levee manager is typically responsible for assembling maintenance records on the levee and its components. Table 4.1 lists some of the types of data produced and used during O&M. It also indicates where in this handbook additional information may be found. Section 5.6 provides more information on how to manage and access data.

Table 4.1 *Data produced and used during operations and maintenance*

Type of data	Description of data	Data could be used in O&M to:	Where to find more information
Operations records	Records may include, but are not limited to: • dates and notes related to the operation of pump stations and gates (if managed by the levee manager) • dates and notes related to trial installations of closure structures • information related to the preparation for floods and other emergency events • information related to the preparation of protocols for stockpiling materials and carrying out of emergency drills.	• ensure operation of the embankment features occurs at appropriate intervals • identify information to reference when conducting maintenance, to flag necessary repairs	Chapter 6
Maintenance records	Records may include, but are not limited to: • periodic mowing of the levee and other vegetation management efforts • periodic coating (painting) of pipelines and pump stations • cleaning and lubrication of mechanical gate structures, including maintaining security fencing.	• ensure required maintenance specified in original agreements, such as the O&M manual, is occurring • record changes such as rehabilitation or other maintenance repairs to supply information for engineering assessments • provide a written record of maintenance completed for reference by future inspectors/maintainers	Section 4.3
Levee logs (similar to a 'legger' in the Netherlands)	A comprehensive record of all items an inspector may expect to find on the levee. This may include, but is not limited to the location of: • all encroachments (non-water retaining objects) – both permitted and unpermitted • all pipes – both permitted and unpermitted (note that when pipes are removed, it is important that the location and previous existence of the pipe be noted. This may be important information if there are future problems in that area of the levee) • areas where the levee has been tested by past events (this includes, but is not limited to, where sandbag rings were required to fight boils, where seepage occurred, and where sandbags or other measures were required to prevent overtopping) • ramps.	• help maintainers to identify all features on their levee so that any unpermitted encroachments can be addressed • help maintainers to know that features requiring maintenance are being maintained • help maintainers anticipate encroachment challenges when performing maintenance	Section 4.3
Legal requirements	Legal requirements and regulations that must be complied with, such as: • standards of protection (if appropriate) • local and national environmental laws and regulations • levee profiles over which the maintainer has legal jurisdiction (for example, 'legger') • rules or laws with which the operator and maintainer must abide (such as the 'keur')	• ensure compliance with legal requirements	None

Table 4.1 Data produced and used during operations and maintenance (contd)

Maintainer- generated visual inspection records	Record the maintainer's monitoring of the levee, including monitoring of any deficiencies and any changes that are noted to historically identified deficiencies.	• help the maintainer to alert appropriate managers of anything of importance regarding the condition and operation of the levee • require the maintainer to monitor the condition of all items mentioned in the levee log and flag items requiring maintenance	Sections 5.3 and 5.4
Engineering- generated inspection reports	Engineering-generated inspection reports may identify deficiencies not found during routine maintenance inspections (including design deficiencies). Of particular importance is data related to the crest elevation, which can be used to monitor subsidence and settlement.	• help operators and maintainers understand the condition of the levee and its components • determine the maintenance activities needed to maintain the integrity of the levee • note changes in the levee condition from previous maintainer-generated visual inspection reports • note future changes in the levee condition from the last engineering generated inspection report • identify encroachments of concern from an engineering standpoint so that appropriate action is taken	Sections 5.3, 5.4, 5.5
Records of encroachments (non-water retaining objects) permitted	A detailed record of all encroachments for which permission has been requested/ granted/denied should: • indicate the location of the encroachment and whether the request was granted/denied • state the conditions under which encroachments were permitted.	• help the maintainer to assure the agency that issued the permit that the conditions in the permit are being upheld (as appropriate) • help the maintainer know how to approach issues with encroachments in the levee (note that the approach taken to a permitted encroachment may be very different from that of an unpermitted encroachment) • help the maintainer keep the levee log updated	Section 4.4
Maps indicating the jurisdiction of the operator and maintainer	A map could be a good visual representation of this jurisdiction, helping levee maintainers to understand the extent of their jurisdiction.	• allow the maintainer to know if they need to acquire additional permission to be on the land where maintenance is required • help maintainers understand the area over which they have control	Section 4.4
River/channel impact studies	Any historical records indicating the effect the river or channel has had on the levee as well as any studies that have looked at the hydraulic effects of the river. While the maintainer may not be qualified to analyse these reports, being familiar with the key findings could be important.	• help the operator and maintainer be aware of any river channel morphology or flow impacts	Sections 7.1.6.3, 7.2.2, 7.3, 7.5
As-built drawings	Drawings indicating the design that was constructed. These include details of all the features that are included in the flood risk mitigation system, such as berms, relief wells, seepage ditches, flood walls, interior drainage pipes and channels.	• help the maintainer know what the original design is that they need to maintain	Section 10.1.5

Table 4.1 ***Data produced and used during operations and maintenance (contd)***

Design memorandums	Detailed records indicating the basis of design. While the maintainer may not be qualified to analyse these memorandums, having them on file and being able to provide them to any engineers who are inspecting or doing work on the levee could be important.	• help the maintainer to provide relevant information to engineers working on the levee	Section 9.3
Past performance data	Past performance data (also called points of distress), the loading at the time the distress was noted, and any effect on the levee could be important information for the maintainer to be familiar with. This may also include studies that predict the way the levee performed in the past if there are no data or studies that indicate where weak points in the system may be.	• include the information in the levee log • help monitor weak areas in the levee more carefully • inform inspections of encroachments • provide information to permitting agencies about whether to permit encroachments or not	Section 6.2.6
Flood response plans	Flood response plans include reference information on how to react during an emergency situation. Details on these plans can be found in Chapter 6.	• provide reference information when acting in an emergency response capacity • provide reference information when completing trial closures of closure structures or performing emergency preparedness drills	Sections 6.2.2 and 6.2.3
Operations and maintenance (O&M) manuals	O&M manuals are jointly created by the operator, maintainer and the designer during the construction phase. As rehabilitation work is done, updates to the O&M manual may be required. Should revisions be required, it is important that co-ordination occurs between the operator and maintainer, the designer and the constructor to make sure everyone is informed of revisions.	• detail inspection frequency (engineering inspections, maintainer's inspections, flood event inspections and when detailed condition assessment is required) • ensure that maintenance detailed in the original agreement as stated in the O&M manual is being performed	Chapter 4 (Section 4.1.4), Section 9.3.6, Table 10.10
Assurance agreements	When a levee is constructed, agreements are typically established that explain who funds the construction of the levee and under what conditions, who does the maintenance, and who pays for repairs to the levee and under what conditions.	• help maintainers know what was originally agreed to, to what extent they may be able to count on additional funding, and what they need to do to maintain eligibility for rehabilitation funding • help all parties know whose responsibility it is to take care of certain elements during the lifetime of the levee including, but not limited to, acquiring land during construction, ensuring the land during construction is free of unauthorised encroachments, paying for rehabilitation of the levee, and paying for routine maintenance of the levee	Section 9.3 and Table 10.10
Contact information	It is good practice for the levee manager, operator and maintainer to have on hand contact information for: • the levee manager, operator, and maintainer • any contractors who are working on the levee or have done work on the levee • flood-fighter staff • people required for emergency drills or trial closures (including those with access to stockpiled materials or those required to put the closure in place).	• help operator/maintainer know who to contact if there is a problem • help operator/maintainer know who to contact to complete emergency preparedness exercises	Sections 6.2.2 and 6.2.3

Table 4.1 Data produced and used during operations and maintenance (contd)

Records data (including piezometric data)	It is good practice for the levee manager/ operator/maintainer to have on hand relevant records data such as: • groundwater levels (and normal variations of that level) • what normal soil deformation and cracking looks like (by gathering information such as soil type).	• help levee manager, operator and maintainer know when to be concerned about: • seepage (if it is due to normal variations in the groundwater table it may not be as much of a concern) • deformation (if the soil is organic, then a few cm of deformation is normal during high water events) • cracking (if the levee cracks every summer then that may not be a concern unless the crack is a certain width or differential settlement is observed)	Sections 4.5.1.6 and 7.9

4.1.7 Scope of O&M and Chapter 4

There are many topics discussed briefly in this chapter that are dealt with more comprehensively elsewhere in the handbook. Table 4.2 shows where to find additional information about these topics.

Table 4.2 For further reading on topics related to this chapter

Topic	Why it may be important to an operator or maintainer	Where to find more information
Asset management	Levee O&M is one component of levee asset management. See Chapter 2 for general principles of asset management and how these apply to levees.	Section 2.3.3
Failure mechanisms	A levee operator and maintainer should be committed to ensuring that the levee they are working on does not fail. Understanding the ways levees do fail, in addition to the deterioration mechanisms (discussed in Chapter 4) may be helpful.	Section 3.5
Topology/parts of the levee and related structures that may affect performance of the earthen embankment	Understanding this topic may help operators and maintainers be aware of their levee's complete cross-section and of the related structures that may affect the performance of the earthen embankment.	Sections 3.3 and 3.4
Visual inspections	A levee manager should use the latest visual inspection report to understand the condition of the levee and its components and to determine what maintenance activities are needed to meet the specifications in the maintenance manual.	Sections 5.3 and 5.4
Monitoring	Monitoring helps operators and maintainers spot changes in the levee, particularly weak spots, and flag issues.	Sections 5.5 and 7.9.8
Emergency management	The levee operator is usually an integral part of any emergency measures that occur on the levee.	Chapter 6
Non-routine maintenance/ rehabilitation/remedial measures	Levee maintainers should know their limits and know when to call in a professional levee designer. Chapter 4 provides guidance in these areas. Chapter 9 discusses them in depth.	Sections 9.8, 9.9, 9.10, 9.11, 9.12, 9.13, 9.15
Designs to be maintained	Levee maintainers normally maintain levees so that they perform as intended by the original design. To learn more about design, see Chapter 9.	Sections 3.3, 3.4, 9.5.5, 9.6

Table 4.2 For further reading on topics related to this chapter (contd)

Design deficiencies	Chapter 4 provides limited guidance to the maintainer on how to identify design deficiencies. Once deficiencies are identified, an experienced professional designer may be required to fix the situation. If a condition assessment is needed, Chapter 5 provides information on how to perform the assessment.	Chapters 5 and 9
Decommissioning of the levee	Chapter 9 discusses the basics of why a levee may be decommissioned and offers some tips to determine if the levee you are working with may be a good candidate. Levee maintainers should become familiar with the entire life cycle of a levee and be aware of potential indicators that the levee's useful life may be coming to an end.	Section 2.3.3

4.2 OPERATIONS

This section discusses the operation of the earthen embankment and only those auxiliary structures that directly affect its operation, such as closure structures. Though the operation of pipes through the levee is discussed to some extent, the discussion is limited to those operations that affect the earthen embankment.

Regardless of the reason for the operation or the type of operable feature, the purpose of levee operation is to assure that the levee performs its flood mitigation role safely and according to its design. Levee systems are operated to fulfil three critical functions:

- to keep water out of a leveed area
- to get water out of a leveed area should it be inundated
- to keep the levee resilient during flood and storm events.

Operable features of levees (eg gates, pipes, flaps, valves, spillways) may be part of the original design, or they may have been added either to adapt the levee to new infrastructure in the leveed area (eg transportation) or to deal with changes to interior drainage. Spillways may be used to pass water at a specified elevation into a floodway, which can relieve pressure from the rest of the levee system. In some cases, flooding the leveed area can also provide flood risk management and environmental benefits.

This section explores some common operational activities and how and why they are performed.

4.2.1 Operating to keep water out of the leveed area

If the design height of a levee section has been reduced to accommodate a roadway, railway or other crossing through the crest, a mechanism is usually installed to close off that area during a high water event or during the severe weather season. While the way that mechanism is operated may vary, eg manually by the levee owner, with the aid of mechanical equipment, or automatically, its primary function is to help keep water out of the leveed area. Sometimes, to improve response times, these closure mechanisms may even be operated by the community living closest to them, as in the example in Box 4.9. O&M of closure structures is covered in detail in Section 4.14.

Gates and valves in culverts and pipes that pass through levees may also require some operation. These may include, for example:

- flap gates on the waterside to keep water from back-flooding through the system
- outfalls from interior drainage pumping stations, gravity drainage, and wastewater treatment plants.

The effectiveness of each type of levee operation depends on a combination of proper design and installation, thorough training of operators, regular pre-event testing, and routine inspections and maintenance. If the levee can protect the designated area from the design event with the intended safety factor preserved, then the levee is operating as intended.

Box 4.9 ***Community engagement and partnership for closure structure installation***

In England and Wales, community engagement and local partnerships have allowed some assets, such as flood gates, to be operated by locals. Local operation allows for faster response times (the operators live close to the gates) and also raises the community's awareness of the asset's benefits. One such example is at Shaldon in the county of Devon, where there is a 20-year legal agreement between the Environment Agency and the Parish Council. The Environment Agency constructed the scheme and provided an operation manual and regular training, together with telemetry and flood warnings. The Parish Council provides the manpower locally to operate the gates. The agreement is reviewed once every five years.

Each type of structure has its own set of unique operational activities that support it before and during its operation. Table 4.3 lists several types of structures and the associated operational activities that may be required to keep water out of a leveed area.

Table 4.3 ***Operational activities that may be required to keep water out of a leveed area***

Type of structure	Operational activities
Stop logs 	• transporting materials to the closure • stacking logs on top of each other to close the physical opening in the levee • covering the structure with plastic sheeting • patrolling the closure if logs are made of valuable material that may be stolen.
Flap gates 	None. A flap gate operates passively. It has a one-way door that closes to keep rising water level fluctuations from flowing through a flood defence. It keeps water out of the leveed area by closing when the water pressure differential is high enough to hold the gate closed.
Slide gates 	Slide gates allow interior drainage to flow from the leveed area out through the flood defence and keep flood waters from inundating the leveed area when water levels are high. They give the levee owner and maintainer close control over how much water is allowed to enter or leave the leveed area. Operational activities may include: • adjusting the gates (manually or automatically) up and down along the track • regularly removing any debris that accumulates in the intake structure. See Section 4.14 for more information on maintenance of closure structures.
Flood gates 	Some flood gates slide vertically in slots along the opening, and some are hinged along one side. Others have hinges on both sides of the opening and seal against each other in the middle. Flood gates may be operated manually, mechanically or automatically. Automatic, rising flood gates are operated by a float system and by the rising flood water (for example at Carrick on Suir in the Republic of Ireland). Flood gate operational activities may include: • ensuring that no one is trapped either by the gate mechanism or by the closed gates • monitoring/remotely controlling telemetry and closed circuit television systems, if included • clearing away any obstructions that may prevent gate closure.
Demountable elements 	In some urban areas, demountable structures are put in place before flood events. Operational activities include: • transport from the depot to the site • assembling and fitting • maintaining security of elements from theft (see Box 4.10).

Table 4.3 Operational activities that may be required to keep water out of a leveed area (contd)

Sandbags	The use of sandbags may be written into the emergency preparedness plan and may be the closure structure for openings in some levees. If this is the case, the emergency preparedness plan should include, at a minimum, the following: • contact information for those who would install the sandbag closure • how much time the operator would likely have to assemble the structure and how long it would take to get all the materials on site and assembled • how to assemble the sandbag structure, including any critical installation instructions specific to that site, such as how to tie into the levee on either side, or whether plastic sheeting is required to cover the sandbag structure.

Box 4.10 Safeguarding closure structures (demountables) from theft

England and Wales have experienced attempted thefts of deployed temporary and demountable flood defences, particularly those elements that are made of aluminium. The Environment Agency recommends employing security staff to patrol these defences once they are in place, both to reduce the risk of theft and to avoid the resultant increased flood risk to the communities protected by the defences.

4.2.2 Operating to get water out of the leveed area

If high levels of water have collected on the landside of the levee, several operational activities may help to get the water out. Table 4.4 lists several system features and the associated activities that may be required to get water out of a leveed area.

Table 4.4 Operational activities that may be required to get water out of the leveed area

Levee system feature	Operational activities
Interior drainage ditch gate control	Change gate settings on irrigation canals and drainage ditches to keep flood waters from reversing flow and further inundating leveed areas (gate changes may be done by the operator or by local communities, provided they have the knowledge and training necessary to do it safely). This allows pumping systems to remove the water.
Interior drainage pipes	• remove any blockages of pipes, if possible • monitor pipes to detect and resolve any issues as soon as possible (see Section 4.15 for more information on pipes).
Interior drainage pumps/pumping stations/movable pumping stations	Pump stations vary in size from very small to very large. They are generally sized to convey design flows from the interior drainage channels and surface areas that drain to the pump. Small pump stations may have an electric pump that discharges small volumes where the leakage or runoff is a concern but does not accumulate quickly. Large pumps may handle drainage and seepage from a network of drainage channels that may be very high volume: • monitor for proper ranges of pump line pressures and temperatures during a flood • monitor diesel generators for proper fuel levels and lubrication when they are used as backups to electric pumps • monitor the discharge point to make sure that the flows are not causing erosion of the levee surface and endangering the structural integrity of the levee • monitor the interior pump intake areas for sand boils if the head differential across the station is high and the duration of high river levels extends over several weeks. Experience has shown that significant problems may develop near or at sump pump areas. As the evacuation of interior water creates high hydraulic gradients and increased seepage rates, soils from beneath the levees begin to mobilise (based on experience during the 2011 Mississippi River flood and prior events in the USA).
Outlet gate control	In some cases, it may be necessary to reopen gates in the levee after the flooding in the river has subsided in order to release water from the flood plain.

4.2.3 Operating to keep the levee standing

Table 4.5 provides a few examples of operational activities that may help improve levee resilience – that may keep the levee from failing due to seepage, slope instability, uncontrolled overtopping or other issues. These activities are meant to illustrate things that could be done and also encourage operators and maintainers to brainstorm additional options if these are not appropriate to their situation. Further details can be found in Chapter 6.

Table 4.5 ***Operational activities that may improve levee resiliency***

Feature of the levee system or levee weakness observed	Possible operational activities
Scour protection on the waterside of the levee	Provide additional protection in advance of anticipated high flow velocities with rip-rap and limit water entrance into deep permeable layers below the levee.
Erosion protection on the levee surface (turf or other)	• a good, short turf cover may provide protection against erosion. Maintenance regimes should ensure that the grass cover is kept to a suitable length • rip-rap may need to be placed during an emergency to repair severe erosion sites and prevent levee failure (See Chapter 6 for possible emergency response options).
Spillways	 Some spillways require action from the operator to function as intended. This may include detonating explosives located in the spillway to bring the spillway into action and allow the water to inundate the floodway, thereby relieving pressure from the rest of the levee system. Others are designed to operate passively (eg they overtop only at the desired water surface elevation). ***The Jargeau Spillway in the Orleans floodplain, France, which was completed in 1882. It is 575 m long and would be activated by a fuse if the spillway was needed***
Relief wells/toe drainage systems	Relief wells and toe drainage systems are designed to control seepage. However, due to the difficulty of maintaining them, they are recommended only as a last resort when designing levee systems. Wells could be pumped if they are not performing, but it is very difficult to determine which well to pump. If wells are not performing during an emergency, it is typically because the coffer (filter) is clogged. As of 2012, there is no known documentation of incidents where relief wells were effectively able to be unclogged during a flood event.

4.3 MAINTENANCE

Maintenance is critical to the long-term performance of the levee. But even a perfectly operated and maintained levee may not provide the intended flood risk mitigation if there are flaws in its design or construction. These flaws are frequently the cause of levee failures. Common design deficiencies include:

- inadequate levee heights
- uncontrolled underseepage
- unprotected erodible surfaces
- unstable slopes.

Though these deficiencies may not be fixable through good routine maintenance, operations and maintenance programs may be reformulated to detect such flaws before levee failure occurs. By training field staff to routinely inspect their levees for erosion, settlement, seepage or slope distress, adverse findings may be reported in a timely manner to levee designers for their assessment and input on recommended repairs.

See Chapter 9 for guidance on addressing design deficiencies, and Chapter 5 for information on levee condition assessment.

It is important to adopt a proactive approach to the program of identification and repair given the need for an efficient and effective use of limited repair budgets. Deterioration needs to be monitored closely and addressed in a timely and efficient way. If the maintainer waits too long and too much deterioration has occurred, it can be challenging to get the necessary equipment into the area to do the repair. But it is also important to be economically efficient about repairs. The money spent on repairs that are made too soon is looked upon as money that could have been better spent elsewhere.

Consequences of postponing maintenance

The consequences of postponing maintenance should be carefully weighed when faced with the decision to do repairs right away or defer them because of costs, environmental regulations, permitting issues or other concerns. Delaying levee repairs or not doing them may heighten the risk to inhabitants and property in the protected area until the repair is made. The maintainer will have to determine whether postponing repairs will:

- cause the levee or its associated facilities to deteriorate further and result in increased repair costs in the future
- cause the levee's integrity to be threatened, eventually resulting in substantial and costly design and repair work to restore it to its originally intended level of flood risk mitigation.

CAUTION

Perform a 'condition assessment' before pursuing costly design repairs.

When a levee problem is so severe or complex that a professional designer is required, it is recommended that a condition assessment be performed first. The condition assessment can help ensure that all issues associated with the problem are known so that a good, workable solution can be designed. Chapter 5 provides guidance on possible ways to assess the levee to determine what action is needed. It should be used in conjunction with Chapter 9. Chapter 4 suggests ways to determine when repairs are beyond the scope of maintenance. The use of a qualified levee designer's services will be required to design a new structure or rebuild an existing levee section that falls outside the realm of operations and maintenance. Information on these topics can be found in Chapter 9.

4.3.1 Issues with earthen levees

Table 4.6 elaborates on the relationship between deterioration processes and failure mechanisms. Many of the deterioration processes could cause most of the failure mechanisms. For the sake of clarity, only the primary failure mechanism associated with each deterioration process is represented. The topics listed are those that appear in Sections 4.6 to 4.12. Encroachments, vegetation, closure structures, culverts and discharge pipe systems, transitions, and flood walls are discussed separately in later sections.

Table 4.6 Primary failure modes that may result from deterioration processes (see Sections 4.6 to 4.12)

		Primary failure modes caused by the deterioration process			
		External erosion	**Internal erosion**	**Instability**	**Further explanation**
Deterioration process	Burrowing animals (Section 4.6)		X		Burrowing animal holes can encourage the passage of water through the levee, which can erode internal levee material (internal erosion). These holes can also cause levee instability either directly or as a result of the internally eroded material.
	Erosion and bank caving (Section 4.7)	X			The deterioration process of scour can lead to the failure mechanism of external erosion, if not adequately addressed.
	Depressions and rutting (Section 4.8)	X			Depressions and rutting on the levee crest and side slopes are a form of external erosion. Though it is unlikely that this deterioration process would cause levee failure on its own, when combined with other deterioration processes (see first column), depressions and ruts have the potential to make the levee crest/slopes undriveable (due to water ponding on the levee and turning to mud), making it harder to flood-fight. Depressions and rutting can also damage the turf cover, making the levee more vulnerable to external erosion.
	Settlement (Section 4.9)	X			Levee settlement significantly increases the risk of external erosion because overtopping will occur at a lower water stage. The settling of the levee crest can also cause issues that lead to the failure mechanism of levee instability.
	Seepage (through-seepage and underseepage) (Section 4.10)		X		Seepage can move fine materials through or under the levee and, eventually, larger soil particles, causing internal erosion.
	Slope instability (Section 4.11)			X	Slope instability can appear as a slump, a slide, a tension crack, an oversteepened slope or as toe erosion. It can progress to the levee failure mechanism of levee instability as additional portions of the levee slope become unstable.
	Cracking (Section 4.12)			X	Tension cracks can indicate that the levee slope is unstable. If not addressed, they can cause the entire levee to become unstable.

4.4 ENCROACHMENTS

An encroachment is any structure not considered part of a levee's design that when placed on, over, under, through or near a levee, may have a negative effect on its structural integrity, its ability to mitigate flood risks, or on its access roads. This includes structures directly beside the levee on both the land- and waterside and any structures that may increase the hydraulic load. Examples of encroachments include utility lines, pipes, boat docks, stairs, homes, swimming pools, power poles, roads, irrigation ditches and railways. Encroachments also include activities performed on or near a levee that are not related to its design function, such as farming and excavating.

Why encroachments occur on levees

If mitigating flood risks was the only function of a levee and the only goal of a levee operator, then controlling encroachments would be simple, as all levee encroachments would be prohibited. But because levees often provide flood risk mitigation for residential or commercial communities or border environmentally sensitive areas other, sometimes competing objectives arise. These may include:

- meeting the special needs of those within the leveed areas, such as demands for water (that require pipelines to pass through the levee) or transportation (that may require a roadway or railroad to pass through the levee)
- preserving historical buildings that were built in or near the levee prism (see example in Box 4.11)
- honouring historical agreements that allowed encroachments no longer permitted today
- meeting special environmental or endangered species needs.

Box 4.11 Preserving historical buildings in the Netherlands

In the Netherlands, the homes shown in Figure 4.5 currently have their ground floors and basements in the levee prism. With a 0.3 m rise in the flood level anticipated within the next 50 years, the cross-section (Figure 4.6) of the levee will need to be enlarged. Since the entire row of homes is not likely to be removed, other alternatives are being considered for future levee improvements, including the proposals set out here.

Figure 4.5 Homes with ground floors and basements in the levee prism

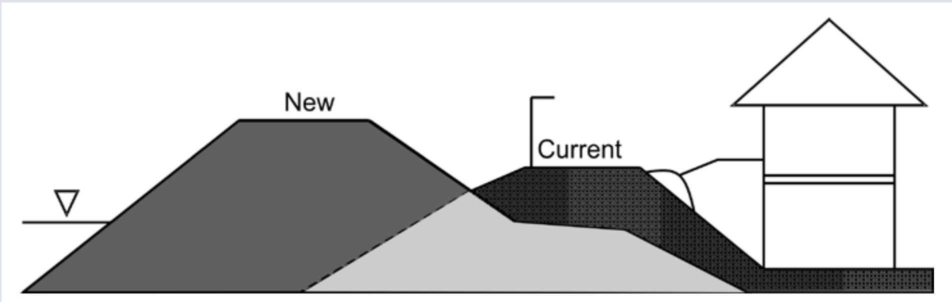


Figure 4.6 Cross-sectional view of the current levee section (Figure 4.5) and one of the new levee sections that is proposed to be built to provide the additional flood risk mitigation required for the future

If new or existing encroachments are going to be allowed on a stretch of levee, however, then those encroachments should be prevented from threatening levee integrity, inhibiting access, or interfering with flood-fighting and inspection. Levee operators should establish both an effective permitting system (a system that reviews and evaluates all encroachment requests and their potential effects on the levee) and a way to enforce its decisions (see the list of elements of a good encroachment control program later in this section).

Why encroachments need to be controlled

The examples in Table 4.7 illustrate some of the adverse effects encroachments may have on levees.

Table 4.7 Adverse effects of encroachments

Type of encroachment	Possible adverse effect on levee
Embankment-related	
Improper excavation	Could create unstable slopes, which may lead to slope failures.
Removal of material from the levee, its foundation, or anywhere within the zone of influence of the levee	Could cause the levee embankment or flood wall to collapse (see Section 4.1, zone of influence).
Directional drilling	Could fracture the levee's impermeable blanket and cause severe seepage issues. Exercise extreme care and monitor continuously during the drilling process.
Pipes passing through the levee	See Section 4.15.
Degrading the levee crown for road, railroad and highway crossings	Require additional flood fight measures to address any areas where the levee was degraded. See Sections 4.2 and 4.14, and Chapter 6.
Encroachments that may cause hydraulic or hydrostatic problems	
Railroad, highway crossings, utility crossings, boat ramps, docks and buildings, bridge piers	Affect a stream's flow distribution at out-of-bank (flood) discharges. Undesirable flow distributions and patterns may result in scour at the levee, which, in turn, may result in project failure at less than design stage and discharge. Undesirable flow distributions and patterns may also increase interior ponding areas or otherwise inhibit interior drainage. All structures or facilities located in the floodway and bypasses should be investigated for hydraulic impacts on flood flow distributions.
Bridges	If not built high enough, may accumulate debris, which could place additional pressures on levee systems.
Any work done in the floodway during the flood season	May impair channel capacity, threaten the ability of the levee to function as intended, and put the people and equipment working in the floodway at risk.
Boat docks	Could interfere with the design channel capacity. Can also threaten levee integrity if they penetrate the levee.
Swimming pools, boring holes, power poles, wells and irrigation ditches located close to the landside levee toe	Could increase the hydraulic gradient and put additional pressure on the levee.
Uncontrolled or improperly controlled groundwater	Could, by hydrostatic pressure and seepage, cause piping, heaving or reduction in the stability of excavation slopes or foundation soils such that they become unsuitable for supporting the structure.

In addition, some encroachments may:

- puncture the levee's blanket, facilitating piping of fine material (when a levee is designed, a low permeability layer is typically placed on the surface of the levee. This layer, which is sometimes made of clay, is called the 'blanket')
- hinder levee inspection and regular maintenance, such as mowing
- complicate future levee improvements or enlargements, especially if encroachments have been allowed too close to the levee
- require immediate repair.

The examples in Boxes 4.12 and 4.13 illustrate just a few of the challenges that are being faced worldwide regarding encroachments on and near levees.

Box 4.12 Encroachment control systems may prevent big problems

Figure 4.7 ***Unauthorised pipe installed by local business had to be abandoned and sealed at its expense***

In Corning, New York, a local business chose to create a drain for its newly constructed parking lot by installing a pipe through the embankment of the adjacent levee (Figure 4.7). The business assumed the levee was high ground and installed their structure without getting a permit. Signage and other levee awareness communication procedures can help prevent this from occurring. The local business interested in installing the pipe through the levee should have worked with the levee sponsor to assure that their design and method of installation would not affect the levee's integrity or its ability to reduce flood risks. In this case, the local business had to pay to abandon the newly installed drainage structure by sealing the ends and grouting the pipe.

Box 4.13 Why sponsors should follow the conditions listed in encroachment permits

In 1996, the sponsor that partnered with the US Army Corps of Engineers (USACE) on a project on the Truckee River requested to do work on Center Street Bridge in the floodway during the flood season. USACE, Sacramento District, advised against this. A long legal paper trail ensued with the sponsor finally indicating that they would do minimal work during the flood season and be able to get all the equipment out in a timely manner. On 1 January 1997, flooding of the Truckee River caused the formwork and all the work that had been done on the bridge to be swept away. Luckily, there was no loss of life, and the formwork that was washed downstream did not cause a breach in the levee. As a result of this incident, USACE, Sacramento, now adds a condition to almost all encroachment permits that prohibits work in the floodway during the flood season, except in rare circumstances.

Figure 4.8 is a good example of a pipe encroachment passing through a levee with a positive closure on the waterside hinge.

Figure 4.8 ***Pipe encroachment passing through the levee, Butte Creek levee, California, USA***

Elements of an effective encroachment control program

The following are the recommended elements of an effective encroachment control program:

1 Concise guidance as to what constitutes an encroachment:

  a Clearly communicate to the encroachment permit applicants when permits are required and how to submit an application, and what types of encroachments are likely to be permitted (see Box 4.14).

  b Keep track of any changes in requirements for encroachments (this may help the levee operator answer questions about why a certain encroachment, such as a fence, may have been allowed previously but is no longer allowed).

  c Give permit applicants feedback on their application in a timely manner. Let them know where their application is in the review process.

  d Clearly communicate to anyone who lives near a levee or who may be considering performing work near a levee the zones where permits are required.

Box 4.14 The Netherlands distinction: needed versus desired encroachments

In the Netherlands, many individuals, real estate firms, water companies, gas companies, energy companies and telecommunication companies want to encroach levees with new homes, pipes, and cables. The encroachment review process decides whether each encroachment is needed or not by determining if it has 'great society importance' or its position in the levee seems logical from a spatial planning point of view. If the encroachment is important (and there is no other suitable alternative location for it), it is accepted under strict rules set forth in an encroachment permit. These rules ensure that the encroachment does not influence the safety of the levee. Such encroachments are not accepted without an encroachment permit, and when they are permitted, every attempt is made to keep them outside of the levee prism, either by over-enlarging the levee or adjusting the minimal prism by inserting a sheet pile wall or other reinforcement. In general, local wishes are only permitted outside the levee prism (including a zone for possible future levee enlargements). Greater needs are sometimes permitted in the original prism and/or the prism is adjusted by installing special structures.

2 An effective enforcement system to prohibit any encroachments that could pose a threat to levee integrity.

  a Establish incentives for the levee operator and potential encroachment owners to keep their levee free of unpermitted encroachments. For example, governments may provide rehabilitation assistance to levee owners that maintain certain standards (USACE, 1965).

  b Regularly monitor the levees to keep track of unpermitted activities. Other entities may be called upon to be the extra 'eyes and ears' of the monitoring program (eg law enforcement, code enforcement, emergency responders, local contractors), as some systems may prove difficult to monitor closely.

  c Understand the legal means available for removing or forcing the removal of unpermitted encroachments.

3 A clear understanding of the maintainer's jurisdiction, the zone of influence of the levee, and the features included in the levee (such as seepage berms, stability berms, flood walls, pump stations, coastal levee protection structures, pipes, and drainage features).

Maps and diagrams are effective tools for visually displaying this information (see Boxes 4.15 and 4.16).

4 A geospatial database for keeping track of all historical and existing encroachments, including records of drawings showing cross-sections of all historical and existing encroachments.

See sample cross-sectional drawings in Box 4.16 (reinforcements added to the levee to accommodate encroachments). As indicated by the example, it is important to keep clear records that indicate when and where reinforcements have been installed in the levee to accommodate encroachments.

The data should indicate the following:

- if existing encroachments have permits or not
- whether permitted encroachments meet the conditions specified in their permits
- how historical encroachments were abandoned
- the latest inspection records that are available (for example, records associated with the video inspections of any pipes).

5 A way of addressing cumulative impacts.

For example, one boat dock may not be a problem, but several boat docks lining the levees of a narrow channel could affect the channel capacity.

6 An inspector on site who could determine if the integrity of the levee is not compromised by any encroachment activities.

7 A clear definition of the roles and responsibilities of the levee operator, any related regulating agencies and the owner of the encroachment, should any encroachment-related issues arise.

Box 4.15 Key maps and diagrams help define jurisdictional area, zone of influence and zone of disturbance

A maintainer's jurisdiction (ie the area within their control) is the area within their ownership or right of access and operation. Maintainers are better able to determine the encroachments that are within their jurisdiction and those that could potentially affect levee integrity by referring to:

- **the O&M manual**, which sometimes states the area (eg a certain distance from the toe of the levee) in which the maintainer is entitled to operate
- **maps indicating the area over which the maintainer's have jurisdiction** (see sample map in Figure 4.9)

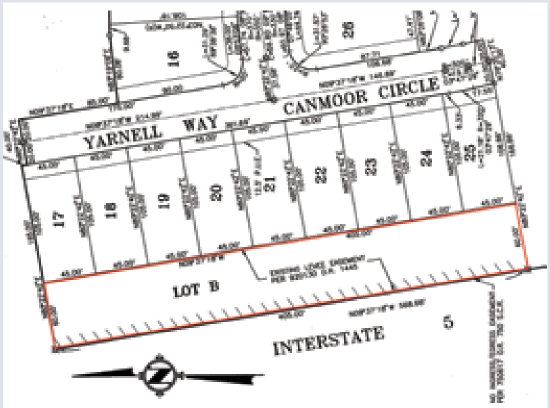


Figure 4.9 Sample map indicating the area over which the maintainer has jurisdiction

- **cross-sectional drawings** (called as-builts) that are generated by the construction company after a levee is built (typical in the USA). Note that it is important when considering an encroachment to take into account any additional features such as berms (see Figure 4.10) and profile margins for future levee improvements

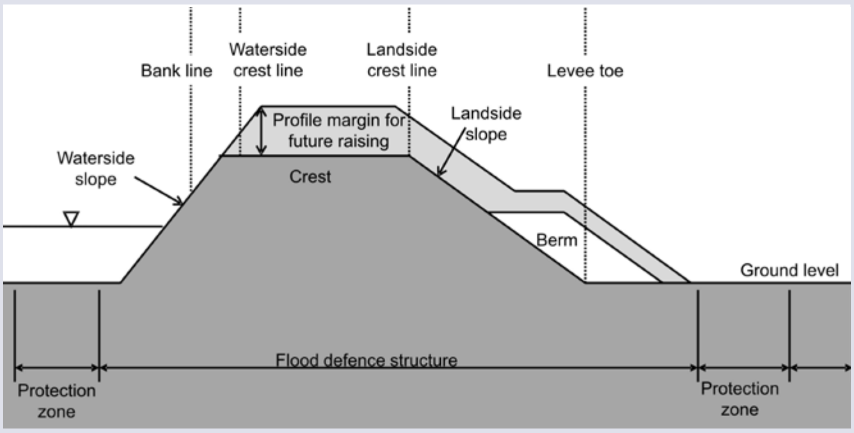


Figure 4.10 Sample cross-section of levee including stability berm and profile margin for future raising of levee

- **the zone of influence**, the area within which actions can affect the levee. In the Netherlands a good rule of thumb for calculating this is a factor multiplied by the height of the levee(for example a distance five times the height from both toes). Note that a detailed analysis of the influence zone is preferred because the five times the height rule of thumb is not suitable for soft (organic) soils or shallow silt/sand layers. A more detailed analysis of the influence zone for each failure mechanism may also be used
- **the zone of disturbance of the encroachment**. In the Netherlands, this is based on the type of encroachment. For example, for pipes the calculation for the area of a zone of disturbance may include what the pipe is carrying (eg gas, water, oil). In the Netherlands, a permit for the encroachment will not be issued if the zone of disturbance of the encroachment overlaps with the zone of influence of the levee (see Boxes 4.6 and 4.17).

Box 4.16 ***How the Netherlands determine the minimum cross-section that should be maintained***

In the Netherlands, levee operators/maintainers consider all the possible ways the levee could fail when determining the cross-section that should be maintained as well as where encroachments are prohibited, permitted on a temporary basis, or permitted under certain conditions. The minimum levee cross-section that should be maintained is the most conservative combination of the following:

- the actual current ground level (indicated by the dotted line in Figure 4.11)
- the minimum distance from the levee that has been calculated to be at a higher risk for encroachments that lead to piping problems (this is sometimes estimated as 15 times the height of the levee from the opposite levee toe, as shown in Figure 4.11, or by a more detailed analysis)
- the minimum levee prism required to avoid slope stability problems (indicated by the solid line in the diagram)
- the minimum levee cross-section required to minimise the risk from other threats to the levee, such as overtopping
- the margin for future levee improvements (only temporary encroachments may be permitted in this area).

All these points are taken into account in combination with the potential zone of disturbance of the encroachment to determine how close the encroachment can be to the levee.

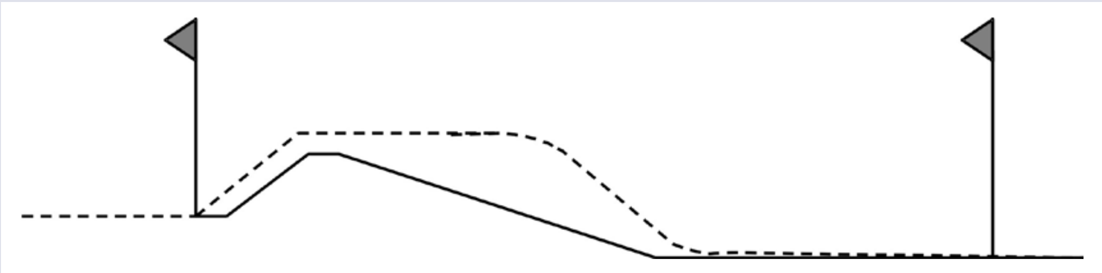


Figure 4.11 ***How to determine the minimum cross-section***

Box 4.17 ***Reinforcements added to a levee in the Netherlands to accommodate encroachments***

The house in Figure 4.12 was spared during a levee improvement project by building a reinforced structure (such as a sheet pile wall) behind the brick wall, placing the home outside the levee's zone of influence.

Figure 4.13 shows possible reinforcements that could be made to a levee to accommodate a pipe. Note that the levee prism or sheet piling must not be affected by the zone of disturbance of the pipe, according to criteria used in the Netherlands.

Figure 4.12 ***Levee along a river – historical house on the riverside of the levee, the Netherlands***

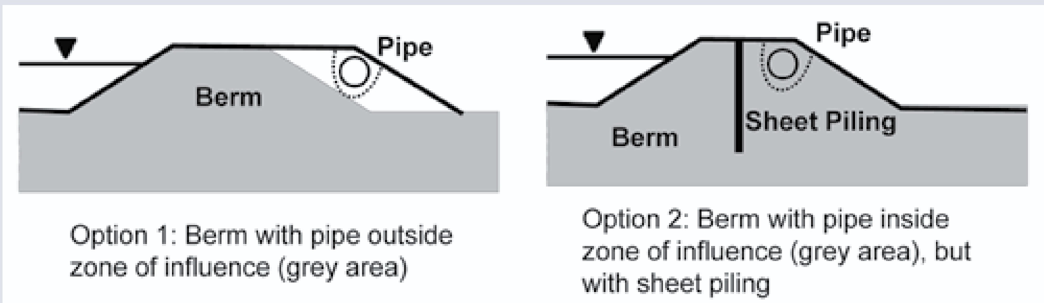


Figure 4.13 ***Possible levee reinforcements to accommodate a pipe***

1 2 3 4 5 6 7 8 9 10

Box 4.18 shows results from testing performed with a wave overtopping simulator, which measures damage occurring around levee encroachments (in this case, stairs) during simulated overtopping.

Box 4.18 Testing shows encroachments can compromise levee resilience

Figure 4.14 Testing of resilience on levee with staircase, the Netherlands (courtesy Y Provoost, RWS, the Netherlands)

In the Netherlands, wave overtopping simulator tests were done on two levee sections with stairways and on a levee section with good turf (sod cover). Both stairways showed erosion at the interface between the stairs and the levee, whereas the levee with good turf demonstrated much less erosion when tested with the same amount of water overtopping the levee. A comparison was also made between the two stairways. One stairway had been treated with herbicides and had no grass along either side. The lack of grass, combined with an unstable foundation, led to extensive erosion on either side of the staircase (Figure 4.14).

Repairs of encroachment issues

Damaged encroachments, encroachments that were installed without permits, and encroachments that have not been maintained in accordance with the conditions of their permits may cause problems on the levee.

Some guidelines for repairing problems with encroachments are as follows:

1. When making repairs, the owner of the encroachment should work closely with the levee maintainer and should make every effort to make all repairs in accordance with the permit.
2. Use repair methods that do not affect the integrity of the levee.
3. Repair any issues identified as quickly as possible. Early repairs may prevent issues from getting worse.
4. Become familiar with the permitting procedures in the area if planning to repair an encroachment problem. Different countries and different regions may have different rules governing when a permit is needed. Some regions require a separate encroachment permit for every action on the levee. For example, one permit may be required for the installation of the encroachment and a separate one for making repairs.
5. Adjust the timing of repairs to the unique needs of each situation. For example, damaged electrical cables that provide electricity to a village may need to be repaired during the flood season even though repairs during that time period may generally not be allowed.

Box 4.19 is an example of how a complex levee encroachment can successfully be approved by a review and permitting process.

Box 4.19 ***Significant encroachment successfully installed: applicant was prepared, application review process was effective***

In Riverside, New York, USA, a bridge was installed through a levee as part of a railroad crossing (Figure 4.15). A potentially challenging permit situation (because of the risk of substantial damage to the levee) was made easier by the applicant meeting all design specifications needed to avoid any negative impacts. The permitting process went very smoothly because there was an effective review process in place and the application was thorough and complete.

Figure 4.15 ***Permitted bridge installation in Riverside, New York, USA***

When repairs are beyond O&M

If the presence, or removal, of an encroachment causes such serious damage that the levee section needs to be rebuilt, a designer should be consulted (see Chapter 9). The designer should be hired by the person who placed the encroachment on the levee, or the person responsible for the encroachment.

4.5 VEGETATION MANAGEMENT

Vegetation management is the systematic and continual control of vegetation on and near levees. The primary purpose of vegetation management on a levee is to preserve levee integrity, performance, visibility, and access in the interest of public safety.

Vegetation, both cultivated and naturally occurring, has been present along the banks of rivers and other waterways long before levee systems were constructed. Woody vegetation, which consists of plants having hard lignified tissues or woody parts, especially stems, is commonly both inevitable and persistent, and must be managed if levee integrity and reliability are to be maintained. Vegetation management, on and near levees, is focused on three levee performance objectives:

1 Protecting the levee from external erosion.
2 Maintaining adequate access and visibility.
3 Preventing the development of vegetation-induced damage or defects.

Vegetation management practices should also seek balance and address sustainability issues by minimising adverse environmental impacts and considering objectives for habitat, aesthetics, and community values.

The presence of woody vegetation introduces uncertainty in the reliability of levee performance and the associated scientific issues have not been fully resolved (for example, see the literature review by Corcoran *et al*, 2011). So different countries have adopted, or intend to adopt, a range of practices for the management of woody vegetation on levees. Many of these practices are common to all countries, but there are interesting differences in the primary concerns that were considered when developing the management practices (eg visibility and access in the USA, seepage and internal erosion along decaying tree roots in France, tree blowover in the Netherlands and France, and functional and operational impacts in Germany and the UK).

There are two primary situations that policies address:

1 Where the levee has been kept free of woody vegetation.
2 Where woody vegetation is present on the levee.

In the first situation, nearly all of the guidelines and standards worldwide recommend only a trimmed grass and turf cover, unless specifically designed planting berms or other measures are incorporated in the levee design. These guidelines and standards include procedures for the care and trimming of grass covers and generally recommend that such levees remain generally free of woody vegetation.

The second situation is more challenging to manage, especially where the vegetation has been allowed to establish and mature, and where such vegetation is considered to be of benefit to the environment, community values, or community aesthetics. There are additional concerns associated with financial constraints, and about how to properly remove woody vegetation without harm to levee integrity. Many countries have allowed woody vegetation to exist on levees for decades and are now addressing these situations. Some agencies allow existing woody vegetation to remain if it is trimmed and thinned, or where oversized levee sections and berms can allow such vegetation to remain without significantly affecting levee integrity.

4.5.1 Protecting the levee from external erosion and maintaining adequate access and visibility

Exposed bare soils in levees are vulnerable to external erosion induced by:

- precipitation
- wind
- traffic (vehicular, pedestrian, and animal)
- scour
- wave erosion
- overflow/overtopping.

Such erosion may lead to many distress mechanisms and potential levee failure modes as described in Section 3.5. Various revetment systems, such as rock armour, asphalt, and concrete layers, have been used over time and in various countries to protect levee surfaces against external erosion. These are described in more detail in Section 4.7 and Chapters 8 and 9.

However, the most common, resilient, and cost-effective surface protection measure against external erosion is to maintain a robust grass covering on the surface of the levee (see Figure 4.16). When properly maintained, grasses and turf (see Box 4.20 for definitions) have been found to be both useful and cost-effective in protecting levees against external erosion.

Figure 4.16 New grass cover growing on recently constructed Bear River setback levee, California, USA (courtesy California DWR)

Box 4.20 ***Grasses and turf***

> Grasses are defined as any plant of the family Gramineae having jointed stems, sheathing leaves, and seed-like grains. Many grasses are cultivated in lawns, used as pasture for grazing animals or cut and dried as hay. Turf (also known as 'sod') is generally defined as the condition where the roots of grasses have developed to the point where the root mass binds the soils to form a stable mat that effectively resists erosion.

In addition to grasses and turf providing a surface that is resilient to external erosion, they have the added benefit of providing an ecosystem-friendly and aesthetically pleasing surface and environment.

A properly maintained grass cover does not inhibit physical or visual access and is resilient to necessary traffic. The most effective grasses are those that form a dense turf and cover. The preferred species to maintain an adequate grass cover for a given location should be selected based on the climate and soils, and usually are species that are native to the region. In most cases, it is generally considered best to avoid using invasive species, even if they may be effective as a grass cover. Sometimes the use of invasive species is even prohibited by regulatory agencies.

Healthy grass cover typically takes four years to develop. Sandy soils on levees can impede the ability of the grass to grow or can cause grass only to grow on the top-most layer of the levee and not provide the necessary erosion protection. Pesticides or other chemicals in the water will also affect the ability of the grass to grow, or weaken the strength of the grass cover.

Trimming of vegetation

Adequate access and visibility are critical to levee performance and reliability. Good management practices ensure that vegetation does not threaten levee integrity by inhibiting or preventing:

- physical access necessary for effective maintenance, repair, and emergency operations
- visual access necessary for inspection. For example, burrowing animals are widely acknowledged to have an undesirable effect on levee integrity (see Section 4.6). Levee surface visibility and access is important to identifying and later managing these animals' burrows.

Proper vegetation management methods to address these objectives often include frequent mowing or trimming of the grass cover, trimming low-hanging tree limbs, and removing trees and other non-grass vegetation. Figure 4.17 shows how untrimmed grasses and woody vegetation limit visibility and access on a levee.

Figure 4.17 ***Examples of limited visibility and access on levees due to untrimmed grasses and woody vegetation in East St Louis Levee, Illinois, USA (courtesy L F Harder) (a), and France (courtesy M Vennetier) (b)***

Effective vegetation management methods associated with trimming grass include:

- mechanical mowing, remote-controlled mechanical mowing and hand mowing
- application of herbicides
- grazing by livestock
- burning.

Good practices for the maintenance of grass are designed to ensure adequate erosion resistance by selecting grass species that are able to resist the erosional forces the levee is designed for, and by developing and maintaining adequate grass density and root depth.

Box 4.21 discusses recent research into grass mowing.

Box 4.21 ***Research into grass mowing***

Grass maintenance aims to increase the erosion resistance of the grass cover by preventing woody vegetation and by encouraging the growth of a diverse mixture of grass species with fully developed root systems. Though optimum grass maintenance will depend on the species and conditions (such as climate and soil), in general, research shows that grazing by sheep or more frequent mowing produces a superior grass cover. A recent UK study by the Environment Agency considered several parameters of turf management practices at three sites in the Anglian Region in the UK (Smith *et al*, 2009). The study found that optimal cutting frequency (three or more times per year) improved surface soil strength and erosion resistance. A Dutch study revealed that grazing by sheep leads to a dense and diverse mixture of grass species, and that mowing leads to deeper root growth (provoked by a slow decline of nutrients) (STOWA, 2013).

Figures 4.18 to 4.21 illustrate the methods of mechanical mowing, applying herbicides, grazing and burning, and Table 4.8 summarises the advantages and disadvantages of each. Mowing may be supplemented by chemical control, as needed, subject to any local restrictions. In Europe, the use of chemical control near rivers is completely prohibited. Close mowing (between 5 cm and 10 cm) of the entire levee is recommended for inspections. Close mowing of the landside slope is often recommended to help effective monitoring during flood events. Waterside mowing should be suspended when flooding is imminent, as somewhat longer grass will provide greater erosion protection.

Table 4.8 ***Vegetation trimming methods***

Method	Advantages	Disadvantages
Mechanical mowing (Figure 4.18)	***Technique*** • very efficient • can be supplemented by selective hand mowing in sensitive environmental areas or on levees with obstructions • useful for moist or dry grass and woody vegetation of limited size. ***Results*** • trimmed height of grass – can be selected by adjusting the height of the mower • excellent for controlling vegetation for access, visibility, and erosion protection.	***Technique*** • limited reach or access for steep slopes or for levees with obstructions such as trees and buildings • equipment can be costly to purchase, operate, and maintain • hand mowing typically takes more time than mechanical mowing and presents additional safety risks to the operator. ***Results*** • leaving cuttings behind can cause bare patches.
Application of herbicides (Figure 4.19)	***Technique*** • very effective in tight or limited space/ working area where mowing or grazing is not possible. ***Results*** • can target specific vegetation types without harming desirable grasses • leaves behind existing beneficial vegetation cover.	***Technique*** • has environmental consequences and limitations in application (prohibited within 5 m of the waterway in most parts of Europe) • can be expensive to purchase and apply. ***Results*** • does not trim existing overgrown vegetation – best used as a supplement to other methods such as mowing or grazing.

Table 4.8 *Vegetation trimming methods (contd)*

Grazing by livestock (Figure 4.20)	***Technique*** • efficient method of trimming grasses while still maintaining both cover and a root system • useful for both moist and dry grasses and brush • use of goat and/or sheep herds can result in trimming of woody vegetation and vines in addition to trimming grasses • careful shepherding is used to ensure that livestock only trim vegetation in desired areas. ***Results*** • manure from livestock may act as a fertiliser for growing grasses and other ground cover.	***Technique*** • careful shepherding is necessary to avoid overgrazing, which can damage grass cover and levee surfaces • sheep and goat herds are not always available or allowed in urban environments. ***Results*** • cattle, if used, may cause surface damage and surface rutting by cattle trails (see Section 4.8) • there may be water quality issues in some circumstances • animals may disturb the surface slopes and cause nutrients from the levee to enter adjacent bodies of water.
Burning (Figure 4.21)	***Technique*** • very cost-effective way to remove dry grass and brush from levee slopes. ***Results*** • method that probably best reveals the condition of earthen levee slopes and surfaces.	***Technique*** • smoke from burning has environmental consequences and is not allowed in many areas. ***Results*** • reduces the density of cover – removes most grassy or turf cover, may also reduce density of enduring live plants, reducing potential erosion resistance • not effective for moist or wet grasses • some vegetation (eg vines) are not susceptible to burning • burning may change soil characteristics and alter the local ecosystem relying on poorer soils.

Figure 4.18

Mechanical mowing of grasses on California, USA, levee slope in dry season (courtesy California DWR)

Figure 4.19

Mechanical application of a broadleaf herbicide on a grass levee slope

Figure 4.20

Grazing by sheep and goats on moist grass and brush on levee in Sutter County, California, USA (courtesy L F Harder)

Figure 4.21

Burning of levee slope in California, USA (courtesy L F Harder)

4.5.2 Preventing the development of vegetation-induced damage or defects

A levee failure can be caused by a deterioration process or a damage mechanism. Deterioration processes (eg blowover, overturning of trees) can lead to a damage mechanism (eg external erosion), which can then cause a levee breach. Damage mechanisms can lead directly to a breach. While slow deterioration processes may be able to be managed or mitigated before serious damage occurs, damage to the levee that may result in a breach should clearly be prevented.

Untrimmed woody vegetation, in addition to potentially blocking levee visibility and access, may in some cases induce various types of deterioration. This deterioration may contribute to damage modes that significantly affect levee integrity. While the science of this process may yet be unclear and the seriousness of the deterioration may vary significantly for different conditions, two things are clear:

1 It is important to prevent the development of vegetation-induced damage and defects.
2 The response to each situation should take into account specific factors such as location, species, soil type etc.

The potential deterioration processes and damage mechanisms are summarised in Table 4.9. Chapter 3 provides a fuller discussion of potential levee damage modes.

Table 4.9 Summary of potential deterioration mechanisms associated with woody vegetation on levees

Deterioration process	Role of woody vegetation	Potential levee damage mechanisms affected (see Chapter 3)
Blowover/overturning	The overturning or blowover of a large tree may remove a large section of a levee or adjoining ground during a flood event. If on the waterside, the resulting pit may leave the levee susceptible to scour.	External erosion, slope instability, and internal erosion (caused by through-seepage, underseepage, and piping)
Root penetration	Roots, especially when decayed, may alter soil permeability or concentrate seepage along root paths.	External erosion and internal erosion (caused by through-seepage, underseepage, and piping)
Woody vegetation weight and wind loading	The adverse effects of woody vegetation weight and wind loading is transferred to a levee slope.	Slope instability (slip surfaces may be deeper than extent of root penetration)
Scour flows	Woody vegetation may cause concentrations or eddies in waterside or overtopping flows.	External erosion
Burrowing	Woody vegetation may attract burrowing animals into a levee.	Internal erosion
Discouraging adequate growth of grass and turf	Woody vegetation may prevent adequate growth of grass and turf by blocking sunlight, absorbing nutrients and moisture or releasing chemicals that act as herbicides, resulting in bare, exposed soil on levee surfaces.	External erosion
Damage to the revetment	If the revetment was not designed for vegetation, the growth of roots and stems may move and loosen the stones, or rigid levee protection elements such as asphalt, grouted stone, or concrete slabs, thus affecting the revetment's interlocking characteristics.	External erosion

While there are numerous potential deterioration mechanisms associated with woody vegetation on levees (see Table 4.9), there are also some benefits that woody vegetation may provide to levees (see Box 4.22). It is important to bear in mind the risks associated with removing all woody vegetation from levees or banks that are vegetated. If vegetation is to be removed, a long-term plan should be implemented that considers river morphology and the potential for increased scour and erosion. Increased risks to levee integrity is one of the reasons that some agencies take a phased approach to the removal of woody vegetation (see Box 4.23, and Vennetier *et al*, 2011). Woody vegetation may also provide soil reinforcement to a slope that may improve slope stability (Box 4.22). Any benefits afforded to the levee by woody vegetation should always be weighed against the potential deterioration mechanisms they could cause. Where such risks exist, a vegetation management strategy should be created that make sense for that levee.

Box 4.22 ***Woody vegetation observed to improve slope stability***

It has been well established through conventional slope stability analyses and instrument monitoring that woody vegetation improves the stability of forest and other vegetated slopes through soil reinforcement and the reduction of pore pressures through evapotranspiration. Studies by Wu *et al* (1979), Ziemer (1981), and O'Loughlin and Ziemer (1982) showed that the reinforcing effects of tree root systems could be represented by a higher apparent cohesion, and that the benefits calculated by conventional analyses matched observed behaviour and that there was an increase in sliding after trees had been felled. Studies by Chok *et al* (2004) and Kokutse *et al* (2006) employed finite element analyses for generalised soil slopes and found similar benefits, even when 3D effects were analysed. Research has shown similar beneficial influences of tree root systems on the stability and reinforcement of streams and riverbanks (eg see Pollen and Simon, 2005, Pollen-Bankhead and Simon, 2010, and Simon *et al*, 2011).

An exception is when the waterside slope already has slope instability issues due to steep slopes beneath the water level, in which case large (greater than 30 cm in diameter) or medium-sized trees (greater than 10 cm in diameter) at the waterside toe were found to decrease slope stability (Folton *et al*, 1998). In France, specific engineering designs were made for vegetated bank stabilising solutions for this case (Bonin *et al*, 2013) using dwarf willows or frequently coppicing trees as shown in Figure 4.22 (Vennetier *et al*, 1998). In California, rip-rap berms have been added to stabilise such eroded slopes.

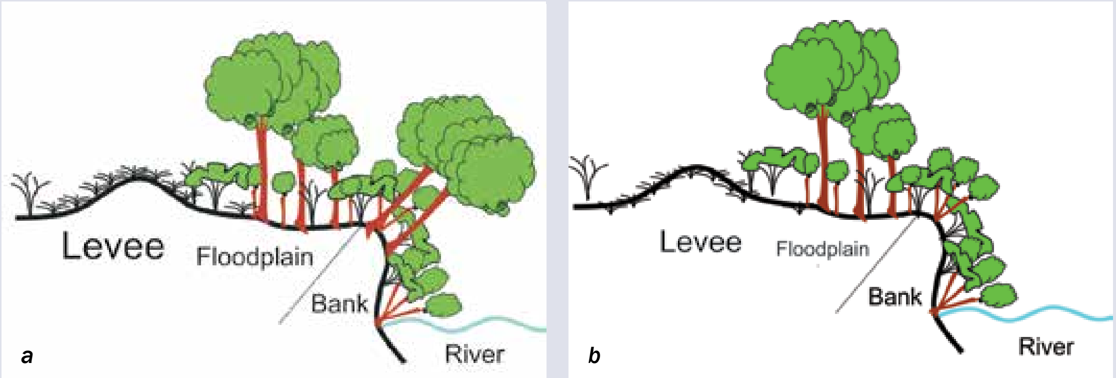


Figure 4.22 ***The French solution to the situation (a) is to only allow the situation (b) with trees low to the ground above the precipice, such as dwarf willows or frequently coppiced trees***

Improper removal of woody vegetation may also increase risks associated with seepage and stability, and many agencies have developed standards and guidelines for the appropriate removal of woody vegetation when necessary (see Box 4.23). Concerns about potential deterioration of levee integrity have led many levee owners and agencies that regulate them (eg USACE and Rijkswaterstaat) to establish policies to keep the levee and the adjoining land free of woody vegetation and allow only frequently trimmed grasses and turf (see examples in Box 4.24). In some parts of the world, this approach has been implemented with great success. Some standards requiring levees to be generally free of woody vegetation go back for more than a century (eg in 1737 City of Murcia, Spain Levee Ordinances, and in 1905 California Levee Specifications).

However, this is a complex subject and there are many different opinions and approaches to vegetation management. Though each agency develops and implements vegetation standards in different ways, and nearly all guidelines and standards generally recommend levees be covered only with maintained grass, many agencies make provisions for exceptions. For instance, all levee regulatory agencies for which information was gathered will allow woody vegetation where engineering accommodations have been provided to assure that such vegetation does not threaten levee reliability and performance. Examples of such accommodations are:

- a significant enlargement of the levee cross-section
- installation of a cut-off wall or some type of barrier element to serve as a functional replacement for a vegetated levee cross-section.

These agencies also have a process for granting exceptions to standards.

Box 4.23 *Removing woody vegetation from levees*

Many agencies agree that the most effective approach for removing woody vegetation is to cut and remove the vegetation, remove the stump and all significant roots by excavation, and then restore the excavation with soil and compaction similar to that of the adjacent, undisturbed cross-section.

However, different agencies and operators have different judgments regarding how much of the root system is significant. Agencies also have varying degrees of concern for the potential deterioration associated with dead roots left behind in a levee. The range of approaches to removal is illustrated through the following:

- German standards (Section 14.2.7, DIN 19712:2013-01, DWA, 2013) require the complete removal of the stump and all roots and reconstruction of the cross-section to current design standards. Exceptions under certain constraints are possible.
- Dutch guidelines (STOWA, 2000) recommend that when trees are removed from levees or dikes, that the roots should be removed as much as possible and replaced with compacted soil. If this is not done, then the probability of seepage, underseepage, and stability problems are believed to increase. However, while it is generally advised to remove the larger roots around the trunk and to replace them with compacted soil, actual practice sometimes results in allowing the residual stump and roots to be left in place after the tree is cut flush to the ground
- guidelines issued by the USACE (2009) recommend the removal of the trunk, stump, rootball and all roots greater than 13 mm in diameter within the levee. The resulting pit produced from the excavation is then filled with soil, meeting both the original soil and compaction specifications or matching adjacent soil and compaction conditions. However, in practice it may be difficult for levee maintaining agencies to comply with the guidance, because it may entail the removal and replacement of a substantial portion of the levee, disturbing utility lines that the roots may be penetrating and destabilising the foundations of major roadways when the levee crest also serves as a road. The costs of compliance may also be uneconomic
- design criteria set out by the State of California (FloodSAFE California, 2012) has a minimum requirement that the root ball and all roots larger than 38 mm in diameter that are within a metre of the perimeter of the tree trunk shall be removed and replaced with engineered fill using appropriate placement and compaction methods
- in France it is recommended to leave both the stump and root system alive to allow for new growth. This helps maintain the life and integrity of the root system if the roots cannot be removed and the levee rebuilt in the near term (Vennetier *et al*, 2011). Over time, root systems are expected to eventually die, and additional measures, such as rebuilding portions of the levee, adding a cut-off wall, or widening the levee on the landside, may be needed (Pinhas, 2011). Research has shown that tree roots will detect defects such as cracks in a slurry wall and will penetrate the wall if given the opportunity (Harder *et al*, 2011). This is anticipated when reinforcing measures are selected (Vennetier *et al*, 2011). In the French approach, a distinction is made between clay soils and cohesionless soils. The risk of piping is much larger for clay soils. In general, to facilitate the inspection, trees are not permitted on or near to the crest, or near the toes.

Also, it is often important to remove the cut log and excavated root ball from the levee, particularly on the waterside. Such debris can float downstream during floods and contribute to logjams and threaten bridges and other structures. In addition to simply removing the debris, common options include crushing or shredding the debris or attaching the cut tree trunks to the waterside levee slope by cables to provide aquatic habitat.

Box 4.24 *Examples of policies prohibiting woody vegetation on or near levees*

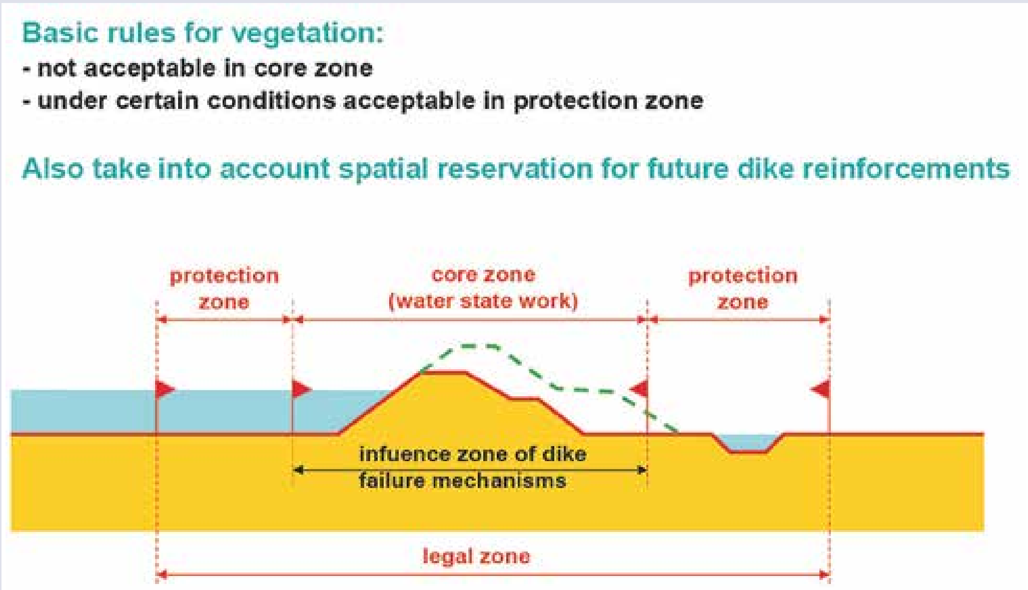


Figure 4.24 ***Basic rules proposed for simple assessment of vegetation on primary dikes in the Netherlands (courtesy Harry Schelfhout)***

Box 4.24 Examples of policies prohibiting woody vegetation on or near levees (contd)

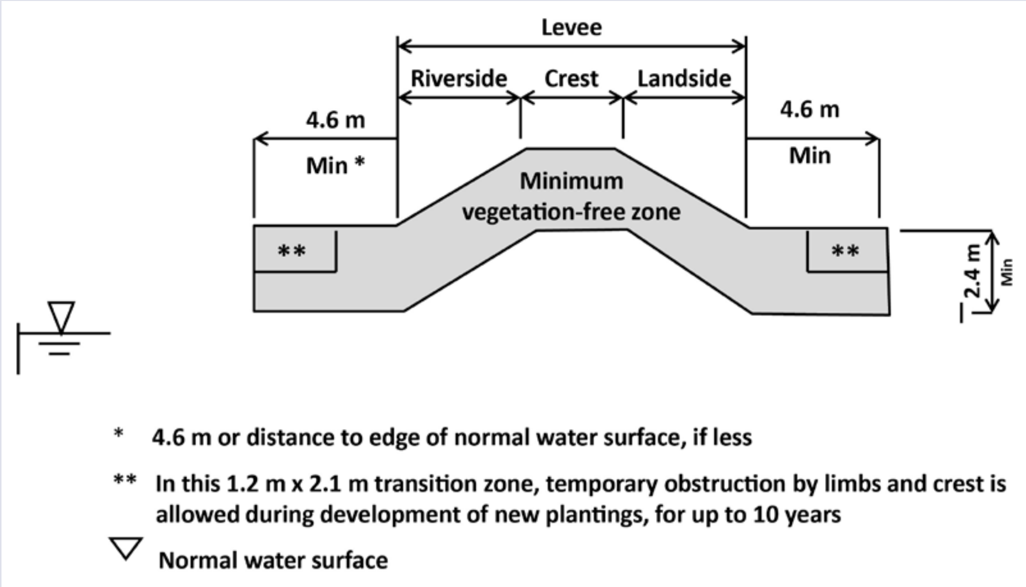


Figure 4.23 Minimum vegetation-free zone on and adjacent to levees recommended by the US Army Corps of Engineers (from USACE, 2009)

4.5.3 Managing existing woody vegetation to minimise environmental impacts

While the preferred or recommended condition for a levee, from an engineering perspective, would be for it to be free of woody vegetation, it is also recognised that, for a variety of reasons, this condition may be difficult or impractical to achieve where mature woody vegetation is already established. In many locations, trees and other woody vegetation, together with associated habitats, have become established on levees. The reasons for the existence of woody vegetation on levee systems include the following:

- the age of the levee. Most levees were constructed many years ago. Some are hundreds of years old and, over time, vegetation has been allowed or its prohibition has not been enforced
- limitations of resources during times when levee maintenance is a low priority (eg in Europe during and post-WWII)
- difficulties in accessing slopes with mechanical equipment
- encouragement of woody vegetation to provide additional erosion protection
- encouragement of woody vegetation for fisheries and habitat
- encouragement of woody vegetation for aesthetics and recreation
- beliefs that woody vegetation provides benefits that outweigh the risks
- concerns that removing existing woody vegetation may cause harm to levee integrity.

Unfortunately, for many of the concerns previously outlined in Table 4.9, formal research and documentation of field performance is lacking. Current understanding of how woody vegetation impacts levee performance is based on two types of knowledge:

1 Observation and experience.
2 Formal research.

Both types of knowledge are fairly limited and sometimes contradictory because of the wide range of variables involved (eg soils, vegetation, climate, and river characteristics). For example, in one part of the world, woody vegetation on the waterside slopes of levees may be discouraged because of a concern for erosion and tree blowover. However, in other parts of the world, trees and other woody vegetation have been intentionally planted on waterside slopes to provide erosion resistance, habitat enhancement and social amenities. Even in areas where there is agreement that woody vegetation is detrimental to levee integrity, it is often not possible to quantify those potential impacts and risks.

As of 2012, research into tree root architecture and behaviour is being conducted by various agencies and universities around the world (eg the US Army Corps of Engineers Engineer Research and Development Center (ERDC), California Levee Vegetation Research Program, University of California, IRSTEA Aix, and STOWA, the Dutch Foundation for Applied Research in Water Management). Findings are preliminary. However, much of the research indicates that there is a large variability in tree root architecture and potential impacts on levee integrity will depend upon tree species, soil conditions, climate, age and health of the tree, and other factors. Figure 4.25 illustrates some of the diverse patterns of tree root architecture that may be encountered in levees. Studies by Zanetti (2010), Vennetier *et al* (2011) and Chung *et al* (2013) provide guidance on tree root characteristics within levees based on environmental factors present.

At the same time, many countries recognise that woody vegetation that has matured on levee systems now provides important environmental and landscape aesthetics and that there are many concerns about simply clear-cutting existing woody vegetation. Chapter 2 outlined many of the environmental principles that should be considered in managing the maintenance of a levee. Among these was the need to consider multiple uses and benefits and to use balanced and flexible approaches in maintaining the integrity of levees. Several countries allow already established woody vegetation to exist under certain circumstances, and accept both the risks and the benefits for doing so. Boxes 4.25 and 4.26 describe general guidelines used by Dutch water boards to assess whether trees on levees in the Netherlands need to be removed.

Box 4.25 Philosophy of management of existing woody vegetation on levees in the Netherlands

"It is inherent in evaluating existing vegetation that the presence of the vegetation entails an extra risk, but that this (for the lifespan of the vegetation) is acceptable if certain conditions are met. The presence of vegetation could lead to damages to the flood defence, but these damages do not immediately result in a failure of the flood defence. This implies that the presence of vegetation is undesirable from a safety perspective, but that existing vegetation can be tolerated if it does 'not immediately result in a failure of the flood defence.'"

Source: STOWA, 2000

a shallow system with many horizontal roots of all sizes including big and long ones and no taproot

b taproot system with one very big taproot (length 2.5 m, diameter = 50 cm at 1 m) a secondary smaller taproot, and mainly small and short horizontal roots

Note

Only the central portion of the root systems are shown – the complete root systems extend to greater lengths (up to over 15 m from stump) and were found to densely occupy a volume of 10 to 25 m^3 for adult trees

Figure 4.25 Typical root system types

c mixed system with a few taproots and several large horizontal roots

d heart system with many small to medium roots in all directions (from Zanetti, 2010, Vennetier *et al*, 2011, courtesy IRSTEA Aix)

Figure 4.25 Typical root system types (contd)

Box 4.26 General guidelines used by regional water boards in the Netherlands

Current practice for assessing the impact of trees on levees (Figure 4.26) is to observe two basic rules:

- the tree, including its zone of disturbance (for example the hole after wind-throw), must be outside the minimum required levee prism
- the levee must be able to withstand possible additional further negative impacts, such as strong winds and shadows on grass vegetation.

The following additional criteria (STOWA, 2010) represent part of a general framework being developed to eventually include all types of levees. They are only applicable to levees where the body of water near the levee changes less than a metre during flood events (eg as in canals), and only apply to a single, individual tree, not a group of trees. These criteria, in the form of the following three questions aim to identify the most important risks and find the balance between 'reliable' and 'do-able' methods, guided by the amount of effort needed to assess a large number of trees one at a time. The criteria are:

1 Does the levee meet the safety standard without the tree? If the levee does not meet the safety standard without the tree, no additional effort is made to evaluate its condition with the tree (the presence of the tree will only increase the safety risks and never improve the outcome of the safety assessment).

2 Is the levee obviously stable? If stability is not at risk, the levee is considered OK regardless of whether or not the tree negatively impacts it. There is no need for further analysis. Stability plays no role if the levee height is less than 0.5 m or if the levee slope is flatter than 1V:8H.

3 Is the tree less than five metres high? It is expected that trees that are less than five metres high are not at risk for being blown over (based on soil and climatic conditions observed in the Netherlands). All trees that are less than five metres in height are permissible on the levee crest, landside slopes and near the levee.

Trees that do not meet these three criteria require a more detailed analysis to determine their potential effects on the levee. That analysis will determine the impact of the tree on the levee for each failure mechanism. Its approach will vary depending on the location of the tree (foreland, slopes, crest, berms etc). As a result of the analysis, trees will either be counted as a load or as a scouring pit after wind-throw is taken into account.

Water boards are also concerned about vegetation on levees because of experiences such as this one:

- in 2011 in the Netherlands, after a very dry period, water seeped through the dikes along the roots of a tree. At one location, sandbags were required and the water level had to be lowered to prevent a failure of the levee likely resulting in flooding.

Note
These trees were determined to be unacceptable because they don't meet the criteria, and the scour pit will cut through the minimal required cross-section.

Figure 4.26 Trees on the landside slope of the regional (secondary) levee along the canal Pijnackerse (courtesy STOWA)

Box 4.27 ***Lessons learned by dealing with the reality of trees in France***

In France there are over 5000 km of levees with large trees. More than 70 per cent of these levees have narrow crests and steep slopes, yet there are limited resources for removing all trees and rebuilding the levees. Approaches that have been used to prioritise work when developing their long-term management plans include:

- in areas where there are invasive, fast growing, light-demanding shrub or tree species, an indigenous tree cover is allowed to remain to keep the fast growing species from invading the area and taking over the levee
- large trees at the landside toe of the levee are prioritised for removal because they have been observed to have negative impacts on the integrity of the levee because:
  - such trees are believed to create favourable seepage paths
  - seepage issues usually occur at the toe of the levee, and if a large tree is there it hinders flood-fighting and may need to be removed in an emergency
  - such trees are believed to 'hide' water within their roots, further exacerbating seepage issues, and making seepage or saturation issues difficult to detect
  - such trees can impair visibility of any problems with the performance of drainage material near the toe
  - such trees can obstruct the drainage material and keep it from functioning properly
- on levees with narrow crests and slopes that are steeper than 1 vertical:1 horizontal, the removal of large trees has been prioritised because they have been observed to lead to landslides. This occurs when the water levels drop quickly after a flood because the levee remains saturated. There is less cohesion and the weight of the tree can cause a large part of the levee to slide away
- limiting the diameter and height of trees on and near the levee helps avoid wind-throw. In France many tree species will re-grow smaller diameter shoots after being cut flush with the ground, and this is a common practice to keep existing trees from blowing over
- cutting only select trees flush with the ground and allowing them to re-grow as smaller shoots has proven effective in decreasing the number of large diameter trees. This practice also keeps the roots alive to avoid both their decomposition and the seepage issues associated with decaying roots. For example, every third tree is cut along the levee every 10 years, allowing the stumps to sprout new shoots so that there are smaller trees in place of the large trees and the roots stay alive
- before the tree is completely dead, good practice in France is to remove as much of the root ball as practicable. There have been over 10 well-documented cases of seepage and at least one complete dam failure that were associated with preferential seepage paths that developed as a result of the decomposition of tree roots
- the French have been incorporating concerns learned from the 1997 Oder River flood in Germany and Poland (Grünewald, 1997) into their long-term management plans. The 1997 flood event in Germany and Poland helped convince the French that vegetation on the levee could be a direct threat to the levee when they are overturned by wind-throw, or an indirect threat when tree roots are uprooted on the slopes, leading to internal or external erosion. Fifteen per cent of the breaches that occurred during the flood were associated with external erosion due to trees. The primary causes were determined to be wind-throw of leaning trees and roots. Several large breaches were directly attributed to large trees growing on the levee crest.

For several countries, the following elements are generally common in their approaches to managing existing woody vegetation on levees to minimise environmental impacts:

- allow a flexible approach considering the value of the woody vegetation, and allow woody vegetation to exist on some levee locations, particularly if specific design features were incorporated, such as an oversized levee (eg the Environment Agency and Rijikswaterstaat/STOWA). For example, if a levee had a robust seepage or stability berm, or was overly wide, woody vegetation may be allowed. Alternatively, woody vegetation may be allowed or tree roots retained if a seepage barrier was added (eg slurry cut-off wall, Pohl, 2011, and Pinhas, 2011). The general presumption is that by either making the levee oversized or incorporating a root barrier, roots would not penetrate into the critical portions of a levee
- perform an initial evaluation or assessment that would lead to removing woody vegetation that poses an obvious or serious threat to levee integrity (eg large or unstable trees on levees with small cross-sections that, if blown over in a strong wind, would remove a significant portion of the levee cross-section, see FloodSAFE California, 2012)
- trim and/or thin woody vegetation in some places on the levee to provide visibility and access (eg Smith *et al*, 2009, and FloodSAFE California, 2012, see Box 4.28)
- consider staged efforts over time – that is, phasing of vegetation removal over long periods of time (Bonin *et al*, 2013, and FloodSAFE California, 2012).

Box 4.28 shows examples from England, Wales, France and the USA of the use of trimming and thinning of existing woody vegetation to allow access and visibility and to maintain levee integrity.

Box 4.28 Examples of the use of trimming and thinning of existing woody vegetation to allow access and visibility and to maintain levee integrity

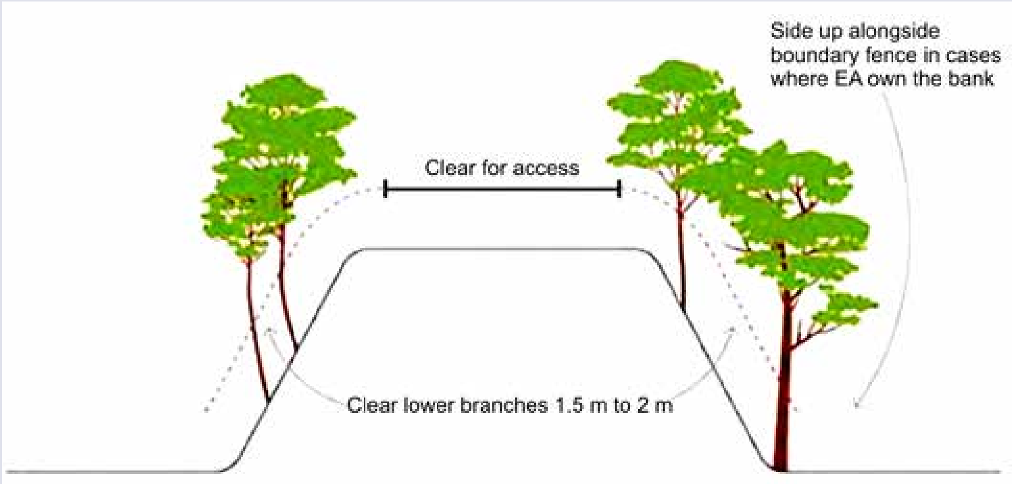


Figure 4.27 Levees in England and Wales (from Environment Agency, 2010)

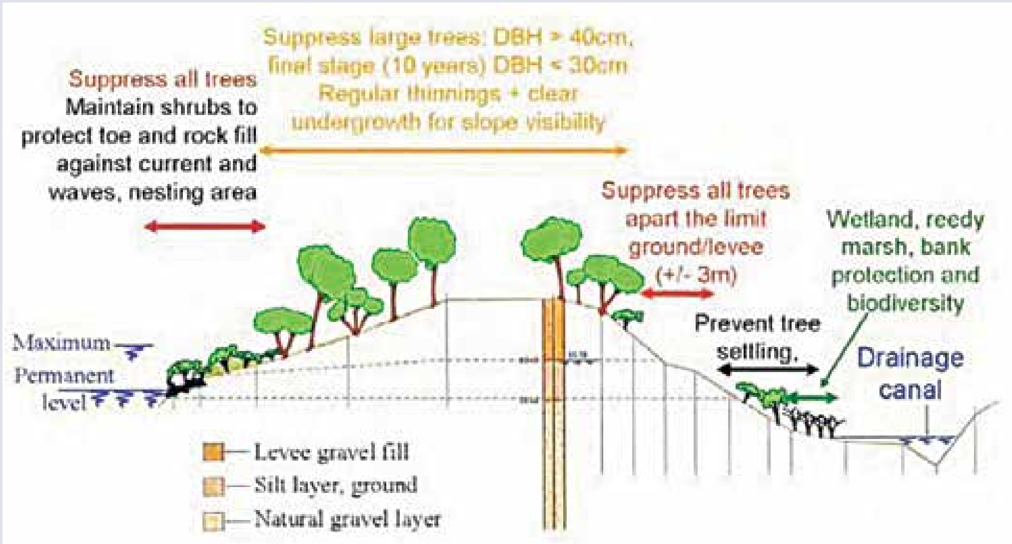


Figure 4.28 Large and wide levees in France (from Vennetier et al, 2011)

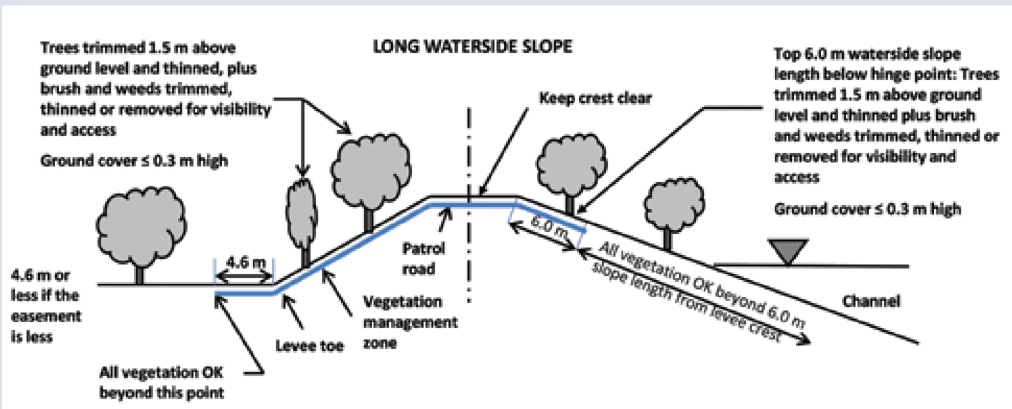


Figure 4.29 Urban levees in California, USA (from FloodSAFE California, 2012)

4.6 BURROWING ANIMALS

Burrowing animals, including mammals, amphibians, reptiles, and invertebrates, dig holes or tunnels into levees for habitation or temporary shelter. These burrows may vary in form from short, single tunnels to lengthy tunnel complexes interspersed with chambers. Burrow gradients may range from horizontal to vertical.

Animals also burrow into levees because:

- food sources are located nearby
- animal control programs in the area are ineffective
- long-term flooding forces them onto dry ground
- abandoned burrows and other voids have not been backfilled
- some animals (eg badgers) like digging in ground with a steep surface.

Figure 4.30 gives examples of holes in levees caused by animal burrowing. The most undesirable burrowing animals are usually large rodents. Box 4.29 and 4.30 give examples of damage caused by insect-related burrowing and animal related burrowing.

Box 4.29 ***Insects are burrowing animals too***

> Sometimes the greatest risk to levee safety comes from small insects. In Vietnam, termites dig holes and small galleries in levees that can occupy as much as 4 m^3 per nest. This was observed, for example, in the Red River delta. Also, in the city of New Orleans, USA, termites caused flood walls to leak by eating through rubber waterstops in the expansion joints. This was observed at several outfall canals (eg London Avenue and Orleans).

Figure 4.30 ***Holes caused by animal burrowing, (a) greater than five holes in a 10 m stretch of levee, and (b) a single hole larger than 15 cm in diameter***

Why burrowing animals are a concern

Animals burrowing in and around levees may cause:

- internal erosion, which may lead to piping (due to shortening of seepage paths)
- mechanical weakening (riverbanks, riverside slope)
- perforations of the impermeability components
- destabilisation of masonry, revetments and roadways
- collapsed areas/unevenness along the crest
- openings below the water line (because the entrance of burrows can be under the water surface even though the main part of the burrow may be above water level)
- direct seepage (through-embankment burrows, see Figure 4.31).

Box 4.30 ***Levee breaches caused by animal burrowing in the Rhone River Delta***

In 1993 and 1994, there were 16 breaches in the Rhone River Delta, and 13 were caused by animal holes. In 2003, two levee failures occurred, but neither was caused by animal holes, because levee maintainers had stopped more than 20 leaks in the levees that were caused by animal holes.

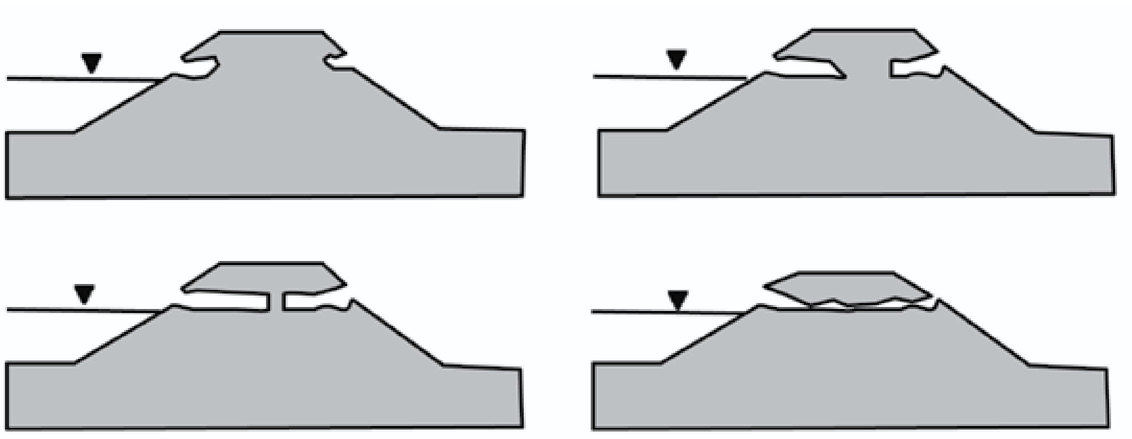


Figure 4.31 ***Levee collapse due to through-embankment burrowing***

The elements of an effective animal control program

Levee maintainers can do the following to create an effective animal control program:

- assess the types of risks posed by burrowing animals (see Table 4.10)
- identify animal species and potential threats relevant to their area (see Table 4.11)
- formulate plans for preventing these threats to levee integrity (see Table 4.12)
- formulate plans for repairing issues as they arise (see Table 4.13).

Tables 4.10 to 4.13 offer guidance in each of these areas.

Table 4.10 ***Assess the types of risks posed by burrowing animals***

Risk	May be caused by
A burrow passing through (or nearly through) the levee may lead to seepage, internal erosion, and piping	Badger, rabbit, beaver, ground squirrel, groundhog/woodchuck, and other animals capable of building large or complex burrows
Removal of significant amounts of material from the levee during the digging of the burrow may cause slope stability and bank caving problems	Ground squirrel, badger, groundhog/woodchuck, coypu and other animals that create large diameter burrows or that colonise in high densities
Multiple holes in a short stretch of levee may threaten embankment stability	Groundhog/woodchuck, badger, coypu, and other animals that colonise in high densities
A single large hole could threaten embankment stability or reduce the height of the levee crest	Badger, beaver, ground squirrel, groundhog/woodchuck, and other animals capable of building large burrows
Underwater entrances may provide a seepage path into the levee	Beaver, otter, muskrat, and American signal crayfish. See Box 4.31 for more information on crayfish problems in England

Table 4.11 *Identify animal species and potential threats relevant to the area*

Animal	Typical burrow characteristics
Badger	Setts (tunnels) are approximately 40 cm in diameter. Digs a network of five to ten tunnels, each 8 m to 10 m in length, complete with air shafts.
Wild rabbit	Warrens (tunnels and chambers) are 10 cm to 20 cm in diameter. Prefers sandy, silty soils. Easily identified by its droppings.
Fox	Limited burrowing activity. Often lives in setts with, or abandoned by, badgers.
Ground squirrel	North America only. Digs burrows typically 10 cm to 15 cm in diameter, which may often cross through the entire levee. A single squirrel may create a fairly elaborate system of tunnels. The activities of a colony may lead to extensive voids.
Groundhog/woodchuck	Excellent diggers. Burrow may measure up to 4.3 m in length, with two to five entrances.
Otter	Otter holts (dens) may be located on land, but have entrances below the water surface.
Coypu (nutria, ragondin, castorino, beaver rat, or river rat)	Invasive and destructive burrower found in warmer climates in various parts of the world. Makes dens 25 cm to 60 cm in diameter at the entrance and several metres in length. In areas where their population is dense, burrows may occur close together.
Muskrat	North America, Western Europe. Digs a network of tunnels with entrances that are 15 cm to 20 cm wide and are always underwater.
Weasels (Mustelidae family, including stoat, polecat, ferret, and European mink)	May live in the burrows of other rodents.
American signal crayfish	Crayfish burrow into the banks to create tunnels and chambers where they take refuge during daylight hours and the overwintering period. These tunnels can create weaknesses in engineered channels and levees (see Box 4.31).
Beaver	Known for building dams, canals, and lodges (homes). Beaver tunnels may be up to 90 cm in diameter and 6 m to 9 m in length. Dens at the end of the tunnel are commonly larger then 90 cm in diameter. These holes may be problematic, since the tunnel enterance is often below the water surface and difficult to find.

The examples in Box 4.31 to 4.32 illustrate some of the damaging effects caused by crayfish and beavers, respectively. The cumulative impact of the holes needs to be considered. In some cases it has been observed that a beaver den on the waterside of a levee has been connected with ground squirrel holes on the landside and has almost caused the levee to fail due to piping.

Box 4.31 *Crayfish threaten the banks of streams in England*

In the UK, the burrows of the non-native American signal crayfish have caused significant seepage problems and even substantial breaches in some locations (Figure 4.32). In Oxfordshire, along the Wolvercote Millstream, which is an engineered channel that conveys water from the Thames navigation above Kings Weir to the Oxford canal, signal crayfish issues have become so severe that piles had to be installed in the centre of the bank to fix several significant breaches. The Millstream runs parallel to another, lower level, channel called the Kingsbridge Brook. The initial crayfish problem on the millstream was identified in 2002, and shortly afterwards, water was observed to be seeping through the bank. Within two years substantial breaches had formed in the banks, allowing significant volumes of water to flow into Kingsbridge Brook. This loss of water was preventing the levels in the Thames from being held at a depth suitable for navigation above Kings Weir. Also, the land adjacent to the lower level channel, an internationally important wildlife site, was becoming too wet to support the specific plant communities. To close the initial breach, 20 m of piles were required in 2005, but in 2009 additional breaches occurred. In 2010, approximately 100 m of additional pilings were installed. In 2012 further breaches were identified and additional measures to repair the bank have been planned.

Figure 4.32

Breach in Wolvercote Millstream bank, England, caused by signal crayfish activity (courtesy Environment Agency)

Box 4.32 ***Beavers pose challenges to levees in the USA and the Netherlands***

In the Netherlands, beavers may seek refuge in river dikes and levees when water levels rise high enough in the floodplains to prevent them from using their dens and forts. Tunnels of six to nine metres in length are common. With the population in 2012 at approximately 700 beavers, the situation is under control. But the population is expected to grow tenfold in the future.

In the USA, the California Department of Water Resources (DWR) has found several beavers in its levees, including one weighing approximately 32 kg. Challenges related to beavers in California levees have included:

- a hole that went straight through a levee
- dens causing large sinkholes to open up along the edges of the water (a problem when the levee toe is adjacent to the water's edge)
- dams and lodges compromising channel capacity and diverting water toward the levee, which increases erosion.

Table 4.12 ***Formulate plans for preventing burrowing threats to levee integrity***

Suggested plan	Benefits/limitations
Mow, cut, and clear vegetation encroaching the levee	• disturbs wildlife, and discourages it from returning • prevents the development of dense plant cover, thereby reducing the temptation for burrowing animals to inhabit the levee • eliminates food sources near the levee.
Install impenetrable mesh	A metal mesh could be placed just beneath the surface of the levee soil so that burrowing animals cannot penetrate the levee prism. See Figure 4.33. When installing mesh, consider: • **durability:** choose the right material, ie metal versus fabric • **placement:** above topsoil or below topsoil, typically better below so it can be placed on a firm surface • **extent of protection, determined based on local experience:** if not placed over the whole levee, the animal's second favourite spot for burrowing may not be covered. Burrows may appear in the areas that are not covered.
Create alternate habitat to lure the animals away from the levee	• environmentally friendly alternative • may not require killing the animals • provides endangered species with alternate housing (avoids issues with them inhabiting the levee) • if animals procreate, their offspring may create burrows in the levee if it is nearby and appears to be an attractive place to burrow.
Use cage traps	Cage traps (versus other types of traps): • have fewer statutory restrictions, because the trapped animal is not killed • are highly selective, ie non-targeted animals are released • are highly efficient for intermittent control, such as for the coypu. Certain animals eventually outwit cage traps. In that case, humane stop snares could be used, which do not kill the animal and are selective.
Eradicate by shooting	Generally governed by laws that may: • require a permit to hunt that allows ownership and use of hunting weapons • prohibit certain types of weapons and ammunition • specify eradication periods (seasons), formalities, and geographic restrictions. Underground terrier work, such as using dogs or ferrets, may also be regulated in many countries
Use chemical control	Some jurisdictions do not permit the use of poisonous substances to eradicate burrowing animals. When authorised, chemical control of rodents has been found to be very effective. Anticoagulants, for example, work by preventing the formation of blood clots and cause smaller capillaries to rupture. The risk of secondary kills is mitigated by the poisoned animals typically tiring and returning to their burrows to die, where their carcasses are not exposed to scavengers

An effective animal control program should include a plan to regularly remove burrowing animals from the levee area and address any voids they have already created. When designing such programs, remember to take into account:

- existing or pending protective legislation
- existing or pending environmental legislation
- whether there is a way to obtain either exemptions from legislation or special licenses
- whether there is a statutory requirement that certain animals be controlled because they are considered pests by the government (eg the wild rabbit in England).

National, provincial, and local legislation (eg the Wildlife and Countryside Act 1981 in England), in addition to guidance offered by government or conservation agencies (eg the Netherlands' former National Coordination Committee on Muskrat Control, now included in the Union of Water Boards), should also be consulted to ensure that the burrowing animal control program is aligned with all applicable laws. Protective legislation tends to be unique to the type of animal. Even in cases where animal protection legislation may protect destructive burrowing animals, it should be followed when removing burrowing animals from the levee. Common provisions of protective legislation include prohibitions against:

- harming or killing the animals
- treating animals in cruel or inhumane ways
- using specific animal control methods, such as trapping or chemical agents (pesticides)
- disturbing shelters (tunnels, dens etc) or feeding and mating grounds
- interfering with general habitats, including wetlands.

Figure 4.33 Metal mesh on the surface of the levee (courtesy Symadrem)

Examples in Box 4.33 illustrate how the UK and Ireland balance the requirements of environmental laws with the need to protect levees from burrowing damage.

Box 4.33 ***Balancing requirements of environmental laws with the need to protect levees***

In Ireland, an active otter holt (den) was found on a levee along the River Brick. The otter is a protected species in Ireland, so care was taken not to disturb the holt. It had to be removed and replaced with an artificial one that had its exit on the landward side at crest level.

In the UK, water voles and their resting places are fully protected under the Wildlife and Countryside Act 1981. It is an offence to deliberately capture, injure, or kill them, to damage, destroy, or obstruct their breeding or resting places, or to disturb them in such places. Prohibited methods may be used under certain circumstances if the issue cannot be resolved by any other means. In England, the Natural England Wildlife Management and Licensing Service administers license applications when public health or the potential for serious property damage are involved. In Wales, licenses are issued by the Nature Conservation Branch of the Welsh Assembly Government. An advisory leaflet, which describes licensing policy for water voles, is available from Natural England.

Repairing damage caused by burrowing animals

Once a levee section has become infested by burrowing animals, the holes they created should be addressed as soon as possible by one of the methods in Table 4.13.

Table 4.13 ***Formulate plans for repairing issues as they arise***

Method	Additional details
Excavate the area around the hole, backfill the hole in 9 cm to 13 cm lifts and re-compact the material to the same compaction as the adjacent levee	One concern with this method is that either the main tunnel or the tunnels branching off the main tunnel may not be found.
Fill the holes with a low pressure, flowable grout, a viscosity that will adequately fill the holes, and is compatible with the local groundwater chemistry (commonly, a 3:1, cement:bentonite solution is used)	When holes created by the rodents are properly backfilled, their habitat is disturbed, which discourages them from returning to the site: • start applying low pressure grout at the lower levels of the levee. This forces the grout upward to fill all voids • make sure the grout is applied under low pressure and not gravity fed. A gravity application of the mixture cannot ensure that all voids are filled • note that the interface between the grout and the soil may create an additional seepage path. If piping is an issue on the levee, this is not a recommended option. See Box 4.34 for an example of how low pressure grout was used in California, USA.

Box 4.34 ***Applying low pressure grout to levees to fill holes, California, USA***

The California DWR uses a low pressure flowable grout made of a 3:1, cement:bentonite solution to fill animal burrows (having bentonite in the mix helps to prevent cracking). In the first year of using this method, the average ground squirrel hole size was 74 litres. In the second year it was 40 litres, and in the third, 11 litres. A cost estimate made in 2011 suggested that excavating levees and backfilling all animal holes in California would be 10 times more expensive than the low pressure grouting (Wagner, 2010).

Determining when burrowing animal repairs are beyond maintenance

If the burrowing animal problem has caused irreparable damage to the levee, a professional levee designer may be needed to address the problem. This may happen when:

- the animal burrow penetrates the impermeable layer, if there is one on the surface of the levee, or if the hole is beneath the levee and is allowing for piping to occur
- the burrows are so large or numerous that they compromise the stability of the levee embankment.

For either of these problems, it is likely that an entire portion of the levee may need to be dug out and rebuilt (replacing the water seal), or a reinforcing structure (such as a diaphragm wall or sheet pile cut-off running along the dike) may need to be installed.

If the waterstop has been compromised, this could open a direct path for seepage through the levee. If not addressed completely, it could lead to levee failure. Similarly, if the burrowing animal problem has compromised the stability of the levee, this could lead to the collapse of the slope, weakening the levee and making it more susceptible to external erosion and eventual collapse. For more information on how these deterioration processes may lead to levee failure, see Chapter 3.

4.7 EROSION AND BANK CAVING

Erosion is the wearing away of a surface above the water line (eg bank, foreshore or embankment) by floods, waves, wind, or any other natural process. Bank caving is the localised slough or slide that occurs when the slope of a levee becomes unstable. Scour is the wearing away of a surface below the water line (eg streambed). See Chapter 3 for more details about erosion and bank caving processes.

Why erosion and bank caving are a concern

Erosion and bank caving may remove materials from the levee in a way that affects its thickness and density. Affected areas may be further weakened by increasing hydraulic gradients within the bank or levee and increasing the likelihood of collapse during extreme events.

The internal material of the levee exposed by erosion or bank caving is not usually designed to resist environmental aggressions. The continued direct exposure of this material to these aggressions may accelerate the levee's deterioration (see Box 4.35). If the problem is not addressed quickly, it could both compromise the levee and become unrepairable by maintenance techniques alone. Maintainers need to be aware that erosion and bank caving do not subside if no actions are taken and that scour can be dangerous to the levee, as it can undermine it. They also need to be aware of instability of the foreshore (intertidal zone) or any portion of the levee under the water. Changes in the ground just below the levee will affect levee stability.

Box 4.35 ***Effects of erosion on a levee slope, Park Creek, New York, USA***

On Park Creek deposits have concentrated the river flow close to the levee. Because the flow is more intense on the outside of the curve, the portion of levee in that area was heavily eroded during the 2006 flood (Figure 4.34). Once the grass cover was removed from the levee and the unprotected earthen material was exposed, erosion progressed very quickly. This site was repaired by armouring the toe of the levee. No damage was done to the levee during the largest flooding event on record in September 2011.

Figure 4.34 ***The effects of erosion on a levee slope at Park Creek, New York, USA***

Table 4.14 lists common causes of erosion and bank caving and suggests ways to prevent them.

Table 4.14 How to prevent common causes of erosion and bank caving

Observations	Preventive measures
Hydraulics of the body of water adjacent to the levee (flow and waves) removes material from the levee	• monitor the levee and its surroundings. Report any change in river geometry, flow, or increase in ship traffic. Pay special attention to convex channel zones and narrowing of the channel • mitigate the hydraulic effects to reduce damage to the levee, if possible limit boat traffic and/or speed if wake waves affect the levee.
Rain runoff	• report any unexpected runoff damage done to the levee • address drainage system malfunctions (eg avoid obstructed or broken culverts or drainage pumps or pipes) • maintain levee surface with grasses or other cover to reduce the erosion of soil particles due to effects of raindrop impact and shallow overland flow • divert all drainage from surrounding areas away from the levee slope.
Fallen trees uproot the bank/slope	See Section 4.5
Frequent access (eg vehicular traffic, foot traffic, grazing livestock)	See Section 4.8
Slope instability (above the waterline)	See Section 4.11
Foreshore instability (or any instability below the waterline)	Place new rip-rap/slack. Slack is an industrial residue such as copper slack, steel slack or phosphorus slack. Environmental regulations should be observed. In the Netherlands, slack is found to be a readily available, easy-to-handle material for bank protection

Obstructions, curves of the river, new construction nearby	• co-ordinate with relevant parties to:
	  ◦ ensure vegetation in the riverbed is appropriately maintained/removed
	  ◦ remove log jams and other scrap materials that could create an obstruction
	  ◦ ask that the potential changes of flow be taken into account during the design process of new construction nearby
	• frequently monitor sections close to recent changes.

Rising water level (flood, riverbed changes, climate change)	• monitor water level changes • maintain a profile that induces a laminary runoff over the levee in case of overtopping.
Toe scour	• monitor any modification of the bank near to the levee • if practical, protect the bank before erosion affects the levee.

Repairing erosion and bank caving problems

Measures to repair erosion and bank caving include:

- replacement of any lost material to maintain the designed cross-section
- replacement or repair of existing bank or levee toe protection, if it has been damaged.

These measures are not usually sufficient to stop subsequent erosion and bank caving. It is important to stop the main cause of erosion and bank caving by addressing it directly with preventive or curative actions.

Determining when erosion and bank caving repairs are beyond maintenance

A condition assessment (see Chapter 5) can help identify the full extent of levee issues that appear as material is removed from the levee or revetment. Permanent solutions (see Chapter 9) to erosion and bank caving (such as toe protection with rip-rap or levee setback) may be required (and may involve a levee designer) if:

- the preventive and repair measures taken are not sufficient to solve the problem, and erosion or bank caving continue to affect the levee

- the affected area near to the levee is still subject to erosion (scouring or deepening) after remedial work has been done
- a permanent change is occurring that the levee has not been designed for (eg building of a new structure, such as a road, on the levee, or a change in the flow of the river)
- the rock level is changing, which could be an indication of an internal erosion issue.

4.8 DEPRESSIONS AND RUTTING

Ruts are long stretches of depressions in the crest, toe, slopes or access ramps of levees and range in size from a shallow depressions of just a few hundredths of a metre caused by vehicular traffic or animal grazing, to pot holes, which may be more than 0.30 m deep.

Why depressions and ruts are a concern

When water ponds in ruts and depressions on a levee's crest or access ramps, it may seep into the embankment and increase the moisture in the levee. This increase in moisture may weaken the embankment and decrease stability, especially for loaded embankments during high water situations. Water ponded on crest or access roads, when combined with high volumes of vehicular traffic, may also make these roads undriveable both for passenger vehicles and heavy flood-fighting equipment.

Table 4.15 lists common causes of depressions and ruts and how to prevent them. If a depression or rut is deeper than, eg 0.15 m (according to a USACE levee inspection checklist), repairs may be needed. If the original levee design was inadequate or flawed, a designer may need to be consulted (some of the details a designer should consider when building or rebuilding a levee are discussed in Chapter 9).

Table 4.15 ***How to prevent common causes of depressions and ruts***

Observations	Preventive measures
Maintenance activities and issues	
Tyre impressions from mowing on wet slopes	• mow in alternating patterns to prevent mower-related rutting. Match mower type with the task at hand • avoid operating heavy equipment on the levee when the levee is saturated with water.
Dry grass	Ensure that a good grass cover is maintained. Indigenous grass species that are well adapted for the local climate are recommended. See Section 4.5
Organic materials in the soil deteriorating over time	Avoid placing organic materials in the levee during construction or repair/rehabilitation.
Rodent activity The mechanism is usually progressive. A rodent hole creates a void in the levee, and additional pressures on the outside or deterioration on the inside of the hole may cause the roof of the hole to weaken then collapse. Depending on the type of animal burrow, the collapse may be localised to the embankment slope or, in the case of larger holes, may migrate under the crest, leading to its collapse.	See Section 4.6
Encroachments	
Leaking pipelines over, through or under the levee surface (eg overhead pipe racks dripping rain water on the levee surface)	See Section 4.4

Table 4.15 How to prevent common causes of depressions and ruts (contd)

Excessive use	
A higher volume of pedestrian, animal, or vehicular traffic on the levee or higher loads than the levee was designed for	• use appropriate signage and traffic control techniques to limit the type, frequency and speed of vehicular traffic on the crest to the levels that it was designed for • consider using proper all-weather driving surfacing on the levee crest, such as granite or limestone chippings (see Boxes 4.36 and 4.37 for alternate solutions) • allow lighter livestock, such as goats and sheep, on the levee, but discourage heavier livestock (eg cattle), which may cause depressions.
Design issues	
Uneven settlement	• compact frequently (most critical measure) • ensure surfaces are properly graded, inspecting during construction for compliance with design.
Poorly draining crest slope and underlying subgrade, increasing likelihood that vehicular traffic may cause ruts	• visually evaluate the levee embankment and access roads after substantial rain events to identify and repair problematic areas of ponding water • repair improper drainage in a timely manner to prevent small concentrations of flow from becoming ruts on the crest or shoulder that may cause erosion or stability issues.
A design flaw in the material used to build the levee or in the all-weather driving material placed on the crest	May need to consult a levee designer or the material vendor/ manufacturer.

Box 4.36 An alternative to heavy fill materials

In the Delta area of California, USA, the California Department of Transportation (CalTrans) wanted to widen and straighten a section of Highway 160 near Rio Vista that runs atop an extra wide, 130-year-old levee built on 10.7 m of soft organic soils. Though the highway required all-weather driving material, using heavy fill materials would likely have caused levee subsidence. As an alternative, CalTrans used wood chips encapsulated in mineral soils as the fill.

Box 4.37 An alternative to all-weather driving surfacing on the crest

In England and Wales, where the vast majority of embankments are covered with grass to reduce erosion and where crests rarely are covered with all-weather driving materials, the Environment Agency uses matting to get to work areas on levees when ground conditions are poor. Access to the levee is generally via maintenance access strips (easements protected by byelaws) along the levee's landward toe, rather than by roadways on the levee crest.

Flood patrols, which tend to be on foot, often use ramps up to and over the levees for access, and synthetic placements or sprays to protect the levee surface. Typical of these are stone pitch and a form of concrete paviors that have regularly spaced openings through which grass can grow.

Repairing depressions and ruts

The following steps for repairing depressions and ruts are suggested:

1. Begin by removing and temporarily storing any turf and topsoil.
2. Scarify the depressed surface and adjacent area to allow proper bonding of new fill material with in-place material. Avoid doing repairs when the ground is frozen otherwise all frozen ground will need to be removed as a separate step.
3. Add suitable fill material, preferably durable material consisting of highly plastic clays (CH) or lean clay (CL), in loose layers no more than 0.15 m thick. Moisture content of the fill materials should be within acceptable ranges before placement. Judging the proper moisture content by the outward appearance of the material is best done by those trained in soils or earthwork construction (eg a levee designer).

4 Compact the fill layers either by hand or with small mechanical equipment. Layers should be added to the low areas until the fill forms a slight mound over the rut or depression such that water will no longer pond, while allowing for a small amount of settlement of the fill materials.

5 Replace top soil and existing turf with material that has the same soil properties as the adjacent soil material. If the original turf is not suitable, a biodegradable erosion control mat may be used along with appropriate fertilising, re-seeding and mulching for adequate erosion control.

For embankment crests and access ramps where the road has a gravel or crushed stone surface, it would be prudent to place a durable road surfacing material over the repair. The surface treatment is recommended for active levees where vehicular access is common on the crest for inspections, high water monitoring, and access by nearby property owners.

Determining when repairs of depressions and ruts are beyond maintenance

Consider consulting a levee designer (to identify the source of the problem and take appropriate action) and relevant design codes if:

- a depression has been refilled numerous times (a possible indication of a sink hole, a seepage problem, an animal burrowing issue, differential settlement, or an issue with a damaged pipe)
- a design deficiency requires correction, such as oversteepened slopes, sloughing on a levee that typically had smooth slopes, or shallow slip failures that may appear to be ruts on the slopes of the levee but upon closer examination have vertical displacement of the levee material
- excessive vehicular traffic is anticipated, such as for a public road or highway. If excessive vehicular traffic is anticipated, the designer should consider the performance requirements for the public road (or highway) as well as the levee. Understanding both requirements can help avoid design issues such as using a thick permeable layer as the sub-base of the road (on the levee crest) that may unintentionally decrease the effective height of the levee.

4.9 SETTLEMENT AND SUBSIDENCE

Settlement and subsidence both result in a lowering of the original ground elevation with consequent loss of the elevation of any overlying levee or other flood defence structure. Settlement occurs from movement of the ground due to some type of loading, while subsidence occurs from movement of the ground due to loss of support in the foundation.

Why settlement and subsidence are a concern

Both settlement and subsidence reduce the levee's height, which reduces the design flood mitigation level. Also, localised and general movement may reduce the crest elevation. Localised crest settlement may be easy to identify visually, but gradual reductions in the levee height that occur over long reaches may not be detectable without a survey. In some cases, settlement may induce transverse cracks, and if left untreated, could result in crack erosion and ultimately in a levee breach. If settlement leads to a sinkhole, it can produce a hazard both to the levee and to anyone driving on the levee.

Settlement is a particularly serious concern on zoned levees and levees with revetment. Zoned levees are typically designed with an impermeable core/cap (see Figures 3.74 to 3.78 for cross-sections of zoned levees). Settlement on a zoned levee could be a fundamental indicator of a blending of the clay and sand layers, which could clog the sand and prevent the levee from performing as intended. If a revetment on a levee settles, the stones may separate in the process. This separation may cause issues, such as a decrease in the levee's ability to resist erosion.

Causes and preventive measures

The causes of settlement may be related to seepage, encroachments, or design. See Table 4.16 for suggested preventive measures.

Table 4.16 *Settlement: causes and preventive measures*

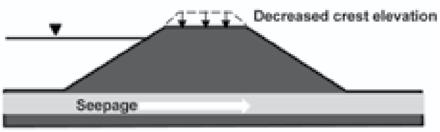
Observations	Preventive measures
Seepage-related	
Loss of excessive amounts of material as a result of boils  Movement of material under and through the levee due to seepage can cause settlement	• if a boil is identified, a ring dike could be placed around it to keep fine particles from piping, preventing settlement if caught early enough • every effort should be made to control seepage runoff so that the loss of material does not lead to settlement • if excessive amounts of material have already been lost in an area of the levee because of a boil, consider installing a cut-off wall (see Chapter 9) See Section 3.2.2.9 on cut-offs for permeable foundations, Section 6.7.2.1 on seepage berms and Section 6.7.3.1 on additional techniques for flood-fighting boils.
Encroachment-related	
Improper compaction of material around culverts, pipes, or other structural features added after levee construction	Place a sand filter and a weighted berm around the culvert at the landside exit of the pipe to prevent the movement of material through the levee at the interface of the soil and the pipe. When installing pipes through the levee, after remedial work, or when doing any work in the levee prism that involves excavating and backfilling, use: • good design detailing (see Chapter 9) • proper compaction around encroachment (see Chapter 9) • good quality control and quality assurance.
Leaking pipes inside the levee	Water leaking from pipes within the levee may create voids that lead to slope instability, internal erosion or other issues. Section 4.15, Culverts/ Discharge Pipe Systems, suggests ways to prevent pipe-related issues, maintain pipes, and repair pipe-related issues, should they occur
Collapse of abandoned pipes	See Section 4.15
Groundwater extraction, oil extraction and mining operations	Extractions and mining operations that weaken the levee foundation may be able to be regulated by a good encroachment control system. See Section 4.4
Improper compaction of original materials used to construct the levee	See Chapter 9 for information on levee design and proper compaction techniques
Design-related	
Settlement of compressible foundation soils in small, concentrated areas (possible causes include animal burrowing or drying out of the levee). See Box 4.38 for example.	• remove, if possible, the area of compressible material, replace with suitable levee material, and build or rebuild the levee on top • if the area of compressible soil is too large to remove and replace, then anticipate the amount of settlement, and design the levee to plan for the settlement. See Chapters 7, 8 and 9.

Table 4.16 Settlement: causes and preventive measures (contd)

Geological subsidence (settlement of foundation strata over large areas)	• if possible, avoid by building levees on solid foundations that are not prone to settlement
	• for existing levees built on strata with known subsidence issues (eg karstic or limestone), or materials in areas known to be unstable or subject to erosion, such as riverbanks, consider:
	  ◦ building a setback levee as extra support for the leveed area
	  ◦ regularly monitoring the levee for settlement below the levee's design height.
	See Chapters 7 and 8.

Compaction of levee fill over time or as the result of: • the first hydraulic loading • the imposition of loading from crest structures • loading from external agents that should have been designed for, such as maintenance vehicles or seismic loadings.	No preventative measure is available within the realm of O&M. Consult an experienced designer and see Chapters 5 and 9. Chapter 5 discusses levee condition assessment. It provides guidance on how to assess the levee to determine what action is needed and should be used in conjunction with Chapter 9.
Sudden or slow rotational or translational failure of the levee and/or the levee foundation	Consult an experienced designer and see Chapter 9.

Repairing settlement

Prompt repair of settlement is important to avoid further deterioration. Good engineering practices suggest elevation surveys be taken and validated every three to five years, depending on surrounding soil conditions and anticipated settlement (see Section 9.12.1). Small areas of settlement may be restored to the design crest elevation by removing the turf (sod) cover, scarifying (ie roughening) the exposed levee materials, placing and compacting the same type of material (pervious or impervious) as the original levee composition, and restoring the turf cover. For longer stretches of settlement, adding material to the top of a levee may be appropriate. If the levee is built on a soft soil foundation, the risk of a resulting slide should be considered. The risk may be reduced where the foundation soil is consolidated, unless there is a significant amount of through-seepage or underseepage. If historical mining occurred under the levee, it may affect the embankment and cause settlement. Although historical mining below the levee may create additional settlement challenges, maintainers are usually able to handle these unless a sinkhole is observed. Box 4.39 provides examples of settlement issues and their solutions.

Determining when repairs after settlement are beyond maintenance

If the levee is built on a soft soil foundation, a designer should be consulted about the risk of a resulting slide. The risk may be reduced where the foundation soil is consolidated, unless there is significant amount of through-seepage or underseepage.

If settlement is observed that exceeds the anticipated settlement in the design, an experienced, professional designer should be consulted because it could be an indication of several serious issues including:

- internal erosion in the levee body or foundation soils
- ongoing primary or secondary consolidation settlement
- desiccation shrinkage of the levee
- animal burrows
- consolidation of foundation soils caused by a lowering of the groundwater table
- external erosion from the overtopping flow.

Box 4.38 ***Managing levees built on soft clay soils in France***

In the French departments of Gironde and Charente-Maritime, levees have historically been built from materials that were on hand. Many levees on ocean-front land in those areas were built on swamps and low-lying areas with loose or compressible soil, such as soft clay or peat (Figure 4.35). Even though the first two metres of the soil is typically compacted, it still has a high water content and a high plasticity. These levees are particularly susceptible to settlement, which can make the levee vulnerable to overtopping and desiccation cracks that occur as a result of the settlement. Careful attention needs to be paid to these cracks (Figure 4.36) and the depressions that occur from a combination of the settlement and cracks (Figures 4.37 and 4.38). Preventive measures, such as an adequate crest drainage system, should be installed whenever possible, because this issue can lead to levee failure by slope instability shown in Figure 4.37.

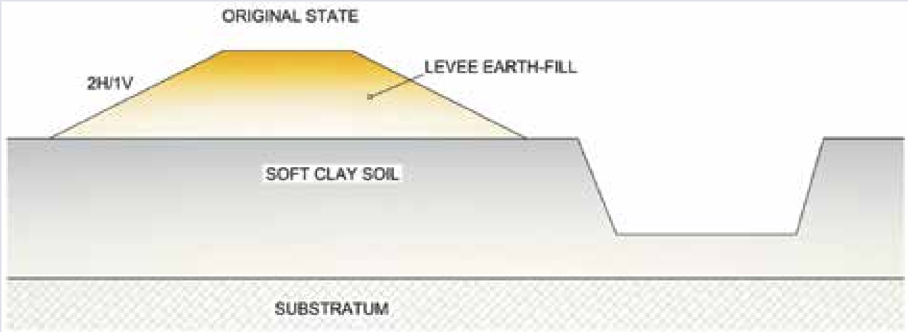


Figure 4.35 ***Historical levee built on soft clay soil***

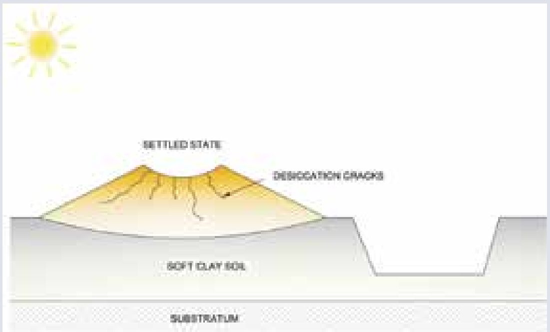


Figure 4.36
Crest settlement due to a soft clay soil foundation

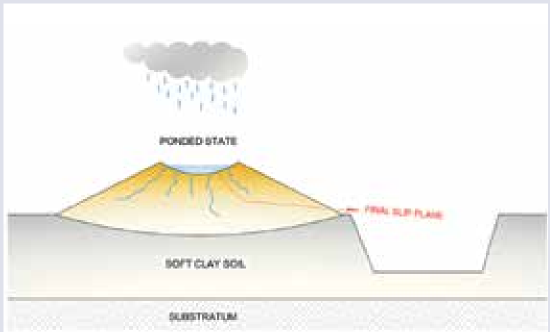


Figure 4.37
Water ponding on the crest of the settled levee. This can cause a slope instability failure (shown in red)

Figure 4.38 ***Cracks in the levee crest such as these should be taken seriously. Preventive action such as the installation of an adequate drainage system is recommended (courtesy Y Nedelec)***

Box 4.39 *Examples of settlement issues and their solutions*

Historic changes in worldwide climate have caused settlement

In some areas of the world, historic climate changes have caused settlement issues. For example, in Northern Europe, glaciers retreated after the last ice age, reducing the weight on the land previously covered by ice. This land is now rising, causing a rotation about the axis of the middle of the land mass and causing other areas of land to sink. This is known as glacial isostatic adjustment. The combination of glacial isostatic adjustment and the rising sea level due to ongoing climate change may reduce the levels of protection provided by existing levees.

A settlement solution to avoid

Placing pervious aggregate on top of a levee to restore it to its design height is a solution sometimes used for settlement. But it is a solution that has proven problematic. In the Delta area of California, USA, where there are many levees, pervious fill has often been added to levee crests to raise the levee height either to maintain freeboard or to provide a road base. Over several decades, the foundation soils beneath these levees have consolidated and settled, and in some cases by as much as 3 m to 6 m (Lund *et al*, 2007). After repeated settlement and repeated applications of aggregate, some pervious crests/road bases have descended to the part of the levee that should be impervious to water. This predisposes the levee to increased seepage problems.

Settlement solutions in the Netherlands

In the Netherlands, levee managers regularly consider future storm events and high water events and try to take precautionary steps to ensure that their levees will be resilient. But managers in Zeeland, the Netherlands, while expecting settlement to occur, anticipate that their levees will not be able to be built higher. These managers are faced with three choices:

1 Accept the settlement and potential overtopping with no additional reinforcement.
2 Accept the settlement and reinforce the inner slope of the levee.
3 Accept the settlement, reinforce the inner slope and build another levee behind the first levee in case the primary levee is overtopped during a high water event.

4.10 SEEPAGE

Seepage is the movement of water through the soil of a levee's embankment or foundation. The source of the seepage may be inside the levee (eg leaking pipes) or outside (eg high river stages). Seepage may occur as through-seepage or underseepage (see Figures 4.39 and 4.40).

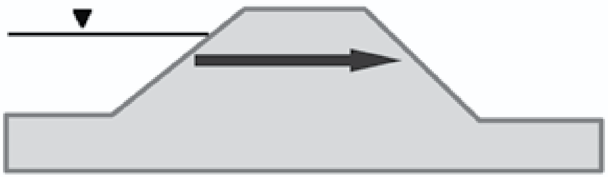


Figure 4.39 *Through-seepage*

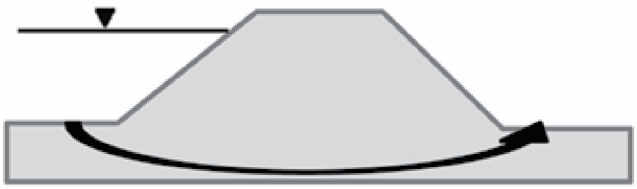


Figure 4.40 *Underseepage*

When seepage is a concern

Seepage is a concern if it causes the levee toe to become saturated (see Figure 4.41), or if it removes material from the levee (piping). The occurrence of light seepage either on the lower portions of the landside slope or near the levee toe during flood stage conditions is not necessarily an unusual or unsafe condition. Depending on the duration of the elevated river stage condition, some seepage is natural and likely to occur.

Note

Seepage may not become visible on either the landside slope or toe area if the flow duration is short and the embankment and foundation are made of low permeability soils such as clays.

Figure 4.41 Seepage at the landside levee toe

Seepage on zoned levees is of particular concern because zoned levees are designed with an impermeable core or an impermeable cap (or both) to try to prevent seepage (as described in Chapter 3). If seepage is observed, it is a serious concern, as it can indicate a design deficiency. Contact a designer to determine the cause.

Levee maintainers and inspectors needs to bear in mind that the best time to gather data (eg about seepage) from a data gathering and inspection point of view is during a high water event when the levee is being challenged. However, from a health and safety point of view, this may be one of the worst times to be on a levee. Always give high priority to the health and safety of staff making observations on the levee.

Causes of seepage issues

When flood stages are higher than the ground on the landside of the levee, water may seep through the soil from the higher elevation to the lower. Differences in elevation cause water to 'seek its own level'. Seepage through the levee may be made worse by the following conditions:

- leaking pipes that are through or beneath the levee and have improperly compacted material supporting/surrounding them, were abandoned and have corroded or collapsed, or have improper drainage control bedding (through-seepage or underseepage)
- dead or dying tree roots that are decaying within the levee and providing a path for seepage (through-seepage) (Aigouy *et al*, 2006, also see Section 4.5)
- pervious materials (used for levee construction) in the levee foundation (through-seepage and underseepage)
- burrowing animal holes, which may provide a shortened seepage path through the levee (through-seepage)
- loss of channel capacity, which may cause water levels in the channel to rise, remain on the levee toe and slope more frequently, and increase the probability of through-seepage, or increase water table levels, which could increase the likelihood of underseepage
- cracks in the impervious layer of the slope
- punctures in the clay blanket caused by activities such as dredging or excavation
- irregular settlement around stiff elements (pipe or other structures) in the earthen levee. This increases the vulnerability of the levee at the interface of the soil and the stiff element.

Preventing seepage-induced damage

Immediately report seepage (especially if any movement of soil particles is observed) to those responsible for inspections, investigations, and monitoring (see Chapter 5) so they can determine whether further action is needed. Document the seepage in great detail. Seepage conditions may progressively worsen over time due to repeated flood loading conditions, so diligence in observing, documenting and reporting any seepage condition is important.

Consider preventing seepage issues by prohibiting dredging and excavation in the zone of influence of the levee.

Table 4.17 explains how to prevent seepage-induced damage.

Table 4.17 Preventing seepage-induced damage

Observation	Preventive measures
Maintenance-related	
Sand boils	Put down a geotextile fabric to allow water to flow through the levee without moving material (see Sections 3.2.2.6 to 3.2.2.9 on seepage berms, filter layers, drainage systems and seepage barriers). Immediately report sand boils observed during any water stage condition as they could be very damaging to the levee.
Trees on the levee (especially fallen or dead trees and decaying roots)	When trees are removed, ensure their roots are removed down to 13 mm diameter and properly backfilled with compacted fill. See Section 4.5 for details about tree-related issues. See Chapter 6 for information about dealing with issues that occur on a levee during emergencies.
Cracking	Monitor the levee embankment and any seepage control features (such as seepage berms) for cracking. Re-grade and fill where observed. See Sections 4.8 and 4.9 for more information and guidance.
Seepage runoff	• control seepage runoff to prevent surface erosion of the embankment • maintain good grass cover on the slopes.
Encroachment-related	
Pipes installed through a levee	• use a sand filter around the pipe to protect the levee from piping along pipes • install all new pipes (especially pressurised pipes) through the levee above the elevation of the design flood stage. Ensure that they are inspected following installation • periodically inspect existing pipes that pass over, under or through the levee profile to identify and address issues as early as possible • see Section 4.15 for more details.
Design-related (may require a levee designer's assistance)	
Seepage water near the landside levee toe. This is a problem because saturated ground at the toe: • hinders vehicle access during normal maintenance or flood-fighting operations • softens the soils and reduces the stability of the landside slope • promotes vegetation growth that may obscure ground conditions and the presence of burrowing animals (note that the presence of moisture-loving plants on the landside toe of the levee may indicate a seepage problem. Look for the source of the water that is feeding the plants. A seepage control system may be needed).	Preventive measures for this condition include: • site grading to promote positive drainage away from the slope • shallow drainage collection/discharge features (ditches) to control the seepage • buried collection drains may be appropriate in some cases. Collected seepage could either be diverted into the internal basin or collected and pumped back into the waterway • lined or unlined drainage features • pumping stations • buried collector drains, which include piping • cut-off walls • relief wells. Seepage control structure design is discussed in Chapter 3, maintenance is discussed in Table 4.18. Design permanent collection features in accordance with the principles described in Chapter 9.
Pervious levee or foundation materials	Levees constructed on or with highly permeable material, such as sand, karst or limestone chippings, are likely to have seepage issues. See Chapter 7 for information on site characterisation and guidance on choosing a levee's location, and Chapter 9 for details on levee design.

1 2 3 4 5 6 7 8 9 10

Repairing damage caused by seepage

To repair sections of the levee that have been damaged by seepage:

- excavate and rebuild the levee section with an impervious material or insert a seepage barrier (see the seepage control systems section in Section 3.2.2)
- excavate levee embankment in disturbed area, re-compact material using proper construction methods. If repair cannot be completed due to an elevated river stage condition, then monitor seepage for soil migrating from embankment and settlement of levee crest or side slopes. Control seepage runoff to prevent erosion of embankment
- build an impermeable blanket on the waterside of the levee.

Determining when resolving seepage problems is beyond maintenance

A seepage issue is clearly beyond the realm of operations and maintenance if there are any visible indications of internal erosion such as cloudy water observed on the landside, a mud patch on the landside, or settlement near the seepage issue. Consult a professional designer for design-related seepage issues such as the presence of a sand boil. A sand boil could be an indication of internal erosion, liquefaction, or a critical hydraulic gradient, all of which are serious concerns and can quickly lead to failure. Recommend that the designer consult Chapter 5, which provides guidance on assessing levees to determine what action is needed. Chapter 5 should be used in conjunction with Chapter 9.

Seepage control structures

Seepage control structures are designed to help control the flow of water from a levee's waterside to its landside without impairing any necessary drainage from the landside to the waterside. They may also improve the levee's stability. Seepage control structures include seepage berms, stability berms, weighted filters, relief wells, seepage relief trenches, seepage collection drains and seepage barriers. Great care should be used if seepage barriers are constructed, as they may redirect seepage flow and force it to other areas of the levee.

Table 4.18 suggests general techniques for maintaining seepage control structures. The overall approach to maintain seepage control structures involves periodically observing any drainage emanating from seepage control systems for changes in performance. Low or no seepage could indicate that the feature is clogged, and increased seepage may indicate that the filter system has failed or that the impermeability of the waterside has decreased. A clogged seepage control system may cause hydrostatic pressure to build up in adjacent locations.

Table 4.18 Good practices for maintaining seepage control structures

Structure type	What needs to be maintained
Seepage berms	• maintain the design shape of the structure to ensure proper drainage and seepage control • regularly cut grass to monitor for animal holes or roots that penetrate the blanket • regularly check drainage if the system has any drainage features • maintain surface drainage to ensure proper drainage of the berm • regularly control nearby vegetation (tree roots may prevent it from functioning as intended, blown over trees could remove a section of the berm) • keep filter layers intact during repairs.
Stability berms	• maintain the design shape of the structure to ensure proper drainage and stability control • regularly cut grass to monitor for animal holes or roots that penetrate the blanket • prohibit removal of any material from the berm (eg by agricultural activities) that may reduce the berm's ability to support the levee • regularly check drainage if the system has any drainage features • keep filter layers intact during repairs.

Table 4.18 Good practices for maintaining seepage control structures (contd)

Weighted filters	• maintain the design shape of the structure • regularly check drainage if the system has any drainage features • regularly control vegetation • keep filter layers intact during repairs.
Pumping facility (for pumping collected seepage back into the waterway)	• test pumps regularly (at intervals recommended by the installer of the pump system) to be sure they are working properly • keep intake and outflow pipes clear of debris.
Relief wells Note that the service life of a relief well is limited. It depends on project-specific conditions such as water and soil chemistry, frequency of flow from the wells, well material type, and quality of maintenance	Problems with well efficiency may be caused by clogging, so: • periodically check the filter pack for migrated formational material • check for bio-fouling of the well screen due to build-up of bacteria • during maintenance, record the relief well effectiveness and compare it to the initial yields during installation and well development. Guidelines for maintenance of relief wells are included in USACE (1992).
Seepage collection drains and seepage relief trenches	• clean out the piping systems, if present • mow the grass in the trenches • keep the trenches free of debris and sediment to maintain their conveyance capacity.
Seepage barriers	Seepage barriers do not generally require maintenance. However, immediately report cracks, sinkholes or other anomalies observed nearby to the agency responsible for technical review. If the seepage barrier is penetrated by any O&M activity, reconstruct the penetration to restore the original design and functionality.

4.11 INSTABILITY

A levee slope is considered unstable when the levee's ability to react to a disturbing force (such as a flood) by maintaining or re-establishing its position has been compromised.

Recognising the signs of an unstable slope

Look for the following visual signs of an unstable slope in both the levee embankment and foundation soils below the levee:

1 **Slumps:** these appear as isolated areas of near-surface soil on a slope that has slightly to moderately dropped down-slope and exposed underlying subgrade (occurring only on the face with no evidence on the crest), see Figure 4.42.

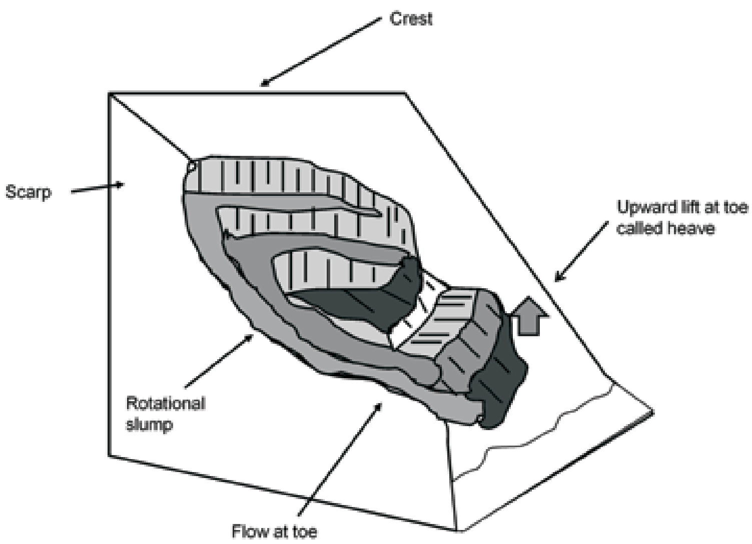


Figure 4.42 Cross-sectional diagram of slump in the levee

2 **Slides:** may be seen when the soil has dropped down a slope, exposing a near-vertical scarp (portion of the inside of the levee) at the top of the slide. This could cover a large area. See Figure 4.43 for a cross-sectional diagram of a slide in the levee.

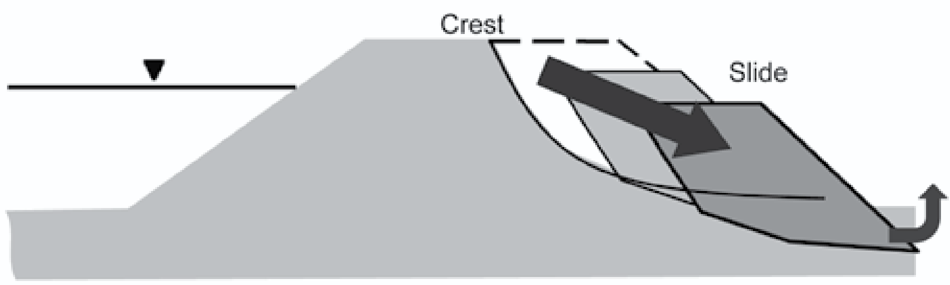


Figure 4.43 Cross-sectional diagram of a slide in the levee

3 **Tension cracks on the slopes or crest:** tension cracks (Figure 4.44) appear either as relatively straight single or multiple cracks parallel to and above the top (scarp) of a slide or slump. These cracks may be several centimetres to over a metre deep and may be as long as the slide. The presence of a tension crack may indicate that material down the slope from the crack could have moved slightly, but may not be obvious. A tension crack is different from a desiccation crack, which may have an irregular shape and run in any direction as the soil material shrinks and gaps open up (see also Section 4.12). However, a tension crack may be aggravated by a desiccation crack.

Figure 4.44 Tension cracks in the levee slope (indicating that slumping is occurring)

Causes of slope instability

A slope's instability may be contributed to or aggravated by surface erosion, over-steepened slopes, toe erosion, internal erosion, saturated levee embankment soil, construction activity, slope creep, and seismic forces. Over-steepened slopes are those that have become steeper than their original designs. Toe erosion is the wearing away (removal) of material at the levee toe. For more information on erosion, see Section 4.7. See Chapter 3 for more information about the relationship of these conditions to slope instability.

Preventing slope instability

Prevent slope instability by:

- inspecting levees regularly in accordance with visual inspection practices specified in Chapter 5
- repairing areas where slope instability or slope instability issues are found

- identifying historic stability problems, identifying their causes, and preparing a mitigation plan with engineering support
- avoiding over-steepened slopes. For example, rather than repairing settlement issues by just placing additional material on the crest within the same levee footprint, take steps to ensure that the side slopes are preserved as well and the footprint is increased as needed.

There can be other forms of mass instability. Box 4.40 discusses ways to prevent instability of peat levees due to dehydration.

Box 4.40 Preventing instability of dehydrated peat levees

During extended periods of drought, peat levees that become dehydrated can shrink and lose their weight – a situation that can predispose them to overtopping and instability (due to the uplift) and ultimately lead to a breach. Peat levees in Wilnis (the Netherlands) and on the Grand Canal in Ireland (1989) have failed for these reasons. In the Netherlands, both structural and non-structural maintenance measures are used to prevent dehydration of peat levees during extended droughts.

Structural maintenance measures: these are designed to reinforce the levee and make it more stable, and include:

- applying a clay cover on the inner slope to reduce dehydration
- filling any landside drainage ditches with soil.

If these measures are not executed properly, however, they may increase the risk of instability due to the rise of the phreatic surface during events with high water after rain. (Note that using impermeable levee reinforcements, such as a sheet pile, may contribute to extensive dehydration of the levee during a drought.)

Non-structural maintenance measures: these include:

- keeping the grass short (to reduce dehydration) by mowing or grazing
- increasing the water level in the ditches on the landside of the levee to help rehydrate the levee
- spraying water on the levee (see Figure 4.45).

Figure 4.45 Spraying water on levee (courtesy Water Board of De Stichtse Rijnlanden)

Avoid over-watering the levee, as it can create additional stability concerns. Non-structural measures can sometimes take the place of costly structural ones, but they require frequent inspections of the levee during a drought, constant monitoring of meteorological data in order to be aware of drought conditions, and extra care in the preparation of emergency measures.

If peat levees become overly dehydrated, they also run the risk of becoming water repellent (see Figure 4.46). This phenomenon is caused by a 'skin' that forms around the soil particles of dehydrated organic soils like peat during a complex process in which micro-bacteria are formed. In this 'hydrophobic' state, water remains on the surface, instead of being absorbed into the ground. Hydrophobic soils can be treated with a wetting agent that reduces the surface tension of the water and allows it to penetrate and wet the soil. For more information on hydrophobic soils, see Sunderman (1983).

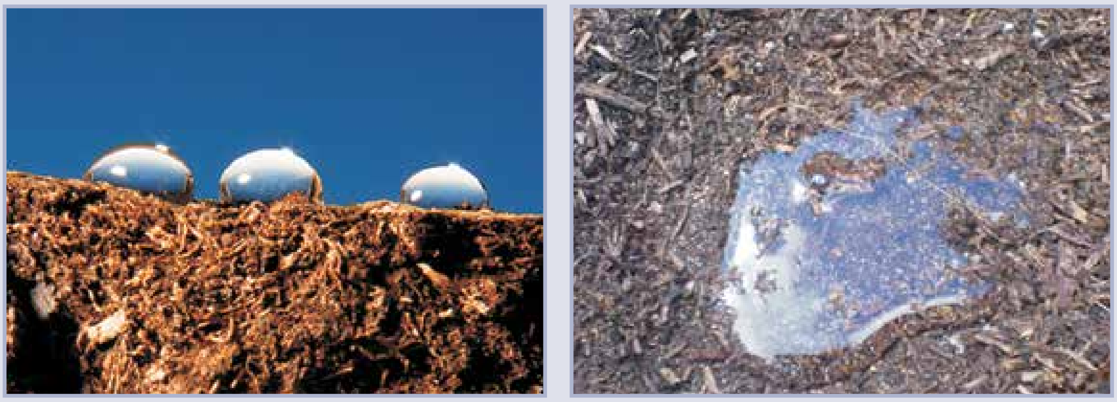


Figure 4.46 Examples of water-repellent peat levees (courtesy of E Elsen (a) and B James (b))

Repairing slope instability

Box 4.41 gives a suggested repair sequence that is applicable if the repairs for slope instability are within the scope of maintenance. See Chapter 6 for emergency repairs and Chapter 9 for remedial measures for rehabilitation. Chapter 5 discusses levee condition assessment. It provides guidance on how to assess the levee to determine what action is needed and should be used in conjunction with Chapter 9.

Though all slope stability issues should be taken seriously, the following guidelines may help maintainers prioritise these issues in terms of the urgency associated with their being repaired.

Most urgent

Any issue that:

- affects the entire crest
- affects the crest level in any way
- affects a significant length of levee
- appears during a flood event
- could significantly reduce the levee cross-section.

Less urgent

Shallow surface slumping occurring over a long period of time (affecting either the landward or waterward slope but not both).

Box 4.41 ***Suggested sequence of activities for maintenance repairs of slope stability problems***

1	Remove the slide debris down into stable levee or foundation material and stockpile. If removal exposes drainage material, a levee penetration, or unstable ground, contact a levee designer for guidance.
2	Achieve a level to slightly inward-sloping ground surface.
3	Scarify, moisture-condition as necessary and re-compact the exposed native ground.
4	Backfill and compact the soil, after repairing the slope instability problem, using current engineering good practice.
5	Ensure the soil is moist. Select soil that has similar properties to the adjacent soil. Compact it in 0.20 m loose layers to 95 per cent of optimum density (see Chapter 9).
6	Perform tests to verify compaction and moisture. Bench the repair area at least 0.61 m horizontally into stable existing foundation or levee material.
7	Over-build and cut back slopes to desired slope geometry. Do not trackwalk slopes. (trackwalking is a field term used to describe a track-mounted bulldozer passing back and forth over newly placed fill, typically on a slope where compaction equipment is much less effective. ‘Tracks’ are the circular, studded metal feet that move the dozer forward or backward. This method is not often approved because the tracks are built for low pressure ground contact, so they do not compact soil deeper than 50 mm or 75 mm. At best, they rough up and tighten up the upper 50 mm or so of the ground surface).
8	Vegetate or armour the rebuilt slope consistent with adjacent slopes.

Determining when repairs are beyond maintenance

When any slope instability issues are observed, contact a levee designer for assistance and to ensure that the issues are adequately resolved. This is particularly important if any soil removal exposes a cut-off, a levee penetration or unstable ground. If the crest is affected by the issue, it should be treated as urgent. If more than half of the required cross-sectional crest is affected, it should be treated as an emergency if the levee is expected to function as intended during the next flood event.

Slope instability issues typically indicate that there are profound underlying issues within the levee that need to be resolved before a permanent solution can occur. For example, localised slope instability observed on a newly constructed levee may be due to poor foundation material (such as a buried channel). Slope stability issues observed during a high water event may indicate imminent levee failure (which may be due to pressures within the levee or foundation material).

4.12 CRACKING

Cracks are narrow openings in the levee embankment that generally occur in one of two main forms:

- structural cracks (also known as tension cracks)
- desiccation cracks (also known as shrinkage cracks).

Structural cracks appear at the surface of a soil mass, often adjacent to a retaining wall or the top of a failing embankment slope. Desiccation cracks occur when the levee is dry and the soil shrinks, typically because of the highly plastic nature of the embankment material. Desiccation cracks are generally smaller but more extensive over the levee surface than structural cracks. It should be noted that structural cracks are indicative of a potentially serious issue that should be investigated to determine the cause and remediated in an expedient manner.

Why cracks are a concern

Water entering the embankment through surface cracks could reduce the soil strength of the levee. Depending on the location of the cracks, their depths and widths, and the composition and condition of the embankment, the weakening of the soil may result in slope slides. Slope slides may compromise the stability of the embankment and, in turn, reduce the ability of the levee to perform as designed during high water events.

Climate, soil gradation, surface drainage and surface protection are the primary factors contributing to the formation of cracks. Cracks are more common in geographic regions where temperatures vary a great deal and where the embankment soils are fine-grained, such as clays and silty clays, as opposed to coarse-grained sands or silty sands. Table 4.19 lists the characteristics of desiccation and structural cracks and additional reasons why these cracks are a concern, while Box 4.42 gives an example.

Box 4.42 Large shrinkage cracks can lead to seepage

The spring and summer of 2003 and 2011 in the Netherlands experienced extreme dry periods. During subsequent levee inspections, many cracks in the length of the levee were observed. Some of the larger cracks were on the levee crest, and some cracks were perpendicular to the levee and water was almost seeping through the levee. On peat levees, a sufficiently deep clay layer can help prevent shrinkage cracks that could lead to seepage, as can a well-maintained grass cover. Section 4.5 provides additional information on maintaining a good grass cover.

Table 4.19 Characteristics of desiccation and structural cracks

Type of cracks	Typical level of concern	Characteristics	Why they are a concern
Desiccation cracks (Figure 4.47)	Low	• vary in size by material type • appearance of the crack varies based on the moisture content (may exhibit a blocky pattern common in very dry or desiccated clayey levee surfaces) • generally parallel to the crest, although they may be parallel or perpendicular • generally narrow and shallow and extend 0.3 m to 0.5 m into embankment and/or adjacent riverbank • normally close during wet periods • typically smaller than structural cracks, but more extensive over levee surface.	• extended dry periods may induce larger and deeper cracks into levee • repeated wetting and drying fatigues the embankment soils, reducing embankment shear strength and possibly resulting in a shallow slope (slough) slide • material or debris may enter the crack when it is open and keep the crack from closing properly when it is wet.
Structural cracks (Figure 4.48)	High	• may run parallel or perpendicular to the levee crest • may result from embankment movement or duress, such as settlement, sliding and/or soil-bearing type failures.	• may appear as small displacement cracks and grow substantially depending on the mechanism for movement or duress • should be brought to the attention of a geotechnical engineer for evaluation of foundation conditions.

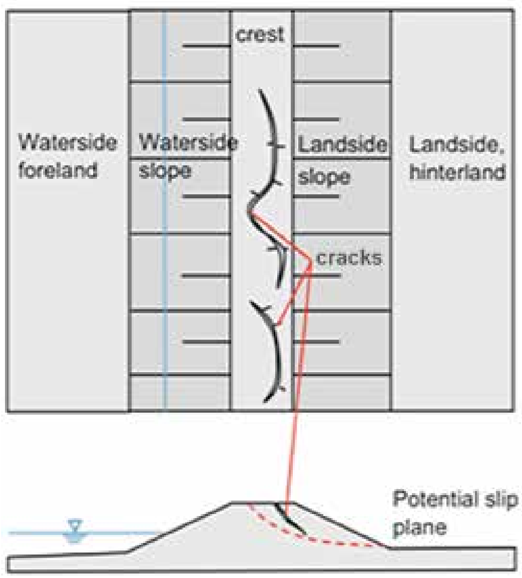


Figure 4.47 Plan and cross section view of desiccation cracks on the crest

Figure 4.48 Structural crack running parallel to levee crest

Preventing cracks

Regular inspection of the levee embankment is an important step toward catching cracks in their earliest stages. Table 4.20 lists several crack-related issues that may arise during inspections and suggests some preventive measures for them.

Table 4.20 Issues with cracks and suggested preventive measures

Observation	Preventive measures
Maintenance-related	
Cracks that continue to grow in length and width	• periodically inspect all cracks to document any increase in length or width • mark growing cracks well and evaluate on a weekly basis for an unloaded condition and at least daily for a loaded condition. Longitudinal cracks are more likely to result in slope slide because they reduce the resistance of the soil mass. Continued growth of longitudinal cracks could be a sign that a slope may be under duress and that an initial mode of failure exists • seek appropriate geotechnical expertise to determine what actions may be necessary to stop further embankment degradation. See Chapter 5 for more information on the visual inspection process.
Ruts and depressions	• use proactive maintenance practices to ensure that the levee is kept relatively free from rutting and depressions. Ruts and depressions may retain water that could seep into the embankment through surface cracks or allow water to pond, then dry, making the crest more susceptible to cracking and more vulnerable if overtopped. See Section 4.8 for more information.
Inconsistent turf cover	• maintain a good stand of surface protection, such as a durable turf that is well suited for the climate • use deep-rooted grasses to provide better protection to the surface soils. A well-established root system helps support the soil mass, reducing the tendency for sliding. See Section 4.5 for additional information.
Cracking on riverbank adjacent to the levee	Monitor and evaluate all visible areas of cracking to ensure they do not develop slope stability problems that could threaten the levee's stability.
Design-related	

Type of embankment materials (ie clays vs. silts and sands)	• knowing the embankment material will make it easier to assess the severity of the types of cracks that are discovered. For example, it is important to keep in mind that highly plastic clays are very susceptible to both:
	  ◦ surface cracking during dry summer seasons (desiccation cracks) when the embankment's water content is reduced and surface soils shrink
	  ◦ structural cracking, which can sometimes reach over a metre in depth during sustained dry periods. This could cause seepage in extreme cases with sudden rises in water levels
	• remember that clays, in general, are the most durable and impervious levee material, while silts and sands are more susceptible to erosion and surface water damage.

Repairing cracks

Table 4.21 suggests repair methods for minor cracking problems.

Table 4.21 How to repair cracks

Observation	Suggested method of repair

Cracking that may compromise the integrity of the levee slope (typically structural cracks)	• dig out the entire cracked area. Rebuild the levee using soil similar to the adjacent soil with appropriate backfill and compaction techniques
	• apply temporary repairs until permanent ones can be made. These may include:
	  • using metal or wood pins to stake high density polyethylene (PE) sheet in place over crack
	  • applying sprays, such as soil cement, that claim to adhere to non-vegetated levee surfaces and act as a surface protectant. These products are new and still under review (as of 2012) but may provide some benefit. For example, they may help to hold moisture in the levee and help keep the surface from cracking.

Desiccation cracks	• typically, desiccation cracks do not require repairs, but they should be monitored. If the cracks grow in length and width or appear to threaten the integrity of the levee, contact an experienced geotechnical expert to determine the appropriate action • dig out the entire cracked area. Rebuild the levee using soil similar to the adjacent soil with appropriate backfill and compaction techniques • if desiccation cracks are of some concern, one method to help prevent their recurrence is to scarify and place top soil over the cracked area and re-seed to establish a good stand of turf.

Determining when repairs are beyond maintenance

For cracks that are not suitable for simple, permanent repairs and for cracks that include vertical displacement (particularly on only one side of the crack), contact a geotechnical engineer for assistance. See Section 9.12.2 for more information.

4.13 LEVEE SLOPE AND BANK PROTECTION

Levee slopes/banks are commonly protected against the erosion of waves and currents by a facing of stone, concrete, vegetation or other natural and environmentally acceptable material. The type of material is chosen based on the:

- level of attack the levee slope/bank needs to be protected against
- environmental impact and the appearance of the levee protection method
- suitability for the location (in particular, coastal or fluvial)
- availability of materials
- construction and maintenance costs
- effect on channel capacity.

On coastal levees and lakes, slope and bank protection is exposed to a tidal range and higher wave effect instead of long periods of high and low water. Storm events are more important than high water events to coastal levees. In many cases the coastal levees are also much larger and the impact of levee failure is much greater. The difference in the types of events that coastal and riverine levee faces change the way a levee has to perform. Along the coast, levees have to resist severe wave attacks and higher water. Along the rivers, levees have to withstand high water pressures both within and under the levees.

Reasons for maintaining levee slope/bank protection

The most common, resilient and cost-effective measure against external erosion is a well-maintained, robust grass covering on the surface of the levee. But levees on fast flowing rivers, on bends in rivers, in tidal areas and along the coast are exposed to larger hydraulic loads, which can easily exceed the strength of grass. The purpose of additional levee slope/bank protection is to maintain a stable channel, coastline, and levee slope and to reduce the destabilising effects of erosion. If required levee slope/bank protection maintenance is not carried out, the levee slope/bank protection may not function as intended during a high water or storm event.

On coastal levees, the monitoring and maintenance of levee slope/bank protection is critical, because a displaced rock can be an early indicator of early issues that can quickly lead to slope destabilisation. A displaced rock can indicate that wave impact is more than the design anticipated or that there are problems on the inner foreshore (tidal zone) such as erosion, settlement, or liquefaction issues. Close attention should be paid to the coastal levee toe protection, in particular, as many slope stability issues start with missing rocks in that area (which can be challenging to spot).

Methods of levee slope/bank protection and their maintenance

Tables 4.22 to 4.24 list a variety of levee slope/bank protection methods in three categories of options:

- stone and concrete
- cage, bag and block
- softer engineering.

The long-term effectiveness of these protection systems against erosion is equally the responsibility of the levee designer (see Chapter 9) and the levee maintainer. The designer needs to provide a solution that takes the circumstances outlined above into account, in particular the suitability of the location. The maintainer's most enduring repairs will be those that:

- keep in mind the environmental principles introduced in Chapter 2
- make every effort to balance the economic, environmental and social (eg recreational) effects of the repairs against the risks of failing to deliver each of these.

Table 4.22 Stone and concrete methods of levee slope and bank protection

Method	What it is and where it could be used	Maintenance needs
Rip-rap (courtesy USACE) 	Rip-rap is carefully placed or dumped natural stone, typically in two layers. The stability of rip-rap depends on the selected stone size. To ensure stability of the rip-rap slope the toe should either extend below bed level or contain additional stone that can be released if further channel erosion occurs. The size of the stone should be chosen with consideration for its stability and the likelihood of it washing out, given the prevailing wave and current conditions. Rip-rap is used on levee slopes/banks, straight or meandering rivers and coastal locations. Rip-rap can be used to cover and reinforce pre-existing revetments.	• rip-rap should require little maintenance except inspection and monitoring for movement, loss of rocks, a change in rock size (especially after cold events), or toe erosion • saplings or vegetation growing between the rip-rap stone should be removed to avoid unduly displacing the rip-rap in the future. Note that one of the benefits of the rip-rap is that some damage or displacement may be allowed for in the design and may be tolerable.
Rip-rap grouted with asphalt or colloidal concrete (courtesy Rijkswaterstaat) 	The stability of rip-rap can be increased by penetrating the stones with asphalt or colloidal concrete. If necessary, the revetment can be covered with lava stone to increase the growth of algae.	• grouted rip-rap needs little maintenance • the rip-rap should be monitored for loss of stones or infill and replacement when needed • any saplings or shrubs should be removed before they become established.

Stone pitching (courtesy Y Provoost) 	Stone pitching involves the careful placing of loose stone (typically in the size range 0.2 m to 0.5 m) onto a prepared sloping levee slope or bank. Each piece of stone is placed so that voids are kept to a minimum. Stones can be set in concrete or carefully placed on earth. Voids are necessary to avoid water pressure and to allow for the growth and well-being of desirable plants and animals. If there is an ecological need, the elements can be covered with lava stone. Voids can be filled with gravel for additional interlocking. Stone pitching needs to be combined with a suitable toe construction and a toe protection system (eg of rip-rap). Pitched stone can be used: • along medium and faster flowing rivers • along coastlines with wave attack not exceeding about 3 m • in recreational areas along the coastline (easy to walk on) • in areas that are ecologically important.	• stone pitching often needs little maintenance • constructions should be monitored for loss of stones or infill, which should be replaced when needed • any saplings or shrubs should be removed before they become established.
Basalt (courtesy Rijkswaterstaat)  	Basalt is natural stone that forms in hexagonal columns and should be carefully placed by hand on a filter/gravel layer (and geotextile). The size of the elements ranges between 0.2 and 0.5 m. Density is approximately 2900 kg/m^3. Voids can be filled with gravel for more interlocking. The resulting construction is easy to walk on. It needs to be combined with a suitable toe construction and a toe protection system (eg of rip-rap). Artificial concrete versions of basalt columns are available. Basalt columns can be used: • along medium and faster flowing rivers • along coastlines with wave attack not exceeding 2 m • in recreational areas along the coastline • in areas that are ecologically important.	• needs little maintenance • monitor for loss of stones or infill and replace when needed • remove any saplings or shrubs before they become established.
Asphalt revetment 	Asphaltic revetments are available in different forms (dense sand asphalt, open stone asphalt) for paving the levee slopes. It is a practical alternative if traffic needs to run on the slope or berm. The thickness of the asphalt can be adjusted depending on the circumstances.	• asphaltic revetments generally need little maintenance • revetments should be monitored for erosion and more regularly the longer it has been in place • the risks of water overpressure underneath the asphalt should be noted. Cracks should be monitored and checked for signs of sand washout

Asphalt, open stone asphalt (courtesy Y Provoost) 		• any saplings or shrubs should be removed before they become established.
Steel sheet piling  ***Sheet piles used as levee slope protection along the Loire River, France (courtesy DREAL Centre, France)***	Steel sheet piling is made from steel sheets driven into the bed vertically and tied back to the levee slope or bank with anchors. Painting the sheet piling can improve its appearance and reduces corrosion. Steel sheet piles are commonly used on vertical banks and fast flowing rivers.	• steel sheet piles should require little maintenance other than repainting in some situations • they should be monitored for movement and corrosion, especially near low water.
Bulkheads 	Bulkheads are structures designed to retain or prevent sliding of land. They are composed primarily of steel sheet piles and are used to prevent loss of land in areas where mitigation against flooding is of less importance.	• steel sheet piles should require little maintenance other than repainting in some situations • they should be monitored for movement and corrosion especially near low water.
Seawalls 	Seawalls are designed to prevent flooding and coastal erosion. They are composed of many different materials including concrete, steel sheet pile and rip-rap and are designed to resist wave attack and overtopping.	• seawalls should be monitored for defects such as cracking, corrosion, rip-rap movement and toe erosion • appropriate repairs should be carried out.
Concrete/pointed masonry-covered levee slopes  ***Waterside levee slope covered with concrete***	Used in reaches of a river with very limited area between the toe of the levee and the channel when the levee is subjected to wind-driven or navigation-generated wave action. Masonry is commonly used in France on spillways because the masonry protects the spillways against water erosion during overtopping.	Monitor the slabs for excessive cracking and erosion under the slab.

Table 4.23 Cage, bag and block methods of levee slope/bank protection

Method	What it is and where it could be used	Maintenance needs
Gabions	Gabions are wire mesh cages filled with loose stone to provide flexible structures for levee slope/bank protection. They are used on steep banks and fast flowing rivers.	• gabions are vulnerable to corrosion of the wire cages, but this can be reduced if the cage wire is protected or manufactured of stainless steel • woody vegetation growth should be removed to prevent displacement of cages • gabions should be monitored for damage caused by vandalism or floating debris during high flows.
Bag work	Bag work bank protection is made of concrete-filled hessian bags laid in a brickwork pattern. Used on vertical banks and fast-flowing rivers.	Little maintenance needed except inspections and monitoring for displacements and toe erosion.
Cellular blockwork	Cellular blockwork is composed of open-type cellular blocks laid either as individual units or as a mattress. Voids are filled with soil and seeded to give a more natural appearance. These can be made of pre-cast concrete or synthetic materials such as certain plastics. Used on fast flowing rivers and meandering rivers.	• inspect regularly for loss of infill • any saplings or shrubs should be removed before they become established.

Table 4.24 Softer engineering options for levee slope/bank protection

Method	What it is and where it could be used	Maintenance needs
Geotextiles	Geotextiles are flexible fabrics and/or mesh matting that can be placed over a slope or area of potential erosion and pinned in place to provide stability. Geotextiles can also provide extra protection against erosion when used jointly with another solution (in this case, geotextiles are placed between the protective material and the levee toe). Geotextiles can also be used to reinforce grass turf. Uses: • on slow to medium flow rivers or on lowland meandering rivers • should be used only rarely and considered temporary.	If not covered, geotextile is susceptible to being torn by floating debris during high flows, and breakdown and deterioration from exposure to sunlight. Close and frequent monitoring is required and occasional re-pinning of sections that have lifted up from the face of the levee.
Faggots	Faggots are bundles of osier willow, poplar or hazel stems of no greater than 25 mm diameter, bound together with string or wire and buried in the levee slope/bank, to allow vegetation to develop over them. Uses: • on slow flowing lowland rivers • can be used to create meanders in straight rivers • on banks where there is no concern that the roots will increase the risk of seepage within the levee.	Faggots are a temporary solution designed to establish natural vegetation of the adopted willow, poplar or hazel. To maintain them: • during the initial growth make sure that the ties, wooden piles and protective grating are not damaged • water the vegetation if there are low-water or hot dry periods • remove any material that is not part of the faggot • plant any cuttings in low density areas • periodically trim growth. This is particularly important on narrow channels where vegetation growth could obstruct flows.

Table 4.24 Softer engineering options for levee slope/bank protection (contd)

Hurdles	Hurdles are wattle hurdles laid flat on the surface of the levee slope/bank and firmly staked. They provide temporary erosion protection until vegetation grows through and the panels rot away. Uses: • localised channel training or narrowing when placed vertically • slow to medium flow rivers or on lowland meandering rivers with sloping levees • banks where there is no concern that the roots will increase the risk of seepage to the levee.	Hurdles are a temporary solution while waiting for natural vegetation to establish. • during initial vegetation growth, make sure that the ties, wooden piles and protective grating are not damaged • water the vegetation if there are low-water or hot dry periods • remove any material that is not part of the hurdle • plant any cuttings in low density areas • periodically trim growth. This is particularly important on narrow channels where vegetation growth could obstruct flows.
Fascines	Fascines are rough bundles of brushwood with willow plants in the bundles. Uses: • to strengthen levees from erosion • on the waterside of levees that are wider than the necessary design, where there is no concern that the roots from the fascines will increase the risk of seepage to the levee.	• during the initial growth, make sure that the ties, wooden piles and protective grating are not damaged • water the willows if there are low-water or hot dry periods • remove any material that is not part of the fascine • plant any cuttings in low density areas • periodically trim growth. This is particularly important on narrow channels where vegetation growth could obstruct flows.
Willow spilling (see Box 4.43)	Willow spilling involves weaving willow withies (thin willow branches) between fresh winter-cut willow stakes to form a fence-like structure. Over time the willow will sprout, providing a living protection and additional stability. Uses: • on slow to medium flow rivers or on lowland meandering rivers with steep or vertical levee slopes/banks • used on levee slopes/banks where there is no concern that the roots will increase the risk of seepage to the levee.	Periodic trimming of growth. This is particularly important on narrow channels where vegetation growth could obstruct flows.

The maintenance needs of a coastal levee generally do not differ from that of a fluvial levee. The maintenance practices are determined by the structure of the embankment and protection rather than location. Harder methods of levee slope/bank protection, such as rip-rap or gabions, require less routine maintenance than softer methods of levee slope/bank protection, such as willow spilling (Box 4.43) or just grass. All levee slope/bank protection should be inspected at a frequency proportionate to the risk and following a major flood or storm event.

Box 4.43 *Willow spilling in England and Wales*

Figure 4.49

Willow spilling in England and Wales (courtesy Environment Agency)

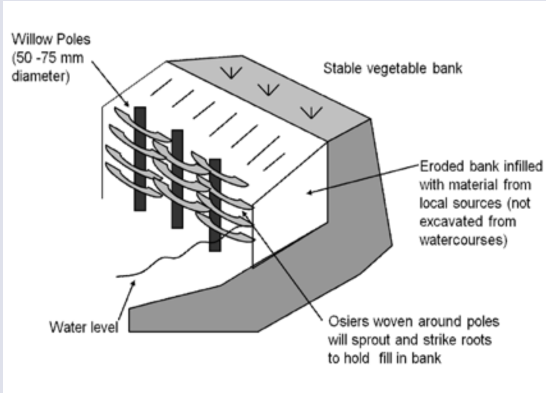


Figure 4.50

Schematic of willow spilling method

A large amount of riverbank in England and Wales is protected by willow spilling (Figure 4.49). Since 1992, willow spilling has become one of the most widely used soft engineering methods for erosion control in England and Wales. Long willow canes are woven around vertically driven willow poles (Figure 4.50). Because structures are living, resistance to erosion increases over time. Willow spilling has, in addition to its geotechnical stabilisation function, numerous ecological and economical benefits for river restoration, habitat enhancement and community engagement.

When to consult a professional levee designer

Consult a levee designer if any of the design attributes listed no longer meet their design requirements and specifically if:

- there is any sudden failure or gradual deterioration of an existing length of rip-rap (unless the original design and historical performance are well documented and the nature of the failure and the characteristics of the event that caused the failure are well understood, it should not be presumed that the original design was adequate. Reinstatement to the original condition may not be sufficient to resist what would be the current design event)
- a new revetment solution is needed for any reason
- the levee or revetment has been moved and a new solution is required
- changes in the geometry indicate that the original revetment solution is no longer working and a new solution is required
- design deficiencies have been identified. For example:
  - fines are observed moving through the levee
  - the existing solution is not sufficiently durable and a new solution will be required
  - rock is undersized. Changed hydraulic/hydrologic conditions could result in insufficient protection from erosion or wave damage
  - coverage of revetment is insufficient. Changed hydraulic/hydrologic conditions could result in the revetment not extending far enough upstream or downstream
  - the quantity of stone in the 'launch' section is insufficient to protect against anticipated scour.

See Box 4.44 for a case study that uses several methods of levee slope/bank protection.

Box 4.44 Case study that uses several methods

Softer engineering options and rip-rap were used on adjacent stretches along the Bouteille levee, which protects the city of Orleans, France. Located on the outer edge of a curve of the Loire River, the levee was very susceptible to external erosion. The flow of water had already begun to damage the downstream portion of the levee (800 m long) and the riverside of the levee slope could not be maintained adequately due to lack of a maintenance trail. Woody vegetation had grown uncontrollably. To restore the levee toe and avoid further damage:

- the downstream banks were covered with rip-rap to protect the levee toe against bank caving and to form a maintenance trail (see Figures 4.51 to 4.53)
- the upstream banks were protected with willow fascines.

Also, woody vegetation was removed from the levee, including the roots, and the levee slope was restored.

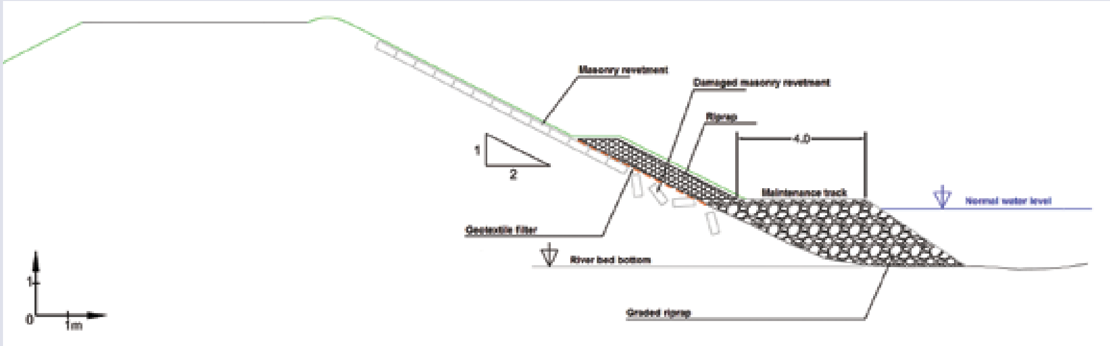


Figure 4.51 Levee cross-section with rip-rap reinforcement (courtesy to Nicolas Auger)

Figure 4.52 Levee cross-section with rip-rap reinforcement

Figure 4.53 Placing rip-rap at levee toe (courtesy DREAL Centre)

The location of each solution was carefully chosen. On areas where the levee is in constant contact with the river, rip-rap was used because of its cost/benefit ratio, its durability and ease of maintenance. For the portion above the water, geotextiles were used (geotextiles could not be used under water, which is why the rip-rap was critical). A portion of the levee that is more than 5 m wide allowed the use of fascines, which would not normally be used, due to concerns that the roots might increase the levee's risk of seepage.

4.14 CLOSURE STRUCTURES

Closure structures are removable, watertight mechanisms used during flood events to seal off levee segments whose heights have been reduced below design grade by an intersection with a roadway, railway or other crossing. Two abutments (a cast-in-place concrete wall perpendicular to the levee or flood wall at each end of the opening) and a sill (the base of the opening that carries the closure's weight) typically frame the structure's opening. Cut-off sheeting is often driven below the opening to prevent undermining or seepage, and sandbags and flexible seals may be used to prevent leaking. The structures are normally left in the open position and are closed (or installed) in advance of a flood event to provide a watertight seal.

Note

For the purpose of this handbook, closure structures are of the order of 10s of metres long rather than 100s to 1000s of metres.

Note

Closure structures do not include demountable defences located on top of a levee, which raise the level of defence at time of flooding. However, the principles of maintenance of demountable structures are similar to those of closure structures.

Types of closure structures

Chapter 3 contains information related to types of closure structures, which are:

- **gated (movable) closures:** include swing-hinged gates (single-, double- or multiple-leaf), overhead vertical operating gates (similar to sluice gates), and rolling gates (wheeled or overhead trolley)
- **assembled closures:** include assembled trusses with purlins (members that span between girders) supporting sheeting panels, pinned frames with purlins supporting sheeting panels, stop logs (with and without intermediate posts), panels supported by purlins without intermediate supports, and single piece bulkheads for pedestrian openings
- **sandbag closures:** are physical barriers created from stacked sand-filled sacks. Some levees have openings in the levee system that are planned for closure with sandbags. Challenges related to sandbag closures can include:
  - not having the appropriate materials close enough to the site to be able to assemble the structure in time
  - improper assembly of the sandbag closure due to filling the bags too full (they should only be filled one-third of the way)
  - stacking the bags on top of each other instead of staggering the bags (the way bricks are staggered in a wall)
  - designs that require sandbag closures for openings that are too tall or too wide for sandbag closures to function properly

  the tallest recommended height for a single stack sandbag closure is 1/3 m (1 m for a pyramid placement) and the widest recommended closure is 30 metres. These are general guidelines. Consider site-specific characteristics such as the number of people able and willing to place the bags, the amount of advanced warning time the operator will have, the consequences of not installing the closure in time, how long it will takes the operator to get the materials to the site, and the strength of the foundation material that the bags are to be placed upon. A test assembly of the sandbag closure should be done annually. See Chapter 6 for additional information on sandbagging during an emergency situation and how to properly assemble and place sandbags
- **earthfill closures:** are filled with compacted, locally available earth and are generally covered with plastic sheeting and sandbags. Properly operating equipment, trained personnel and other components are all integral to the successful operation of a closure structure.

Wide openings with assembled closures may have removable intermediate posts and bracing to withstand flood loadings. For truss-supported or pinned frame-supported wide openings, the supporting components are fixed in place by cast steel anchorages that are deeply anchored into the concrete sills. These anchorages are accessed by removing their robust cover plates. The trusses or pinned frames are fixed in the anchorages by steel shear pins. See Figures 4.54 and 4.55.

Rubber, neoprene, or composite belting provides a continuous perimeter seal between the fixed structural elements (sill and abutments) and the movable closure components. After the structural components have been installed, plastic sheeting and sandbags are placed on the flood side.

Preventing closure structure failures

Table 4.25 describes four main causes of closure structure failures and how they may be prevented.

Figure 4.54 Truss shoes in their cast steel anchorages

Figure 4.55 Intermediate truss supports pinned in place

Table 4.25 Common causes of and preventive measures for closure structure failures

Observations	Preventive measures
Alarm failure	
Alarm alerting the operator to close the structure does not work	• regularly test the alarm and check the operator notification system to ensure they are working as planned • check may include practice calls to the operator or to the contractor who installs the closure.
Mobilisation failure	
Debris or soil in the sill and recesses prevent efficient assembly	• regularly clean sill and maintain recess seals, including anchorage boxes • perform regular practice installations. As crews become well-practiced, the clean-up process should become more efficient • maintain seals or fill recesses with foam plastic inserts to prevent accumulation of water etc • use equipment to remove the debris, when needed • remove pack rust mechanically • lubricate moving parts on a regular cycle • install the closure using shims, if necessary.
Debris, corrosion, or improper lubrication prevents gated closures from closing	• regularly remove debris • lubricate moving parts • maintain a protective coating system, such as paint, to prevent corrosion damage.
Installation is improperly done	• maintain installation instructions, diagrams and parts lists • conduct trial closings.
Multi-closure system is installed in the incorrect order	Maintain order of installation lists that are tied to flood stages, water levels etc.
Component failure (storage issues)	
Stop logs and other components are not marked or ordered properly, resulting in a fit that is not watertight	• ensure components are properly marked and stored to allow proper sequence of installation • use weld beads to permanently inscribe parts designations (beads remain if paint fails) • provide clear, simple checklists for order of assembly and materials.
Components are deteriorating: wood is warping, steel parts are corroding	• inspect storage vaults for leaks or surface drainage problems • replace wood parts at regular intervals as they decay, warp, and twist • store components on blocking to keep them dry in a secured, dry, and ventilated storage location • sandblast and re-paint corroded steel components • inspect welds for cracks and signs of distress • cover installed wooden stop logs (that are subject to leakage) with plastic sheeting, sandbagging or other methods to seal gaps • consider replacing wooden stop logs with aluminium or glass fibre ones. They require less maintenance and are easier to handle, but are more expensive.

Table 4.25 Common causes of and preventive measures for closure structure failures (contd)

Sandbag and sand quantity and quality are inadequate	• inspect empty bags regularly to ensure their viability and quantity, replacing deteriorated bags • low quality sandbags disintegrate over time • provide good quality, local sand supply.
Labour force is inadequately trained	Create, carry out, and document regular training programs
Stop logs or installation equipment are not available close to site	• store stop logs or other components near to the structure in a weatherproof enclosure, if practical, or if kept off-site, on a trailer for ease of transport. It may be necessary to offload equipment to assemble the closures • equipment needed to move components into the closed position should be readily available, well-maintained, and transportable to the closure site on short notice.
Structural failure	
Structure malfunctions during an emergency due to missing or damaged parts (pins, bolts, nuts, washers)	• store (in well-organised bins) and lubricate small parts, including pins, bolts, nuts and washers • replace damaged parts (most original components were fabricated from standard structural members and shapes) • maintain a list of closure parts • take regular inventories • maintain a supply of spare parts.
Anchorages are damaged or heavily corroded	• seal anchorage recesses to prevent intrusion of water or soil • provide positive drainage from recesses • repair damaged anchorages.
Gated closures that are permanently stored unprotected have been damaged by vehicles	• repair damage • consider installing bollards or other protective barriers to prevent damage by vehicle impact.
Closures do not seal properly. For example, sill is paved over with asphalt or railway track grades have been raised so that closure does not properly seal, differential settlement is leading to leakage around the structure, damaged or desiccated perimeter seals, damaged concrete sills (by spalling) are preventing proper bottom seals.	• inspect seals and replace as necessary • ensure that the roadway or railway owner understands the need to co-ordinate repairs and modification with levee owner/operator • work with a designer to repair differential settlement.

Closure structures should be well maintained and operators sufficiently trained. Closure malfunctions during an emergency may compromise an entire flood risk mitigation system. Trial installations should be conducted on a regular basis and maintenance records kept with the system's operation manual (see Boxes 4.45 to 4.47, and Section 6.2.3).

Box 4.45 ***US Army Corps of Engineers inspection and trial closing policies (USACE, 2010)***

The installation of a closure structure requires either human or automated action, so there is always the risk that an error will prevent proper and timely installation. Regular trial closings can lead to more efficient installation during an emergency, especially on structures that may be used infrequently, such as during higher magnitude floods. Test closings provide training and practice for operating staff and may identify any deficient materials or procedures.

An effective test closing should include:

- a detailed test procedure and an inventory of components and equipment as part of the system's operation manual (the O&M manual instructions should also be available during the test)
- timely co-ordination with the roadway or railway owner. Busy rail lines will likely require extra co-ordination to suspend service on the track to perform a test closing
- an adequate work crew whose members are familiar with the closing procedure
- an accurate timing of the test closing operation, including the effort expended to block the roadway or railway and gather the equipment, staff, and components at the site. The amount of time required to complete the installation in advance of rising waters should be established during the test
- having the test video recorded by the agency responsible for the closure installation. This video can then be used for training of new crew members, record keeping, and after-action review.

Once the test closing is complete, visually check the perimeter against the concrete abutments and sill for a tight seal.

Box 4.46 ***An example illustrating the importance of trial closings in the USA***

The intensity of the 2011 Ohio River flood revealed several maintenance issues with closure structures:

- the perimeter seals on some older closures had become less flexible and leaked excessively
- differential settlement had occurred at several closure sills and, as a result, the steel/aluminium panel components did not sit flush on the sill surfaces, resulting in high rates of leakage
- on one closure, the removal of overlying asphalt pavement (the sill had been paved over) destroyed an embedded steel anchorage.

Box 4.47 ***Competing for the fastest closure structure installation***

In Pennsylvania, USA, the Sunsbury Municipal Authority holds an annual competitive event to exercise their closure structures.

Background

The authority has six closure structures of two varieties:

- two sandbag closures
- four aluminium panels.

Closure structure installation is an extremely laborious task. During the 1996 flood event, 40 men were required for 5.5 hours to install one of the sandbag closures. Following the flood of 1996, the authority secured national government grant money to modify existing structures and permanently close those that were no longer needed, reducing manpower requirements. Modifications included redesigning one of the closures from a stop log closure to an aluminium panel closure, redesigning another closure from a sandbag closure to an aluminium panel closure, and removing one of the closures. The modifications resulted in a significant reduction in manpower requirements and installation times. For example, the closure that was previously executed with sandbags can now be installed with three men (an equipment operator and two labourers) in less than 12 minutes.

The competitive event

Each year in the autumn, the authority co-ordinates a simultaneous closure structure training exercise at various locations throughout the city. The exercise is scheduled as a competitive event where authority crews and volunteer crews from several fire departments compete against each other to beat previous installation times. The event rotates authority and volunteer crews in an effort to reduce response time during an actual flood event. Following each exercise, crews meet to discuss ways to improve on their time for the next training event. To date, training records indicate that each year the rotating crews have been successful at beating the previous year's installation record. In many cases, installation times have been reduced to just minutes.

Maintenance methods and Repairing closure structures

Table 4.26 suggests maintenance methods for several common closure structure conditions.

Table 4.26 Common closure structure conditions and related maintenance methods

Observation	Suggested maintenance methods
Closure sill and abutments are damaged from deterioration, settlement or seepage. Wide sills have settled differentially	Fabricate new stop logs or other components to provide a functional and sealable closure. If settlement needs to be repaired by pressure grouting below the sill, contact a designer. Care should be taken with pressure grouting to ensure that environmental considerations are taken into account and that the correct amount of pressure is used, so it does not cause an unanticipated increase in horizontal or vertical stresses.
Spalling or otherwise damaged concrete sills	See Section 4.17.
Railway track passes through an opening, causing a poor seal between sill and closure structure	• if top of rail is flush with top of sill, maintain the sill cut out on the inside of each rail to accommodate the wheel flanges • if the top of rail has a higher elevation than top of sill, maintain the elastomeric insert plugs that seal these openings.
Openings are no longer used or maintained by the user or flood risk mitigation system manager	Replace abandoned closure structures with properly designed levee or flood wall sections. See Box 4.48 for additional ways to seal abandoned structures.

Box 4.48 Removing abandoned closure structures, USA

If a crossing is no longer needed, eg a rail siding is abandoned, the levee owner/operator should not simply close the opening with the gate or stop logs. A permanently assembled closure structure is subject to corrosion, vandalism and other deteriorating effects. Instead, the owner should consult a levee designer about having the closure structure removed and replaced with a new section of levee or flood wall. A common method for permanently closing an opening is to fill it with a reinforced concrete slab designed to span between the abutments. The slab may require intermediate counterforts (or buttresses) for support. Another option is to mechanically roughen the concrete sill and abutment surfaces and fill the opening with properly moisture-conditioned, compacted clay. In such cases, it is good practice to overbuild the fill so that no portion of the original concrete abutment or sill is directly in contact with flood waters. This measure minimises seepage along the interface between the concrete surfaces and the clay fill.

Swing or slide gates may be permanently welded in the closed position, their steel components encased in pneumatically applied concrete.

Determining when repairs are beyond maintenance

Components of gated and assembled closures are often fabricated from rolled steel shapes and plates. Lack of proper corrosion protection systems, such as paint, and inadequate storage may lead to corrosion of the steel. Structural elements with a measurable loss of section should be inspected by a professional levee designer to ensure the integrity of the structure meets all safety requirements. Damaged parts should be replaced or reinforced.

Other situations in which a professional levee designer should be contacted include when:

- slab-jacking or similar procedures are required to repair settled closure sills or abutments. A levee designer should prepare the specifications and be present during the operation
- hydrologic studies and stream gage data may be required to create an order of installation plan for a multi-closure systems (if the document does not already exist)
- a permanent closure (a reinforced concrete wall or levee section) needs to be designed to replace an existing opening
- settlement issues cannot be resolved by creating new stop logs or other components to provide a good seal.

4.15 CULVERTS AND DISCHARGE PIPE SYSTEMS

Culverts and discharge pipes are pipe systems that are built under, over or through a levee to:

- provide drainage as part of the flood risk mitigation system (typically from the landside to the waterside). These culverts and pipes need to retain the flood water and are discussed in Part 1
- function as utility pipes for gas, irrigation, water supply or electrical/communications cabling. These pipes should not leak or damage the integrity of the levee in any way and are discussed in Part 2.

4.15.1 Culverts and discharge pipes

This section covers:

- why culverts and discharge pipes are a concern
- maintenance – avoiding internal erosion
- maintenance – avoiding external erosion
- maintenance – avoiding levee instability
- repairing culverts and discharge pipes
- maintenance of culverts and discharge pipes system components.

Why they are a concern

Culverts and discharge pipes are a concern when they are not properly maintained. Lack of proper pipe maintenance increases the risk that the levee system will fail as a result of internal erosion, external erosion or levee instability – the three failure mechanisms discussed in Section 3.5. The following sub-section suggests preventive measures and repair techniques that can be used when maintenance issues are observed. These measures and techniques are grouped according to the failure mechanisms they are intended to prevent, which are internal corrosion, external corrosion and levee instability. Also included is a separate section on maintenance of culvert/discharge pipe system components, which includes flap gates, sluice gates, slide gates, air relief valves and trash racks.

At the heart of proper pipe maintenance is a program of regular levee inspections. These inspections identify and report maintenance issues before they result in such serious concerns as pipe deformation, leakage or internal obstructions. For details on conducting and documenting inspections, see Section 5.3.

Maintenance: avoiding internal erosion

Pipe-related internal erosion may come from water seeping along the exterior of the pipe (as a result of improper soil compaction during construction) or from leakage caused by the pipe's deterioration over time. Deterioration in the pipe (eg holes, weakened joints) can shorten the levee's seepage path, resulting in the piping of soils, internal erosion and the eventual failure of the levee. The movement and loss of embankment soils may also make the levee slopes unstable.

Safety precautions for visual inspections

Safety precautions: note that the detection of these issues will typically require a visual inspection. This can be done either by trained staff physically entering the pipe or by video inspection. If people are to enter the pipes for inspection purposes, it is important that they have the proper training and they take all necessary precautions. For example:

- the pipe needs to be large enough and considered safe for entry
- staff need to have received proper training for working and performing inspections in confined spaces
- appropriate personal protective equipment (PPE) must be worn at all times
- gas monitoring equipment should be used if appropriate
- inspectors should not work alone.

Routine visual inspection of all pipes through the levee is highly recommended for early detection and prevention of pipe leakage issues in the levee. See Boxs 5.28 and 5.29 for details and examples. Table 4.27 lists the types of pipe leakage issues that may lead to internal erosion. See Table 4.30 for descriptions and images of how the suggested repair techniques should be done.

Table 4.27 Pipe leakage issues that may lead to internal erosion

Potential failures by internal erosion		
Observation	***Suggested preventive measures***	***Suggested maintenance/repairs***
Joint separation in concrete pipes 	After construction is completed, a levee owner/operator cannot prevent this issue from occurring. See Chapter 9 for correct placement of pipes within a levee embankment. Proper placement can reduce the risk of joint separation.	Depending on the degree of separation and accompanying erosion, the pipe may need to be completely replaced using conventional open cut methods. If the separation has not progressed to the point that external erosion of pipe backfill materials has occurred, the pipe can be slip-lined using trenchless technology or pressure grouted at the separated joints.
Deterioration/leakage of pipes 	Periodically inspect pipes that pass over, under or through the levee profile to identify and address early signs of deterioration or leakage.	Monitor seepage and soil migrating from the levee embankment.
Root intrusion through joints 	Remove all vegetation from the levee embankment to prevent the intrusion of roots through the joints.	Remove all roots that have penetrated the joints after vegetation from the levee embankment above these areas has been removed.
Corrosion holes in corrugated metal pipes 	Corrugated metal pipes are considered to have a 50-year design life, but it is good practice to inspect them often for corrosion holes (eg in the USA, the USACE requires inspections every five years). Some factors that may decrease the design life are: • deterioration of the pipe materials • construction defects • adverse environmental conditions.	• open cut and replace or slip-line the pipe using trenchless technology • weld new metal sections in place as patches • cover holes with a cement grout or concrete • replace lost bituminous coating inside pipe.

Maintenance: avoiding external erosion

Pipes that are not adequately maintained may not be able to convey their design capacity because of the build-up of sedimentation, the intrusion of roots, or the ovalling of the shape. A reduction in the pipe's capacity can:

- increase the velocity of the discharge, causing uncontrollable outflow onto the levee surface and external erosion
- cause ponding on the levee's landside toe that cannot be drained away. This ponding can saturate the toe, decrease the cohesion of the soil on the landside and lead to erosion of the levee slope
- cause fast-flowing water that should have been drained through to the waterside of the levee to discharge on the landside instead, resulting in external erosion to the landside slope.

Table 4.28 lists several situations that can lead to external erosion if not addressed. See Table 4.30 for descriptions and images of how the suggested repair techniques should be done.

Table 4.28 Issues that may result in external erosion

Potential failures by external erosion		
Observation	***Why it's a concern/suggested preventive measures***	***Suggested maintenance or repairs***
Erosion near pipe outlet on waterside	May be caused by inadequate channel or slope armouring near the pipe outlet Consider proper channel armouring/protection on the discharge/waterside of pipes.	Place properly moisture-conditioned, benched and compacted backfill materials in areas of erosion. Compacted backfill should be covered with filter stone and rip-rap designed to resist the exiting velocities from the discharge pipe.
Silt/sediment build-up in pipe 	Can reduce pipe's drainage capacity and cause saturation of the levee toe. The saturated toe may induce erosion or allow landside water velocities to be greater than what the pipe was designed for, causing external erosion. Keep the pipe's outlet channel graded to allow for drainage away from the pipe, preventing sediment build-up within the pipe. Consider installing a sediment trap to minimise future inflow of sediment. See Chapter 9 for proper placement and slope of pipes within a levee embankment to provide proper drainage.	Clean out silt/sediment that has built up inside the pipe after re-grading the outlet channel. This can be accomplished by jetting (desilting) the interior of the pipe.

Table 4.28 Issues that may result in external erosion (contd)

Ice formation within pipes  **Note** This figure shows an operative at the entrance to a confined space. If the operative were to enter the confined space, he would need to implement specific health and safety control measures, such as having gas monitoring equipment available and not working alone. See the safety precautions box at the start of Section 4.15.	Can reduce the pipe's drainage capacity, interfere with the operation of flap gates/ sluice gates/slide gates and cause saturation of the levee toe. The saturated toe may induce erosion or allow landside water velocities to be greater than what the pipe was designed for, causing external erosion. In cold climates, once a pipe is constructed within a levee embankment, a levee owner/operator can no longer prevent this issue from occurring. See Chapter 9 for proper placement of pipes within a levee embankment in cold climates. In moderate climates and levee projects along slow-rising rivers where outfall invert is clear of the water level, the flap gate can be chained up	• remove the ice by steaming it away from the gates • flush warm water though the pipe to melt the ice • schedule frequent pipe joint inspections to look for signs of pipe separation.
Damage caused by fires near or inside of pipe 	Can reduce pipe's drainage capacity and cause saturation of the levee toe. The saturated toe may induce erosion or allow landside water velocities to be greater than what the pipe was designed for, causing external erosion. To prevent damage to drainage pipes and the levee, crops should not be planted in the vicinity of pipes that may have flammable liners.	• open cut and replace pipe if damage collapses pipe or causes interior coating to be significantly damaged • slip-line pipe using trenchless technology • remove burned liner and re-slip-line • prohibit use of large-diameter high density polyethylene (HDPE) to minimise the risk of fire in pipe.
Root intrusion through joints: these can reduce pipe's drainage capacity and cause saturation of the levee toe. The saturated toe may induce erosion or allow landside water velocities to be greater than what the pipe was designed for, causing external erosion (see Table 4.27).		

Maintenance: avoiding levee instability

Improperly maintained pipes can collapse or become oval-shaped, which can lead to localised sediment or cracking on the levee surface. Any movement of a significant amount of soil can result in slope instability. If pipe deterioration causes significant internal erosion, a substantial loss of levee material can also lead to slope instability. Table 4.29 describes how changes to a pipe's shape may be prevented or repaired. See Box 4.49 for an example of how the collapse of a pipe can damage the crest of a levee.

See Table 4.30 for descriptions and images of how the suggested repair techniques should be done.

Table 4.29 ***Issues resulting in levee instability***

Potential failures by levee instability		
Observation	***Why it's a concern/suggested preventive measures***	***Suggested maintenance or repairs***
Pipe is losing its shape (ovalling or crushing) 	As this loss of shape occurs, it can cause significant displacement of the levee soil on top of the pipe, destabilising the levee above the pipe If soils are not properly compacted when a pipe is placed within a levee embankment, it could collapse due to lack of bridging of soils to handle the overburden load. See Chapter 9 for proper placement of pipes within a levee embankment to ensure proper bedding and backfilling is done. Also: • give extra attention to restoring protective coatings when corrosion is noted on the inside of a pipe • establish a pipe inspection program to regularly evaluate coatings.	In most cases, once a pipe begins to oval/flatten at the crown of the pipe or has lost more than five per cent of its original interior height, it should be replaced using conventional open cut methods
Erosion near pipe outlet on waterside: this can lead to instability if significant portions of the levee are destabilised by the erosion (see Table 4.28)		

Box 4.49 ***Pipe failure causing a sinkhole in the levee crest on the Jeffersonville/Clarksville levee system, Indiana, USA***

On the Jeffersonville/Clarksville levee located in Indiana, two 1.8 m diameter corrugated metal pipes collapsed in 1996 beneath the 10.7 m tall levee embankment. This section of levee is within the jurisdiction of the USACE Louisville District. The pipes were 50 years old. Had the pipes failed during a flood event, the resulting sinkhole (Figure 4.56) might have caused the levee to fail due to internal erosion or an overtopping breach. To repair the levee, the entire levee section was cut open and the pipes were replaced.

This experience inspired the Louisville District to create an inspection program to ensure that flood control system pipes passing through the levee are not threatening the levee's integrity. Inspections are done by physically entering the pipes or by remote video inspection.

Figure 4.56 ***Sinkhole created when two 1.8 m diameter corrugated metal pipes collapsed under a levee in Indiana, USA***

Box 4.50 describes the use of an anti-seep collar used by the US International Boundary and Water Commission (USIBWC) to prevent piping in culverts.

Box 4.50 A preventive measure that worked well for the USIBWC

An anti-seep collar should be included as part of the installation of a culvert to prevent piping along the length of culvert. On corrugated pipes, the anti-seep collar can be aluminium or plastic. One type of culvert that has performed well on USIBWC land is a rubber gasket pipe. The pipe is very sturdy and ridged for added strength, and a cradle can be installed in conjunction with the anti-seep collar. Aluminium and steel corrugated pipes do not perform well in coastal areas due to the salinity of the water. In these environments, the USIBWC has found that plastic corrugated pipes or concrete pipes are better suited to preventing rust.

Repairing culverts and discharge pipes

Table 4.30 describes several common ways to repair culvert and discharge pipe problems. Consult a professional designer if:

- an issue requires excavation and removal/replacement of the pipe
- the pipe inspection reveals a pipe separation or deterioration around the pipe joint
- the pipe repair also requires an extensive form of bank protection at the outlet end due to water exiting the liner pipe at increased velocities
- the solution requires the pipe to be sealed and abandoned (assuming the pipe is no longer required as part of the flood risk mitigation system).

Table 4.30 Types and descriptions of repair techniques

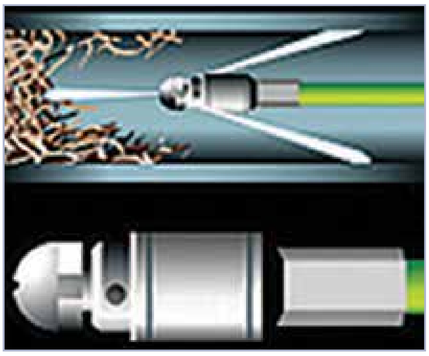
Type of repair	Description of repair
Slip-line trenchless technology 	Slip-lining is the installation of a liner pipe into a larger deteriorated host pipe
Pressure grouting at the joints 	Pressure grouting at separated joints (by isolation of the joint) may prevent internally eroded embankment soils from entering the pipe. Chemical grout is forced through joints and into the surrounding soil where it solidifies with the soil to form a waterproof mass that cannot be pushed back into the pipe (ICM, 2012)
Augering 	An auger can be used to cut and remove roots and other obstacles from the interior of the pipe. The drain snake auger is a corkscrew-shaped auger attached at the end of a motor-driven, bendable metal cable. It rotates in the clogged pipe until it hits the obstruction, at which point the corkscrew will scrape away the clog. It is especially effective when used in conjunction with video inspection (SewerTV, 2012)

Table 4.30 Types and descriptions of repair techniques (contd)

Jetting (desilting) 	Jetting is a common way to remove silt/sediment build-up from pipe interiors. A hose introduced into the pipe fires powerful water jets at up to 28 MN/m^2 (4000 psi). A nozzle at the hose end converts water into laser-like cutting jets that strip deposits from pipe walls, flush out waste and restore full flow (DrainPower, 2012)
Channel armouring 	Channels near an outlet on the waterside of a levee are armoured/protected with stone or rip-rap to prevent erosion from fast flowing water exiting the pipe

Other methods of pipe rehabilitation that may be acceptable for use in levee systems include cured-in-place pipe, fold-and-form pipe, spray-on lining and horizontal-auger boring. All methods of pipe rehabilitation should be reviewed and approved by the controlling authority for their suitability for levee embankments. There are limits to some of these methods that may be unacceptable to a levee embankment. For more detailed information about good practices for performing these remediation measures, see Chapter 9.

Maintenance of culverts and/discharge pipe system components

Regular maintenance of pipe system components (eg flap gates, sluice gates, slide gates, air relief valves, trash screens), in addition to the actual pipes, is a good practice for ensuring the daily operation and reliability of gate and valve closings, as well as the longevity of pipes. Table 4.31 lists common maintenance practices for these components.

Table 4.31 Culvert and discharge pipe components and their maintenance

Component	Common maintenance practices
Flap gates, manually operated gates and remotely operated gates (including slide gates, sluice gates and valves)	• examine, grease and trial-operate at least once every 90 days • wire brush and check the bearing surfaces of the flap gate and seat to ensure there is continuous contact and a proper seal • examine bushings or hinge pins for excess wear. Replace if flap gate and seat rings do not line up properly • sandblast and paint (with an appropriate protective coating system) steel parts associated with these components when they show signs of deterioration resulting from corrosion • inspect and trial-operate sluice gates. Because sluice gates are generally furnished with a visible position indicator, take care when opening and closing the gate so as not to over-run the travel of the gate. Over-running can damage the stem mechanism or the mounting anchor bolts for the gate operator. Carry out with either two people for a complete visual inspection or a remotely operated closed circuit television camera mounted on an extendable rod.

Table 4.31 Culvert and discharge pipe components and their maintenance (contd)

Flap gates at ends of pump discharge pipes	• see previous entry • consider installing energy-absorbing spring-bumpers to limit the gate's travel. When the pump begins discharging water, the pressure may cause the flap gate to violently swing open, slamming the gate body up against its discharge headwall. In this scenario the gate body may get damaged.
Air relief valves	• test-operate at least semi-annually • verify that required maintenance has been performed • ensure that valves are not painted shut • repair broken valves (valves may be subject to impact damage from mowing equipment and maintenance vehicles because they are generally installed on the levee crest). If valves are frequently broken, consider installing protective bollards around air relief valves.
Trash screens (at the inlet of gravity pipes)	Though not all trash screens are easily visible, they need to be inspected and maintained. Many pump stations are constructed with gravity flow bays passing through the structure, which may also have trash screens. • remove any accumulated debris to allow the water to freely pass into and through the pipe • repair any corrosion or damage (galvanised steel will corrode heavily if continuously submerged).

4.15.2 Utility pipe and line systems

This section covers:

- why utility pipes and line systems are a concern
- maintenance of utility pipes and lines
- repair of utility pipes
- determining when repairs are beyond O&M.

Why they are a concern

Utility pipes are used to convey water (eg for irrigation), natural gas, hazardous chemicals, petroleum products or sanitary sewage. Lines transmit electricity, cable television, the internet, or phone service (often via fibre optics). Since pipes and lines are not part of the flood risk mitigation system, they are considered encroachments and need to be managed as such (see Section 4.4). Pipes and lines often introduce additional risks to the levee system. Their deterioration, improper installation, and malfunction can negatively affect the levee system, whether they are permitted (ie approved by permit) or not. For these reasons, they should be inspected regularly to ensure that they are not threatening the levee's integrity. The following are examples of possible threats posed by utility pipes and lines:

- a stream of water released from a pressurised or non-pressurised water pipe inside the levee (see Figure 4.57) can cause internal erosion
- a leaky pressurised gas line inside the levee may create a gas pocket that either deforms the levee or causes an explosion
- hazardous chemicals or sanitary sewage pipes leaking at the waterside of the pipe or in the levee can create environmental issues
- if an electrical utility line is damaged, the supply of power to pumps or emergency equipment related to the flood risk mitigation system may be interrupted.

Box 4.51 illustrates how redevelopment in a leveed area can increase demand for utility access through the levee.

Figure 4.57 Evidence of water supply pipe leak – ponding of water at landside levee toe

Box 4.51 Permitting utilities under, over, or through levees in the USA

In the USA, many riverfront areas in communities with flood risk management systems were used as sites for heavy manufacturing and transportation industries until about the 1970s. As the USA economy shifted from manufacturing-based to service-based, redevelopment occurred in many of these riverfront areas, resulting in construction of recreation, dining and entertainment facilities. This development has driven the increase in requests for permits for utility pipes under, over, or through levees and flood walls.

Maintenance of utility pipes and lines

Some agencies have found that making the pipe owners responsible for pipe inspections and repairs instead of the levee maintainers has worked well for them. Whatever the preferred arrangement, frequent communication between the owner of the pipe and the levee maintainer is critical. So if either notices an issue with a pipe, the other should be notified, and the pipe should be closed until the issue is resolved. If a pipe is pressurised, the owner of the pipe should have a process in place to detect the loss of pressure and a way to quickly notify the maintainer of a possible leak or other issue.

Table 4.32 offers possible preventive methods and solutions to issues with pipes that are being newly installed and pipes that already exist in a levee. Note that a powerful leak can breach the embankment in several minutes. Preventive measures should be taken. Once a powerful leak has been detected it is too late for most of the solutions suggested in the table.

Table 4.32 Issues specific to utility pipes passing over, under and through levees

Issues	Preventive measures	Possible solutions
Difficulty inspecting pipes (usually due to property/access issues with landowners near the pipes)	• the permit for a pressurised pipe that crosses the levee should include real estate information about proper access for inspection (see Section 4.4, and Chapters 5 and 9 for more information) • work with the lawmakers to investigate creating a law that would allow levee inspectors access to all facilities that may impact the levee, such as pipes.	Investigate easements or securing easements/rights of entry where needed.

Table 4.32 Issues specific to utility pipes passing over, under and through levees (contd)

No shutoff valves	The levee manager/maintainer and pipe owner should co-operate to retrofit pipes with shutoff valves within 15 m of the landside and waterside levee toes to ensure that these lines can be isolated in the event the pressure line ruptures or maintenance is required. Shutoff valves should be able to be closed before or during a flood-fight. This placement ensures that there is a way to control the flow of water through the levee during a high water event. In the worst case, if the flow were stopped by using such a valve and there was a leak in the pipe on the waterside, it is possible that the waterside slope would be lost. Sacrificing the waterside slope is preferable to having water continue to leak through the entirety of the levee.	***Retrofit pipes with shutoff valves on either side of the levee embankment***  ***Positive closure on Butte Creek Levee, California, USA***
Difficulty acquiring inspection reports from the entity responsible for the pipe (usually the pipe owner)	• require the party responsible for the pipe to provide inspection results/reports meeting the specific national government regulation • consider relocating the pipe outside the levee embankment if responsible party does not wish to provide this information.	• acquire the pipe inspection reports from the responsible party (see Chapter 5 for pipe inspection techniques) • threaten to withdraw the owner's permit for the pipe (if applicable) • consider acquiring additional staff, equipment and funds for the maintainer to complete the inspection (ideally paid for by the owner of the pipe).
Difficulty locating existing pipes within the levee	Require that installers of new pipes follow the guidelines for installing new pressurised pipes.	Develop a plan for identifying all pipes passing over, under or through the levee using: • historical documents • field verification • ground-penetrating radar • sonar techniques • develop an effective system for tracking encroachments, including pipes (see Section 4.4).
Sanitary sewer lines (concrete pipes) are prone to joint separation	Avoid irregular settlement by not placing heavy loads on portions of the levee with pipes (eg fill material or buildings).	If the separation has not progressed to the point that external erosion of pipe backfill materials has occurred, the following may be options: • slip-line trenchless technology • pressure grouting at the joints • if the damage is extensive, use conventional open cut methods. Table 4.30 provides descriptions and images on repairs
Deterioration of pipes and leaky pressurised pipes, which can be detected by observing: • liquid coming out of levee embankment • ponding at levee toe in area of known pipeline crossing.	Consider relocating the pipe outside the levee embankment if responsible party does not wish to provide this information.	When leaky pipes are detected: • ask the responsible party for the pipe inspection reports • excavate, replace and encase the utility pipe • do not allow a pipe to be replaced within the levee embankment once a leak has been detected. Proper abandonment and rerouting of the pipe should be required. Or the pipe should be constructed in an overbuild section above the levee profile.

Utility pipe repair

Table 4.33 matches suggested repairs or recommended actions with a selection of utility pipe issues. As a guideline, the repair method selected should pose the least risk to the flood risk mitigation system. In general, open cut methods are the least desirable for repairs to utility pipes or lines that pass under or through a levee. See Box 4.52 for an example of the decision process used for a gas pipe repair.

Table 4.33 Utility pipe issues and possible repairs

Observation	Suggested repair or recommended action
Pipe is leaking	• repair typically consists of excavating, replacing and possibly encasing the newly replaced section of pipe • should be repaired immediately.
Joint separation	See Section 9.15.4 for possible remediation techniques
Pipe at end of service life	• responsible party may decide to request a permit to seal and abandon this line in-place and build a new line at a different location. Co-ordinate these decisions with the levee manager and maintainer • good practice is to require written permit approval by the levee manager/maintainer.
Pipe does not have a gate valve	Retrofit pipe with pinch or gate valves (valves help isolate the section of pipe that crosses the levee so that internal erosion can be minimised in the event of a leak).
Damaged utility pipes or lines	May be filled with a cementitious grout with a shrinkage-compensating admixture, or a dense expansive foam (eg in the USA, SEMCO PR-82 or equivalent).

Utility pipe repairs should be pressure-tested according to specific pipe manufacturer's criteria (eg in the USA, the American Water Works Association (AWWA) standard) before backfill and compaction around the repair area of the pipe to verify that the repair was successful. If a pipe has been abandoned in-place and filled with grout, the volume of the pipe to be grouted should be precisely computed to determine whether or not the pipe has been completely filled. If the pipe takes a different volume of grout than that computed, the pipe may be partially clogged or a void may exist. If this occurs, an open cut repair may be needed. See Figure 4.58 for an example of an open cut repair.

Determining when repairs are beyond O&M

If a repair requires excavating, removing, and replacing the utility pipe, a professional designer associated with the utility company should be involved. After a pipe or line is installed, it becomes an O&M concern, so it is important during the design stage of a pipe crossing to fully understand and anticipate all considerations for long-term service and maintenance. Also note the time of year the proposed repairs are to be made, as major repairs undertaken during the high water season increase the risk to the community behind the flood risk management system.

The utility company may have its own professional designers who use proprietary standards to assemble drawings and specifications for moving, replacing or abandoning a utility pipe. These standards should be reviewed and approved by the levee owner/maintainer to ensure that the changes to the levee would not adversely affect its performance should it become fully loaded.

Figure 4.58 *Open cut repair of leaking water supply pipe that penetrated a levee (least desirable repair technique due to cost and disturbance to the levee prism)*

Box 4.52 *Gas pipe repair and abandonment: the decision process, Cincinnati, USA*

The original levee included a 0.6 m steel casing directly embedded in the concrete sill of a roadway closure, through which passed a 0.45 m high pressure gas line. The gas company determined that the original gas carrier pipe had deteriorated and had to be replaced. The decision process for repairing this situation was as follows. If the proposed replacement pipe was equal to or smaller than the original pipe, the annular space between the embedded casing and the new carrier pipe would have to be filled with an annular seal material or product (see Figure 4.59). If the proposed replacement pipe was too large to fit within the original casing, then the repair options would be more complex. Partial demolition of the concrete closure sill might be required so a new, larger casing could be cast into the sill (Figure 4.60). This option would expose the levee owner to increased flood risk during flood season and greatly increase the duration of the roadway closure. The owner had the option to deny the gas company the opportunity to replace the pipe and could have demanded that once the original pipe had been removed, the original casing was to be filled with grout and properly abandoned *in situ*.

Figure 4.59

Polyethylene (PE) high pressure gas line within steel casing (encased within the concrete closure sill). Annular space filled with an expanding foam

Figure 4.60

Restored concrete sill and new asphalt over where gas line entered the sill

4.16 LEVEE TRANSITIONS

A flood risk reduction system may include levees and other structures such as walls and gates, in which the geometric configuration of the continuous line of defence differs. A levee transition is a location on a levee characterised by a (often visible or measureable) change in geometry of the levee cross-section, material type, loading or strength (see Chapter 3.4.3). Examples of transitions are locations at which:

- the geometry (levee slope angle) or alignment of the levee changes

- the type of levee changes, for example from an earthen levee to a concrete flood wall, rock revetment or the embankment meets natural (high) ground
- objects are situated on the (slopes of the) levee, eg a bridge pier, buildings and stairways
- the earthen levee meets other structures, eg closure structure
- the outside edges of rock revetments transition to levee slope protection.

The interface between the earth of a levee and the surface of (partially) embedded structures (like pipes, culverts etc) can also be considered as a transition. Issues related to these transitions are discussed within the sections on these structures (see Sections 4.15 and 4.17).

Figures 4.61 and 4.62 illustrate two types of transitions.

Figure 4.61

Transition from flood wall to levee (Rushford, Minnesota, USA)

Figure 4.62

Levee tying into high round (Halstad, Minnesota, USA)

Transitions are a well-known weak point in the levee. Failures have been known to occur for two main reasons:

- **external erosion:** water flowing against a levee (a riverine or stream levee or rock revetment protecting a shoreline from wave action or tidal movement) may develop a turbulent flow transition zone when it comes in contact with a change in material. The turbulence may displace rock, pull bedding material out from under a rock revetment, or cause erosion of the earthen embankment to the point that a failure could occur if the levee is not monitored and repaired. Figures 4.63 and 4.64 show examples of transition damage following a flood event in Rushford, Minnesota, USA
- **internal erosion:** the flow of groundwater through the levee is impacted by the presence of structures/objects with a different (often lower) permeability. Concentration of groundwater flow may locally increase hydraulic gradients, increasing the risk of internal erosion.

If structures in or on the levee cause a rise of the groundwater table inside the levee, their presence may even affect slope stability.

Deciding whether a transition zone is problematic

Methods of evaluating a levee system's transition zones are unique to each country. Box 4.53 lists three examples.

Figure 4.63

Transition between levee and rip-rap. Erosion occurred upstream and downstream of rip-rap area, and bedding was pulled out from under rip-rap

Figure 4.64

Scour on the landside of a transition between the levee and the flood wall as a result of overtopping

Box 4.53 *Country-specific approaches to determine if a transition zone is problematic*

USA: the US Army Corps of Engineers (USACE) uses criteria outlined in its inspection guidelines or levee checklists to evaluate transition zones. The USACE also closely monitors all erosion that has progressed into the levee section or into the extended levee footprint that may compromise the stability of the levee's foundation.

England and Wales: though transitions are not specifically addressed in the condition assessment manual (Environment Agency, 2006), any transition zone-related defects would be captured during routine visual inspections done to identify weaknesses in each levee section.

The Netherlands: some transition features are specifically included in the mandatory safety assessments. Guidance is provided for a range of transition types including levees to dunes, levees to high ground, and the transitions between different revetment types.

Preventing internal and external erosion at transitions by maintenance

If at all possible transitions should be avoided to limit problems of internal and external erosion. So an effective encroachment control and permit system is important.

Once the levee transition is there, the maintenance may not be able to prevent erosion. However, routine maintenance of the levee near transitions should take care not to increase discontinuities across the transition, or weaken the levee near the transition. For example:

- embedded structures typically require sound foundations and may require use of non-cohesive materials. However, if such materials are also used for any backfilling close to the levee surface, they may increase the vulnerability to erosion
- the use of herbicides to keep stairways over a levee free of weeds weakens the sod/turf immediately adjacent to the stairs, increasing the vulnerability of the levees to erosion there.

Repairing erosion at transition zones

Table 4.34 recommends repairs for damage that has already occurred.

Table 4.34 Preventing further/future erosion at transitions

Observations	Cause of observed phenomena	Possible repair solutions
Surface erosion at transition locations caused by water movement (wave/tidal action, river flows)	Gradual water movement (eg tidal areas, areas with minimal wave action, or slow flow velocities)	Repairing erosion with earthen material: 1 Remove and scarify the eroded surface and adjacent area to allow for proper bonding of the new fill material with in-place material. Avoid doing repairs when the ground is frozen, otherwise all frozen ground may need to be removed as a separate step. 2 Add suitable fill material, preferably material consisting of highly plastic clays (CH) or lean clay (CL), in loose layers no more than 0.15 m thick. Moisture content of the fill materials should be within acceptable ranges before placement. Judging the proper moisture content by the outward appearance of the material is best done by those trained in soils or earthwork construction. 3 Compact the fill layers either by hand or with small mechanical equipment. Layers should be added to the low areas until the fill forms a slight mound over the eroded area. 4 Replace top soil and existing turf. 5 If turf is subjected to repeated erosion, soil reinforcement could be used to assist in stabilising the transition area. Could use soil reinforcing poly mats with anchors that extend into the earth embankment. Grass and vegetation grow within the mat openings to help stabilise the earth and provide minor erosion control. Poly mats work well at the base of flood walls, transitions between flood walls and levees, and the outer edge of rock revetments, or any location that is not subject to higher flow velocities.
	See Figure 4.64	Possible options include using sand/gravel bedding material, rock revetments, and articulated concrete block (interlocking or concrete mattresses). Rock protection and bedding material selection begins with the levee designer selecting the size of sand/gravel bedding material and rock based on the flow velocity or wave action. • based on the designed bedding and rock thickness, remove and scarify the eroded surface and adjacent area to allow for proper bonding of the bedding material with in-place material • the sand/gravel bedding material is placed and compacted by hand or small mechanical equipment before placing the surface rock. For high velocity flow areas or area where the bedding material has been known to be pulled from under the surface rock protection, a layer of geotextile fabric could be placed between the sand/gravel bedding material to provide additional protection. If the rock protection is higher than the surrounding earthen embankment, additional soil stabilisation such as a poly mat, should be considered at each end of the rock revetment to help stabilise the soil that is subject to additional erosion and turbulence as the water flows over the rock.

Table 4.34 Preventing further/future erosion at transitions (contd)

Surface erosion at transition locations caused by water movement (wave/tidal action, river flows) See Figure 4.64	High flow velocities or wave action that would subject an earthen embankment to continual erosion	Possible solutions include large rock revetments, articulated concrete block, or other method to divert the flow away from the leveed areas such as wing dams in river systems or offshore barriers. When these conditions exist, careful engineering evaluation is recommended: 1 Remove and scarify the eroded surface and adjacent area to allow for proper bonding of the new bedding sand with in-place material. 2 Compact the fill layers either by hand or with small mechanical equipment. Place poured concrete or articulated (interlocking) block. When placing a concrete pad or articulated block, care should be taken to ensure the free surface is level with the surrounding earth/turf. If the finished surface is higher or lower than the surrounding area, a new transition zone could be formed, resulting in erosion along the transition line.
Internal levee erosion at material transition zones due to uplift pressures.	Excessive seepage, fracturing of the levee clay blanket, or blowout of a rock revetment observed	Should request an engineering evaluation be conducted immediately to determine the proper method of repair. An interior/uplift pressure evaluation and repairs are beyond normal levee O&M. When excessive seepage, soft soils, or piping occur on the landward side of the levee, an on-site engineering evaluation should be conducted to determine the best course of action to reduce the water movement. For sand levees with a clay blanket, the on-site investigation should focus on holes that have penetrated the clay layer. These holes may be caused by the roots of trees or unwanted vegetation, rodent activity, or unauthorised encroachments. These holes should be filled with compacted clay. Internal erosion within the levee foundation should only be evaluated by an engineer to determine the best method to slow or eliminate it. For more information on internal erosion, see Chapter 3 of this handbook and/or Section 3.1.1 of Morris *et al* (2012).

Determining when repairs are beyond maintenance

A levee designer and appropriate design codes (see Chapter 9) should be engaged for any of the following conditions:

- **internal erosion, piping and/or sand boils:** during a flood event, excessive seepage, soft/saturated material, and soil transportation along any of the previously described transition zones are causes for concern and a design review. Seepage in conjunction with material transportation could rapidly deteriorate the interior of a levee or levee foundation, resulting in excessive levee settlement and potential failure
- **rock and bedding material displacement:** reoccurring rock or bedding material displacement at transition zones following a flood event may mean the rock protection and bedding material is undersized for the flow velocity or wave attack. An engineering review should be conducted to verify the rock and bedding material is properly sized
- **erosion along flood walls and rock revetments:** reoccurring erosion/scour along a flood wall or along the edges of the rock revetments is the result of flow turbulence. If erosion/scour becomes an issue, buffer zones such as poly mats, concrete pads, or articulated/interlocking block may need to be installed to prevent future erosion. These should be properly designed to ensure that there is a good filter between the mat and the underlying soil and that they transition into the levee without creating additional problems.

More information on levee condition assessments can be found in Chapter 5.

4.17 FLOOD WALLS

Flood walls (see Section 3.4.1.2) are flood risk mitigation structures of either gravity or cantilever design. In the context of levees they are used when space does not allow increasing the levee cross-section, the right of way is not available, or the levee foundation cannot support the weight of the additional earthfill. A well-designed and constructed flood wall can provide long-term service while requiring minimal maintenance and fiscal resources.

Clear zones should be established and maintained along both sides of a flood wall for inspection, maintenance, and flood-fighting purposes. A minimum zone of 4.6 m measured from the face of both sides of the wall provides adequate width. The zone should be kept clear of all vegetation other than maintained turf. Planting in this zone should be prohibited. Root systems can induce external forces that may lead to wall instability or localised deterioration. Also, this clear zone should not be excavated or used for storage or for structures. Walls can be particularly susceptible to severe fire events, so flammable materials should never be stored adjacent to flood walls.

Preventing flood wall deterioration

Table 4.35 lists common flood wall problems, their potential impact on levees, and suggested preventive measures. An early response to the listed observations can prevent further deterioration and high maintenance costs.

Table 4.35 Common observations and preventive measures for floodwalls

Observations	Potential impact on levee	Preventive measures
Concrete		
Erosion of cement paste on floodwall surfaces (exposes more durable aggregates beneath, resulting in a roughened or textured surface)	No immediate impact, but can induce scaling, spalling and micro-cracking (spalling is the deterioration of concrete by flaking and crumbling)	Consider applying protective coatings, by spraying or painting, to retard this process (horizontal surfaces may otherwise retain moisture, accelerate the freeze/thaw processes in colder climates and affect vertical expansion joints)
Efflorescence along small cracks (efflorescence is the white calcium precipitate left by mineral-rich water seeping through the concrete and evaporating on the floodwall surface)	No immediate impact. Indicates moisture intrusion. Long term impacts could affect the levee if the floodwall continues to deteriorate	• regularly inspect the floodwall to detect efflorescence • determine source of moisture intrusion and coat or patch to prevent reduction in strength or an increase in porosity of concrete
Exposed steel reinforcement	Exposed reinforcing steel has reduced tensile capacity and can corrode (when the steel corrodes, it creates internal expansive pressures leading to increased concrete delamination and loss of steel cross-section)	• regularly inspect the floodwall to detect exposed steel • identify and treat the cause of problem • patch or repair exposed steel, as concrete deterioration is likely to continue while steel is exposed.
Delaminating and spalling concrete	May lead to more serious and expensive maintenance and repairs, including corrosion of reinforcing steel. Not necessarily a floodwall safety issue	• during inspection, identify delaminating by noting a hollow sound when suspect areas are lightly hammered • identify spalled areas • determine and treat the source of problem; patch the defect to prevent further deterioration.
Cracking from shrinkage, deterioration, settlement, overstress, impact, or other cause	May lead to more serious and expensive maintenance and repairs, including corrosion of reinforcing steel	Generally, routine maintenance cannot prevent this type of cracking, but routine inspections can reveal the problem.

Table 4.35 Common observations and preventive measures for floodwalls (contd)

Cracking/spalling of concrete related to ice formation within cracks and monolith joints	No immediate impact. Long term impacts could affect the levee if the floodwall continues to deteriorate	• inspect floodwalls and monolith joints prior to onset of cold weather • seal all cracks larger than 0.3 mm and seal the joints.
Joints, wall movement, and settlement		
Deterioration of joint material and waterstop	May result in spalling, cracking or excessive leakage at monolith joints	• establish a regular inspection program for joints • replace deteriorated joint material • consider that differential settlement (horizontal or vertical) may have damaged waterstops.
Joint separation (generally results from settlement)	May result in excessive leakage at monolith joints and increased potential for scour of adjacent ground	• (see differential settlement, next)
Differential settlement between floodwall monoliths (ie tilting) where one monolith leans landward and the other leans to the waterside (generally results from poor subgrade preparation)	Compromises the waterstop between the monoliths, resulting in excessive leakage at monolith joints and increased potential for scour of adjacent ground	Routine maintenance is not effective in preventing differential settlement. If differential settlement is noted, it should be brought to the attention of the levee designer immediately
Gap on riverside of I-wall at ground interface	A heavily loaded I-wall can rotate landward. In fine-grained soils, a gap will form between the soil and the foundation, potentially leading to instability of the wall	Routine maintenance is not effective in preventing gap formation. If gaps are noted, they should be brought to the immediate attention of a levee designer.
Scour and erosion		
Inadequate scour protection to resist wave action at the base of a floodwall	May cause instability	Scour protection (eg rip-rap or concrete slab) should be adequate to resist wave action on the seaward side and wave overtopping on the landside. The protections should also be resilient
Foundation erosion, scour, or bank caving (scour often occurs at wall/levee transitions); see Figure 4.51 (plan view of transition)	May cause instability, including instability on the landside from overtopping. Erosion and scour of a levee may result in instability of a floodwall integrated into the levee or even the breach of a levee system	• repair or retard erosion and scour damage when it occurs • prevent foundation erosion by plugging small joint leaks with coal dust, sawdust, or other expansive material • use layered pervious fill (at least 0.33 m thick) confined by sandbags to restrict flow through foundation and to add stability • use sacked-earth rings for sand boils.
Scour holes undermining foundation where stream flows near the floodwall	May cause instability or a breach	Inspect stream bank for scour holes that threaten to undermine the floodwall. Repair as foundation erosion
Encroachment		
Encroachment of debris, unauthorised equipment and structures, and excavations	May inhibit operations/maintenance and impact integrity of levee. Construction of an encroaching building with a basement is of particular concern, since the basement may shorten the seepage path or decrease the sliding resistance along a foundation failure plane	• regularly inspect for encroachments (See Section 4.4) • maintain an encroachment review/removal program (See Section 4.4) • maintain a clear zone that is free of unauthorised excavations, structures and debris.

Table 4.35 Common observations and preventive measures for floodwalls (contd)

Vegetation encroachment and animal burrows	May cause seepage or settlement; roots may intrude into toe under-drains	• regularly inspect for these conditions (also see Sections 4.6 and 4.5) • maintain a grass-only or paved clear zone of predetermined width on both sides of floodwall.

Repairing common flood wall problems

Table 4.36 identifies common flood wall problems and suggests possible repair or maintenance methods.

Table 4.36 Common floodwall repair or maintenance methods

Observations	Maintenance methods
Concrete	
Delamination	• mechanically remove small areas of delamination and patch with a cement-based repair mortar • refer more extensive and advanced repairs to a professional levee designer who specialises in concrete repairs.
Spalling	• determine the cause of spalling and design a repair system to treat the cause • repair spalling using routine concrete repair techniques (see EN 1504-1:2005) • coat surfaces to prevent water intrusion.
Cracking (from shrinkage, deterioration, settlement, overstress, impact, or other cause)	• determine the cause of the cracking and design a repair system to treat the cause • seal cracks to prevent further deterioration from water intrusion • coat the horizontal surfaces of flood walls with a soluble reactive silicate concrete treatment (eg EN 1504-1:2005).
*Joints, wall movement and settlement**	
Deterioration of joint material and waterstop	• refill and reseal joint • if a flood wall monolith moves laterally or vertically more than approximately 25 mm relative to the adjacent monolith, the waterstop has probably failed and an external waterstop should be designed for its replacement • use a router to remove the old joint sealant, install a backer rod and apply new sealant (the purpose of the backer rod is to accommodate a thin joint sealant application that is more pliable with a thicker section adhering to both concrete panels) • a contraction joint with a visible crack should also be sealed with a joint sealant to restrict potential water migration into the joint.
Joint separation	• replace the sealant if it has cracked, exhibits holes, or has pulled away from the surrounding concrete • use filler compounds to temporarily repair the separation.
Differential settlement	• a designer is always recommended for settlement problems • also see *Deterioration of joint material and waterstop*.
Scour and erosion	
Inadequate scour protection	Depending on the severity, routine maintenance techniques are probably not adequate to repair damage. Contact a professional levee designer.
Foundation erosion, scour (including scour holes in stream bank) or bank caving	• consult a designer • consider performing soundings or a bathymetric survey (for scour holes) if scour appears to be affecting a large area • repair with compacted soil and/or rip-rap • reseed earthfill.

Table 4.35 Common observations and preventive measures for floodwalls (contd)

Encroachment	
Encroachment of debris, structures, excavations, vegetation, and animal burrows	Remove unauthorised encroachments and vegetation (see Sections 4.4 and 4.5); use animal control program (see Section 4.6).

Note

* **Movement of floodwalls:** floodwall panels may settle, rotate, or slide in relationship to an adjacent panel. Notably, this movement may result in discontinuity or misalignment at a vertical joint or tilting of wall panels. Other gross movements may alter the original width of an expansion joint. Movements may have occurred years ago and become static or they may be dynamic. Discontinuities should be monitored by regular measurements taken in different seasons but at the same location.

Floodwalls are not static by nature. Differential solar heating and cooling causes minor fluctuations. An active movement trend should be investigated immediately by a professional levee designer.

Determining when repairs are beyond maintenance

In general, a professional designer should be consulted when a flood wall is subject to impact damage, overstress, or movement – depending on the degree of severity. Cracks, spalling, or other concrete deterioration may provide evidence of this type of damage, if it is not otherwise immediately apparent. This is of particular importance for I-walls, where even a small amount of lateral movement may be an indicator of dangerous large-scale failure. Differential settlement, lateral I-wall movement and repairs of high-degree erosion, scour, or undermining should be reviewed by a designer after performing any necessary repairs mentioned here to address the situation.

Some types of concrete deterioration, such as spalling due to aggregate reaction or premature or deep cracking, may require the attention of a designer with concrete expertise. Ties or props tend to fail first, so any issues with them should be taken seriously – a designer should be consulted.

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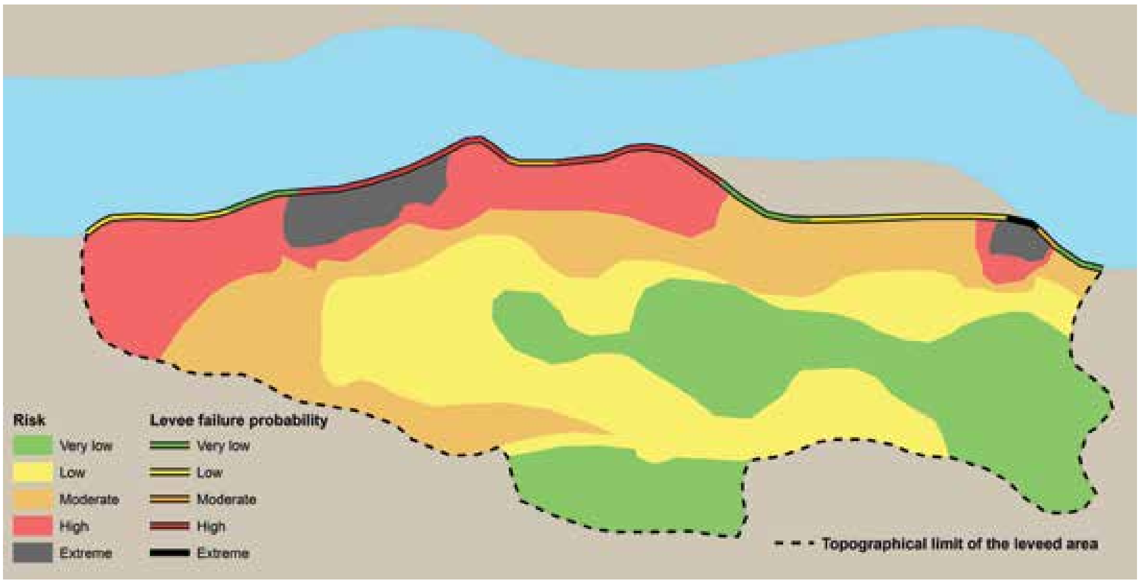
5 Levee inspection, assessment and risk attribution

Courtesy Rémy Tourment, Irstea



Courtesy Bruno Beullac, Irstea

CHAPTER 5 CONTENTS

5 LEVEE INSPECTION, ASSESSMENT AND RISK ATTRIBUTION

Chapter 5 introduces levee performance assessment and flood risk analysis. These support all decisions about levee management.

Key inputs from other chapters

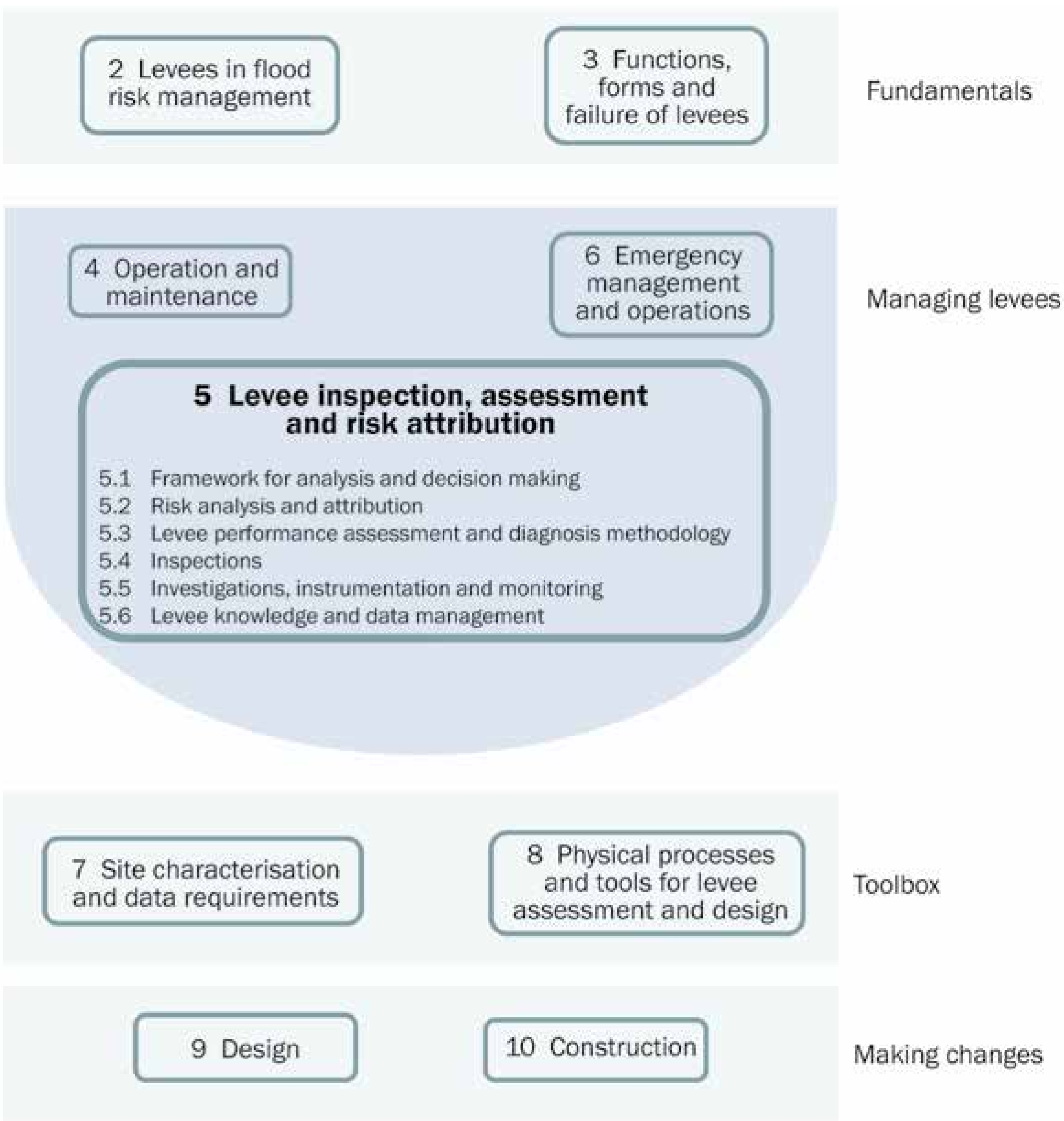
- **Chapter 2** ⇨ basic concepts
- **Chapter 3** ⇨ forms, functions and failure mechanisms
- **Chapter 4** ⇨ operations and maintenance
- **Chapters 7 and 8** ⇨ toolbox (data and models)

Key outputs to other chapters

- inspections ⇨ **Chapters 4 and 6**
- decision making ⇨ **Chapters 4, 6 and 9**

Note: The reader should revisit **Chapters 2 and 3** throughout the levee life cycle for a reminder of important issues.

This flow chart shows where to find information in the chapter and how it relates to other chapters. Use it in combination with the contents page to navigate the handbook.



CHAPTER CONTENTS AND TARGET USERS

This chapter is divided into six sections, providing an overview of levee performance assessment and flood risk analysis, along with a discussion of related data gathering and management techniques.

Framework for analysis and decision making

Section 5.1 introduces the general concepts for the whole chapter and the relationship between the different activities detailed within it.

Risk analysis and attribution

Section 5.2 provides details on how risk analysis can be used to evaluate the flood risk in a leveed area, and how this risk can be attributed to various segments of the levee system. The elementary components of a risk analysis are detailed, as well as subsequent tasks such as risk attribution and evaluation.

Levee performance assessment and diagnosis methodology

Section 5.3 explains the framework and the principles of the different possible methods for conducting levee performance assessments, including the activity of diagnosing the main failure mechanisms. The data required for these activities, and the way the data are used, is presented, linking them to failure mechanisms. The important concepts to be considered in conducting these tasks are synthesised here integrating information from different national approaches.

Inspections

Section 5.4 presents the different types and frequencies of inspections, and details the underlying principles for managing, conducting and reporting inspections. The different types of features to be observed are presented and linked to the possible deterioration and damage mechanisms.

Investigations, instrumentation and monitoring

Section 5.5 explains the importance of conducting investigations and monitoring (including use of instrumentation) in order to gather data for failure mechanism diagnosis and levee performance assessment, linking to the subject matter in Chapter 7 (site characterisation and data requirements).

Levee knowledge and data management

Levee performance assessments and risk analysis rely on analysing of data, which come from many different sources and is useful throughout the life of the levee. Section 5.6 details the principles of levee related data management, including use of information systems, such as GIS-based systems. Levee related data are essential for assessments, but also to other levee or flood risk management tasks.

5.1 FRAMEWORK FOR ANALYSIS AND DECISION MAKING

Levees are intended to protect an area against natural flooding events or sea storm surges. This protection is naturally limited to a certain flood or stormwater level, according to the height of the levee. Even this level of protection is limited, as there is the potential for failure of the levee before water reaches the intended height of the levee system. In fact, the presence of a flood protection system, including levees (see Chapters 2 and 3) transforms a natural hazard into a combination of natural (flood/storm) and technological (levee breach) hazards. In some cases, due to the increased water velocity through a potential levee breach, the actual level of risk in the 'leveed area' is higher because of the levee than it would be without it.

5.1.1 Levee performance assessment tools

Levee system managers seek to make good investment decisions that minimise whole life costs and maximise environmental gain, while ensuring communities are appropriately protected from flooding now and in the future. To achieve this it is important for them to ascertain:

- the actual performance (or reliability, or safety) of a levee or levee system
- the (remaining) risk associated with different events in the 'leveed' area.

Chapter 2 introduces the basic concepts of flood risk identification, analysis and evaluation (see Section 2.1.2 and Figure 2.5) and should be read as a general background to this chapter. Within this general context, three closely-related tools have a role in assessing the levees themselves:

1 **Levee (or levee system) performance assessment:** this is the process of understanding the anticipated structural performance or integrity of an existing levee or levee system given its current state. The most comprehensive levee performance assessments should be based on a diagnosis of the actual or possible initiating causes of failure in order to identify means to remediate or prevent these causes. Levee performance assessments are also inputs into flood risk analyses
2 **Flood risk analysis of a levee system:** this is a process that determines the overall level of residual risk in the leveed area associated with the levee system, given the inputs of the levee performance assessment and the potential impacts in the leveed area
3 **Risk attribution to levee segments:** this is an output of the flood risk analysis and identifies the contribution of each levee segment within the levee system to the residual flood risk in the leveed area (for example, those levee segments contributing most to flood risk).

In order to guarantee the long-term safety of a levee or of a whole flood defence system, these tools should be used on a regular, periodic basis, as well as on special occasions, such as during or immediately after loading events (floods, storms, earthquakes etc). Use of consistent analysis tools and techniques help to support decision making at all levels providing:

- an improved understanding of the role that an individual levee plays within a larger levee system
- a better understanding of the impact of uncertainty within the estimated risk
- the ability to progressively refine the analysis.

A risk analysis of the levee system, taking into account the levee performance assessment and the people and physical assets in the leveed area, helps levee system managers prioritise the actions that need to be taken after the assessment process (and hence optimise their investment strategy). These actions can include, for example:

- carrying out an emergency response or procedure
- conducting a complete diagnosis of some part of the system (most likely based on differentiation of levee segments according to their performance) in order to design and implement remediation of structural problems

- undertaking some 'routine' maintenance repairs
- doing nothing special but keep on inspecting and assessing the levee system.

Figure 5.1 illustrates a decision making process from system analysis to action (note that the part of the figure enclosed in the dotted box represents the initial system assessment/analysis.)

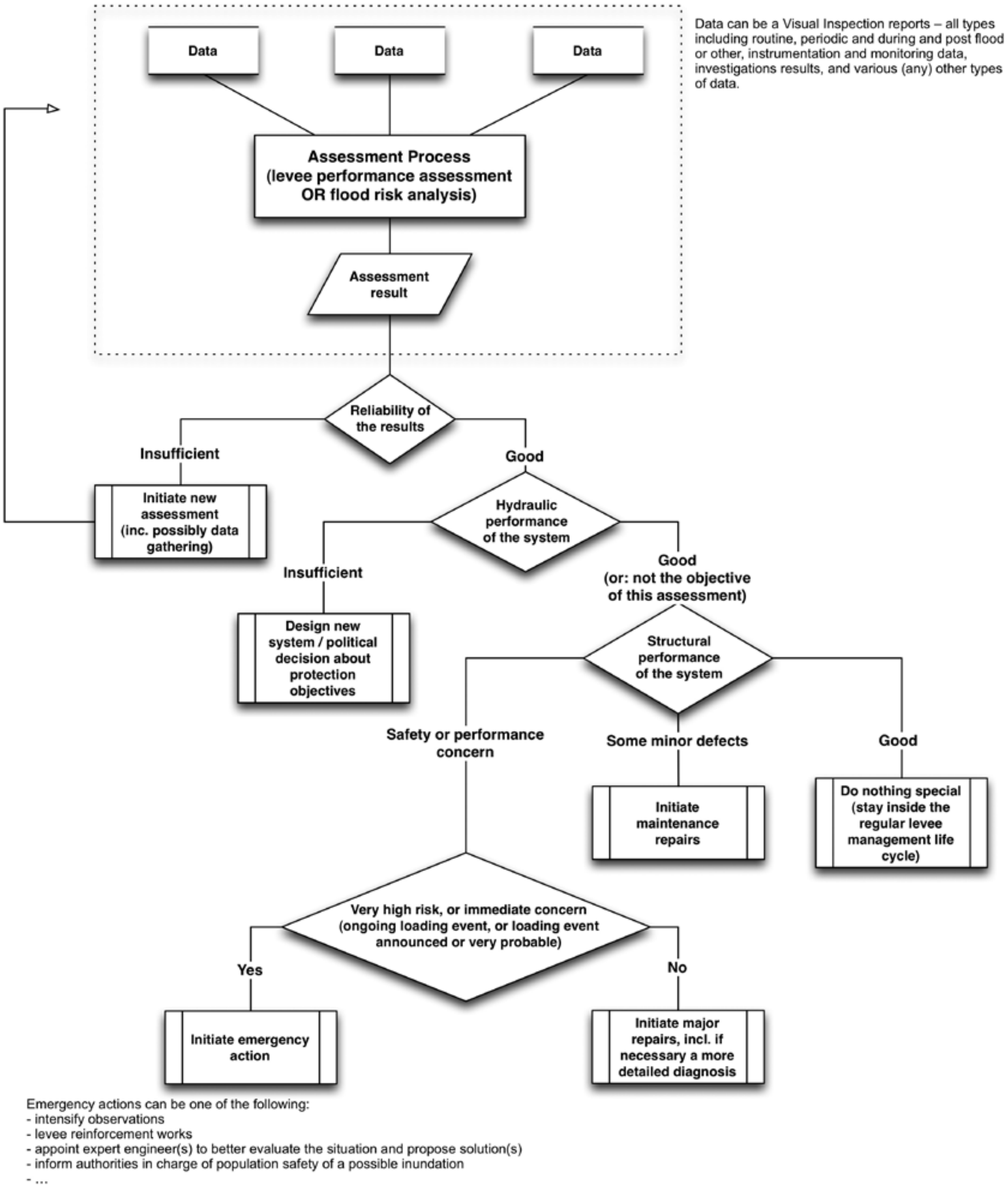


Figure 5.1 Assessments (levee performance assessments and flood risk analyses) and decision making for levee managers (courtesy R Tourment)

The reliability of the levee performance assessment should be discussed in any reporting and will depend on the:

- stage within the levee life cycle at which the assessment has been performed
- available data used, including its relevance and age
- treatment/combination method(s).

The reliability of the performance assessment directly affects the possible measures adopted following the assessment. For example, less reliable results are more likely to suggest further data gathering and assessment rather than remedial works.

5.1.2 Role of data in levee performance assessments

All three types of assessments discussed in Section 5.1.1 rely on the processing of data to reach a conclusion. Figure 5.2 shows how the different sections of this chapter are integrated in terms of the data they use/produce/manage. Some data may already be available at the start of an assessment process, for example from **monitoring** using installed **instrumentation** (see Section 5.5 and Chapter 7). Missing data can be gathered during the levee performance assessment process either during a specific **inspection** (see Section 5.4) or during a more detailed **investigation** (see Section 5.5 and Chapter 7). All data has its place in the **information system** (see Section 5.6).

To clarify the terminology here:

- **inspections** are visually manmade observations (during a field visit), including 'aided' methods like video cameras, registration on laptop/smart phone
- **investigations** are technical measurements (or sets of measurements) gathered during or for an assessment process
- **monitoring** is regular technical measurements (or sets of measurements) or observations carried out frequently
- **instrumentation** are the measuring devices and equipment used to collect data. These devices may be installed permanently, temporarily or intermittently and may be operated manually or automatically.

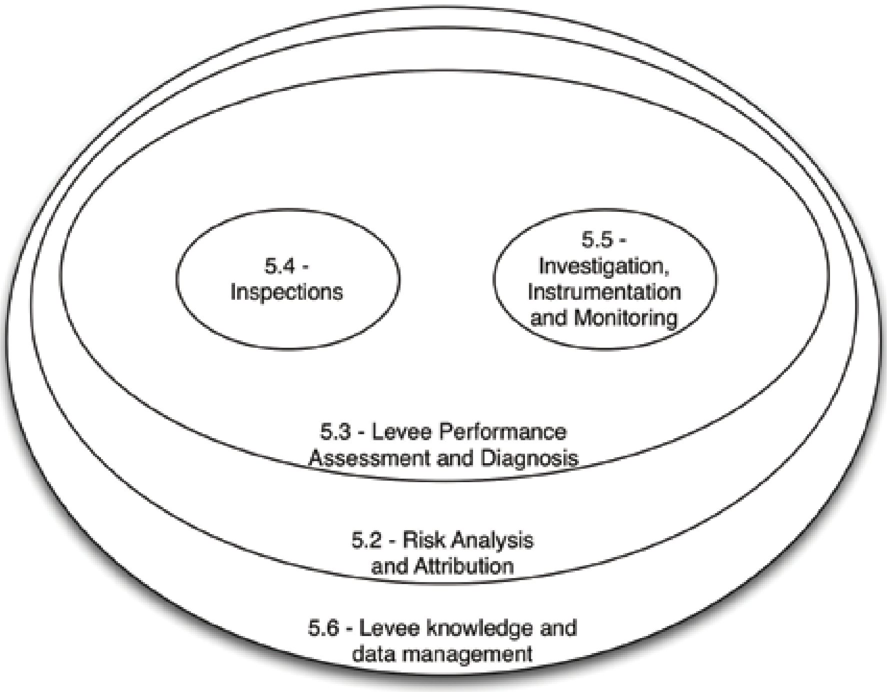


Figure 5.2 Integration of the data handled by each of the activities described in one of the sections of Chapter 5

5.1.3 Links to other parts of the handbook

In order to help levee managers to make good assessments and decisions, this chapter is linked to the remainder of the handbook. The role of Chapter 2 in introducing the basic concepts has already been discussed in Section 5.1.1. Figure 5.3 gives the details of how the remaining chapters of the handbook are linked to the various interrelated sections of this chapter (note that Chapter 10 is not mentioned in this diagram as it only indirectly relates to the assessment process).

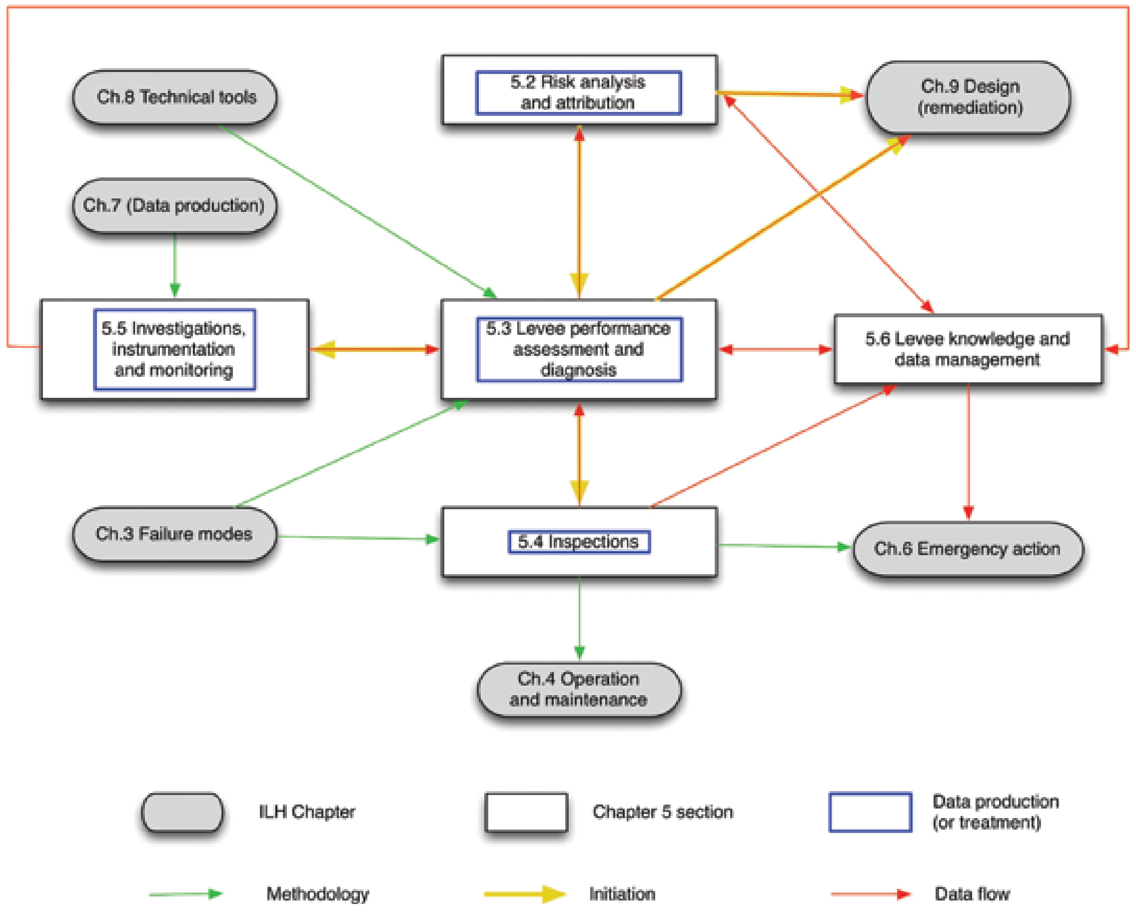


Figure 5.3 Links between sections in Chapter 5 and other chapters within the handbook

5.2 RISK ANALYSIS AND ATTRIBUTION

5.2.1 Overview

Flood risk analysis identifies the combination of the probabilities and consequences of flooding (see Chapter 2) and is conducted in order to:

- explicitly evaluate the level of risk if no action is taken to mitigate those risks
- identify the monetary and non-monetary costs and benefits of mitigation options for reducing flood risks
- account for uncertainty and variability in possible outcomes to better inform decision making.

This section explains how the flood risk analysis may be carried out and the amount of risk associated with an individual levee system or levee segment identified.

The analysis of flood risk should take into account the probability distribution of all flood levels and the likely **consequences** of all possible floods. However, this is not always practical and an appropriate framework should be selected to undertake an adequate and reasonable analysis of flood risk or its elements. For example:

- a **tiered approach** can be adopted to flood risk management, as discussed in Section 2.1.3, in which the outcomes of an initial risk analysis is used to justify further action or inaction. Tiered risk analysis of levee systems is not necessarily a set of distinct levels – rather it is a progression from the simpler approach to the more complex – depending on the requirements and level of risk (see Figure 2.7)

- a **phased approach** can be used for reduction of uncertainty in the data to be used in the risk analysis. In essence, where appropriately detailed or accurate data is not available then surveys, studies or measurements may be undertaken to obtain additional data and information of the required quality and coverage. For example:
  - where information about levee materials is required, desk study and simple sampling and testing may be used initially, but may be followed by subsequent use of more detailed investigation methods
  - printed maps may provide initial data on the receptors, but accuracy of this data could be improved by use of a digital property dataset to improve information on the number of properties at risk.

Further information and examples of phased data acquisition methods are given in Section 5.5 and Chapter 7.

Depending on the tier or phase concerned, various approaches to risk analysis can be applied including:

- **qualitative risk analysis (or semi-quantitative analysis)** may be used as a first step for flood risk analysis of all levee systems and might use word form, descriptive or numeric rating scales to describe the magnitude of potential consequences and the likelihood that those consequences will occur. Its result can typically be represented in a risk evaluation matrix as shown in Figure 5.4. The descriptions of the chance of failure (rare, unlikely, possible etc) can be matched with numeric probability bands (0.01–0.1, 0.1–1 etc) if required. The advantage of the qualitative approach is the short period of time needed. A disadvantage is the subjectivity of the assessor, so that the outcome might not be stable and less satisfactory than that from a more rigorous analysis
- **quantitative risk analysis:** while qualitative analysis methods may be sufficient where consequences and/or the probability of system failure are considered to be low, it may be more appropriate to apply a more time-intensive, complex quantitative method where the qualitative risk analysis indicates that the risk is high. This approach is based on numerical values of the potential consequences and likelihood, the intention being that such values are a valid representation of the actual magnitude of the consequences and the probability of the various scenarios that are being examined. The advantage of the quantitative approach is its objective nature, while the disadvantages might be the time needed and availability of data.

In a quantitative risk analysis, the numeric value of risk is intrinsic to a given flood risk system, including all potential hazards, any levee system and the people and assets in the floodplain. Risk is a socially constructed concept and the way that the various aspects of risk are quantified and evaluated depends on the stakeholders' perception of the defended assets and their own vulnerabilities to flood. Damage to economic, environmental, social and architectural/heritage aspects can be measured and evaluated using different methods/indicators/measurements such as numbers of lives lost, numbers of jobs lost, monetary damage, and economic consequences in the long-term. Evaluation of the significance of such quantified risks is sometimes carried out using a graph such as that in Figure 5.5.

1 2 3 4 5 6 7 8 9 10

Likelihood of failure/ inundation	Consequence of failure/inundation				
	Insignificant	*Minor*	*Moderate*	*Major*	*Severe*
Almost certain	M	H	H	E	E
Likely	M	M	H	H	E
Possible	L	M	M	H	E
Unlikely	L	M	M	M	H
Rare	L	L	M	M	H

Rating risk level	E	Extreme risk
	H	High risk
	M	Moderate risk
	L	Low risk

***Figure 5.4** Example of a risk evaluation matrix that can be used in a qualitative risk analysis*

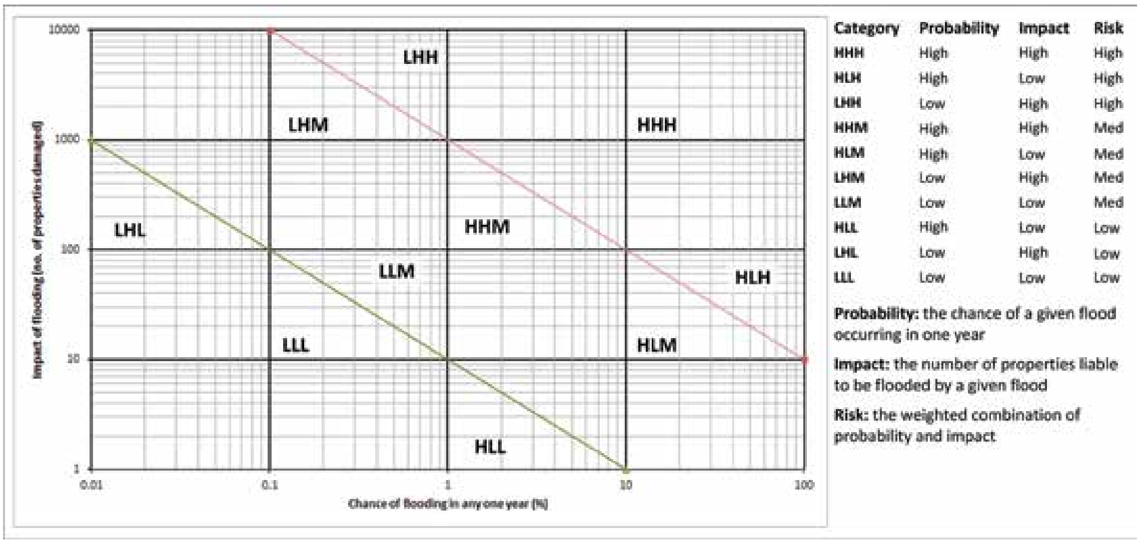


***Figure 5.5** Graphical evaluation of quantified risks (from Environment Agency, 2000)*

Detailed quantitative risk analysis approaches using event trees (see Section 5.3) are valuable for providing insight and understanding failure modes and providing specific estimates of risks (probability and consequences) for stakeholders. However, uncertainties in input values and outcomes need to be taken into account and communicating uncertainties to decision makers is vital. There is no one universal method of developing event/fault trees or quantitative risk analysis in general and they should only be undertaken by experienced practitioners.

5.2.2 Knowledge gaps and uncertainty

Knowledge gaps and uncertainty are prevalent in all types of assessment and risk analysis activities and their sub-tasks. It is important to recognise these and to record them where known.

Risk analysis is often undertaken despite there being 'gaps in knowledge' that, if filled, could influence the outcome. The risk analysis process should recognise this and at least identify where known knowledge gaps exist either in the **data** or in the **methods of analysis** used for the assessment.

Such gaps could exist in any of the 'data' used in the risk analysis itself or in the parameters used to derive certain scenarios. For example data on the incumbent hydrodynamic or hydrologic conditions, ground topography and structure geometry, soil typology and parameters, and data on receptors can all vary in completeness, level of accuracy and detail. Such variances can create imprecision in the results from using the data, which may need to be improved to reduce uncertainty in the outputs of the risk analysis.

'Methodological uncertainty' is the "lack of knowledge or ability to measure or calculate and gives rise to potential differences between the assessment of some factor and its 'true' value" (Samuels, 1995). Knowledge of the flood system is inevitably incomplete, as is understanding of the impact that interventions have on the system. As more aspects of a system are modelled and multiple models are encapsulated into a single risk assessment (for example using a global climate model, hydrodynamic model, defence failure model, human response model and impacts assessment model), the need to handle uncertainty in a robust manner becomes ever more important. There is an increasing demand on decision support systems, and a wide range of decision stakeholders with often conflicting aims and interests. So it is essential to provide some measure of the uncertainty associated with the overall data, analysis and outputs. This information is invaluable as it provides some level of confidence in the various output risk metrics.

In addition to the uncertainty associated with the approach, is the gross uncertainty associated with predictions of future flood risk – which is significant. These are addressed through evaluating strategic alternatives in the context of the full range of possible future scenarios, ie the robustness criterion.

Dealing with uncertainty in flood risk analysis

In traditional deterministic analysis uncertainties (where known or suspected) are accounted for by some kind of sensitivity analysis in which variations in input parameters are made using engineering judgement. In probabilistic analysis, the approach is to explicitly recognise data, analysis and output uncertainties and appropriately disaggregate them by source. This requires more rigorous approaches, which tend to be computationally expensive. So, as a minimum, upper and lower bands on source, pathway and/or receptor terms should be included. Four levels of uncertainty analysis can be identified to guide the approach:

- **Level 1:** include uncertainty in water level and wave conditions and propagate this information through the analysis to provide an output uncertainty.
- **Level 2:** represent the greatest source of uncertainty in the source- pathway-receptor model and propagate through to provide an overall uncertainty:
  - source terms: uncertainty in any element upstream of the first management intervention, eg precipitation, coastal water levels
  - pathway terms: uncertainty in, for example, levee crest levels, levee fragility, ground model etc
  - receptor terms: uncertainty in, for example, damage estimates for properties or infrastructure
- **Level 3:** a more rigorous uncertainty analysis, for example, a variance-based sensitivity analysis or a Monte-Carlo style analysis. These provide a greater insight into the variance on the output due to the variance of a given input. Methods exist for considering both correlated and non-correlated input variables, ie to handle any inter-dependencies.
- **Level 4:** as for Level 3 but also includes a more thorough analysis of the uncertainty in the selected methods and physical processes for the modelling. For example, comparison of the outcomes derived from different flood spreading, breach or river models adds a much larger degree of complexity. This can be a substantial undertaking, and is unlikely to be realised in decision support tools, which typically adopt a single risk analysis approach.

Presenting uncertainty

Uncertainty analysis is meaningless without careful consideration of how the information is presented to users. This is closely allied with the need for clear guidance on its use and interpretation. For example, it may be more appropriate to provide simple visualisation techniques to less expert users and enable more thorough data and model explorations for expert users (Environment Agency, 2009).

Acceptable levels of uncertainty

The level of uncertainty that is acceptable will depend upon the application of the risk analysis and on the perceived receptors at risk. So, the determination of the appropriate level of analysis will need to be ascertained through a tiered approach to risk assessment as described in Sections 2.1.3.3 and 5.2.1. This review should include determining the requirements of the risk assessment and setting the risk criteria. The process should question basic assumptions as to the applicability of the data used (age, resolution, original purpose of use etc), how expert review and judgement should be used, and how the proposed approach compares to other risk assessments.

5.2.3 Components of risk analysis

An analysis of flood risk requires identification and examination of all component factors that influence the risk of flooding in a system (see Figure 5.6). The process must be able to evaluate all these components and integrate them. Figure 5.7 illustrates the different components of the flood risk in a leveed area.

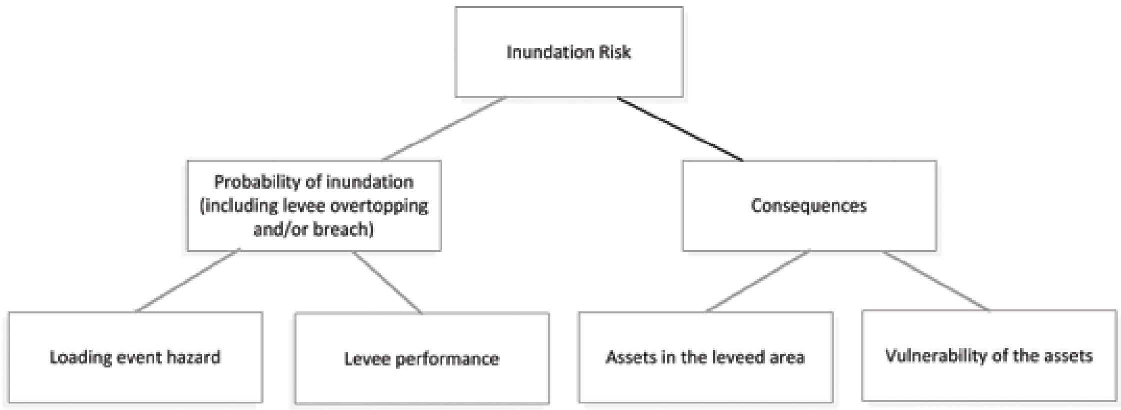


Figure 5.6 The different components of flood risk in a leveed area

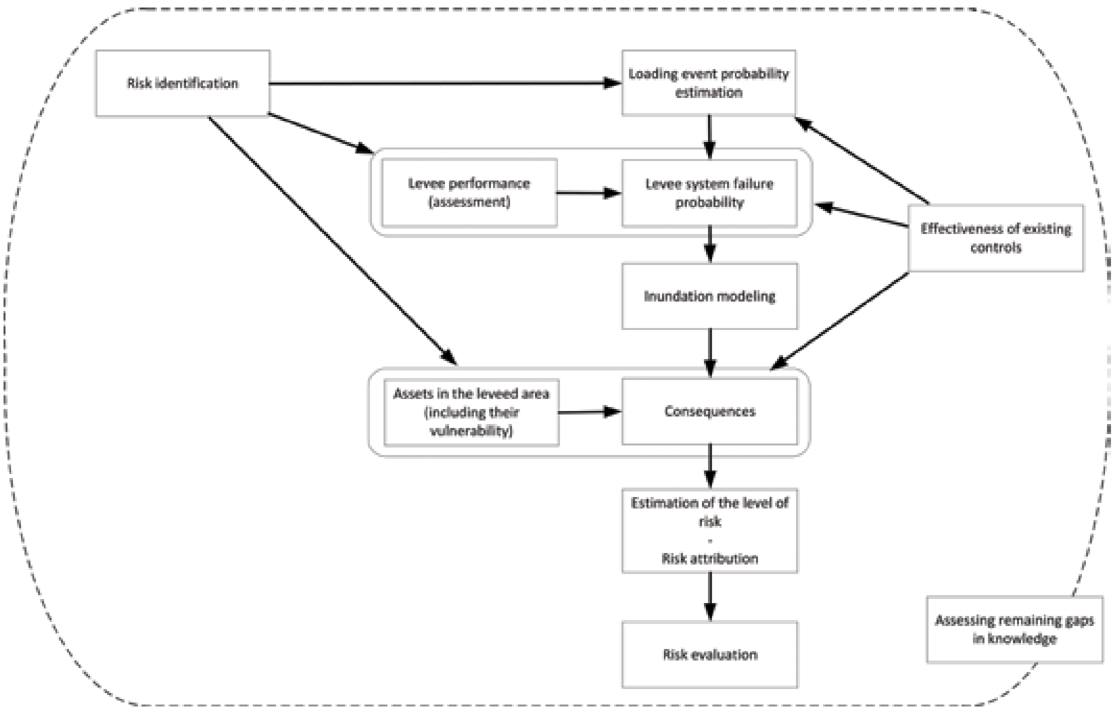


Figure 5.7 A framework for the analysis of different components of flood risk in a leveed area (courtesy R Tourment and M Wallis)

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1 **Risk identification (Section 5.2.4):** to analyse risk, first the factors affecting risk must be recognised and recorded to identify what might happen and what situations might arise. These factors include those sources-pathways-receptors of the flood system (see Figure 2.2).

2 **Event probability estimation (Section 5.2.5):** floods are episodic events. Large floods are rarer than medium sized or small floods. The probability of each size of event can be characterised as the chance that it will occur in any one year (its annual probability).

3 **Analysis of levee failure (Section 5.2.6):** the flood risk in a leveed area depends on the performance of the levee system. Where a failure in a levee system occurs will partly (in conjunction with the topography of the leveed area) determine the receptors that are affected by floodwaters. How and where a levee system might fail is an important consideration in estimating the level of risk. This activity is directly related to the assessment of the performance of the levee during a flood (see Section 5.3). In a risk analysis the result needs to be expressed in probabilistic terms (see Section 5.2.6).

4 **Inundation modelling (Section 5.2.7):** in order to assess potential damage or to prepare evacuation plans, information is needed on inundation patterns, including water depths, flow velocities, and timing of inundation. This information can be derived using inundation models, ie computer programs that simulate inundation along rivers, coasts or even urban drainage systems.

5 **Consequence analysis (Section 5.2.8):** a 'consequence' results when a vulnerable person or property is actually exposed to a flood and suffers some actual harm. Consequences may be a direct result of flooding (eg casualties, damaged buildings and/or contents) or indirect (eg health and social impacts, loss of business earnings due to recovery time). So, an analysis and evaluation of the likely consequences of a flood event needs to be estimated in order to determine the potential magnitude of the impacts of a flood event.

6 **Estimation of level of risk (Section 5.2.9):** an estimate of the level of flood risk is calculated by taking into account the probability of a flood event occurring and the potential consequences of that event derived from the previous steps. This step produces the results of a risk analysis

7 **Effectiveness of existing controls:** controls are measures, either structural or non-structural, taken in order to limit the possibility of occurrence of an inundation, or its consequences. They can apply either to the source (eg breakwaters, and upstream flood management including dams), pathway (eg levee maintenance, monitoring, and emergency management) or receptor (eg flood warning, population evacuation, and resilient buildings) parts of the system. Existing controls can and should be taken into account in the estimations of the event probability, of the levee failure, and of the consequences of the inundation.

Risk attribution and risk evaluation are further optional steps that can be undertaken at the scale of individual levee segments or for levee systems – whichever is appropriate to the scale of the risk analysis and to the objectives of the risk assessment.

- **risk attribution:** levees work together in a system to reduce the risk of flooding. However, all levee systems leave a residual flood risk within the leveed area. Risk attribution is a method of attributing that residual risk in the leveed area to individual levee segments, following the previous risk analysis methods (and allows prioritisation of investment between levee segments)
- **risk evaluation:** risk evaluation is not strictly a part of a levee manager's risk analysis as commonly agreed benchmarks for flood risk acceptability are not always available. Also, it is the broader society that dictates to the technological community the acceptability or tolerability of the levels of risk as it does for many other risks to society (see Section 5.2.11). Communicating the evaluation to decision makers is important to allow them to determine whether or not to proceed further with risk reduction measures.

5.2.4 Risk identification

To analyse risk, the source, pathway and receptor components affecting risk must first be recognised and recorded to identify what might happen and what situations might arise. The actual risk can be analysed by identifying a chain of causes and effects such as:

- rainfall or storms causing high water levels that in turn either increase the load on levees or inundate the floodplain
- the increased loads on the defences may cause failure of a levee (see Section 3.5.2), which may result in breach growth/progression and inundation of the leveed area
- the inundation may lead to casualties (loss of life, serious injury etc) and devastation of property.

So any risk identification process should consider the following factors:

- loading conditions (floods and other hydro-meteorological events, such as ice, earthquakes, unanticipated physical impacts etc) and their likelihood/probabilities
- likelihood/probability of flood inundation without a levee breach (ie loading event exceeds levee crest and/or due to hydraulic or non-structural failure – see Section 3.5.1)
- levee condition and its probability of breach under load (ie levee reliability) resulting in inundation
- characteristics of floodplain and inundation (depth, velocity, geographical extent etc)
- nature, extent and vulnerability of receptors (human, environmental, economic) to inundation
- existing risk control mechanisms and measures, and their effectiveness (eg emergency response)
- uncertainty (in knowledge about, and data on the factors in this list).

To determine the factors listed above, one may use knowledge of floods in the past, but for rare events this may well not suffice. In any event, the circumstances, for example the condition of the flood defences, and the occupation and receptors in the defended area, may have changed (see Section 2.1.4). So, by research, it is necessary to investigate the:

- probabilities and magnitudes of all possible floods
- probabilities and effects of any changes to the pathway
- consequences of impacts on receptors of flood risk.

With this information, the subsequent risk analysis generally adopts one of two approaches:

1. Creating specific scenarios by selecting particular combinations of, for example, loading conditions, failure probabilities, flood inundation characteristics and human responses. These scenarios may not be prescribed and can lead to variable analytical outcomes making comparisons between different risk assessments difficult. If consistency is required then guidance should be sought from the responsible authorities and/or appointed national or regional bodies. An example of prescriptive event scenarios adopted in France when using this approach is given in Box 5.1.
2. Assessing all possible combinations of loading, levee state (overtopped and breached) and resulting inundation, using Monte Carlo simulation and rapid inundation modelling and combining the results according to the individual probabilities to generate an overall assessment of flood risk expressed in economic terms (the approach adopted in the UK).

Box 5.1 ***An example of prescriptive event scenario requirements in France***

NOR: DEVQ0814392A (2008) defines the content of French levee risk survey and states that:

"The French levee risk survey is based on a risk analysis to identify the causes, combinations of events and scenarios that may cause important accidents. This assessment is based on crossing the levee design/state* and the effect of identified hazards.

Mainly natural, the identified hazards can be: floods, storms, earthquakes, landslides and avalanches, erosion of riverbanks and morphological changes of the river bed or sea/coast line.

For levees, the main danger to consider is the accidental release of water in the leveed area resulting from:

- a breach in a levee segment
- overflowing without breach of the levee
- a malfunction of a component of the levee."

2010/04/16 ordinance (NOR: DEVP1009801C) about French levee risk survey states that:

"The study of failure scenarios probability should, wherever possible, position their occurrence at three levels:

In the case of a levee without a spillway:

- before the overflow level over the crest
- close to the overflow level over the crest
- after the overflow level over the crest.

In the case of a levee equipped with spillways:

- before the spillways functioning level
- close to the spillways functioning level
- after the spillways functioning level: by overflow over the levee crest."

Note
* Levee risk surveys have to be conducted for both existing and projected systems.

5.2.5 Event probability estimation

The probability of floods can be estimated by applying various statistical interpolation/extrapolation techniques to records of past flood events. An interpolation example for a lowland river is given in Box 5.2, but the exact approach varies (see Chapter 7) depending on the source event (riverine, coastal etc).

Box 5.2 ***Example of an event probability estimation for a lowland river***

For a lowland river where there are about 100 years of fluvial flow records available, it is possible to assess by interpolation the discharge that corresponds with the 10 per cent annual probability flood by interpolation of the data. Figures 5.8 and 5.9 show examples of how the analyses of such events can be represented.

The same approach can be used to extrapolate to more extreme events than those for which records are available, but clearly the uncertainties grow the more outside the data range the extrapolation is taken.

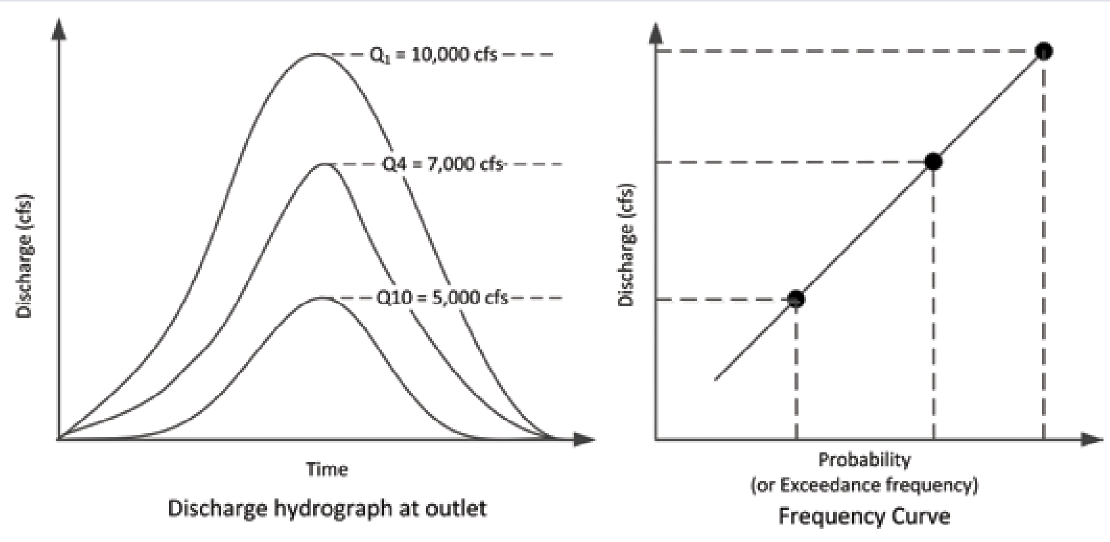

Figure 5.8 ***Example of a discharge hydrograph and a probability curve***

Box 5.2 Example of an event probability estimation for a lowland river (contd)

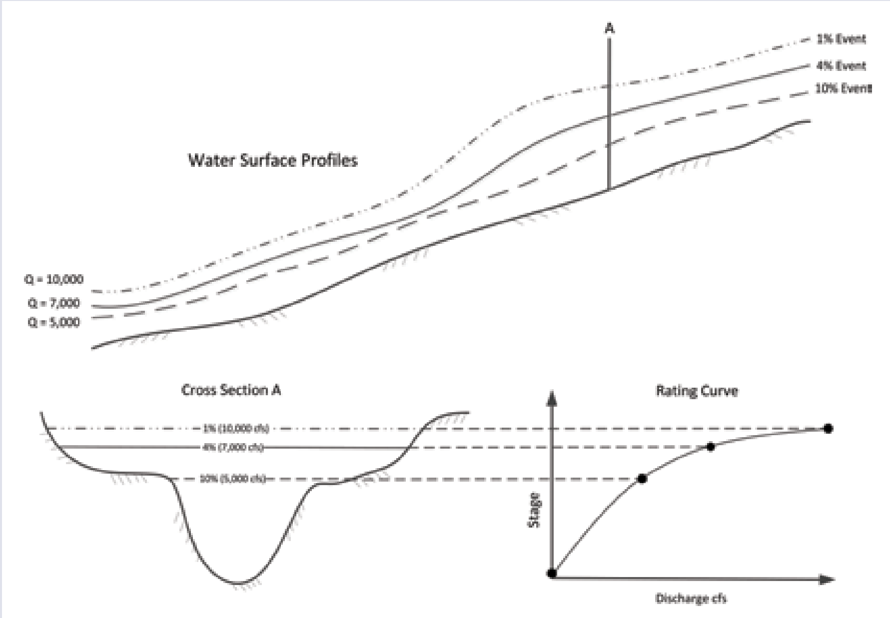

Figure 5.9 Example of water surface profiles derived from selected event discharges

The characterisation of the event in terms of hydraulic loading on the levee system, in order to perform a levee performance assessment is detailed is Section 5.3.3.2.

5.2.5.1 Probability of rare events

Establishing the size (discharge or height) of rare events, for example the 0.1 per cent per annum flood, requires statistical 'extrapolation' to derive a full magnitude-frequency relationship beyond the available data. Other considerations for estimation of extreme events include:

- **extrapolation errors:** while statistically extrapolation of homogeneous data should not extend more than three times the measurement period, extension beyond these limits to assess more rare events is still common practice. For such estimates 'error bands' showing the degree of uncertainty around the central estimate should be given. For example, at coastal locations where decades and even centuries of tide records and wave data have been collected, the water level associated with the one per cent or the 0.1 per cent per annum event can be estimated by statistical analysis, where necessary using extrapolation. Where there are no tide gauges, further interpolation from nearby locations with records may be used. Each additional step clearly widens the error bands that need to be assessed
- **use of synthetic data:** additional discharge data derived from modelling exercises are sometimes added to the recorded data to overcome the statistical limitation of short record periods. Examples of this so-called 'synthetic' approach include:
  - **the derivation** of river flow statistics by computer simulated weather conditions over prolonged periods (by randomly picking from measured data for individual days), followed by rainfall-runoff modelling of rivers, or the generation of sea state conditions to produce the distributions of river discharge or sea state
  - **the direct generation** of sea state conditions by random sampling from the developed joint density functions (eg of wave heights and water levels) (see Section 5.2.5.2)
- **rainfall statistics and flash floods:** rainfall statistics for small catchments are generally more plentiful than flow records, so they are often used in combination with catchment characteristics to generate extreme flow statistics (note that the effect of catchment characteristics means that probability of an extreme fluvial flow occurring will not be the same as the probability of the rainfall event that causes that flow).

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5.2.5.2 Joint probability of events

Estimation of flood events often requires the assessment of more than one causal factor. For example, two unrelated hazards may occur in conjunction to result in a worse flood threat (for example a higher water level), than if only one had occurred. Analysis of the resulting 'joint probability', ie the probability of two or more conditions occurring at the same time, is common practice in coastal engineering for the estimation of:

- total coastal water levels as a result of storm surge and astronomic tide
- combinations of wave heights, periods and directions with total water levels.

However, similar joint probability analysis may be carried out for combined events such as landslides and earthquakes, riverine flood stage and ice loading, failure of upstream dams and levees, or coincident failure of levees on the opposite bank of a river.

The essential elements of a joint probability extremes assessment are the **distribution** of each variable, the **extreme values** of each variable, and the **dependence** for each variable-pair, coupled with a method to combine all of this information in a meaningful way. One of three types of approach is typically used:

- **analytical approach:** feasible if the extreme values correspond to fitted distributions
- **Monte Carlo simulation:** more practical where there is a combination of empirical distributions, fitted statistical models and/or imposed extreme values
- **desk study approach:** more appropriate for non-specialists, in which the extreme values are applied to joint probability conditions, pre-computed in terms of marginal return periods, for a number of different levels of dependence.

The three methods most commonly used to present the results of a joint probability extremes analysis are joint probability density, structure function and joint exceedance extremes (single-variable extremes will also be calculated for checking purposes). Further details are given in Hawkes (2008).

While there is no absolute upper limit on the number of variables to consider in a joint probability analysis, the calculations become more complicated for each extra partially dependent variable considered. In practice, most joint probability analyses are reduced to just two primary variables (Hawkes, 2008). Additional secondary and/or conditional variables can be incorporated into the analysis in other ways (Defra/Environment Agency, 2005).

Further information on estimating the probability of flood events, and combining source events, is provided in Chapter 7, Section 7.4.

Effectiveness of the existing controls, like flood retention dams, groynes, beach nourishment having effect on the loading event, including an estimation of their efficiency, also should be taken into account into this analysis.

5.2.6 Analysis of failure of levees

The subject of the analysis of the failure of levees is detailed in Section 5.3. The purpose of this section is to show how the result of a levee assessment is used as an input into a wider flood risk analysis of a levee system, particularly as this result then needs to be expressed as a probability.

5.2.6.1 Probability of levee segment failure

Levees are rarely uniform in materials, methods of construction, geometry, reliability etc (Section 3.3) and this variability influences the probability of failure. So, the likelihood/probability of failure for a levee system must be evaluated for each levee segment using a functional analysis of the levee (as shown in Figure 5.10) including the levee components, the components' functions, and the functionally homogenous parts of the levee length.

Differences between the probabilities of failure of various levee segments in a levee system can then be identified. Methods of identification of failure modes (see Section 3.5) are given in Section 5.3.3.

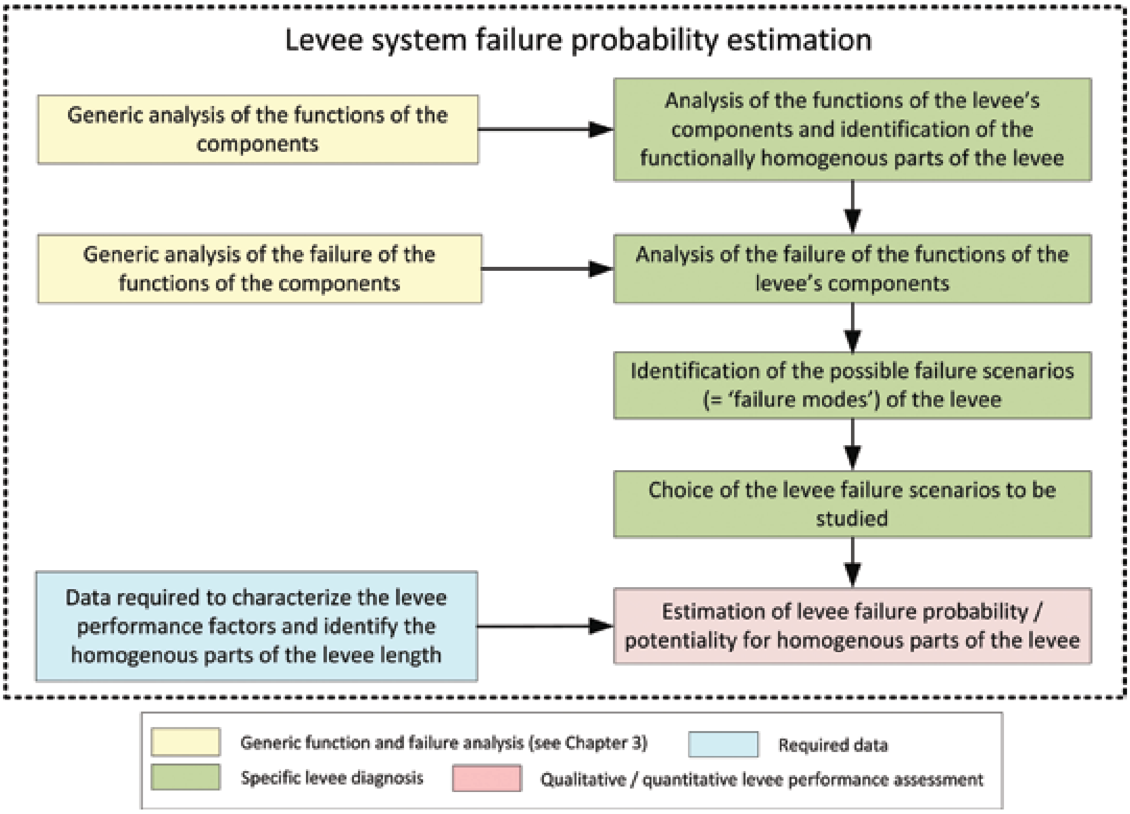


Figure 5.10 Levee system failure probability estimation (courtesy B Beullac and R Tourment, Irstea)

5.2.6.2 Probability of levee system failure

The flood risk in a leveed area depends on the combined performance of all the levee segments in the levee system. The failure of any one component of a levee system may be dependent on the performance of another component, or it could be completely independent or partially dependent. For a (quantitative) risk analysis of a levee system, these relationships should ideally be expressed in probabilistic terms. An example of a calculation method for analysing the potential failure of a levee system is shown in Box 5.3. In this case, the levee segments are assumed to be long enough to be independent.

In a risk analysis, such a method can be used to complement the 'raw' assessment result of individual levee performance to make the link between the levee failure and the resulting inundation scenario being studied/evaluated. This will depend on, among other things, the location and size of the breach within the levee system. Further information on breach analysis is found in Section 8.10.

Control measures taken in order to limit the probability of failure of a levee during a flood (eg organisation of monitoring and repairs), including an estimation of their efficiency, should be taken into account in this analysis.

Box 5.3 A method for analysing levee system failure (from Gouldby et al, 2008)

Figure 5.11 shows a conceptual river channel separated from the floodplain area (flood area) by a series of discrete levee sections ($d_1, d_2, \dots, d_n$). Each levee section has an independent and different resistance to flood loading. These are characterised by, for example, different types of structure, crest levels or condition grades.

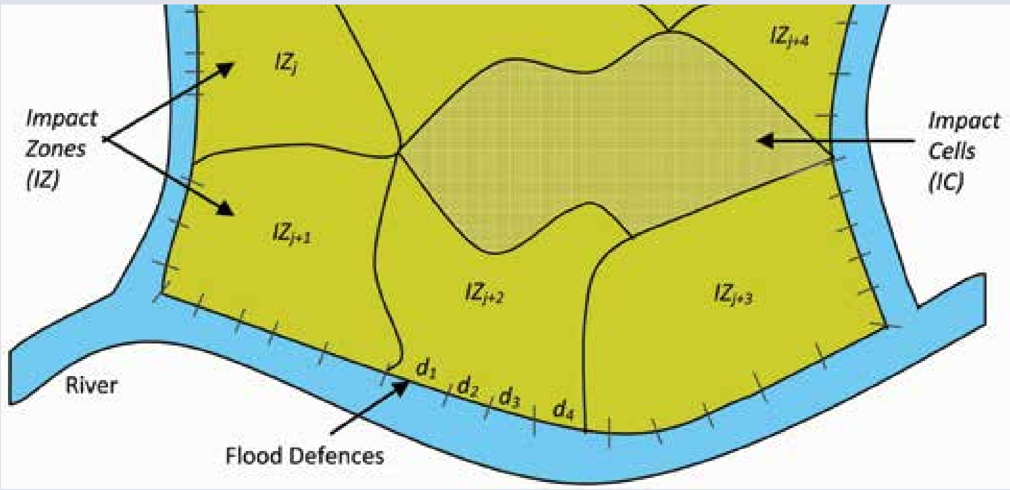


Figure 5.11 Conceptual diagram of the model backdrop (from Gouldby et al, 2008)

Occurrences of extreme loads are defined as continuous random variables (L) associated with each levee, and individual levee section failure (structural failure) is defined as a continuous random variable, conditional on load (these distributions are commonly referred to as fragility curves (see also Chapter 3). During any flood event each individual defence section can exist in two possible states (ie they are defined as Bernoulli Random Variables), failed or not failed (d_i, $\bar{d}_i$), with the likelihood of any particular state obtained with reference to the fragility curves.

Within any given flood area, the continuous line of levee lengths form a defence system. So, the potential number of levee system states (combinations of failed/not failed defences within the flood area), for any specified hydraulic load (l), is 2^n. The levee system state, derived from the failed/non-failed state of each levee section, is a discrete random variable (D) whose conditional probability mass function (pmf) is simply:

$$p_{D|L}(d,l) = P[D = d|L = l] \tag{5.1}$$

where:

D = discrete random variable that represents the levee system state

d = any particular combination of failed and non-failed levees that comprise the defence system (d is a vector that comprises the state of all the levees in the defence system)

The performance of consecutive levee lengths are assumed to be independent of one another, so the probability of any particular defence system state, for example, $d_1,\dots d_k, \bar{d}_{k+1}\dots\bar{d}_n$, occurring on any given hydraulic load (l), is, through the multiplication rule:

$$p_{D|L}(d,l) = P[D = d|L = l] = \prod_{i=1}^{k} p(d_i|l) \prod_{i=k+1}^{n} [1 - p(\bar{d}_i|l)] \tag{5.2}$$

5.2.7 Inundation modelling of the leveed area

Levee failure and/or overtopping results in inundation of the leveed area and can cause casualties and damage. The characteristics of the inundation and the topography of the land will determine where the water goes, which receptors will be affected and to what extent. Inundation modelling software for rivers, coasts or even urban drainage systems is often used to simulate:

- inundation routes and patterns
- water depths
- flow velocities
- timing of inundation.

Inundation modelling can be undertaken for levees that have been overtopped by floodwater or have been breached (whether or not overtopped). For the overtopping scenario, volumetric inflow can be derived using wave run-up and overflowing calculations as described in Section 8.2. For levees that have breached, a breach analysis may be undertaken as described in detail in Section 8.10, to determine the volumetric inflow of water into the inundation model for the defended area. Further details on different types of inundation (or 'flood spreading') models from simple to complex are given in Section 8.11, which also describes different ways in which breach and inundation models can be coupled.

Box 5.4 illustrates how repeated runs of a simplified inundation model for different overtopping and breach scenarios can be used within a flood risk analysis to establish the probabilities of flooding and the related inundations characteristics in various parts of a leveed area.

As well as its use in flood risk analysis, inundation modelling is useful for flood event management planning (see Chapter 6). Plans for warning, evacuation and traffic routing are often based on computed flooding patterns, water depths and arrival times of the floodwater. The anticipated impact on the receptors within the leveed area described in the following section may depend on these plans.

Box 5.4 ***A method for inundation modelling as part of risk analysis***

Following on from the method for analysing levee system failure shown in Box 5.3, the floodplain area can then be discretised into a series of impact cells (z_1, z_2,..., z_m) (see Figure 5.12). Any specified impact cell can be influenced by floodwater discharged through any of the (n) defences within the flood area.

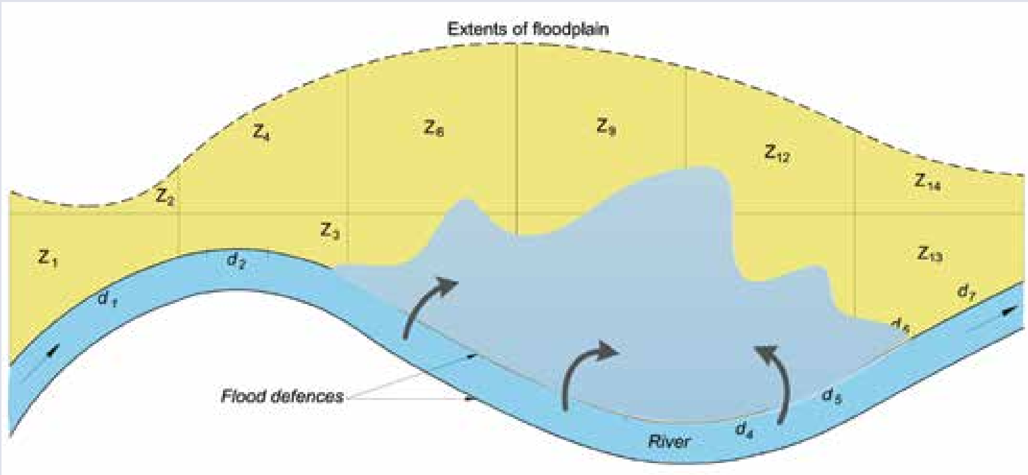


Figure 5.12 ***Conceptual diagram of the inundation model backdrop (adapted from Gouldby et al, 2008)***

Flood volumes (V) discharged through (or over) any given levee section under any specified load (l) are function of the defence system state. The flood depth Y in any impact cell is a function of the flood volume discharged into the floodplain through the levees and thereby a function of the defence system state (Y is a function of D and so a discrete random variable):

$$Y=g(D) \tag{5.3}$$

The problem now is for any particular impact cell, to determine the probability of exceeding any particular flood depth (y) under any specified loading condition (l):

$$\boxed{P(Y > y|l) = P[g(D) > y|l]} \tag{5.4}$$

If the function that relates D to Y is readily evaluated then solution of this problem is trivial, simply involving summation of the probability mass function over realisations of D that yield flood depths greater than y (this region is denoted as (A)).

$$\boxed{P(Y > y|l) = \sum_A p_{D|L}(d_s, l)} \tag{5.5}$$

Evaluation of this function involves running a hydraulic model, for example the rapid flood spreading model (RFSM). While such models are increasing in capability they typically still prohibit solution by enumeration, particularly given that the vector (d) is likely to exceed 100 separate elements (ie a defence system can comprise more than 100 defence lengths (ie more than 2^{100} defence system states) and a range of loading events are to be considered.

The preferred choice for solution of this problem is a conventional Monte Carlo simulation. The elements of the defence vector are sampled (with reference to the defence specific fragility curves) so each realisation of the sampling process comprises a defence system state. A vector (V) comprising the volume discharged from each defence system into the floodplain can then also be evaluated.

Box 5.4 ***A method for inundation modelling as part of risk analysis (contd)***

The vector (*V*) comprises the boundary condition for the hydraulic flood spreading model. This model then generates the maximum flood depth in each impact cell from the input flood volumes. The probability of exceeding a specified depth, given a specified loading event (l), in any particular impact cell, is now estimated through:

$$P(Y > y|l) \approx \frac{m_{Al}}{m_l} \quad (5.6)$$

Where m_{Al} is the number of realisations from the sampling process that result in depths greater than y under loading condition , and m_l is the total number of simulations undertaken for loading event *l*.

To obtain the unconditional annual probability of exceeding *y*, the continuous loading distributions are discretised into *q* levels of *l*: l_1, l_2, l_3,... l_q, associated with specified return periods.

$$P(Y > y|l) \approx \sum_{i=2}^{q-1}\left[p\left(L \geq \frac{l_i + l_{i+1}}{2}\right) - p\left(L \geq \frac{l_i + l_{i+1}}{2}\right)\frac{m_{Al_i}}{m_{l_i}}\right] \quad (5.7)$$

An important consideration when applying the Monte Carlo simulation is the number of simulations required to stabilise the estimated quantity (see Gouldby *et al*, 2008).

5.2.8 Consequence analysis

Evaluating the consequences of inundation in the leveed area (see Figure 5.13) requires combing the net results of hydraulic modelling of potential inundations (see Box 5.4) and the estimated vulnerability of the different assets identified and located in the leveed area.

A leveed area can contain many different types of assets, including:

- people
- buildings
- natural/undeveloped areas
- agriculture
- factories/business
- critical infrastructure: transport, utility and communications networks
- recreational areas
- nature conservation areas.

The extent of the impact on these receptors of flooding in the leveed area depends on key inundation characteristics such as:

- depth of the floodwater
- flow velocity of the floodwater
- duration of the inundation
- speed of rise of the water levels
- the time from breach to impact.

The impacts are also dependent on and interact with the characteristics and quality of the water (salt/fresh, temperature, turbidity, pollutants etc).

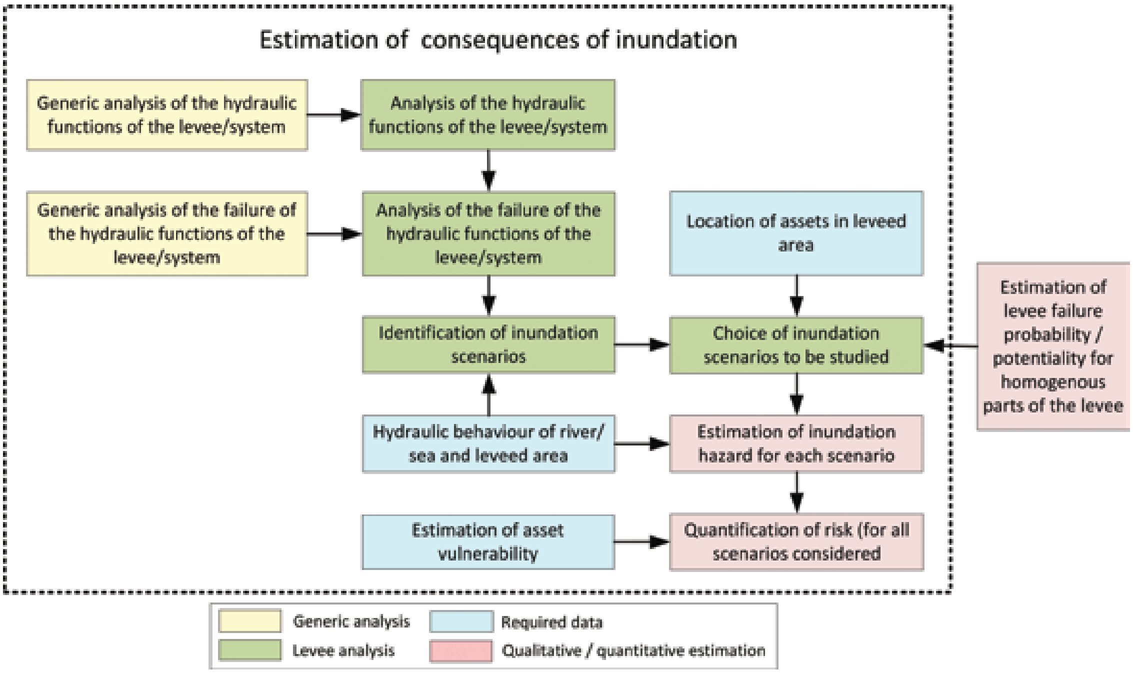


Figure 5.13 An example of an approach to the estimation of the consequences of inundation (courtesy B Beullac and R Tourment, Irstea)

To evaluate impacts, the people and assets in the leveed area should be identified and geographically referenced. Their vulnerability also needs to be assessed. Vulnerability of an asset is a function characterising its damage according to the hydraulic characteristics of the inundation (ie water level, flow, duration).

Control measures taken to limit the consequences of an inundation, like flood warning, organisation of evacuation, shelters, including an estimation of their efficiency, should be taken into account in this analysis.

5.2.8.1 Characterisation of potential impacts

The potential impacts on people and assets can be characterised in terms of:

- human health, casualties or life loss (or social consequences)
- cultural and archaeological heritage
- economic consequences
- environmental consequences.

The social criterion may just be the number of detrimentally affected persons, but normally impacts are differentiated into categories such as loss of life, health effects, stress, safety, equity and community.

Damage to cultural heritage including damage to assets such as historic buildings, parks and gardens, ancient monuments etc can be damaged by floodwater. Cultural and archaeological heritage can be included in flood risk multi criteria analysis in a similar way to environmental receptors through a simple yes/no damage function. An example is shown in Table 5.1. Other indicators/criterion could be added.

Table 5.1 An example of cultural and archaeological evaluation criteria

Indicator/criterion	Potential damage (risk)		Explanation/notes
	Yes	*No*	
Damage to site	1	0	Where inundation has damaged but not destroyed the site. Site and/or historic artefacts are recoverable.
Loss of site	1	0	Where inundation has destroyed the site and removed cultural value and/or historic artefacts.
Final assessment	Σ	Σ	

The most common economic criterion is the expected annual flood damage. This is normally simplified to the direct losses arising from flood damage, but indirect losses (eg due to business or transport interruption) may also be considered.

Environmental criteria can measure, for example, the impact on fauna and fauna habitats, water quality and quantity, soil quality or the effects on landscape scenery. It should be noted that within this criteria, flooding can provide both positive and negative effects.

Sometimes technical criteria are listed, for example hydraulic effects. However, it should be remembered that these do not necessarily measure risk or risk reduction effects – but merely one component of it.

Some difficulties arise because impacts on receptors can often be studied in several ways. For example the consequences of an inundation for a flooded factory can be approached in terms of different criteria:

- economic losses
- social issues (eg health, stress) for employees due to temporary or permanent unemployment
- environment (eg pollution, habitat loss).

Some flood risk assessments have attempted to overcome these difficulties by adopting a multi-criteria approach which can include both non-monetised as well as monetised criteria (see Box 5.5).

Box 5.5 The application of multi criteria analysis in flood risk management

Meyer (2007) found that the application of MCA for flood risk management is still rare. However, there are examples such as:

- Brouwer and van Ek (2004) who evaluated long-term flood risk management options in the Netherlands using the DEFINITE software (Janssen *et al*, 2003)
- RPA (2004) who evaluated the applicability of MCA procedures for flood risk management decision making in the UK
- Penning-Rowsell *et al* (2003) who include a section on multi criteria evaluation of flood protection measures as part of damage evaluation guidance for England and Wales
- Socher *et al* (2006) describe the use of a basic point-based MCA approach for the prioritisation of flood defence structures in the federal state of Saxony, Germany
- Costa *et al* (2004) used the MACBETH approach for the evaluation of alternative flood control measures in Portugal
- Akter and Simonovic (2005) describe use of MCA for flood risk management in the Red River Basin in Canada.

Although these studies focus on methodologies for incorporating the opinions of multiple stakeholders, they do not consider the spatial dimension of flood risk. Very few examples exist of the application of spatial MCA specifically in the field of flood risk analysis and management. In an analysis of the spatial distribution of the multiple effects of different flood protection alternatives in the Red River Basin, Tkach and Simonovic (1997) used a GIS-based variant of the compromise programming (CP) MCA technique that they called spatial compromise programming (SCP). This approach was extended by Simonovic and Nirupama in 2005 by integrating fuzzy set techniques in order to deal with uncertainties in the evaluation criteria. A similar approach was used by Thinh and Vogel (2006) for land use suitability assessment in the Dresden region.

The selection of appropriate evaluation criteria is an important step in MCA. Besides the aforementioned publications on the flood risk problem there are also some publications with no particular MCA background that give a good overview of potential criteria (eg De Bruijn (2005) and Olfert (2006)).

5.2.8.2 Risk to life

The nature and extent of the impacts of flooding on human life and health are significantly affected by the type of flooding, including the depth and velocity of the floodwater, the duration and degree of exposure and the amount of warning given. In mountainous areas 'flash' floods can pose additional problems as they may contain large amounts of mud and other large debris. Flooding from the sea can be equally devastating and impacts can occur for long distances along coastlines.

Risk to life can be 'modelled' (see Box 5.6 and Table 5.2), but there are considerable difficulties. Many of the existing risk to life models are designed to predict fatalities for either large-scale floods caused by flood defence failure in low-lying areas (eg Jonkman *et al*, 2002 and 2008) or dam or levee breach scenarios (eg Waarts, 1992, and Graham, 1999). However, flood events vary greatly and can be quite different, so these models may not necessarily be applicable in different situations. These models also largely involve mortality rates based on empirical observations, which are then applied to the exposed population according to flood severity and other parameters such as flood warning and/or awareness.

Box 5.6 A risk to life model (Jonkman et al, 2008)

A number of models have been developed as a means to calculate the potential fatalities from flood events. Jonkman *et al* (2002) found that they frequently only addressed some of the factors that can cause death. Jonkman *et al* (2008) attempted to overcome this problem by developing a risk to life threshold model, which attributes a 'fatality factor' dependent on various characteristics such as flood depth and velocity hazard, vulnerability, and structural damage. They characterised it as follows:

Risk to life in Europe = *f*(F, Ex, Pv, – M)

where:

F = flood hazard characteristics (eg depth, velocity)

Ex = exposure to the hazard (related to the nature of the area, whether people can avoid direct contact with the floodwaters without being threatened by building collapse)

Pv = people vulnerability (the importance of this variable will depend upon the severity, for example in some rcumstances, such as very severe floods, this variable is redundant)

M = mitigating actions (is there sufficient warning to enable people to evacuate the area entirely or seek appropriate shelter from the floodwaters?)

It is possible to construct threshold models highlighting the consequences of flooding at different depths and velocities using the depth-velocity product. Table 5.2 combines the thresholds for people directly exposed to the floodwaters and information about whether particular areas are vulnerable. It then illustrates these thresholds and identifies the risks associated with floodwaters at each of the different levels. The model provides four different risk levels each highlighted by a different colour: extreme risk (red), high risk (orange), medium risk (yellow) and low risk (green).

It is also possible with this model of risk to life to provide some indication of the dominating factors leading to injuries and fatalities from flooding of difference levels. However, due to the complexity of the factors leading to death, and particularly in relation to those areas in the most vulnerable zones where physically vulnerable properties are found due to poor construction or unsuitable materials, this can only be a broad assessment.

Table 5.2 Threshold model indicating risk to life from flooding (Jonkman et al, 2008)

Depth × velocity mid-range	Outdoor hazard	Nature of the area	Structural damage	Risk to life from flooding	Fatality factor
>7 m^2s^{-1}	Extremely dangerous for all	**3 High vulnerability** (including mobile homes, campsites, bungalows and poorly constructed properties)	Total collapse may occur. Structural damages probable in particular for properties with poor quality building fabric	Risk to life in this scenario is extreme as not only are those in the open very vulnerable to the effects of floodwaters but those who have also sought shelter are very vulnerable due to the fact building collapse is a real possibility.	Hazard and building collapse dominated
		2 Medium vulnerability (typical residential area mixed types of properties)			
		1 Low vulnerability (multi-storey apartments and masonry concrete and brick properties)			
1.10 to 7 m^2s^{-1}		**3 High vulnerability** (including mobile homes, campsites, bungalows and poorly constructed properties)	Structural damages possible	All those exposed to the hazard outside will be in direct danger from floodwaters. Those living in mobile homes will be at risk from the high depths and velocities and those in single storey dwellings will be at risk from not being able to escape to upper floors. Those in very poorly constructed properties will also be vulnerable from structural damages and/or building collapse.	
		2 Medium vulnerability (typical residential area mixed types of properties)		All those exposed to the hazard outside will be in direct danger from the floodwaters. Damage to structures is possible. Those in unanchored wooden frame houses are particularly vulnerable. With very deep waters there is the risk of some not being able to escape.	Hazard dominated
		1 Low vulnerability (multi-storey apartments and masonry concrete and brick properties)		All those exposed to the hazard outside will be in direct danger from the floodwaters. In this scenario those residing in these properties have the lowest risk although structural damage is still possible in wooden properties	
0.50 to 1.10 m^2s^{-1}	Highly dangerous for most	**3 High vulnerability** (including mobile homes, campsites, bungalows and poorly constructed properties)	Structural damages and collapse possible for properties with poor quality building fabric	Those outside are vulnerable from the direct effects of the floodwaters. In addition, those in single storey dwellings will be vulnerable in deeper waters. People will also have little protection in mobile homes and campsites. Those in very poorly constructed properties will also be vulnerable from structural damage and/or building collapse. Vehicles are also likely to stall and lose stability.	

Table 5.2 Threshold model indicating risk to life from flooding (Jonkman et al, 2008)

<table>
<tr><td rowspan="2">0.50 to 1.10 m^2s^{-1}</td><td rowspan="2">Highly dangerous for most</td><td>2 Medium vulnerability (typical residential area mixed types of properties)</td><td rowspan="2">Structural damages – less likely and less severe</td><td>Anyone outside in the floodwaters will be in direct danger. It is at this point where behaviour becomes significant as structural damages are less likely. Those inside should mostly be protected. Vehicles are likely to stall and lose stability. Are people undertaking inappropriate actions such as going outside when it is not necessary?</td><td rowspan="2">Behaviour dominated</td></tr>
<tr><td>1 Low vulnerability (multi-storey apartments and masonry concrete and brick properties)</td><td>Anyone outside in the floodwaters will be in direct danger. It is here at this point where behaviour becomes significant as structural damages are less likely so those inside should be on the most part protected. Vehicles are likely to stall and lose stability. Are people undertaking inappropriate actions such as going outside when it is not necessary?</td></tr>
<tr><td rowspan="3">0.25 to 0.50 m^2s^{-1}</td><td rowspan="3">Moderately dangerous for some</td><td>3 High vulnerability (including mobile homes, campsites, bungalows and poorly constructed properties)</td><td>Structural damages possible with poor quality building fabric</td><td>Only the most vulnerable should be in direct danger from floodwaters (eg children and the elderly). In this category the shelter may not protect them. Motor vehicles may become unstable at these depths and velocities. Those in very poorly constructed properties may also be vulnerable from structural damages.</td><td rowspan="3">People vulnerability dominated though some behaviour-related fatalities</td></tr>
<tr><td>2 Medium vulnerability (typical residential area mixed types of properties)</td><td rowspan="2">Unlikely</td><td>Only the most vulnerable should be in direct danger from floodwaters (eg children and the elderly). Motor vehicles may become unstable at these depths and velocities. Those who seek shelter should be safe.</td></tr>
<tr><td>1 Low vulnerability (multi-storey apartments and masonry concrete and brick properties)</td><td>Only the most vulnerable should be in direct danger from floodwaters (eg children and the elderly). Motor vehicles may become unstable at these depths and velocities. Those who seek shelter should be safe.</td></tr>
<tr><td rowspan="3"><0.25 m^2s^{-1}</td><td rowspan="3">Low caution</td><td>3 High vulnerability (including mobile homes, campsites, bungalows and poorly constructed properties)</td><td rowspan="3">Unlikely</td><td rowspan="3">A very low risk to adults either out in the open or who are in a property. There may be a threat to the stability of some vehicles even with these low depth-velocity factors.</td><td rowspan="3">Low risk</td></tr>
<tr><td>2 Medium vulnerability (typical residential area mixed types of properties)</td></tr>
<tr><td>1 Low vulnerability (multi-storey apartments and masonry concrete and brick properties)</td></tr>
</table>

Vulnerability relates to the susceptibility of people and assets in the leveed area to physical or emotional injury or damage, given their exposure to an event. Some locations will have a higher degree of vulnerability and potential for damage than others, either by their character or by the presence of a large number of vulnerable people (eg children and/or elderly or sick people):

- areas with campsites, locations of mobile properties or areas with large open recreational spaces will provide little shelter from direct contact with floodwaters may be particularly vulnerable
- urban residential areas or other locations with buildings should, in theory, provide a higher level of shelter from floodwaters. In cases of severe flooding the integrity of shelters can be compromised by either structural damage or in some instances total collapse:
  - the degree of protection will vary according to the building type, the quality of construction and the number of storeys
  - factors that affect the ability of a building to withstand floodwaters include the materials that the building is made from (eg timber, brick, stone or a mix of materials), the quality of the original construction, construction methods, and the age and condition of the property.

5.2.8.3 Economic damages and their estimation

Flood damages to receptors are generally categorised for economic purposes into direct and indirect damages and then into tangible and intangible damages (Smith and Ward 1998, Parker *et al*, 1987, Penning-Rowsell *et al*, 2003, and Messner and Meyer, 2005):

- **direct flood damage** covers all varieties of harm that relate to the immediate physical contact of floodwater to humans, property and the environment. This includes, for example, damage to buildings, economic assets, loss of standing crops and livestock in agriculture, loss of human life, immediate health impacts, and loss of ecological goods. Direct damages are usually measured as damage to stock values
- **indirect flood damages** are those caused by disruption of physical and economic linkages of the economy, and the extra costs of emergency and other actions taken to prevent flood damage and other losses. This includes, for example, the loss in production of companies affected by the flooding, induced production losses to their suppliers and customers, the costs of traffic disruption or the costs of emergency services. Indirect damages are often measured as loss of flow values
- **tangible damages** are those that can be easily expressed in monetary terms, such as damages on assets, loss of production etc
- **intangible damages** are those that are more difficult to assess in monetary terms and include casualties, health effects or damages to ecological goods and to all kind of goods and services that are not traded in the market (Messner, 2007).

The 'estimation' of tangible economic damages is often related to the depths of water calculated by the inundation modelling described in the previous section. Typically depth/damage curves are used to estimate damages under a variety of inundation events of different exceedance probabilities (eg 1:10, 1:25, 1:50, 1:100, 1:200, 1:500). This allows a damage-frequency function (Figure 5.14) to be estimated. The area under this function represents the expected annual damage (EAD), which is the expected value or mean of all possible values of damage and can be used as an estimate of benefits in cost–benefit analysis (CBA). In this regard 'damages' includes all economically assessable negative consequence of flooding, including social and environmental. Estimates of EAD should ideally not only be prepared for mean values of damage estimates, but also for the minimum and maximum.

Differences in economic flood damage evaluation methods often relate to the:

- damage categories adopted (see Table 5.3) such as property damage, income loss, traffic disruption
- degree of detail
- geographic scale of analysis
- application of basic evaluation principles (eg replacement cost versus depreciated cost)
- application or non-application of results in cost–benefit and risk analyses.

Depending on the geographical scale of the assessment different data sets might be used. For large-scale or national assessments proxy land use categorisation and official statistics on regional or national levels may be used to derive parameters such as number of inhabitants, housing density etc (eg net value of fixed assets for different economic sectors). For local scale analyses actual figures and features may be known or estimated based on locally held datasets and/or derived from maps of the area.

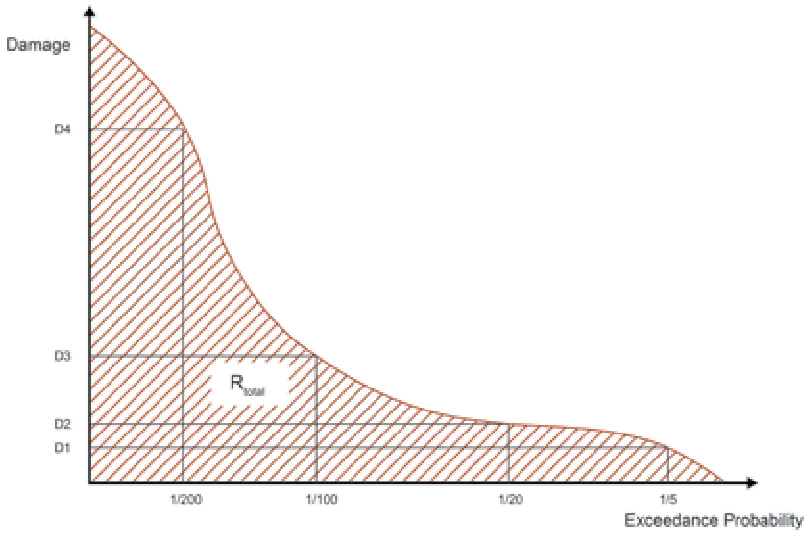


Figure 5.14 Computing the mean EAD

Table 5.3 Receptor categories considered in England (from Volker, 2007b)

Damage category	Macro scale	Meso scale	Meso/micro scale	Micro scale
Direct, tangible damages				
Residential buildings	M	M	M	M
Household inventory	M	M	M	M
Vehicles/cars				
Non-residential buildings, fixture and fittings, movable equipment	M	M	M	M
Inventories	M	M	M	M
Livestock				
Infrastructure				
Streets				
Railways				
Ground values	Agricultural land			
Indirect losses				
Loss of value added				M
Agricultural production	M	M		M
Emergency cost			M	M
Traffic disruption	M		M	M
Other			Flood warning (benefits)	Surrogates costs: house renting, drying out process
Intangible loses				
People	Q	Q		(under development)
Health				M
Environmental losses			Q	M
Recreational losses				M

Table 5.3 Receptor categories considered in England (from Volker, 2007b) (contd)

Cultural goods				
Toxification				
Other	SFVI	SFVI		SFVI

Notes

M = monetary
Q = other quantitative units
D = descriptive, qualitative assessment
SFVI = Social Flood Vulnerability Index (this is a measure for the coping capacity of the flood affected population including indicators for vulnerable groups and persons, eg elderly people, lone parents, people with pre-existing health and financial deprivation problems).

Evaluating impacts to 'critical infrastructure' (such as power, fuel and water utilities, communications, transport links, and medical, fire and police services) is difficult to include routinely or consistently as it can involve subjective/qualitative as well as quantitative measures. The effects of disruption to critical infrastructure can result in serious direct and indirect impacts, both tangible and intangible, of flood inundation. Impacts can be felt far beyond the flooded area itself. After a flood, the subsequent disruption to power, water and communication facilities can afflict recovery operations of people and commerce alike. Such delays can result in significant further economic losses as well as social wellbeing and health problems.

5.2.8.4 Environmental impacts and their evaluation

Flood inundation can have adverse (and sometimes beneficial) effects on environmental receptors. Evaluation of these often requires subjective judgements about exposure and vulnerability of particular habitats and species to flooding. Sometimes, given the diversity of impacts a semi-qualitative scoring approach can be useful, as illustrated in Box 5.7. Alternatively approaches to monetising the impacts are possible.

Box 5.7 Example approach to evaluation of environmental impacts (Tapsell, 2008)

An example of one approach to evaluating inundation impacts on environments is shown in Table 5.4. A simple yes/no damage function can be applied for each criterion, depending on whether the area is affected or not. Provided the criteria are different in terms of their impact functions, which could occur simultaneously during one unique flood event, then the sum of the values given for each can be used to estimate the environmental impact potential of the flood. The formula risk = probability × consequence can then be used to provide an annual average environmental consequence expressed on a point scale.

Table 5.4 An example of some environmental evaluation criteria

Indicator/criterion	Potential damage (risk)		Explanation/notes
	Yes	*No*	
Erosion	1	0	Where erosion of fine grain material occurs pollutants might be mobilised and transported (pollutants = heavy metals bond to clay minerals and organic matter – nutrients such as phosphorus).
Accumulation	1	0	Same as erosion but creation of new polluted sites due to accumulation of the transported material.
Inundation of oligotrophic biotopes	1	0	A longer inundation (>one hour) of oligotrophic biotopes might negatively affect these biotopes in form of eutrophication or drop of the number of species.
…	…	…	…
…	…	…	…
Final assessment	∑	∑	

To monetise environmental damages and benefits for use in a cost–benefit analysis various approaches are possible, but all involve some attempt to internalising what are ‘non-monetised’ intangibles into market values. Approaches include:

- **ecosystem services approach:** this is a framework for assessing the goods and services provided by ecosystems, where environmental effects relate to a loss or gain of one, a group, or all of the services of the ecosystems
- **contingent valuation methods:** this approach measures changes in wellbeing via the trade-off between money and changes in the quality or quantity of a resource, as revealed by the preferences of individuals (so-called willingness to pay or willingness to accept methods)
- **economic valuation methods:** provide techniques for estimating the economic value of changes in goods and services such as those associated with ecosystem services and potentially affected by flood or coastal erosion management schemes. These include market prices, revealed and stated preference methods, although, depending on the nature of the good in question, the extent to which these provide a full account of total economic value varies
- **value transfer (or ‘benefits transfer’):** this allows existing economic value evidence to be used to estimate the monetary value of environmental effects associated with flood or coastal erosion management schemes. Value transfer is used extensively and is a valuable tool in the overall appraisal process but it does depend on matching suitable existing valuation evidence to the context of each particular scheme. This can be a complex and time consuming process.

Internalising these intangibles into market values through such methods is not typically conducted on a project scale and reference values for the purpose of flood risk analyses are not readily available. Consequently intangible impacts are typically incorporated in flood risk analysis using MCA approaches (see Box 5.5).

5.2.9 Estimating the level of risk

As explained earlier (see Box 5.2) the level of risk depends on the chance of a flood event occurring and the potential undesirable consequences should the event occur. Depending on the level of assessment required or undertaken (see Section 5.2.1), this can be represented qualitatively or calculated. For example, a qualitative level of risk can be expressed by plotting the likelihood of failure and the relative consequences on a risk evaluation matrix as shown in Figure 5.4. This section focuses on quantitative methods.

There are various methods for quantifying risk. A typical approach (illustrated in Boxes 5.3 and 5.4), might involve the integration of:

- a full range of loading conditions (extreme water levels for fluvial/tidal defences, or extreme overtopping rates for coastal defences)
- the performance of defences in terms of overtopping and probability and nature of breaching (eg represented by fragility curves)
- a 2D flood inundation simulation method to determine water depths and their probabilities
- the calculation of economic consequences, for example using depth-damage relationships.

Example outputs from this type of method are depicted in Figures 5.15 and 5.16. Section 5.2.8.3 discusses an integration of economic damages, which can also be transposed to other types of consequences. Similar results can be calculated and displayed for other categories and measures of damages and impacts (such as likely loss of life), where sufficient data is available and where flood parameter/damage relationships are, or can be, established.

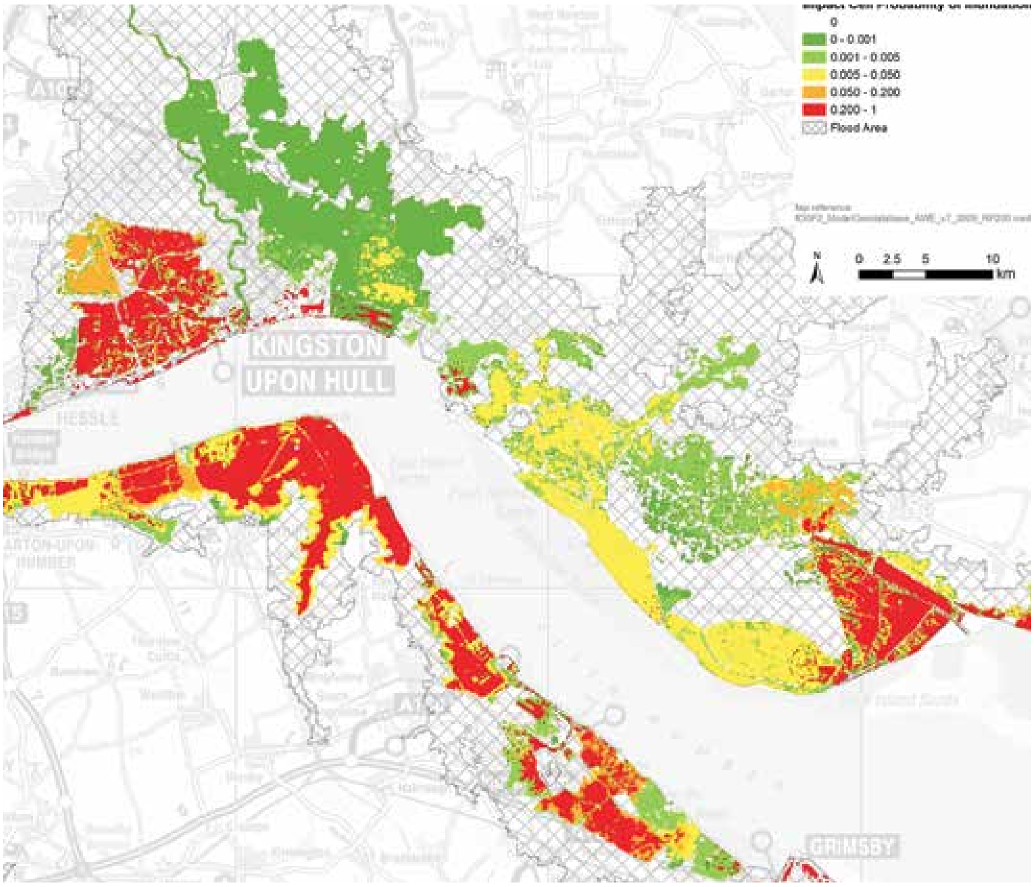


Figure 5.15 Example of a probability of inundation map in the UK (courtesy Environment Agency)

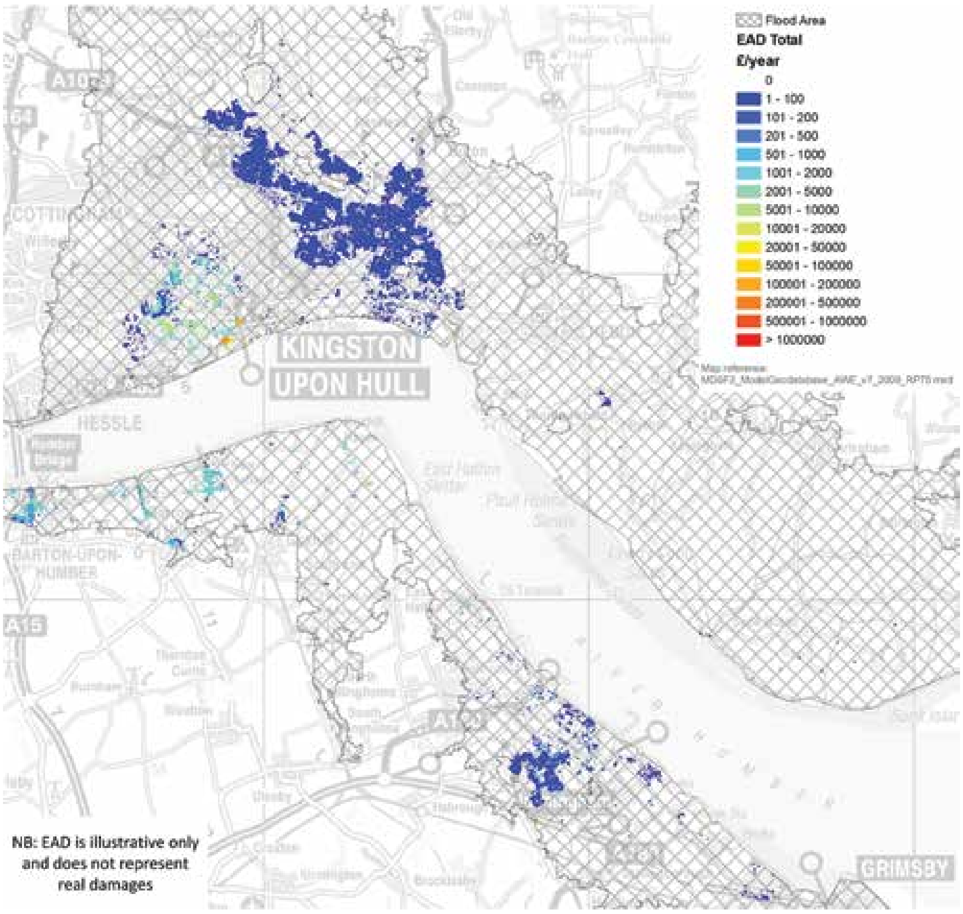


Figure 5.16 Example of an expected annual damage map in the UK (courtesy Environment Agency)

Levels of risk can be calculated for different risk scenarios as explained in Section 5.2.3. Such scenarios may include those in which potential flood risk management measures (increased crest heights of levees, creation of flood storage areas etc, see Section 2.2.1) have been introduced to reduce the level of risk as part of the flood risk management approaches discussed in Chapter 2.

Once a risk analysis has been completed, in order to be able to assess the results, an evaluation of the risk (ie its significance) should be conducted before considering measures and instruments to reduce flood risks. Risk evaluation is discussed in detail in Section 5.2.11.

5.2.10 Attributing flood risk to levee segments

Even though levee segments work together in a levee system, they are not all equally reliable and so do not contribute the same level of risk reduction to the whole. This is because:

- some levee segments may have lower or more variable crest levels than others, so may overtop more readily
- some levee segments may be weaker structurally than others, so may breach more readily
- some levee segments may have less efficient maintenance, monitoring, or emergency management.

The attributed flood risk associated with a particular levee segment is the residual risk arising from inundation of the leveed area (in terms of flooded area, water levels, time, flow velocities and depth etc) as a result of the probability of overtopping or breach of that particular levee segment. So, risk attribution is the process of quantifying the level of this residual risk associated with different levee segments.

Risk attribution can be undertaken for each inundation scenario (for each part of the levee length). One method of risk attribution is described in Box 5.8. Analyses are typically presented in a combined map and tabular format. Figure 5.17 shows an example of risk attribution mapping/tabulation for a part of the Humber shoreline, UK (note that Figure 5.17 is a fictitious example using trial data and future climate scenarios).

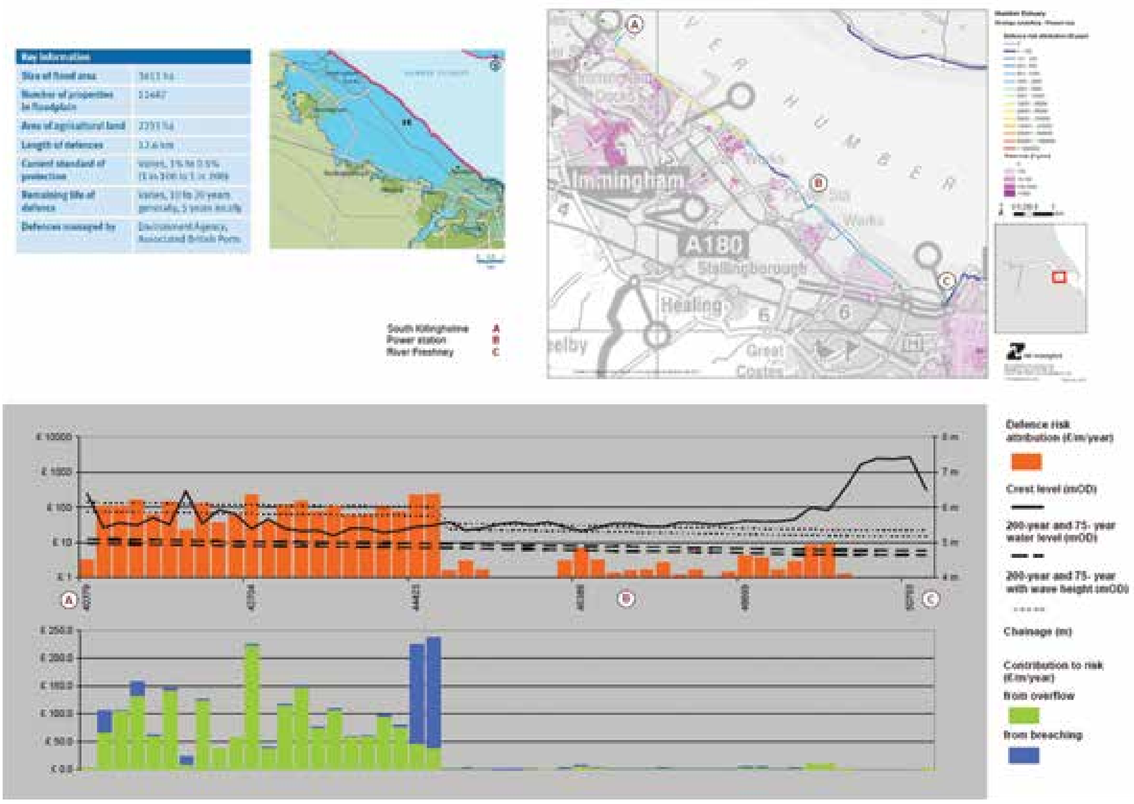


Figure 5.17 An example map and histogram showing risk attributed to defences on the Humber, UK (courtesy Environment Agency)

Box 5.8 Methodology used in UK for estimating defence contribution to risk (ie risk attribution)

A methodology for estimating defence contribution to residual risk is a component of the systems modelling approach described in Boxes 5.3 and 5.4 (Gouldby *et al*, 2008). The method involves establishing a relationship between the quantity of water discharged through each individual defence and the economic consequence of flood events. This relationship is formed through impact zones (an impact zone is a group of flood cells that form the basis of the RFSM – see Box 5.3). More specifically, a relationship is formed between the defences and adjacent impact zones (ie impact zones that are adjacent to defences into which floodwater is directly discharged) and then between 'adjacent' and 'non-adjacent' impact zones.

The defence vector comprises subsets of defence groups. The defence lengths in any defence group all discharge floodwater into the same adjacent impact zone (*IZi*), for example:

$$\left(\underbrace{d_1, d_2, d_3}_{IZ_i}\ \underbrace{d_4, d_5, d_6, d_7}_{IZ_{i+1}}\ \underbrace{d_8, d_9}_{IZ_{i+2}}\ \underbrace{d_{10}, d_{11}}_{IZ_{i+3}} \ldots d_n\right) \tag{5.8}$$

So, it is possible to analyse the volume of water discharged into each adjacent impact zone as:

$$Y = g_1\left(\underbrace{v_1, v_2, v_3}_{IZ_i}\ \underbrace{v_4, v_5, v_6, v_7}_{IZ_{i+1}}\ \underbrace{v_8, v_9}_{IZ_{i+2}}\ \underbrace{v_{10}, v_{11}}_{IZ_{i+3}} \ldots v_n\right) \tag{5.9}$$

On each modelled flood event (ie each realisation of the Monte Carlo sampling process), the proportion of flood volume contributed by each defence to each adjacent impact zone is calculated (see Equations 5.8 and 5.9).

Through a flood spreading model it is possible to associate the volume of water discharged into each adjacent impact zone with the depth and therefore economic consequences) in other non-adjacent impact zones as the floodwater is tracked as it propagates across the floodplain area. The economic consequence associated with each (non-adjacent) impact zone is then apportioned to each of the adjacent impact zones accordingly (ie the total economic damage for the flood area is expressed only in terms of the adjacent impact zones). The defence contribution (c_{d1}), for example for defence number d_1, to the economic damage is, for each flooding scenario, determined from analysing the proportion of volume that is contributed from each individual defence section.

Also of importance is the relative quantity of defence residual risk associated with breaching events (failed defences) and overtopping events (not failed defences) respectively. As the volume discharged into the floodplain is a function of the defence system state, the state of the defence (failed/not failed) is also monitored. So the individual defence contribution to risk can be disaggregated into breaching and overtopping contributions.

5.2.11 Risk evaluation

Since risk cannot be entirely eliminated, the tolerability of the level of residual risk determined by the flood risk analysis should be evaluated using societal, regulatory, legal, owner and other values. However, it is not normally the responsibility of levee owners or operators to formulate risk tolerability standards. Instead the broader society dictates to the technological community the tolerable levels of risk that should be met by levee and flood risk systems – as it does for many other risks to society.

Tolerable risk is defined by HSE (1995) as that risk "which for the purposes of life or work, everyone who might be impacted is prepared to accept assuming no changes in risk control mechanisms".

Criteria on which the tolerability of risk can be evaluated fall into three groups (Morgan and Henrion, 1990):

1 **Equity-based criteria** are founded on the premise that all individuals have unconditional rights to certain levels of protection. This often converts to fixing a limit to represent the maximum level of risk above which no individual can be exposed. If the risk estimate from the risk assessment is above the limit and further control measures cannot be introduced to reduce the risk, then the risk is held to be unacceptable, whatever the benefits (HSE, 2001)

2 **Utility-based criteria** compare the incremental benefits of the measure to prevent the risk of injury or detriment, and the cost of the measure (HSE, 2001). The balance between benefits and cost, both expressed in monetary terms, can be deliberately skewed towards benefits by ensuring that there is gross disproportion between costs and benefits. Examples of such criteria can include deterministic and probabilistic cost-benefit, cost effectiveness (including cost per statistical life saved), bounded cost, maximising a multi-attribute utility function and others (HSE, 2001)

3 **Technology-based criteria** are founded on the idea that a satisfactory level of risk prevention is attained when state-of-the-art control measures (including technological, managerial and organisational) are employed to control risks whatever the circumstances (HSE, 2001).

Most of the countries that have contributed to this handbook use some form of utility criteria, in particular:

- **cost-effectiveness analysis (CEA),** which seeks to identify the least-cost option that satisfies some performance requirement – primarily risk tolerability, although there are often other constraints in terms of environmental standards and socio-political acceptability of the proposed measures. In practice there can be a range of risk thresholds derived from government guidance, insurance availability and social tolerance levels (see Box 5.10)
- **cost-benefit analysis (CBA)** is a method that expresses as many costs and benefits of the options as possible in terms of the monetary or other value placed on them by society, deriving the net benefit as discussed in Section 5.2.8.3, and then assesses whether the expected benefits of a specific risk-reducing option outweigh its expected costs. The major shortcoming of this approach concerns the fact that all benefits and costs are quantified in monetary terms and aggregated to a single number, while some impacts such as environmental effects, which are more difficult to quantify, are not considered.

Inevitably, hybrid evaluation criteria for tolerability of risk can emerge and are often apparent. The HSE TOR framework (HSE, 2001) uses an equity-based criterion for risks in the unacceptable region and a utility-based criterion for risk in the other two regions (see Figure 5.18). Technology-based criteria may be used to complement the other criteria in all three regions (ICOLD, 2005).

In flood risk management, different countries use a combination of these techniques. For example:

- **the Netherlands:** a national economic optimisation model is used to determine the level of tolerability as shown in Box 5.9. However, the resulting criteria are enshrined in Dutch law (TAW, 1990), effectively as an equity criterion for all citizens
- **the UK and USA:** relative tolerability of residual risk is judged on the basis of project-by-project cost–benefit comparisons, although loss of life considerations linked to equity-based criteria come into play in regard to mitigating the more extreme risks. There are also indicative equity thresholds that influence matters such as land-use management and insurance (see Box 5.10).

Although the general framework in Figure 5.18 gives a first impression on how risk acceptance can be approached, it should be stated from a social science point of view that the realms of tolerability and non-tolerability may vary over time and differ significantly between individuals. Also, a public consensus on risk tolerability may not exist. Defining what constitutes unacceptable harm to people and the environment is a difficult task and ultimately depends on what relative values society places on loss of life and damage to buildings, infrastructure, and ecosystems.

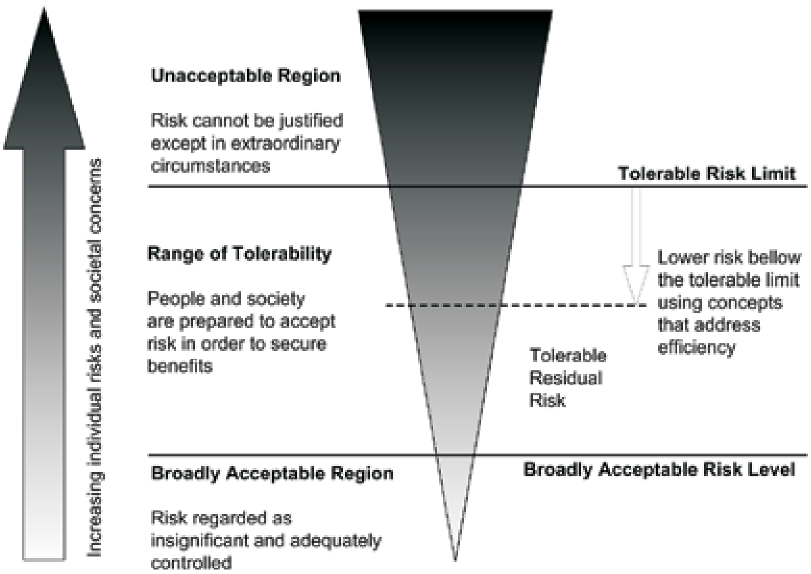


Figure 5.18 Acceptability of risk (adapted from HSE (2001) by Munger et al (2009))

Box 5.9 ***Example from the Netherlands of the economic optimal safety model***

In the 1960s the Delta Committee developed a semi-quantitative approach for determining the required protection level of the many highly urbanised flood-prone areas of central Holland (Delta Committee, 1961). This approach analyses the overhead costs for construction or improvements on existing levees and compares these costs to the levels of protection and safeguard from damages that the improvements would provide. Damage of flooding was calculated as direct and indirect economic damages and then multiplied by two to attempt to quantify fatalities and non-monetary damages.

Figure 5.19 adopted by the first Delta Committee, shows the basic concepts of cost–benefit analysis as it applies to levee improvements. An obvious trend is the direct relationship between cost increase and levee heightening. As the amount of heightening increases there is also an exponential decrease in damages due to flooding. Economic optimum of levee heightening is where the sum of the investment in levees and damages caused by flood inundation, is at a minimum.

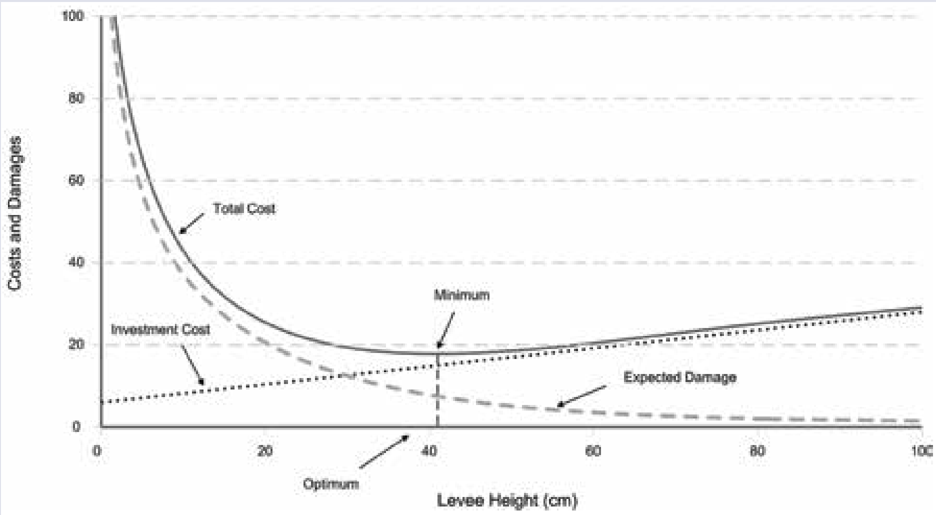


Figure 5.19 ***Graph indicating the economic optimum safety standard (Delta Commission, 1961)***

Box 5.10 ***UK flood probability thresholds for development and insurance in floodplains***

In the UK, guidance for development in floodplains introduces event probability thresholds at one per cent for river flooding and 0.5 per cent for coastal flooding. Properties are currently assessed and insured against flood probability in the following categories:

1 **Significant:** the probability of flooding in any year is greater than 1.3 per cent (1 in 75).
2 **Moderate:** the probability of flooding in any year is 1.3 per cent (1 in 75) or less, but greater than 0.5 per cent (1 in 200).
3 **Low:** the probability of flooding in any year is 0.5 per cent (1 in 200) or less.

A relationship between insurance availability and flood probability also exists in the USA, but in this case federal flood insurance comes into play if the probability of flooding (allowing for the presence of levees) is greater than one per cent (1 in 100).

5.3 LEVEE PERFORMANCE ASSESSMENT AND DIAGNOSIS METHODOLOGY

5.3.1 Introduction to levee performance assessment and related principles

With the exception of levees that retain water either permanently (eg along canals) or periodically (eg along tidal seas), most levees are rarely loaded, so it is important, (but difficult) to be able to foresee the performance of the levee during the future loading event(s). The levee performance assessment process is the use of one or more methods of treating and combining data in order to obtain an evaluation of the performance of the levee system, according to its main function (protect against flood) or its reliability (possible failure modes). This can be done in different ways. Assessment methods categories are presented in Section 5.3.3. The generic assessment framework developed in the European project FloodProBE is presented in Figure 5.20.

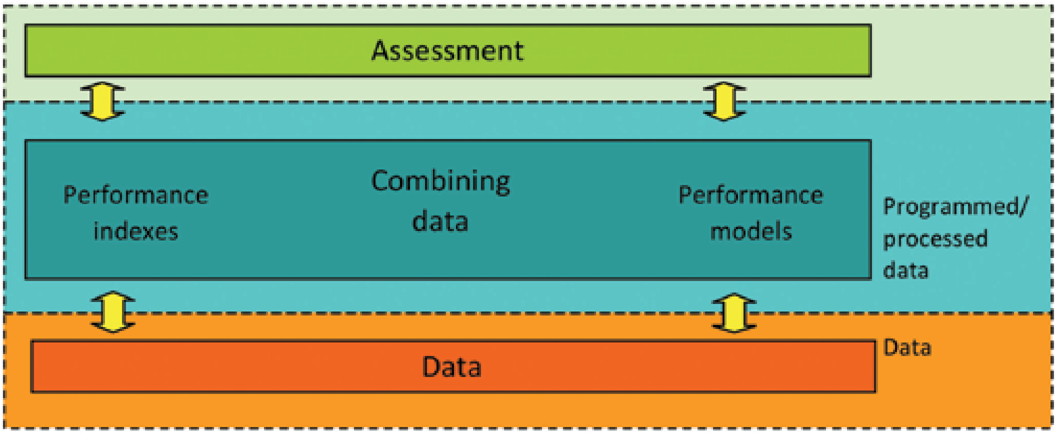


Figure 5.20 A framework representing the levee performance assessment process (from Van der Meij et al, 2013)

Good understanding of the levee, its behaviour and its (relative) vulnerability for different failure mechanisms during flood conditions will assist in:

- directing inspections on what specific features to look for (and where)
- the diagnosis of observed features (causes), ie the identification of the cause or nature of a defect or element of deterioration of a levee (this relates to failure modes and processes)
- the prognosis of the performance of the levee during potential future flood events, the need to take emergency measures (and which measures).

A complete assessment should include a diagnosis of the actual or possible causes of failure, in order to remediate or prevent them. This means that all potential failure modes for the levee and their relative importance to the overall performance have to be determined.

The outcome of a levee performance assessment will be a prediction of how the global levee system and the individual levee segments will perform under a range of loading events. Once assessments have been completed, follow-up actions will be required (see Sections 2.2 and 5.1) and these may well include maintenance (see Chapter 4) and designed rehabilitation (see Chapter 9).

Levee performance assessment methods – types and results

There are different levee performance assessment methods, all based on a combination of data, using expert judgment, index based methods, mathematical models – physical and/or empirical models. Levee performance assessment methods are presented in Section 5.3.2.

There are several different possible results of a levee performance assessment:

- threshold (a limit load)
- conditional chance of failure (for a given load)
- fragility curve (conditional chance of failure given for a range of loads)
- safety factor
- index (eg on a 0–5 or 0–10 scale)
- qualitative (eg very good, good, fair, poor, very poor).

The form of the result depends largely on the used method, but also on the way it will be used thereafter. It is possible to build equivalences between the different types of results.

Uncertainties, incompleteness, imperfections can be integrated into the assessment process, in order to produce an assessment result in a probabilistic form, or in other forms qualifying its uncertainties.

5.3.2 Diagnosis and performance assessment in the levee management cycle

The detailed objectives and degree of detail of a levee performance assessment can vary, according to the role of the organisation responsible for it and the stage in the levee life cycle. The two main types of roles for organisations needing regular levee performance assessments are the levee management organisation and the regulatory authorities. Organisations in direct interaction with these also generally need to have access to the results, eg for funding purposes, but do not necessarily have to commission independent assessments. Usually, levee management organisations will need to have both assessments and diagnosis results, while regulatory authorities and other organisations may only be interested in assessment results.

As for risk analyses, a tiered approach (see Sections 2.1.3.3 and 2.3.3.1) can be adopted to optimise the resources to the risk level. All along the levee life cycle, various assessments (including inspections with conclusive reports as well as complete risk analyses) should be conducted, with different levels of expertise and detail. The result of any of these is both an assessment score (in whichever form) but also a measure of the reliability of the result. This reliability measurement can be a clearly expressed result of the assessment, depending on the method used to produce it, but it can also be implicit, and function on the type of assessment and its level of expertise and detail.

As in the higher levels of assessment some specific data gathering (investigations) has to be commissioned. This is another point to analyse in terms of a tiered approach, to design the right level of investigations.

In order to fulfil its duties (related to management policy and/or regulations), a levee management organisation has to perform regular assessment related operations (eg inspections), remembering that these do not have the same level of accuracy as the less frequent detailed assessments (eg diagnosis and risk analyses). That is why, very often, operation and maintenance (O&M) instructions and/or regulations include various levels of inspections and assessments. Figure 5.21 presents a typical example of a program of regular inspections and assessments for levee managers and their respective levels of detail and frequencies.

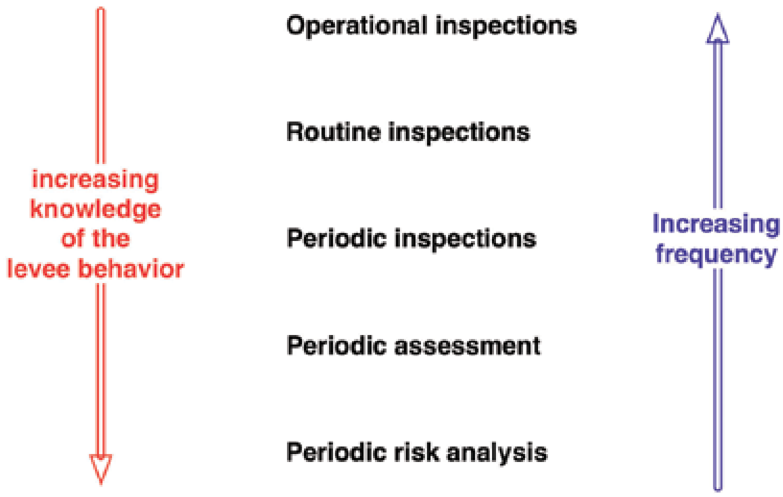


Figure 5.21 Regular assessments, levels of detail and frequencies (courtesy M Wallis and C Neutz)

Risk analyses depend on levee performance assessments. Risk analyses combine levee performance assessment with the analysis of the consequence of a levee failure (see Section 5.2.3). It is possible to update a risk analysis after a levee performance assessment update, without updating the consequences analysis itself.

In the levee life cycle, different assessments follow one another. Every assessment should take into account the data and results of the previous ones. Figure 5.22 presents an example of integration of the various types of assessments in a general levee management life cycle, with an optional part (on the left) for old levee systems having to be rehabilitated before going into a more regular life cycle.

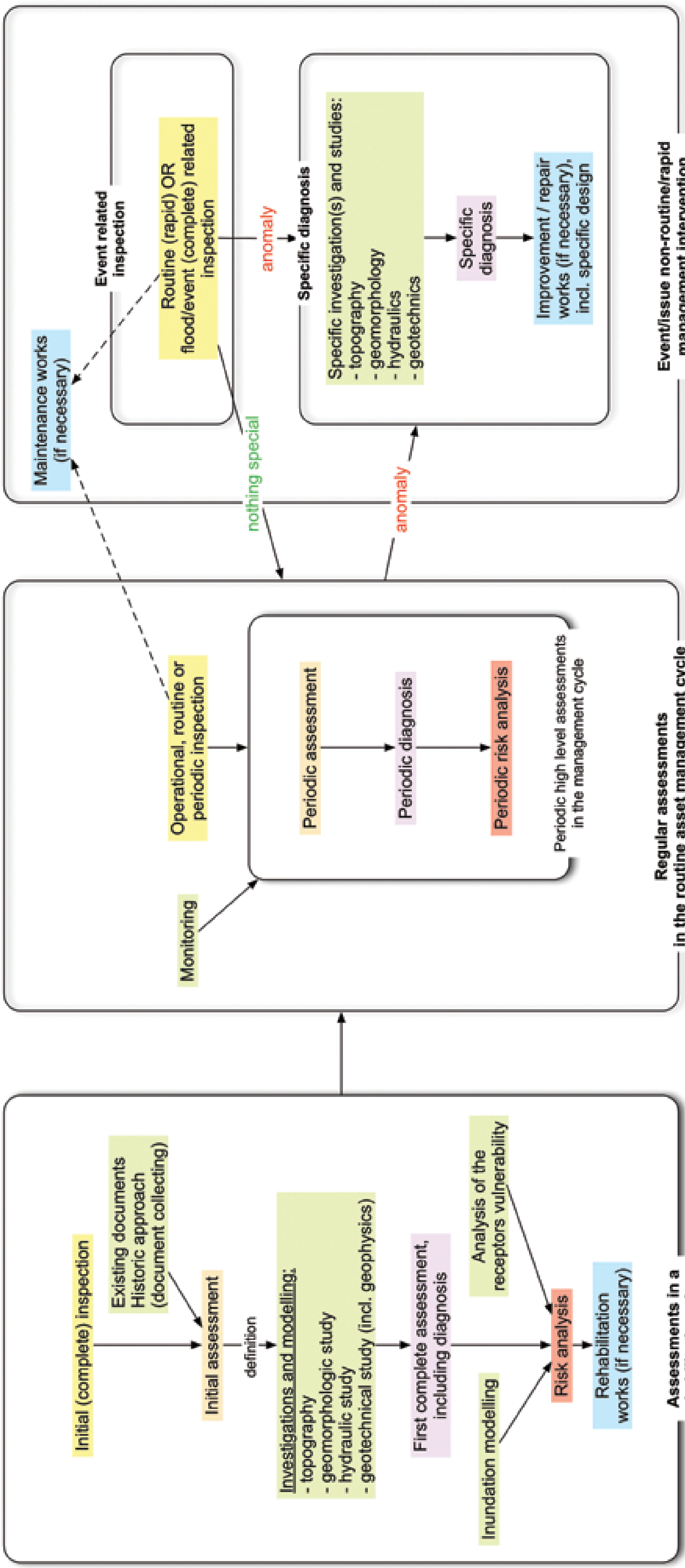


Figure 5.22 Assessments in the levee life cycle and the way they use previous assessments results, as well as specific data (courtesy R Tourment)

1 2 3 4 5 6 7 8 9 10

The level of detail of both the data and of the method for combining them used in a levee performance assessment depends on both where this assessment is in the levee life cycle and previous results. See Chapter 2 and Section 5.2 about tiered, gradual and risk based approaches on the subject of adapting the resources to the risk or other input data.

Diagnosis itself is difficult, as observed features do need to be linked to possible failure mechanisms and this may well require the support of a qualified and experienced engineer (see also Sections 5.4 and Chapter 9). Section 5.3.3 presents the general links between data used in levee diagnosis and performance assessment with the various failure mechanisms. Sections 5.4 and 5.5 illustrate these links more specifically.

5.3.3 Assessment methods

For each or all potential failure mechanisms in a levee, the assessment process must provide an estimation of the potential for failure under one or more different loading events. In an initial desk study, the potential for failure of the levee should be evaluated for all levee segments of the leveed area. In a subsequent more detailed levee performance assessment, it is likely that effort will initially be focused on those segments of most concern. Less detailed but faster levee assessments can be made using direct analysis of inspection data.

In a complete levee performance assessment, the failure modes of the levee system components should be identified for each functionally and structurally homogenous part of the levee, as part of a diagnosis. This identification also includes different theoretical levee system failure scenarios. The scenarios chosen are then evaluated in terms of probability/likelihood according to the performance of the levee systems component functions and of the chosen loading event/hazard(s). For instance, the loading event to be associated with a river flood should be issued directly from its hydrological characteristics.

5.3.3.1 Data and failure modes

It is necessary to produce a list of the possible failure modes related to a given levee, which relates to its longitudinal homogeneity and cross-section. This can be rather simple (if levees are quite similar and well known and a few failure modes have to be checked) or complex (as in France, where there are many heterogeneous levees with many different types of cross-sections).

So, knowledge related to the failure modes and the associated elementary mechanisms is essential. It is important to be able to identify laws, physical or empirical, governing the mechanisms, and design methods to analyse failure modes in terms of scenarios. Presently, some mechanisms are well characterised in terms of physically based models (eg sliding) or empirically based ones (eg regressive internal erosion), but for many others the quantitative characterisation still has to be developed (eg for internal contact erosion).

Expert judgement based on individual and/or collective experience will allow failure modes and mechanisms to be judged, even where formal evaluation is difficult. Whatever the method used to combine data in an assessment process, it is beneficial to list all data type associated with any failure mode and/or mechanism, Sections 5.4 and 5.5 present tables that link data and failure modes.

Data can be categorised by its nature, which helps to clarify its relation to the relevant failure mechanisms of the levee. The different data categories might include:

- topographic data
- geotechnical data (including geophysics)
- hydraulic data (including hydrology)
- morphodynamic related data
- levee environment data (including stakes in the leveed area)
- structures data (including encroachments).

The same data can be classified in various categories.

- geology
- granulometry
- classification test results
- piezometric levels
- drains discharge
- permeability
- shear test results
- compressibility
- density
- penetrometer
- CPT
- erodibilty (HET, JET etc).

For each data category, a list of data types can be built. An example list for geotechnical data might include:

It is also important to consider the different sources, as well as the type and nature of data. Usual sources of data are:

- information system of the levee manager
- historical information (not already available in the levee manager information system)
- topographic survey (eg classical, LiDAR) (see Chapter 7)
- visual inspection (see Section 5.4 and Chapter 4)
- specific investigations (see Section 5.5 and Chapter 7)
- monitoring (see Section 5.5 and Chapter 7)
- previous assessments results (and initial data)
- outside sources (maps, databases, reports).

5.3.3.2 Loading and levee performance assessments

In an assessment result or in the process, the loading conditions should be defined. Different types of results from levee performance assessment can include:

- a single performance assessment result for one loading condition
- a single assessment result for all loading conditions (for example, annual chance of failure)
- an assessment result for each load in a list of loading conditions
- a fragility curve, which is a curve (or function) linking the assessment to a range of loading conditions.

Quantification of the probability of loading events is discussed in Section 5.2.5. The quantification of the impact of the loading events on the levee system in terms of a probability of failure is the object of the performance assessment. This probability will then be used as an input to the risk analysis, as detailed in Section 5.2.6.

System response functions, such as levee fragility curves, allow an estimation of the relationship between flood stage at a given levee location and the probability that levees may fail and are defined at various points throughout the system. System response functions are often important components of the hydraulic analyses used to assess the water resource system's response under various loading conditions (ie floodwater surface elevations).

Box 5.11 The different loading conditions to be considered during an assessment in France (courtesy Irstea)

Irstea, in France, propose the use of three different loading conditions to assess the performance of a levee or levee system:

1. **Protection level:** the loading condition below which there is no flooding of the leveed area.
2. **Safety level:** the loading condition up to which there will be no major damage to the levee system (a flooding, in controlled conditions can occur between the protection level and the safety level).
3. **Danger level:** the loading condition above which the risk of breach in the levee system is probable.

These loading conditions can be objectives, in the case of a project or where the objectives have been previously specified by the authority responsible for the levee. They also can be findings of the assessment or risk analysis process.

These levels can be expressed either in probability of the loading event and/or in terms of altimetry. Equivalence between the altimetric levels and the probability of the loading event is a complicated matter as different events (or combination of events) can lead to the same altimetric level.

5.3.3.3 Levee performance assessment process

A generic performance assessment process for a levee segment may include some or all of the following steps:

- identifying the possible failure modes and mechanisms for the specific levee system
- evaluating the possible methods to analyse performance of the levee relative to each of these failure modes and/or mechanisms
- finding all available data useful for the assessment
- building a specific investigation program to complement existing data
- combining data for each loading condition and every failure mode
- combining results for all failure modes and eventually all loading conditions.

This may be followed by:

- a combination of conclusions for all levee segments to the entire leveed area
- making conclusions in terms of proposals for subsequent actions.

Examples of implementing some of these steps are given in Boxes 5.12 to 5.14.

Many data combining methods can be used in a levee performance assessment. The typical data combining methods are:

- models (see Chapter 8 for all applicable models), using both physical or empirical based equations (see Box 5.12)
- index-based methods, using predefined combination of index rating different observations or parameters (see Box 5.14)
- expert judgement, direct or using one or more of the previous methods as pre-processed data.

In the same assessment a combination of these different types of methods can be applied. Ultimately, it is to be expected that, given the variability of materials and parameters in existing levees, some level of expert judgement will be added to the conclusions of any assessment report. Expert judgement allows to take into account any data, even not used as input into any model, when it is relevant to a given failure mode.

Functional analyses and failure modes analyses can be used to help build index-based methods (see Box 5.14).

Box 5.12 Example of levee performance assessment in Orléans (river Loire, France, 2011–2012)

Assessment method	Application of mathematical models on fixed lengths of levee, based on physical or empirical laws (according to the different failure mode), and adjusted according to other data. Evaluation of failure probability for different hydraulic loads, taking into account levee properties, condition, for scenarios composed of one or more mechanisms, including a distinction between probability of appearance and probability of evolving into a breach. Use of both mathematical (physical and empirical based) models and expert judgment based formalised rules.
What is combined?	Geometric data, geotechnical data, hydraulic loads, results from visual inspection (eg animal burrows, trees) and other databases (eg pipe encroachments) as well as organisation (levee monitoring during floods).
Failure mode(s) and/ or mechanisms	Overflowing, internal erosion, slope sliding followed by another mechanism, external erosion followed by another mechanism, hydraulic uplift followed by regressive internal erosion.

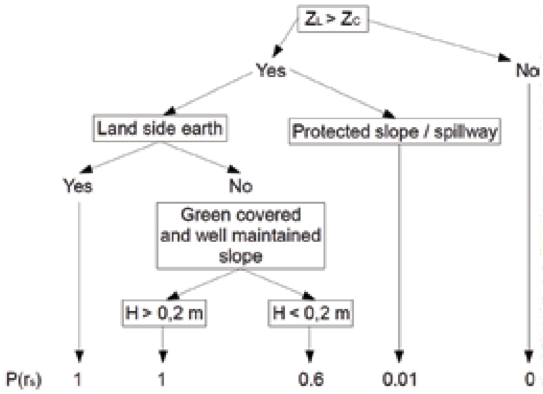


Figure 5.23

Probability assessment for overflowing caused breach, once overtopping has occurred

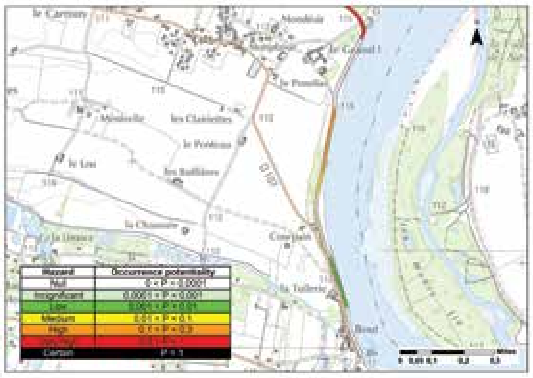


Figure 5.24

Final assessment result

Explanation	Five main failure modes have been identified and kept for analysis for this assessment. These failure modes are either composed of one single mechanism (overflowing, internal erosion) or two or three mechanisms (slope sliding followed by internal erosion, external erosion followed by internal erosion or by collapsing and then internal erosion, uplift followed by regressive internal erosion). Each of these failure modes (or breaching scenarios) has been evaluated on fixed lengths (50 m) of the levee, for different floods (Q50, Q100, Q170, Q200, Q500). First the probability P(A) of the first mechanism was determined. For each probability flood grade P(QT) of interest the probability P(r) that this event will cause the levee to breach, (either by the continuation of the first mechanism up to the breach or through the involvement of other mechanisms, was determined. The result calculates the breach probability with a specific failure mode: P(R) = P(A), P(r), P(QT) Different models have been applied for the each of these terms on each of the sections for each flood. These models are different for each mechanism and failure mode, taking account either physical models (slope sliding etc), empirical models (Sellmeijer internal erosion for initiation of this mechanism etc) (see Section 8.5.1.2), as well as formalised expert rules (resistance to overflowing erosion see Figure 5.23, taking into account visual inspection for countering internal erosion etc). Data comes from both the levee manager GIS data management system (SIRS Digues), a specific visual inspection, and a search through historic information (reports) related to geotechnics and a DEM (created by a LiDAR Survey, see Royet *et al* (2013). The model was developed in a spreadsheet. A table for equivalence between probabilities and their qualification in terms of common language (ie possible yet extremely unlikely event, very unlikely event, unlikely event, likely event, usual event) has been developed, helping to either produce probabilities starting from expert judgement based rules, or to express in words the probabilistic results.

Box 5.12 ***Example of levee performance assessment in Orléans (river Loire, France, 2011–2012) (contd)***

Explanation	Once the breach probability has been assessed for each failure mode and flood grade, these results will be used as such by the work manager or in combination for producing the following global probabilities for: • each flood grade (all failure modes) • each failure mode (all flood grades) • all failure modes and all flood grades. Suitable probability-based methods for producing such combinations are still to be determined. Whatever the expected type of results, these can be graphically expressed, using a GIS, on a map displaying the levee breach hazard along the considered length of the levee (Figure 5.24). They can also be stored in a GIS data management system.

Box 5.13 presents the integrated use of fragility curves in assessments, as well as a method and a way to present the results

Box 5.13 ***Example of levee performance assessment on the River Thames, UK (2006)***

Example name	Assessment of Dartford Creek to Gravesend levees, conducted in 2006
Location/pilot	Thames Estuary, London, UK
Analysis method	Single cross-section reliability method
What is combined?	Geometric data, geotechnical data, hydraulic loads, topographical data, historical data, measurements, visual inspection, economic receptors
Failure mode(s)?	Overflowing, piping, slope instability, uplift, fissuring/cracking, settlement/crest lowering, fluvial bathymetric changes, third party activities/damage

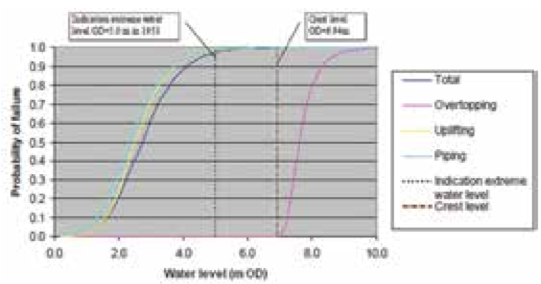


Figure 5.25 ***Fragility curve for a section of the Gravesend levee. The failure mechanisms driven by a combination of uplift and piping, which dominates the total fragility curve***

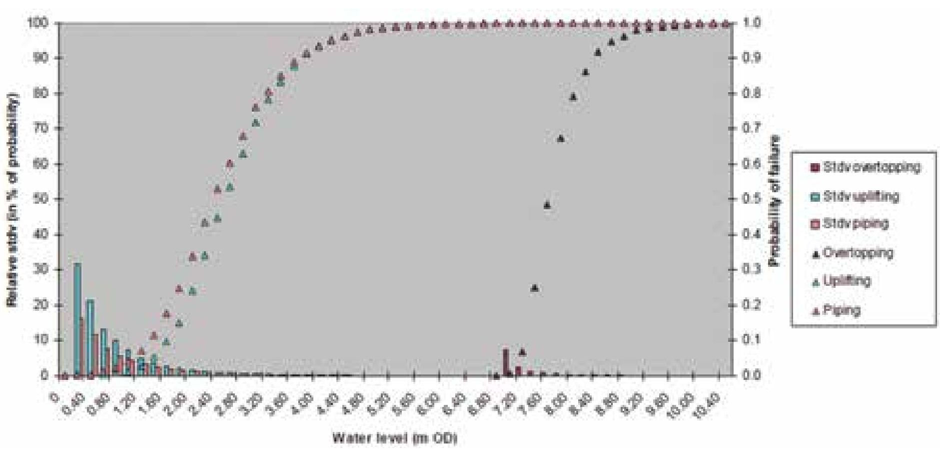


Figure 5.26 ***Plot of the relative standard deviation of the fragility for overtopping, uplifting and piping***

1 2 3 4 5 6 7 8 9 10

Box 5.13 Example of levee performance assessment on the River Thames, UK (2006) (contd)

Explanation	The 'fragility' provides insight in the likely behaviour of the flood defence given different source conditions. A steep curve signifies more certainty about the conditions under which the flood defence will fail. A shallow curve relates to a larger range of uncertainty about the conditions under which the flood defence will fail. In addition it shows the most prevalent failure mechanism under different circumstances. Its practical significance is closely tied to the type of application: ● design from scratch requires consideration of all failure mechanisms for a range of relevant design standards ● maintenance monitors the flood defence to check whether the fragility is within an acceptable envelope ● improvement and repair requires insight of which part of the fragility does not meet the acceptable envelope and which failure mechanisms are causing the problems. The effect on fragility of different improvement options can subsequently be compared within a cost–benefit framework ● evacuation requires information about the likelihood of a flood given a storm situation. The 'total probability of failure' allows comparison of the reliability of flood defence sections among different locations in contrast to the fragility. Fragility does not incorporate the likelihood of the local hydraulic boundary conditions in the probability of failure. Two flood defence sections can have the same fragility but suffer from different hydraulic loading and so will have a different annual probability of failure. Figure 5.25 shows the fragility curve for the section analysed. The failure mechanism is driven by a combination between uplifting and piping, which dominates the total fragility curve. The probability of failure is plotted against the water level, rather than against other hydraulic boundary conditions. Wave conditions occur in the wave overtopping discharge, which is only considered for water levels below the crest level. It is evident from Figure 5.25 that the probability of failure due to overtopping for water levels below the crest level is negligible. As a point of reference, an indication of the highest recorded water level, which is believed to correspond with the 1953 Thames Estuary flood, is given in the figure. According to these results, during a big storm it is very likely that there will be problems with uplifting and piping at this location. Figure 5.26 presents the standard deviation of the fragility for overtopping, uplifting and piping given the choice of 10 000 Monte Carlo simulations for each water level step. Below a water level of OD+1.2 m the standard deviation of the fragility is between 5 and 30 per cent of the fragility. To improve the results, the number of simulations could be increased for these water levels. To make the calculation of fragility more efficient, the number of Monte Carlo simulations can be varied for different water level intervals. The choice for the number of simulations per interval depends on the required level of accuracy.

Box 5.14 presents an example of index based assessment method, relying on a functional analysis and failure mode analysis of the levee components.

Box 5.14 Example of levee performance assessment relying on functional analysis of levee components in France (2011–2012)

Analysis method	Combination of indexes (status indicators, function criterions, performance indicators) with the help of rules based on formalised expert knowledge. Combining different types and nature of data (geometric, geotechnic, hydraulic, morphodynamic, visual observations etc) within a specified methodology
What is combined?	Information from visual inspection, geometry, geotechnology, morphology, database (encroachments structures etc)
Failure mode(s) and mechanisms	The example details the method for internal erosion, but the method also exists for overflowing, external erosion and sliding, and the generic principles for the method are presented.

Box 5.14 *Example of levee performance assessment relying on functional analysis of levee components in France (2011–2012) (contd)*

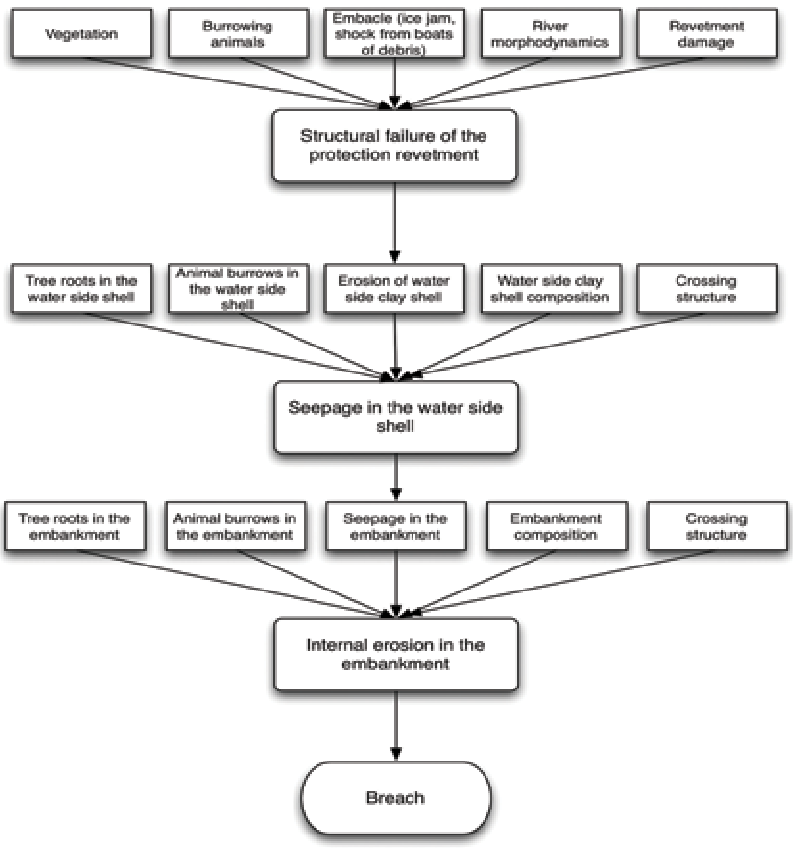


Figure 5.27 *Example fault tree for internal erosion on a levee with waterside impervious shell, including relevant status indicators (courtesy D Serre and L Peyras)*

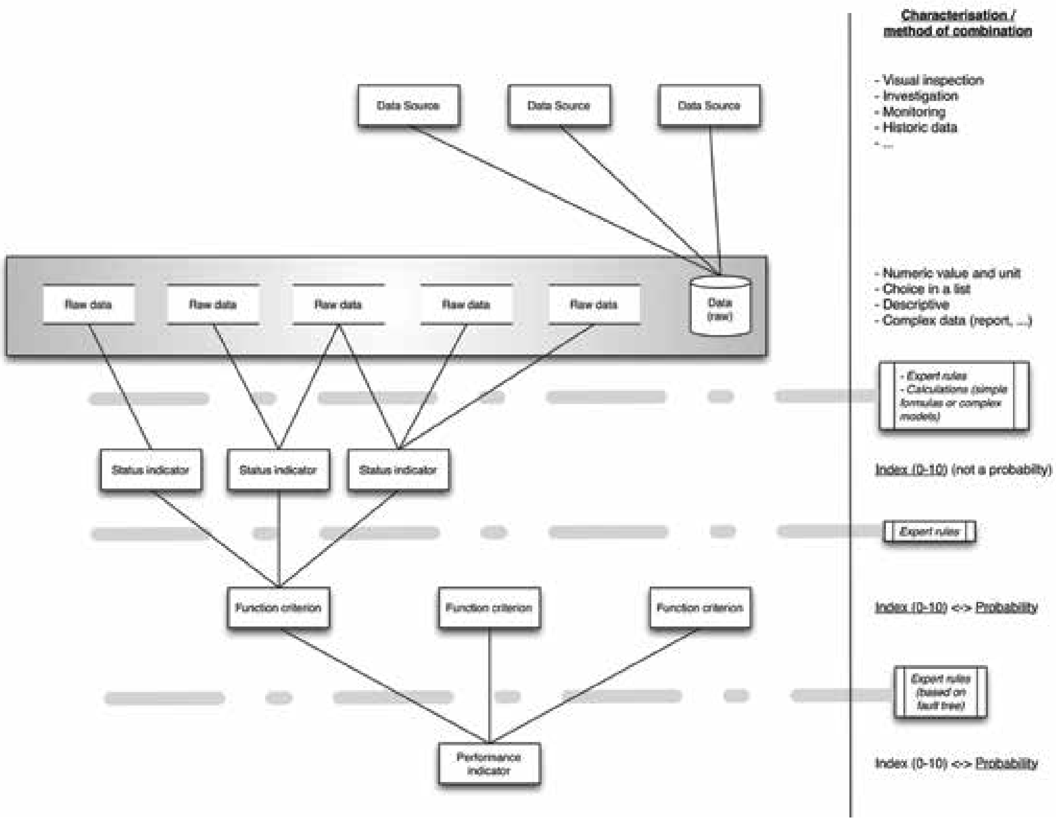


Figure 5.28 *General assessment process using the index based method from this example (courtesy R Tourment)*

Box 5.14 *Example of levee performance assessment relying on functional analysis of levee components in France (2011–2012) (contd)*

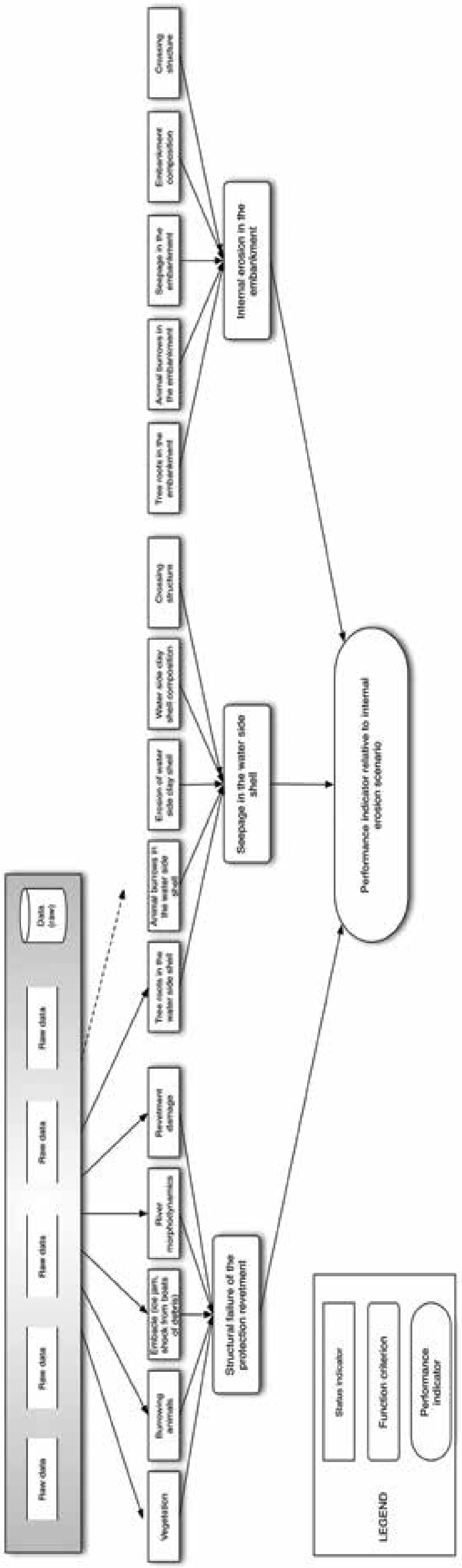


Figure 5.29 Assessment process for internal erosion on a levee with waterside impervious shell (courtesy R Tourment)

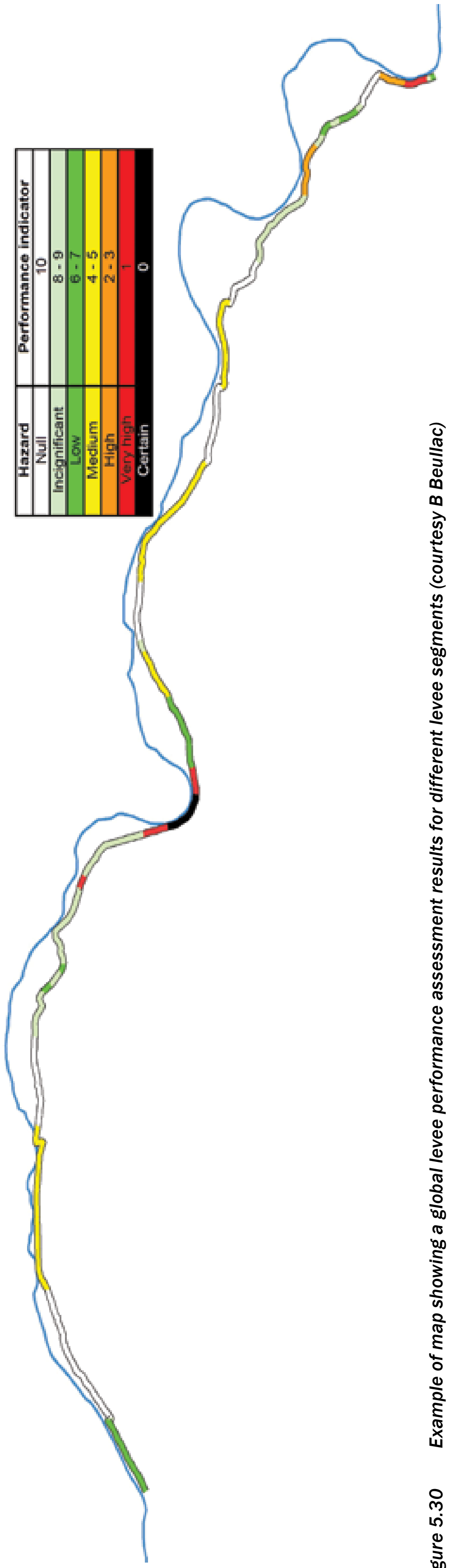


Figure 5.30 Example of map showing a global levee performance assessment results for different levee segments (courtesy B Beullac)

Box 5.14 ***Example of levee performance assessment relying on functional analysis of levee components in France (2011–2012) (contd)***

Explanation	Based on a functional analysis of levee components, in each cross-section, failure scenarios are identified and analysed, linking essential functions (such as protection, imperviousness, stability, filtration) and components. Figure 5.27 presents such a failure scenario. The data are first used to establish the different status indicators. Values of these indicators result either directly from raw data, from pre-processed data, or from combined data. They are relative to one function, and a single component associated with the function. For example the nature (or state) of the protection revetment of the riverside slope. Function criterions calculation are then made, based on the combination of status indicators values. They are relative to one function in relation to the studied failure mode. For example resistance of the revetment to external erosion. Finally, as shown in Figure 5.28 (general) and Figure 5.29 (application to internal erosion), the performance indicators evaluation is based on a combination of functional criteria. They are relative to one failure/breach scenario (or 'failure mode') for a given cross-section. For example performance of the levee segment relative to the 'internal erosion scenario'. The final assessment takes into account all failure modes. Expert judgement is used, in a formalised way to produce the aggregation rules for status indicators to function criteria, and from function criterions to performance indicators. A GIS is used to manage all available data related to the levee to: • make available each data relevant to all status indicators • pre-process the data to produce the status indicators for those who have established expert rules • pre-process the combination of the status indicators then function criteria to produce the function criteria then the performance indicators • represent the assessed performance both for global performance indicator and individual failure mode performance indicators. Figure 5.30 presents the resulting map.

5.3.3.4 Using fault and event trees to examine levee failure scenarios

Deterioration mechanisms can combine in different ways to produce a structural failure of the levee (Section 3.5). Scenarios of events leading to a failure can be rather complex. In order to analyse the failure or the potential failure of a particular section of levee, it is then necessary to produce a detailed analysis of the function and the form of the levee.

In a systemic approach, two main techniques exist to map the paths from deterioration to breach, which are fault tree analysis (FTA) and event tree analysis (ETA). These approaches can be combined in a 'Bowtie' tree scheme. For better results it is recommended to base the application of these methods on the failure modes and effect analysis (FMEA) of the studied system. Based on a functional analysis of the studied system, such an analysis allows to define and identify causes and consequences of failure, to identify links between failures of components of the studied system, and then to build scenarios or chains of events.

A synthesis of these main techniques to perform failure analyses and some simplistic examples are presented in the following subsections. More detailed information about these concepts can be found in ICOLD (2005) and Baroth *et al* (2011).

Fault tree analysis

Fault tree analysis is an expert and deductive method to identify every combination of causes (failure scenarios) that can explain the occurrence of a final event (failure of the system).

Fault trees analysis starts from the final event or state of failure (breach) and conducts a back analysis first to find the originating causes of the breach, then the causes of those originating causes and so on. The aim of this analysis, conducted step-by-step, is to identify all the conditions, factors and mechanisms that have allowed the final event or failure to occur. A fault tree analysis ends with the identification of the original causes, which are defined as causes external to the studied system.

The analysis and the modelling of these fault trees are usually built from the top to the bottom or from the right to the left, with the final failure event being presented at the top or on the right side, and the original causes at the bottom or at the left side. Each level of the analysis corresponds to a failure of a specific function of a component that can result from numerous causes. These causes may be combined (through AND gates) or may be independent (through OR gates).

In the case of levees, a breach situation can be studied to infer mechanisms and external events (initiators and contributing factors) that have generated the failure. An example of simple fault tree analysis is presented in Figure 5.31.

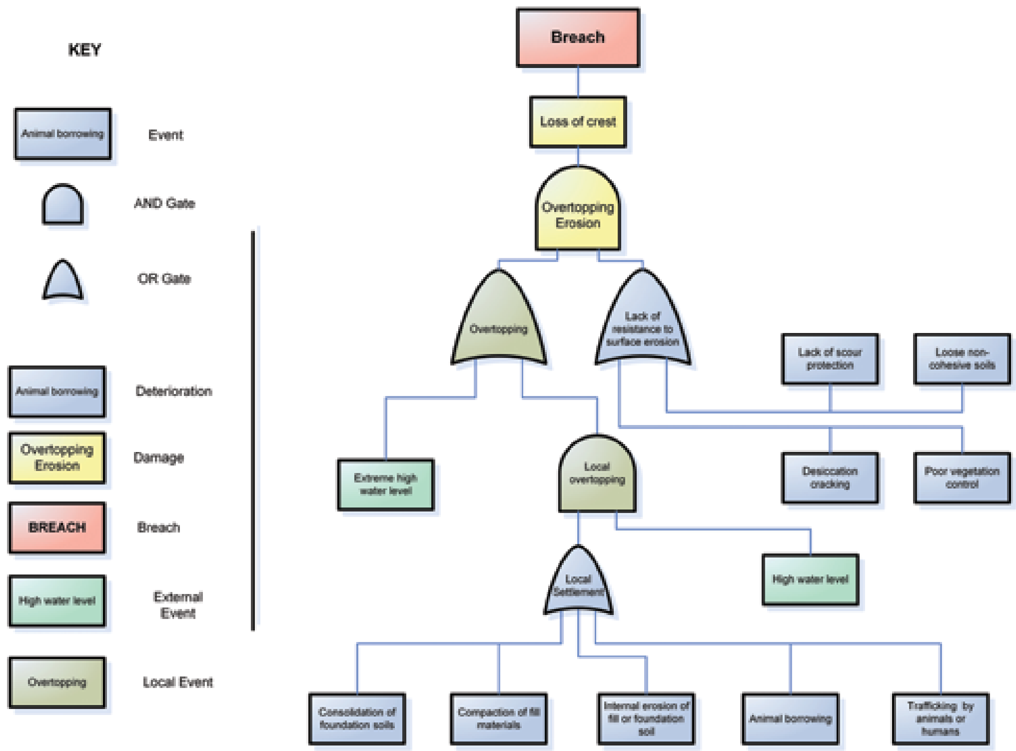


Figure 5.31 Example of fault tree analysis of a breach in a Levee (courtesy P Smith, Royal Haskoning)

Event tree analysis

Event tree analysis is used to identify the possible final outcomes starting from an initial unwanted event. The resultant mechanisms are inferred and combined to define and describe the expected consequences. From the initial event, all the resulting scenarios, or chains of events, and contributing factors are identified, described and may be associated with a probability. The modelling of the event tree is built through a binary (function running/function failed), discrete (events or time punctual evolution) and chronological method. It is presented by a series of linked nodes and branches starting from the left to the right, from the initial unwanted event to the final different consequences. Each node represents an uncertain event or condition. Each branch represents one of the possible binary outcomes of the event or one possible state that a condition may assume. After the identification of the scenarios by expert analysis, the ones that are not physically possible are rejected (Figure 5.32).

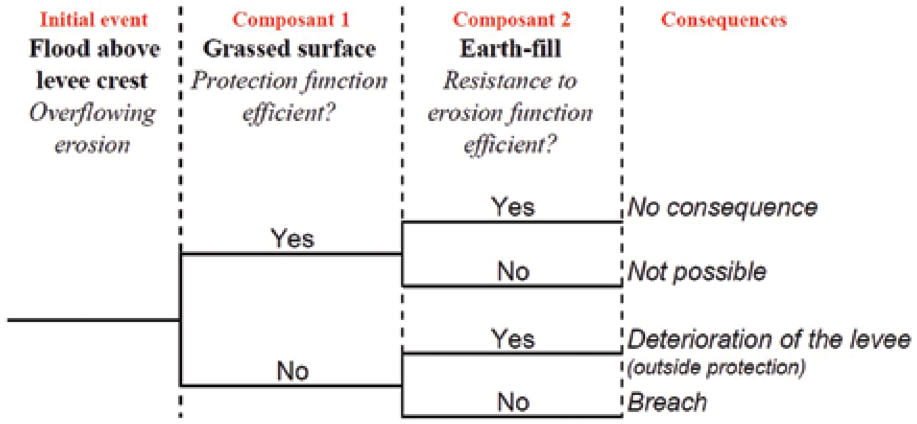


Figure 5.32 Example of simple qualitative event tree analysis (courtesy B Beullac and R Tourment, Irstea)

In a quantitative approach, probability value can be associated to each branch resulting from an event (node). Assuming an independence of the different events that comprises a scenario, a scenario consequence probability may be calculated by multiplying the probability values affected to its different branches.

An example of quantitative event tree analysis is presented in Figure 5.33.

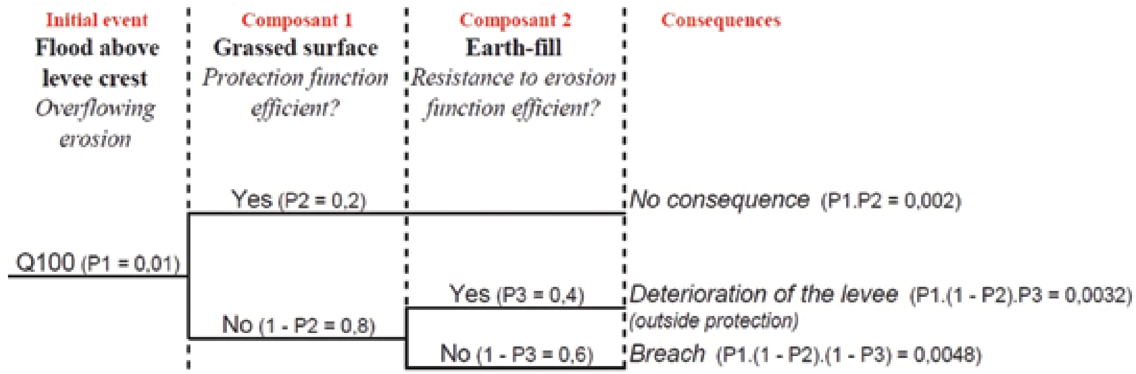


Figure 5.33 Example of simple quantitative event tree analysis with probabilities (courtesy B Beullac and R Tourment, Irstea)

Bowtie tree representations

A bowtie tree combines a fault tree and an event tree in a unique sketch, where the final event of the fault tree is the unwanted event of the event tree. This representation is usually built horizontally. The studied feared event (state of failure) is in a central position, the possible causes are developed to the left and the possible consequences are detailed to the right. Figure 5.34 presents both generic and specific examples of bowtie tree representations of a coastal levee system during a specific loading scenario (combination of events), in which 'inundation of the leveed area' represents the central event.

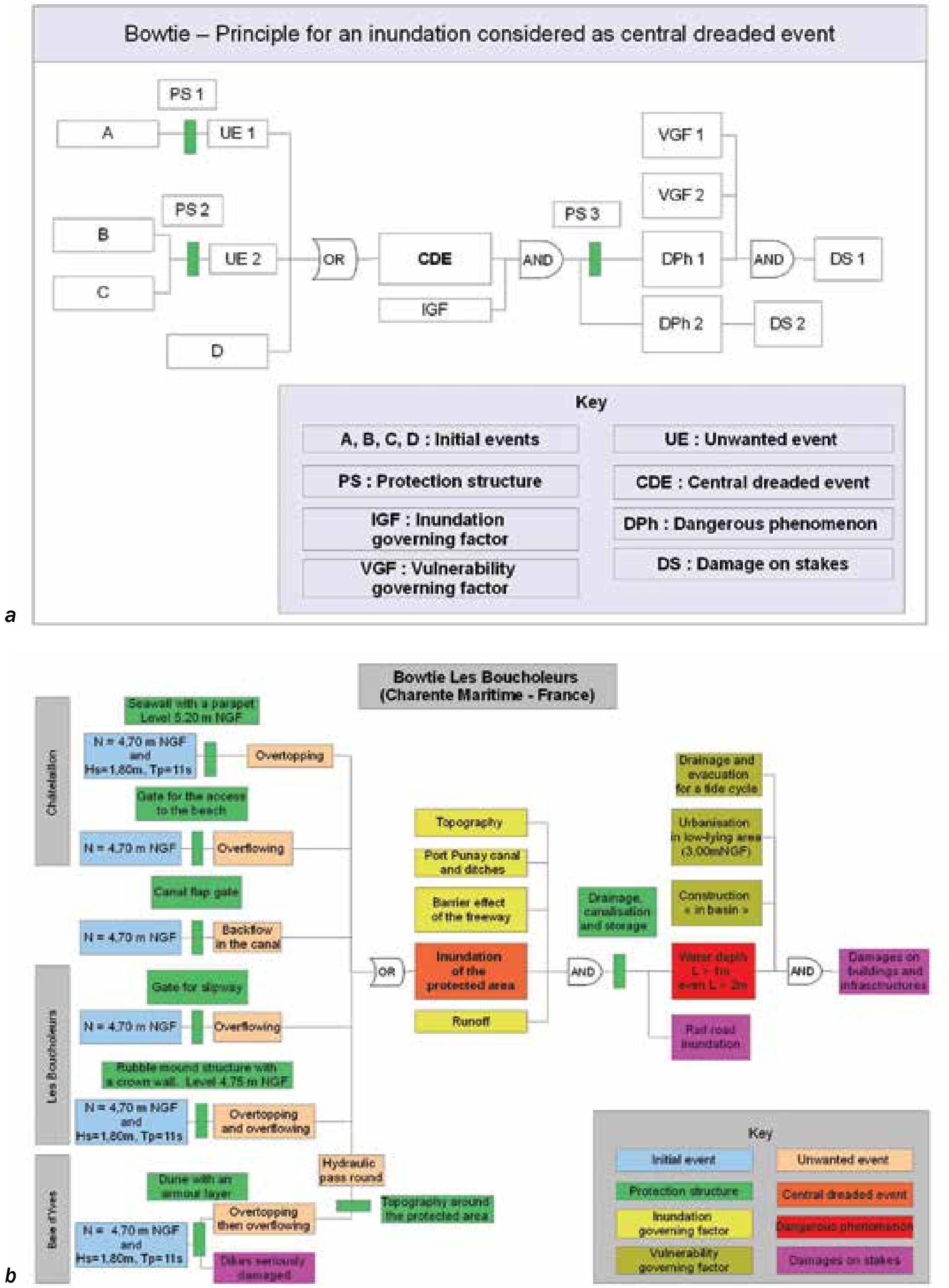


Figure 5.34 Examples of bowtie tree representations of coastal levee systems (a) generic (b) 'Les Boucholeurs' levee system, France (courtesy M Igigabel, CETMEF)

5.3.4 Assessment report

As part of conducting any assessment, it is essential to produce a specific report, presenting:

- the different data used during the assessment
- a description of the assessment method
- the results
- a conclusion.

The different data used during the assessment should be presented, whether they were previously available or produced by specifically conducted investigations or inspections. The engineer responsible for the assessment should judge the quality and reliability of all data, and present this information in the report. Specific reports about the production of data and pre-processing may be appended to the main assessment report or simply referenced, but the main data used for the assessment should be presented in the main body of the report.

Assessment can be a complex process (and not always standardised), so it is also necessary to present a clear and complete description of the method(s) used for combining the data (see Section 5.3.3) in order to produce the assessment results.

All results of the assessment, both intermediary and final, should be presented in the report. These results can be presented as text, tables, maps and/or graphics. The intermediary results can include performance assessment for specific events and/or specific failure modes or mechanisms.

Any assessment should reach some conclusion and, after the summary of the results, present clear recommendations in terms of any follow-up to the assessment (see Figure 5.1), which should be presented in the report itself.

Finally, all assessment reports should be archived and referenced in the levee managers' information system (see Section 5.6) in order to be available for future reference and to facilitate subsequent assessments.

5.3.5 Regulations

Assessments have to be conducted for multiple reasons:

- as part of a good management policy (rational data in the decision making process)
- because of bounding conditions (between a levee manager and the levee owners or sponsors)
- in order to justify management actions and results (toward stakeholders or other actors, see Sections 2.4.1 and 2.4.2)
- because they are enforced by national regulation.

Box 5.15 presents an example of such regulations.

Box 5.15 French regulation and levee performance assessments

French regulation on levees safety is based on decree no 2007–1735 of 11 December 2007, and subsequent texts. Levees are classified into four different classes, according to their height and the number of people living in the leveed area. Class D levees are either less than 1 m high or protect less than 10 people. Levees higher than 1 m high are class C if they protect less than 1000 people, class B if they protect more than 1000 and less than 50 000, and class A if they protect more than 50 000 people.

Different types of assessments are mandatory, according to the levee class:

- an initial assessment had to be produced, for all levee classes, before 31 December 2009
- detailed inspections (including report and conclusions, see Section 5.3) have to be conducted yearly for classes A and B, once every two years for class C and once every five years for class D
- a risk analysis (see Section 5.1) has to be conducted once every 10 years for all levees from classes A to C
- a safety review also has to be conducted once every 10 years for all levees in classes A and B
- state authorities (prefects) are granted the right to prescribe a specific complete assessment for any levee whereby security is at stake.

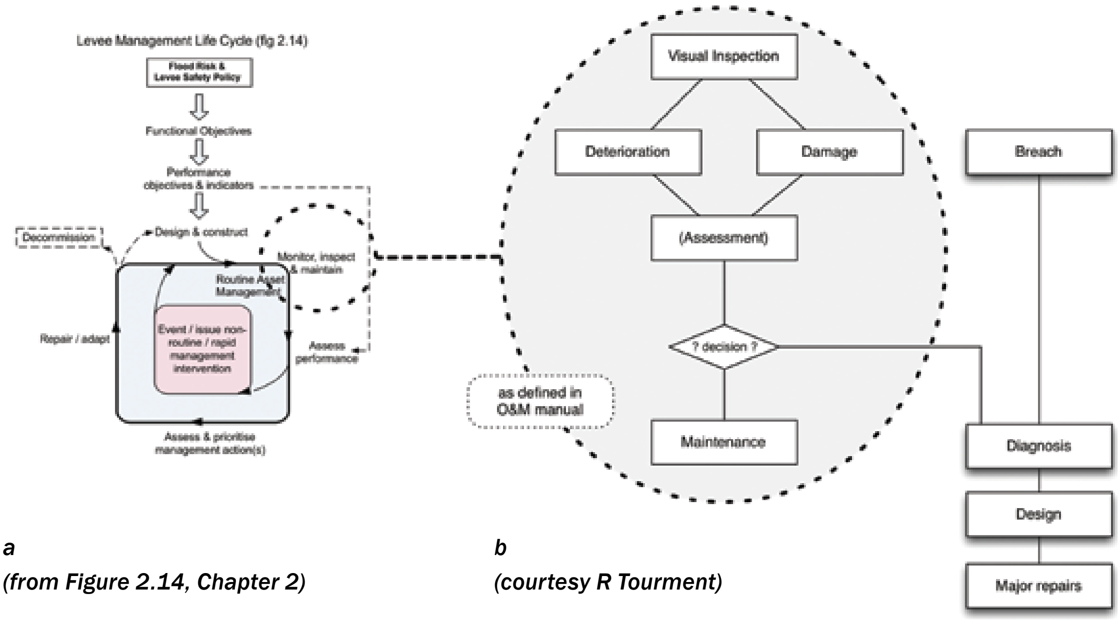
These different studies are defined in term of objectives, but the actual method for conducting them is not defined by the regulation.

A higher-level law (a civil code) also makes the owner of a structure responsible for any liability caused by its property, be it from a design or construction fault, or a maintenance shortcoming.

5.4 INSPECTIONS

Inspections provide valuable data input for assessments. Levee systems are vulnerable to deterioration and eventual failure. Inspectors strive to interrupt this continuum by observing deterioration before failure occurs, so that the intervention prevents failure.

If features of deterioration or damage are observed during inspections, a decision is required as to whether repairs can be simply maintenance works, or need design by a qualified engineer after a specific diagnosis. Such decisions are in fact a first level of assessment, often made by the regular management team. The decision requires a clear description of the observed phenomena, with subsequent necessary actions being indicated by the operation and maintenance (O&M) manual. Figure 5.35 presents this decision process (b) and its place in the levee life cycle (a) (see also Figure 2.14 for a magnified version). Chapter 4 describes, for several features of deterioration and damage, the kinds of action that may be required, distinguishing between maintenance or repair, which require engineering (Chapter 9). Recurring deteriorations (especially if at the same location) need an (qualified) engineered solution. Similarly, for specific actions related to features observed during flood conditions, Chapter 6 suggests some emergency measures.



a
(from Figure 2.14, Chapter 2)

b
(courtesy R Tourment)

Figure 5.35 Use of assessments in regular maintenance to decide on the nature of repairs

5.4.1 Inspections in the assessment process

Inspections are performed to observe the condition and operation of the levee system. They also help to understand the ability of the levee system to keep the chance of flooding from a river/stream/coastal waters to the landside of these systems at least at the desired/prescribed chance of exceedance level. Inspections provide information concerning the location, type and severity of deficiencies for differing failure mechanisms and should be followed by assessments. Inspections are normally conducted on a regular and reoccurring basis, and may be conducted relative to a loading event (pre-event, during the event, or immediately after the event). Inspections also should be conducted after certain special events (seismic, river barge accident etc) in order to check the levee integrity and possible failure initiation.

Inspections are either part of a wider assessment process (a data gathering process like any other one) or a specific 'operation' that is in itself a simple type of assessment, in the sense that conclusions and decisions can (and must) be made at the end of the process.

A number of items should be carefully considered while developing an inspection program such as:

1 Failure to perform quality inspections could allow slow progression of levee deterioration and eventually result in a levee failure.
2 Timing of an inspection will depend on the type of information needed. Inspections while loaded will more readily identify vulnerable areas while inspections in the dry reveal project observations related to deferred O&M activities and better opportunity to observe riverside conditions.
3 Timing of an inspection can reveal if there are any unauthorised modifications of the levee system that have occurred without knowledge, which could affect its integrity to perform if the project were to be 'loaded'.

5.4.2 Inspection methodology

To ensure that the underlying goal of the levee inspection is fulfilled, the most appropriate methodology should be selected. An annual operational inspection (conducted to ensure that routine O&M are being performed) will have a significantly different approach to that of a thorough periodic inspection of each part of the levee (aimed at ensuring continued functioning). Another approach might be taken during an inspection following a major loading event where some defect has been observed. This defect might lead to a more detailed/specialist investigation. When risk increases, the intensity of the inspection also increases. So, the end user of the inspection data should play a key role in planning and executing the inspection.

Inspections commonly consist of arm's length visual examinations of all aspects of a levee system. However, airborne inspections (see Figures 5.36 and 5.37) may be useful, or necessary, if there was a flood event that caused a breach to occur in the levee and there was no other means to make an inspection of the levee system.

Figure 5.36

Airborne method of inspection (courtesy Louisville District, USACE)

Figure 5.37

A levee breach from an airborne inspection (courtesy Louisville District, USACE)

Airborne inspections allow decision makers to rapidly assess the extent of major flooding events and to locate problem areas.

Other forms of inspections beyond the scope of this section include investigations, which are evaluations of physical parameters of the levee system by specific methods, and instrumentation and performance monitoring, which may be read in the field or remotely. Each of these forms of inspection provides information from which determination of a levee's condition and ability to perform satisfactorily can be made. Their use related to assessments is presented in Section 5.5 and the methods are presented in Chapter 7.

The general methodology of a levee inspection or surveillance consists of observing the entire length of the levee (ideally on foot, if practical) and recording all visual information about existing or presumed issues/anomalies affecting any of its parts. For extremely long systems, this may be performed incrementally, and more critical parts may be inspected more frequently than less critical parts that have a good service record. The following is a list of the general approaches to implementing detail of an inspection needed to be performed:

1. A tiered approach to levee inspections is appropriate, the attention to detail and depth of knowledge to be obtained from an inspection is dependent upon the ultimate goals of the ensuing performance assessment or risk analysis (see Sections 5.3 and 5.2).
2. Deficiencies identified in a particular levee system or a part of that system may have implications for more than one failure mode.
3. Consequences should be heavily weighted when determining the thoroughness of a particular inspection in the levee life cycle.

The 'levee items' to be inspected are determined by what is possible to assess visually and also by the failure modes that need to be considered. Performance features of the items can represent failure modes either directly or indirectly. Consistency of the inspection method, wherever possible, should also be considered in the identification of appropriate performance features. It is necessary to produce a set of performance features that balance accuracy of inspection with implementation issues such as the workload associated with inspecting an individual item. If a set of performance features is too large, however, possibly producing a more accurate condition assessment would more than likely increase the duration of an inspection beyond reasonable levels (time and budget).

It is important that inspections should be conducted by appropriately trained and experienced staff. It is inefficient to send a high level expert to inspect/evaluate the general condition (O&M condition) of a levee system, and it is inappropriate to assign a novice inspector to evaluate a critical issue that poses a functional risk with potentially high consequences. Routine or annual inspections do not require a full team of discipline specific experts to evaluate the general condition of the levee system. However, inspections or investigations conducted to accumulate detailed information associated with defined failure modes require experienced specialists and cannot typically be performed by general staff/inspectors.

An inspection of a levee system entails observing and noting the surface condition (turf cover, rutting, animal burrows, slope stability etc) of the levee. The inspection should also include a performance verification of levee system parts (such as, but not limited to, operation of flap/sluice/slide gates) associated with reducing the risk of exterior floodwaters from entering into a specific area (leveed area) as well as removal of rainfall from the interior to assist in eliminating damage to structures/property.

'Individual levee item performance' is the building block of the levee inspection process, which ultimately leads to an overall levee condition assessment. Items can be listed in general categories of a levee system that can be but are not limited to the following:

- river specific items
- sea specific items
- earth levee segments
- concrete wall segments and other structures
- conduit, culvert and pipe items.

These represent general categories of a levee system that can be further defined in a more comprehensive list shown in Table 5.5. It is important during an inspection to document observations that have the potential to cause or are causing issues/damage to the levee and even issues that do not appear to be a negative impact to the overall levee performance if it were to become hydrostatically loaded. Some of the elements in the list may not be the direct responsibility of the levee owner/manager, but it will be important to identify in general terms any defects in those elements that raise concerns about their consequences for the levee itself.

Table 5.5 List of items to be checked (or considered) during an inspection

Earth levee segments	
Sod/grass cover	Surface protection (grass, rip-rap, stone facing)
Vegetation, (trees, brush, grasses – invasive species)	Erosion protection
Encroachments (structures, utility crossings etc)	Animal burrows
Relief wells/toe drainage systems	Saturation/pooling
Seepage	Overtopping (evidence of/historical record of)
Signs of slope instability	Transition points
Settlement	Repairs
Depressions/rutting	Livestock use
Erosion/bank caving	Human activity (digging etc) – either permitted or unpermitted
Translational or longitudinal cracking	
River specific items (with potential to affect levees)	***Coastal specific items***
Channels	Seawalls
Weirs	Beaches
Bridge abutments	Groynes
Conveyance capacity	Dunes
Sinuosity/erosion	Saltmarshes and saltings
Sluices/barrages/barriers	Sea outfalls
Navigation structures (locks/lifts etc)	Offshore breakwaters
Spur levees or groynes	Wharves
Concrete wall segments and other structures	***Interior (inland) drainage items***
Vegetation, (trees, brush, grasses – invasive species)	Vegetation (trees, brush, grasses – invasive species) and obstructions
Encroachments (structures, utility crossings etc)	Encroachments (trash, debris, unauthorised structures, excavations etc)
Monolith joints	Ponding areas
Sealant/water stop, (concrete)	Fencing and gates – security and accessibility
Insect/fungal attack, (timber)	Rip-rap or other revetments at inlet/discharge areas
Deterioration of ashlars, (masonry)	
Staining, (concrete) – caused by oxidation of steel rebar	***Conduit, culvert, and pipe items***
Closure structures	Concrete surfaces (such as gatewells, outfalls, or intakes)
Relief wells/toe drainage systems	Foundation of concrete structures
Honeycombing	Monolith joints
Anchor ties and plates	Conduit, culverts and pipes – inspection frequency
Cracking	Conduit, culverts and pipes – condition rating
Mass instability – rotational or translational sliding	Sluice/slide gates – essential item
Settlement/uplift	Flap gates/flap valves/pinch valves
Erosion protection	Trash racks (non-mechanical)
Backfill	Other metallic items
Animal burrows	Pump station intake and discharge pipes
Transition points	Utility pipes – pressurised (regulated and non-regulated)
Repairs	
Human activity (digging etc) – either permitted or unpermitted	

Performance of these items will typically possess a number of attributes. These attributes are listed below in order of importance:

1 **Items:** can be linked to at least one failure mode, and also to the performance of the levee. For a performance-based visual inspection, the primary objective is to inspect items for their likely performance. If the visual condition of an item plays no role in levee performance, it need not be inspected.
2 **Visible:** the item to be inspected should be easily assessed on a visual inspection. For example, many performance models have geotechnical parameters that cannot be observed on a visual inspection. A visual assessment requires there to be a visible item on the surface of a levee that directly or indirectly relates to a parameter of a failure mode process.
3 **Gradable:** in addition to being visually identifiable, the current condition of the item should be able to be assessed visually. There should be sufficient visual indicators correlated to the performance of the item in order to assign the range of condition values associated with that item.
4 **Mutually exclusive:** ideally, performance of items should be mutually exclusive. There should be no chance of mistaking the condition of one item for another. However, if necessary, this attribute can be relaxed to some degree in order to satisfy the more important attributes of points 1, 2 and 3.

Item performance can apply to a single element of a levee system and so can be repeated for each element where relevant, or can apply to the whole levee. Performance of an item could relate to a number of failure modes or be associated with a single failure mode. Performance of an item that is uniquely associated with a single failure mode is important in the differentiation of the most likely failure mode and usually will have a high contribution rating linking it to that failure mode. An example of this is the class of items relating to deformation of the levee structure or cross-section. Items that could be associated with the deformation of a portion of a levee cross-section could be settlement, depressions/rutting, and conduit/culverts/discharge pipes – condition. See Figures 5.38 and 5.39, which show how these items would be related an internal erosion failure mode.

Figure 5.38

Complete section loss in the bottom of a corrugated metal pipe (courtesy Louisville District, USACE)

Figure 5.39

Depression in the levee crown directly above a deteriorated corrugated metal pipe (courtesy Louisville District, USACE)

In Figures 5.38 and 5.39, a large diameter corrugated metal pipe can be inspected directly by walk-through or using video equipment. Signs of deterioration or failure of a pipe can also be observed in the form of subsidence and sinkholes of the overlying earthen embankment. A video inspection will provide evidence of corrosion, holes, blockage, joint misalignment, and failed coatings. The indirect inspection will provide evidence that failure is actively occurring and soil is being lost from the embankment into the pipe. In this case, soil could also be piping along the surface of an intact and structurally sound corrugated metal pipe, so an indirect observation must be followed up with a more detailed investigation

– beginning with a video inspection. The potential failure mode associated with these observations would be internal erosion.

5.4.2.1 Inspection types and frequency

Countries and organisations around the world use various types of inspections, and each will have differing objectives for those inspections. Also, the frequency of inspections varies from country to country. Generally though, inspections should be adjusted to both the size of the leveed area, the value of the lands/infrastructure in the leveed area and the risk level (see Section 5.1). Also, in a single country, different inspection types are conducted. Regular and reoccurring inspections should provide a record of progressive problems so that patterns can be identified and the need for intervention evaluated. The following types of inspections may be carried out in relation to the levee management cycle. The type of inspection will depend on the timing and the regulatory framework either by the levee manager or the regulatory authority (see Figure 2.14):

1. **Initial inspections** should evaluate and document the condition of the entire levee, either soon after construction or when a levee system is upgraded to enter a regulatory program or when a 'forgotten levee' is incorporated into a good management scheme. An initial inspection helps to determine the capability of the existing system to perform satisfactorily under full hydraulic loading. Initial inspections should require the operation of all aspects of the levee system to set a baseline understanding for the condition of the levee system as a whole. Initial inspections are usually part of a broader (initial) assessment.
2. **Operational inspections** are performed by the levee manager's staff. During routine O&M, items of deficiencies discovered should be corrected as soon as possible or scheduled for corrective maintenance by the levee manager. Deficiencies beyond the levee managers understanding require additional inspection, review and consultation with applicable regulatory authorities.
3. **Routine inspections** should provide documented evidence that the levee continues to meet minimum acceptable standards for O&M and which should relate to acceptable performance levels. Routine inspections are normally performed without the use of special inspection equipment.
4. **Periodic inspections** are typically performed by specialists from various engineering disciplines (hydraulic, geotechnical, structural, mechanical, electrical engineering etc). Special inspection equipment is required (CCTV cameras, man-lifts (hoists), non-destructive testing equipment, electrical and vibration instruments etc).
5. **'Pre-flood' inspections** should be performed shortly before an approaching loading event on a specific levee, to verify that all parts are in order.
6. **'In-flood' inspections** are performed while the system is loaded and are extremely valuable in identifying weak or susceptible areas that could lead to a potential failure in the future in order to plan emergency repairs and/or initiate population evacuation.
7. **'Post-flood' inspections** are crucial in observing any damage that may have occurred and evaluating the ability of a levee to withstand a future loading event. This type of inspection is also used to validate, verify and to add to (which is invaluable because it is hard to get otherwise) information collected during the flood event. This could lead to establishing a program for urgent work.
8. **Special inspections** for modifications, assessments, or special parts are necessary when a change from the original levee design is requested or a specific concern is raised.

Section 5.4.2.2 provides examples of how these inspections are performed in several countries, divided by the levee manager or regulatory authorities.

Every visit to a levee can be an inspection, regardless of the reasons for the visit, and is an opportunity to make observations and note anything out of the ordinary. For example, a site visit is often in response to a phone call received by the regulatory authority or responsible engineer in a specific country from the local owner/manager asking for assistance with an issue that has been noted relating to an unusual condition occurring with respect to the levee. Box 5.16 is an example of such an issue.

Box 5.16 Every site visit to a levee can be an inspection

In the USA, USACE Fort Worth District staff made a site visit to the Dallas levee system in Texas to observe slope movement within the levee embankment. This was a special site visit to note and record this specific issue and possibly determine the cause of the slope movement, and to illustrate a slope stability failure mode. Every visit to a levee provides the various levee actors the opportunity to inspect specific features of the levee.

Figure 5.40

Slope movement on the Dallas levee system, Texas, USA (courtesy USACE Fort Worth District)

There are no perfect inspections. Inspections performed in the dry (with no water loading) cannot provide the inspector with detailed knowledge of how the levee will perform when hydraulically loaded, for example, seepage cannot be observed. Conversely, inspections during floods do not provide an opportunity to observe waterside features (levee toe, discharge pipe outlets etc). Based on historical performance of the levee and its associated items, the inspecting authority should determine the extent of the examination required and the relevant types of inspections. Figures 5.41 and 5.42 illustrate two specific instances of inspections that yield condition assessments of specific items such as the outlet works of a pipe during low river conditions and a trial installation of a road closure.

Figure 5.41 Inspection of a culvert outlet (courtesy USACE Louisville District)

Figure 5.42 Inspection during a closure installation (courtesy USACE Louisville District)

5.4.2.2 Levee management and regulatory authority inspections

Levee management organisations are required by regulation or by practical necessity to continually monitor the levee condition and safety to effectively discharge their duties. In order to achieve this, regular and different levels of condition assessments, including inspections, have to be organised and performed. These inspections can be performed by levee management staff or by hired engineers and specialists. Planning of these inspections should consider:

- tiered approach and the levee life cycle (see Sections 5.2 and 5.3)
- general inspection methods (see Sections 5.4.2, 5.4.3 and 5.4.4)
- reporting of inspections to regulatory authorities and levee managers/local sponsors (see Section 5.4.4).

The regulatory authority also inspects the general levee condition and safety, as well as the management organisation, documenting actions to ensure that the management organisation is managing the levee system appropriately, and to understand the ability of the levee to perform under full hydraulic load and under its design/as-built level. These types of inspections ensure the levee meets acceptable standards related to design/construction and post O&M efforts. The tiered approach and the levee life cycle should be considered during the planning of these inspections (see Section 5.1) and the general inspection methods and reporting (discussed in Sections 5.4.2, 5.4.3 and 5.4.4) should be followed.

Boxes 5.17 to 5.26 illustrate types of inspections performed by levee management organisations and regulatory authorities in various countries at various times during the levee management cycle.

Box 5.17 Inspections performed by levee management authorities in France

- **complete initial inspection:** this inspection is needed in order to initialise the knowledge about the levee. Part of the initial assessment. Helps as a reference for future inspections, up to the next complete inspection
- **operational inspections** by levee guards
- **detailed inspections** performed by engineers, either part of the manager staff or hired contractors. Frequency can vary between one and five years based on the magnitude of the protected population
- **complete inspections** including hidden parts (underwater and inside pipes) performed by engineers including possibly specialists, either part of the manager staff or hired contractors. Once every 10 years and only for class A and B levees (see Box 5.15). Helps as a reference for future inspections, up to the next complete inspection. Part of the periodic safety review process
- **pre-flood inspections** are not required, but useful
- **during flood inspections** are performed by the levee manager, possibly with help from local authorities and levee owners
- **post-flood inspections** are required, and can be a good source of information not normally available (ie seepage, closure integrity)
- **special inspections** as part of a specific assessment, triggered because of a special event (accident, seismic), because of planned works, or other specific reason.

Box 5.18 Inspections performed by Rijkswaterstaat (levee manager) or the water boards in the Netherlands

Primary and regional levees

Inspection type and frequency depend on the category of levee to be inspected:

- **primary levee system:** flood defences along the sea, rivers, main lakes including closure dams and storm surge barriers
- **regional levee system:** flood defences along inland canals and lakes.

The State regulates (set norms and controls the levee managers) the primary flood defences and the provinces regulate the regional defences.

Levee managers are either water boards or Rijkswaterstaat (national body). Water boards maintain the primary and regional levees (dikes, dunes) and related structures, ie they organise the inspections (either by own staff or by contracting it out). Rijkswaterstaat (state body) maintains the large dams and storm surge barriers.

- **inspection in spring (after flood season):** systematic and detailed inspection, in which the actual condition of the levee after the flood season is determined, including the required (repair/maintenance) actions to be implemented before the next flood season
- **inspection in summer:** inspection of the repair and maintenance activities
- **inspection in autumn (before flood season starts):** check on implemented activities and determining the actual condition before the start of the flood season
- **daily inspection:** unregulated inspection carried out by the water board members during their daily work
- **inspection during floods:** inspections by the levee employees during a (river or sea) flood and immediately thereafter, to identify the actual state and determine urgent repair activities
- **periodic detailed inspections:** safety assessment with calculations for all failure mechanisms, and based on soil surveys etc to determine the actual strength of the levees under up-dated hydraulic loading characteristics, and including an in-depth inspection of the levees. The frequency is under consideration, but is at present six years for the primary defence system and five to 12 year for levees of the regional defence system (with more or less continuous inspections).

Box 5.19 Inspections performed by the levee manager/local sponsor in the USA

- **operational inspection:** carried out by the levee manager/local sponsor to determine where O&M issues exist on the project. Inspection intervals are twice a year
- **in-flood inspections (performance inspection):** carried out by the levee manager/local sponsor and may/may not be in conjunction with USACE staff and possibly with help from other organisations – sponsor staff, local authorities
- **post-flood inspections (performance inspection):** carried out by the levee manager/local sponsor and may/may not be in conjunction with USACE staff. This type of inspection can provide information not typically available during a non-loading state. Inspections include identifying weaknesses, sand boils, leaks or other vulnerabilities.

Box 5.20 Inspections performed by the levee owner/association in Germany

For (smaller) rivers of second order within a federal state, supervision and/or organisation are made by the local authorities. In some federal states there are special levee associations who organise and fund the fluvial flood protection.

On river levees (DWA, 2011):

- **regular, periodical inspection** carried out at least annually by the levee owner to determine where O&M issues exist on the project. This inspection is to ensure and monitor the safe function of the flood protection works. For class 1 and 2 (large and medium) levees inspection intervals are once per year typically in the spring. For class 3 (small) levees inspection interval may be every five years
- **post-flood inspections (performance inspection)** carried out by the levee owner. This inspection is to note the damage to the levee and then decide on the repair needed, which is typically performed as soon as possible
- **monitoring programs** with measuring instruments only for class 1 levees (not mandatory) and in special cases (eg special problems, seepage, subsidence)
- **levee crest survey/surveillance** every 10 years
- all technical data and all findings during the inspection have to be documented as a status (or better state?) report in the 'levee book' (levee records are also electronically possible). One copy has to be kept by the owner/operator and one has to be submitted to the authority.

On coastal levees:

- inspection of coastal levees is performed by the levee owner twice a year.

Box 5.21 Inspections performed by the local authorities (LA)/levee owners in Scotland

Local authorities have duties under the Flood Risk Management (Scotland) Act 2009 to assess the condition of bodies of surface water from time to time to ascertain whether its condition gives rise to a risk of flooding. Any artificial structure that forms part of the bed or banks of a body of water, eg levees, are included in the assessment and where repairing such works would substantially reduce that risk, the local authority must prepare a schedule for repair works. Levees may be part of a statutory flood prevention scheme, so the local authority is responsible for its maintenance. Where levees may have been constructed by other persons, mainly agricultural riparian owners for their own purposes, the primary responsibility for maintenance of the levee lies with them.

- **routine inspection** of body of water by LA inspector (not engineers) (frequency is risk based, at least once in every six year flood risk management plan cycle, could be weekly for very sensitive receptors)
- **technical inspections** of levees (statutory schemes) by LA engineers – at least annually or otherwise in accordance with designer's recommendations
- **technical inspection** of levees (non-statutory) by LA engineers – frequency determined by risk
- **special inspections** (includes inspections as part of a specific assessment, triggered because of a special event (eg accident, flood) or because of report from routine inspection or planned works or other specific reason)
- **survey** to check crest levees are carried out every five years or so by the LA engineers.

Box 5.22 Levee control authority inspections performed by the France state authorities

- **initial inspections:** in France the control of levees by the state is quite recent and many levee systems are yet to be inspected for the first time. During the initial inspection, the control authority has to obtain all pertinent information about the levee and its management, and check the safety of the levee through the level of knowledge and organisation of the levee manager and the result of its own assessments
- **periodic inspections:** for class A, B and C levees (period varying from one to 10 years)
- **in-flood inspections:** performed by control authorities when managers' report safety issues
- **post-flood inspections:** performed by control authorities when managers' report safety issues
- **special inspections:** control authorities can perform special inspections, particularly when there are safety issues or planned modifications.

Box 5.23 Levee control authority inspections performed by the state organisations in the Netherlands

- check reports of all primary levee managers and report findings at national level to Parliament (state)
- check report of levee managers under jurisdiction of the province and report findings to regional Parliament (province).

Box 5.24 Levee control authority inspections typically performed by USACE district offices in the USA

- **complete initial inspection (initial eligibility inspection, IEI):** this inspection is needed in order to collate knowledge about the levee. Part of the initial assessment acts as a reference for future inspections, up to the next complete inspection. During the initial inspection, the relevant USACE district office has to obtain all pertinent information about the levee and its management, and check the safety of the levee through the level of knowledge and organisation of the levee manager/local sponsor
- **routine/annual (operational, continuing eligibility inspection, CEI):** inspection by the USACE. Inspection intervals are every one to two years based on land use in the leveed area
- **periodic (detailed, multidiscipline performance inspection) inspections (performed by the USACE or hired contractors):** inspection interval is every five years. This also requires a levee manager/local sponsor to provide inspection reports related to pipes
- **in-flood inspections:** performed by control authorities/USACE district office only when levee managers/local sponsors report life, property and safety issues
- **post-flood inspections:** performed by control authorities/USACE district office only when levee managers/local sponsors report life, property and safety issues
- **levee system evaluation (LSE) inspections:** periodically levees are required to have an inspection to determine a community's eligibility for flood insurance as administered by the National Flood Insurance Program (NFIP). This evaluation can be accomplished by a licensed professional engineer or an agency responsible for levee design (USACE) at the request of the local sponsor. In order for a levee system to remain in a positive status, this evaluation for eligibility will be required every 10 years per the USACE guidance
- **special inspections:** control authorities can perform special inspections, particularly where there are safety issues or planned modifications. The levee manager can report safety issues directly or indirectly through reports related to one type of assessment or another
- **project turnover/final inspection:** a post-construction inspection to obtain a baseline inspection of the project. It occurs at the end of construction (once only) by the USACE.

Box 5.25 ***Levee control authority inspections performed by the supervising authority in each federal state in Germany***

For the coastline and the (large) rivers of first order, the flood protection is supervised and/or organised by the federal state authorities.

On river levees (DWA, 2011):

- annual inspection by the supervising authority to review the project to ensure and monitor the safe function of the flood protection works. Typically the levee owner will accompany the supervising authority. Inspection intervals are once per year.

Box 5.26 ***Inspections performed by the Environment Agency in England in its role as both owner/operator and regulatory authority***

(Note that the Environment Agency visually inspects all levees that are integral in providing protection to communities against flooding from a Main River* and the sea)

- as-built surveys for new and improved levees are produced and the details from these are added to those of existing levees in the national database that maintains a risk-based programme of visual inspections
- routine operational inspections by the O&M teams. This can include pre-flood, during flood and post-flood inspections of critical levees
- a risk-based detailed visual inspection programme that assesses all elements of assets, including levees against standard performance and condition criteria set out in a Condition Assessment Manual. These are carried out by accredited inspectors who have passed a course of theory and practical asset inspection. These form the basis of reports to the Government of the condition of the nation's assets
- detailed inspections and surveys that are either part of a proactive programme or that are reactive (triggered by the visual inspection or flood event). These are carried out either by in-house or consultant engineers on Environment Agency owned and/or maintained levees (note that if there are concerns about the condition of third party maintained levees then the Environment Agency will contact the third party advising that they have more detailed assessments carried out)
- post flood inspections and where appropriate detailed investigations into asset performance.

* Main River is a legal term and relates to watercourses that have the potential to pose a significant flood risk. They are marked as such on maps held by the Environment Agency and its sponsoring government department.

5.4.3 Managing inspections (planning for, and inspectors' training and qualifications)

Good planning and management help ensure consistent, efficient and thorough inspections. Poorly planned and managed inspections cost more, take longer, provide less quality information, and don't deliver consistency over time. Also of great importance is the training and competency of the inspection team. The increased consequences of failure would require an increase in the education and experience of the inspector.

Before any inspection, the levee owner/manager (if it is not the organisation planning the inspection) should be notified so they can make arrangements necessary for the inspection. Such arrangements may include mowing the grass, securing operations staff and equipment (ladders, safety harnesses, lifts, traffic control, special vehicles to traverse the levee etc), and consideration to areas of restricted access (confined space entry, gated/locked areas and/or high water) necessary for complete inspection.

The levee managers' operations staff should have mandatory training that includes comprehensive knowledge of how to operate and maintain the levee system in accordance with the O&M manual. The training should include hands-on experience with operating and maintaining pumping stations and sluice gates, installing closures, performing insulation resistance testing of electrical motors, cables, and power distribution equipment, performing or contracting for arc flash hazard analysis, and all other levee components. The staff should also be trained to identify concerns that may develop during high water periods while patrolling the levee system. The operations staff should also have hands on training to be able to understand the proper maintenance and condition of the levee system and its associated items (see Tables 5.5, 5.6 and 5.7). This will help to identify concerns during an inspection to be brought to the attention of the levee manager, which could trigger a more detailed type of inspection, or other actions. Adequately trained operational staff can assist a levee inspector in determining a more accurate levee condition assessment.

1 2 3 4 5 6 7 8 9 10

A levee inspector (provided by the regulatory authority) must also be competent, adequately trained and have the appropriate guidance to ensure accuracy, consistency and reliability in the inspection results. The inspector will need to understand the reasons for an inspection as well as understand levee 'performance' and how a levee may deteriorate and fail. Training courses such as those taught through structured classroom training, by remote or online training, or by a combination of these can aid to the education of an inspector. New inspectors should have this training in conjunction with 'on the job' guidance from a qualified and experienced inspector before an inspection by the inexperienced inspector is validated. Guidance for the inspector could be in the form of a document produced to reinforce and broaden the knowledge gained during training. It also provides reference to other documents that might also aid them in the inspection of levees, and encouraging the inspector to 'self-learn' and broaden their knowledge and understanding. An inspector must also be capable of accessing and locating the levee 'in the field', which requires spatial awareness and an appreciation of geo-referencing and map reading. Typically, computer literacy is required as well for entering collected data into portable computers (PDA's, laptops etc) and uploading the inspection results to a database.

Box 5.27 provides an example of a method used in England and Wales to train staff to perform inspections of their flood defence systems.

Box 5.27 ***Asset inspection training in England and Wales by the Environment Agency***

In England and Wales staff involved in inspecting flood defence assets for the Environment Agency must be accredited. Each trainee inspector has a mentor and has to complete an e-learning course and e-assessment followed by a probationer/journeyman period of mentored inspections and then an on-site assessment by external accreditors. This aims to provide experience of a wide ranging set of assets including both linear defences and point structures in fluvial, tidal, coastal, rural and urban settings. Inspectors are also trained in the use of relevant interactive software for recording and transferring data. There is a requirement for experienced inspectors to keep a continued professional development (CPD) record to ensure they remain up-to-date with developments in the field of asset inspection and to have a five yearly 'health check' by an external accreditor to ensure they are still fully competent. In 2012 the training and supporting tools, such as the Condition Assessment Manual, were extended to include assets that protect against coastal erosion for inspectors employed by maritime local authorities.

5.4.4 Conducting and reporting inspections

As part of conducting an inspection, it is important to have a structure in place in regards to recording observations in standardised format, a method of reporting the results, and a central location for managing this data (see Section 5.6). Figure 5.43 is a simple flowchart (adapted from Figure 5.1) denoting the results of an inspection and actions to be taken if necessary.

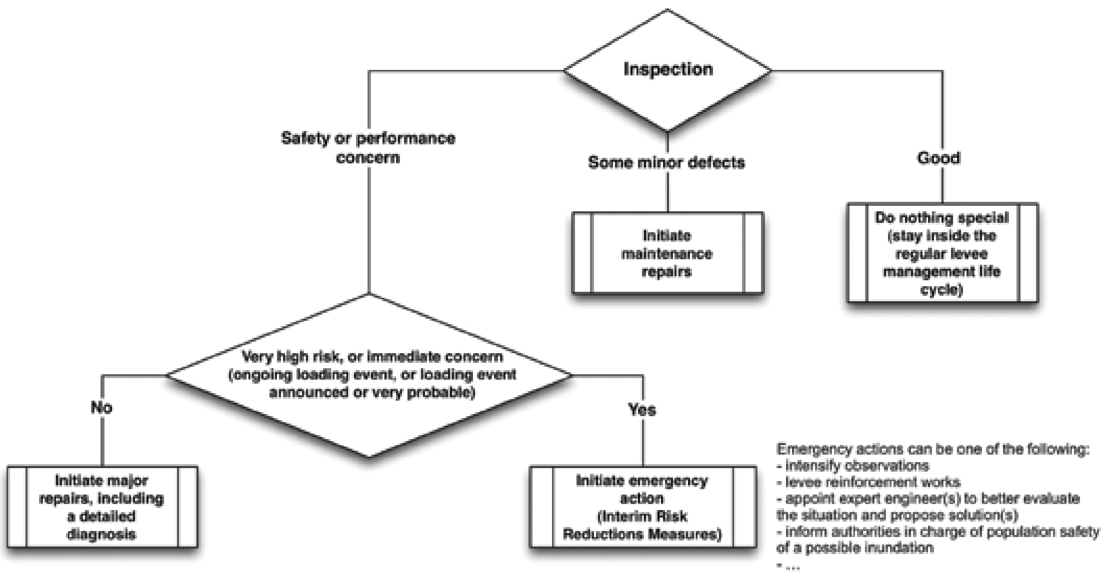


Figure 5.43 ***Flowchart of the follow-ups according to inspection results (courtesy R Tourment)***

Inspections generally consist of evaluating the existing condition of items (see Table 5.5) along a levee system. If a levee is properly maintained according to the O&M manual, it is implied that it will function as designed. There are various factors that may make this assumption incorrect and so a more thorough inspection would need to be undertaken.

A routine inspection is conducted to get a condition assessment for the levee on the day of the inspection. A more thorough inspection (such as a periodic inspection) is a performance assessment inspection, which requires a multi-disciplinary team to evaluate the levee system. This performance inspection would look at the details of the mechanical, electrical, structural and piping components along with the levee embankment and consider how the overall system would perform with hydraulic loading to the top.

Other detailed inspections include assessing the condition of:

- hydraulic steel structures such as roadway and railroad closure gates
- masonry or concrete structures associated with flood walls
- pump stations or gate wells (gate houses)
- sluice gates or flap gates
- pumps and motors
- culverts/discharge pipes.

These can be done by using either walk-through techniques or remote inspection tools such as closed-circuit television (CCTV), sonar, laser methods, and other special devices (see Box 5.28 for an example of culvert/discharge pipe inspection in the USA). All of these items are vital parts of a levee system. Inspection of many of these features requires the use of fall prevention devices and the assistance of confined space equipment, monitoring and special trained staff to ensure safety of the inspectors. The key reasons for levee inspection and reporting the results are:

1. Inspections are conducted to ensure adequate maintenance is being performed and there are no ongoing deficiencies that jeopardise the levee system from performing satisfactorily during a loading event.
2. Inspections are useless without sufficient documentation. All features of the levee need to be available for inspection and operation if deemed necessary.
3. Documentation methods for the level of inspection conducted should be consistent over time.
4. Reporting the findings of the inspection should be sent to the levee managers who maintain the levee systems and to the regulatory authority. Communication of findings is typically by letter to these groups with a copy of the inspection report (see Sections 5.4.4.2 and 5.6).
5. Conclusions of an inspection can require subsequent actions for higher level assessments to be performed, including possibly investigations, installing instrumentation and initiating monitoring (see Sections 5.2, 5.3 and 5.5).
6. Failure to inspect or completion of substandard inspections are likely to lead to deficiencies being overlooked or their significance not appreciated as a problem. This may allow further deterioration and lead to failure.
7. Failure to communicate/provide detailed documentation of inspection findings may result in subsequent failures to identify problem areas or at least the inability to determine when the deficiency started.

Box 5.28 Inspection of culvert/discharge pipes in the USA

In the USA, pipes that pass through or under a levee that are part of a FRMS within the USACE portfolio are typically inspected every five years using CCTV or walk-through procedures. Pipes less than 1.2 m (48 inches) in diameter are CCTV-inspected, along with pipes that are hazardous to enter due to their deteriorated condition or the presence of a hazardous atmosphere. Large pipes in good condition can be inspected and photographed by walk-through procedures.

Many levee managers/local sponsors in the USA use the Pipeline Assessment and Certification Program (PACP) promulgated by the National Association of Sewer Service Companies (NASSCO, 2010). The PACP establishes protocols for inspection, assessment, and rating of pipe condition. The PACP ratings range from 1 for an excellent pipe to 5 for a pipe that is failing. In the USACE Louisville District, PACP ratings are converted into acceptable, minimally acceptable, or unsatisfactory (A, M, or U) ratings for tabulation on the inspection of completed works (ICW) checklist (to rate condition of items during a levee inspection) based on an engineering assessment review of the PACP ratings, CCTV video and video report.

USACE also recognises that other methods are useful for pipe inspection. Sonar and laser point cloud techniques are two other such methods that can also be used.

There are other types of pipes known as 'utility pipes' that pass under, through and over levee systems that are **not** part of the successful operation of these systems. Box 5.29 provides an example of how these types of utility pipes and lines are inspected in the USA to ensure that any potential deterioration does not adversely affect the performance of the levee system.

Box 5.29 ***Inspection of utility pipes and lines in the USA***

In the USA, utility pipelines owned by third parties are inspected by the owners, who must provide details about the pipe design, construction, and operation. Pipes containing hazardous materials are regulated by the Pipeline and Hazardous Materials Safety Administration (PHMSA). Pipeline owners regulated by PHMSA must certify that the pipeline meets regulatory standards and has been identified as a high consequence area where it passes nearby or through the levee system.

Owners of pipelines that are not regulated by PHMSA must identify any regulatory agency having jurisdiction (or that none exists), and must establish and maintain a pipeline integrity management plan that explains how the pipeline is inspected.

Inspections of these pipelines are accomplished using in-line inspection (smart pigging), hydrostatic pressure testing, direct assessment, or other technology that provides similar information. Testing of these pipelines must be performed at least every five years.

5.4.4.1 Conducting an inspection

A typical inspection is performed in the following sequence:

1. Walk the length of the levee system as necessary depending on the type of inspection.
2. A minimum of two people perform the inspection to ensure the completeness and relevance of data collected and for safety reasons.
3. Inspect all parts of a levee and record condition data either digitally or on paper logs.
4. Photographs are taken and GPS co-ordinates are obtained related to issues/anomalies.
5. A report of findings is produced and the results communicated with the levee manager/local sponsor.
6. Data reports resulting from the recording of information during an inspection will be archived for future use (see Section 5.6).

Example procedures prior to and during an inspection

If the levee has a long history of inspections, the general rule for conducting a new inspection would be to look for changes that have occurred since the last inspection. When an inspection is conducted, the previous inspection report should be reviewed and items noted on that report given special attention during the current inspection. For example, during an assessment, the inspector would review the previous inspection report which recorded that a monolith in the flood wall had tilted relative to its neighbour by a 1.3 cm (0.5 in). If the current condition has not changed and the difference between the two still measures 1.3 cm (0.5 in), then the item would not be a critical issue. It would be noted that there was no change since the last inspection. Also, the inspector should interview the levee's operational staff to determine what maintenance work has been done since the last inspection and whether or not the levee has experienced a flood event and how it performed.

As discussed in Section 5.4.3, in order to execute an effective and consistent inspection program, the inspectors need to be trained in aspects relating to levee condition as well as all structural elements related to the proper functioning of a levee. The standards used should be clear and easily understood by both the inspectors and the management responsible for levee system maintenance. A guideline should be used to make sure each aspect of the levee has been looked at and that the ratings given to each item are fair and consistent. Figures 5.44 and 5.45 show the guidance document and a detail sheet within that document used to grade inspection items on levees in England and Wales. Figure 5.46 is a guidance document used to grade inspection items in France. Some countries, such as the USA, use a digital checklist form (developed by the USACE) used by levee inspectors to perform visual inspection of levees. In the USA, the Levee Inspection System (LIS) tablet is used to record the data into the digital checklist. Figure 5.47 shows methods and equipment used by various countries for levee inspection documentation. The trend is towards real time digital recording of the data. A checklist is a simple method for ensuring that the data needed for a visual levee inspection is consistently collected following a fixed procedure. The checklist for a levee inspection can be fairly extensive and detailed. A checklist will typically require the inspector to provide a series of answers to fixed questions that build-up to provide the condition assessment needed. These questions will involve simple yes/no responses or for the inspector to categorise certain features or findings on site. This method of guidance is a good way to ensure consistency of inspection but can require the assessment and recording of a large amount of data, which increases the duration of the inspection process. There is a danger that any observation from the inspection that does not fit into the checklist may be ignored, however they should be recorded in some way. If there is a question as to whether a certain item was rated correctly or has a large effect on levee performance, then the inspector should make detailed notes about the findings along with pictures and bring them back to the office for analysis and review by a professional engineer. Some items may be rated as unacceptable by the inspector based on the rating criteria, but they may not designate a levee as unsafe or pose a risk to the leveed area. These items should also be evaluated by a professional engineer and given appropriate discussion when evaluating the overall system performance.

Box 5.30 ***Example guidance documents in England and Wales, and France used for levee inspections***

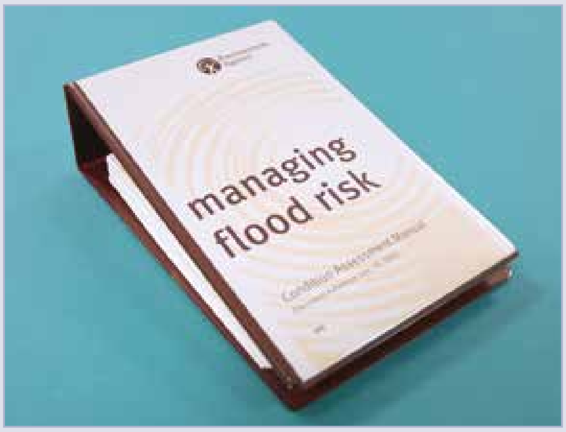


Figure 5.44

Condition Assement Manual used to rate levee inspection items in England and Wales (courtesy Environment Agency)

In the UK, the Environment Agency uses a Condition Assessment Manual (Environment Agency, 2006) containing condition grade descriptions and images used in their current method of visual inspection for all flood defences including levees.

In France, a handbook by Mériaux and Royet (2007) is, so far, the only guidance about levee inspection methodology, and is widely used by most of managing organisations as well as engineering firms.

Embankment slopes Condition 2: Good

General: Minor defects that will not reduce overall performance of the asset

Specific description: In reasonable condition. Minor defects. Minor disparate animal burrows (in clay embankment). Stable side slopes well vegetated but may be non-uniform. Evidence of minor overtopping (see insert).

Key features: Light foreign objects present but not contributing to heave or erosion. No crest parallel cracking, no loss of fines. Stone revetment at toe in good condition – only minor movement. Minor pooling of water at toe of outer face but no uplift.

Figure 5.45

Guidance document used to rate levee inspection items in England and Wales (courtesy Environment Agency)

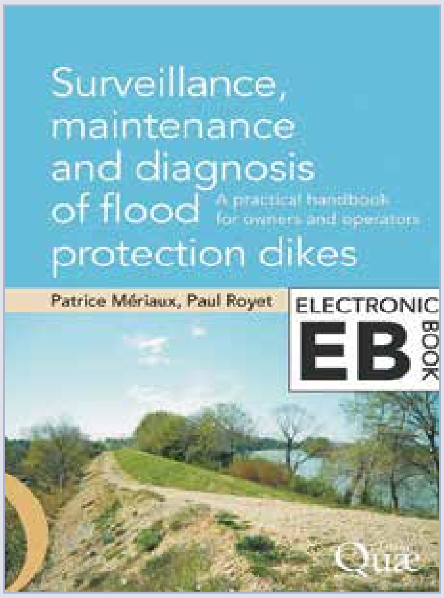


Figure 5.46

Guidance document used to rate levee inspection items in France (from Mériaux and Royet, 2007)

Box 5.30 Example guidance documents in England and Wales, and France used for levee inspections (contd)

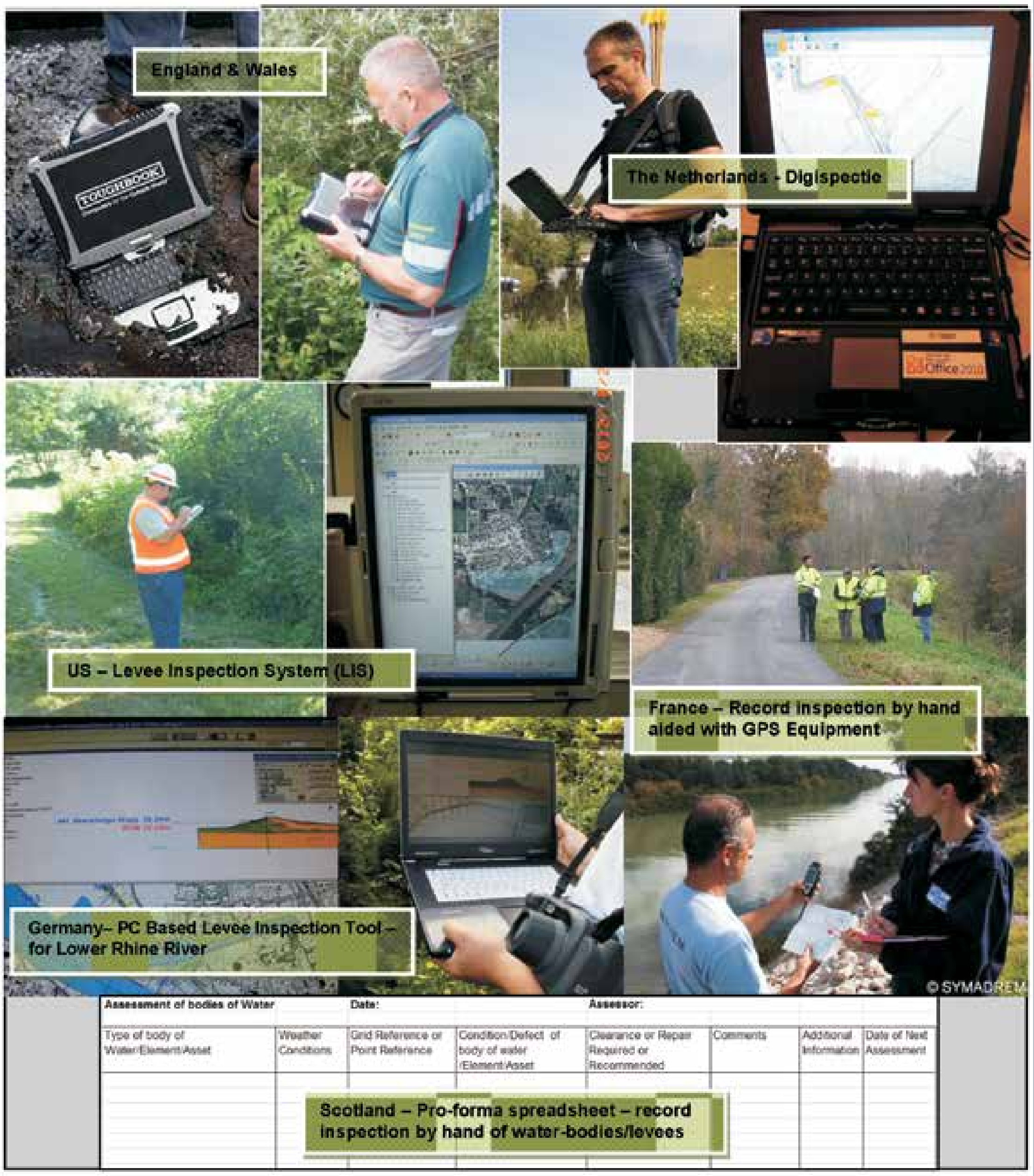


Figure 5.47 Methods and/or digital recording equipment used during levee inspections

There are other data gathering type equipment that are also being used today to create a faster and more efficient way of collecting and managing of field data. This is with the use of smartphones. Box 5.31 is an example of how this technology is being used in the USA.

Box 5.31 Use of smartphones for field data collection and analysis

Researchers at the USACE Research and Development Center (ERDC) in Vicksburg, MS have created a faster, more efficient way for collecting and managing field data using one of the most common technologies in today's market – a smartphone. Using the ERDC – developed Mobile Information Collection Application (MICA) software, data can be captured digitally, saving hours of writing forms and inputting data into spreadsheets. Most smartphones are equipped with cameras, GPS, compasses, WiFi and computer processing. Smartphones were loaded with the MICA software for use during the 2011 flooding along the Mississippi River to help facilitate quicker turn-around of information back to the USACE district offices to assist commanders with making decisions to keep citizens safe.

Figure 5.48

A USACE emergency operations worker uses the MICA application to capture levee data (courtesy USACE)

Another guidance method or standard that could be used in performing a levee inspection is a flowchart. A flowchart can provide a highly structured mechanism for assigning condition to items reviewed during the levee inspection. The increased structure and fixed pathways in a flowchart ensures greater consistency of an inspection than a textual description alone, which suffers from the ambiguous nature of natural language. Flowcharts can display the process of condition assessment across all condition grades within a single chart. This allows the inspector to understand the differences between the grades of condition more easily. The production of flowcharts, however, is a more complex process than a text description and should be extensively trialled to spot any errors and/or omissions. Training should emphasise the use of flowcharts as guidance material and not as a replacement for the inspector's knowledge. Inspectors should apply their own judgment when the on-site reality does not match up with the generalised nature of the flowcharts. Flowcharts should combine the level of assessment detail needed without becoming overly complex in structure. Some logical processes are not best represented by flowcharts, sometimes tables or matrices are more suitable. With either of these (checklist or flowchart), seasoned inspectors should provide feedback on procedures to improve them.

Once a levee inspection has been completed, an asset management system should be used to analyse the inspection results and categorise them according to their criticality in the functioning of the levee system and residual flood risk. Where items of the levee system are determined to be 'critical' for the reduction of flood risk, further investigations may be required. Examples of further investigations may include:

- expert structural assessment
- non-destructive inspections or testing
- other testing or monitoring.

See Sections 7.7 to 7.9 for details of these further investigations.

Surveys, field and/or laboratory tests or specialist analysis may be required to complement the visual inspection if issues or damage is found. Survey and technical specialists should be competent in their specific field. Experience of employing such techniques will vary with each specialism and the technical survey or inspection of different items will demand different competencies. Section 7.9 gives the details of these further investigation techniques.

Table 5.6 is a summary of the grading used by the various countries or entities within a country to rate a particular item (see Table 5.5) observed during an inspection as well as provide the overall levee system condition grade. By understanding the condition of the levee related to specific items will help the levee manager/levee sponsor or regulatory authority determine the likely performance of the levee system as a whole.

Table 5.6 ***Grading by country used for inspection items (see Table 5.5)***

France	
France does not have a standard way of condition grading. Levee managers use SIRS Digues (see Section 5.6), which has a condition grading scale built into it: 0 Does not affect the stability of the structure and is not likely to change. 1 Does not affect the stability of the structure but is likely to evolve. 2 May affect the stability of the structure. 3 Destabilisation of the structure. Other condition grades used in France:	
Netherlands	**United Kingdom**
Well, Reasonable, Moderate, Bad	(1) Very Good, (2) Good, (3) Fair, (4) Poor, (5) Very Poor
United States	**Germany**
Acceptable (A), Minimally Acceptable (M), Unacceptable (U)	Not currently available
Scotland	**Ireland**
Engineering judgment used on the condition of the levee. If a levee inspection by the local authority (LA) engineer did not reveal any item such as crest failure, subsidence, scour at the toe, areas of de-vegetation, failure, any cracking or fissures opening on the embankment, and evidence of animal burrows that needed attention then the condition of the levee is not an issue.	Very Good, Good, Fair, Poor, Very Poor

Other condition grades used in France (scale):

Nonexistent		Poor		Mediocre		Tolerable		Good	
0	1	2	3	4	5	6	7	8	9

Table 5.7 provides a list of observations that may be detected during an inspection and their related possible failure modes (note that this list is not all-inclusive). This table can also be cross referenced with Table 5.8 regarding mechanism(s) and/or failure mode(s).

Table 5.7 ***Various inspection observations and associated failure mechanisms***

Visual indicators (field observations)	**Description (examples of condition grading)**	**Associated mechanism(s) and/or failure mode(s)**	**Description of mechanisms**
Earthen levee			
Vegetation: woody vegetation that would affect or obstruct the operations or integrity of the levee and/or the presence of weeds	• no woody vegetation or weeds • minimal woody vegetation or weeds with a stem diameter of less than 5 cm (2 in) measured at 15.2 cm (6 in) above ground level • significant woody vegetation with stem diameter greater than 5 cm (2 in) measured at 15.2 cm (6 in) above ground level and/or weeds.	External erosion due to: • scouring and instability issues. Internal erosion • seepage issues.	An uprooted tree removes a substantial mass of soil and exposes the unprotected levee slope to erosion and eventual slope instability. Trees with invasive root systems penetrate the levee embankment or foundation to promote seepage and piping of embankment/foundation materials causing a void to develop and then collapse leading to the lowering of the levee crown.

Table 5.7 Various inspection observations and associated failure mechanisms (contd)

Grass (sod) cover: sufficiently dense low level vegetation for soil protection	• regular (>90% coverage) over a 152 m (500ft) levee section • irregular (75%–90% coverage) over a 152 m (500 ft) levee section • poor/none (less than 75% coverage) over a 152 m (500 ft) levee section.	External erosion due to: • overtopping • overflowing • waves • currents or tides against the levee surface. Slope instability	Lack of grass (sod) cover can cause erosion in the levee prism due to the velocities of moving water where there is no erosion resistant coverage (sod). If enough soil within the levee has been removed near the levee toe, an initiation of slope instability could occur.
Deformations: depressions and/or rutting of the crown or side slopes	• minor scattered shallow ruts, pot holes or other unrelated depressions of the crown or side slopes unrelated to levee settlement. Levee cross-section is well established and drains properly • infrequent depressions of the crown or side slopes that could pond water and impact levee integrity • depressions of the crown or side slopes that could pond water and is likely to impact levee integrity.	External erosion due to: • overtopping • overflowing. Slope instability	Deformations within the crest provide opportunities for overtopping during extreme high water events and rutting allows long-term water retention that slowly deteriorates the levee integrity and could promote slope instability.
Erosion: the removal of sod, soil or rip rap that has invaded near or into the levee prism	• no areas of erosion near or within the levee footprint • minor erosion near or within the levee footprint • erosion areas near or within levee footprint that diminish the function or integrity of the levee.	External erosion Slope instability	Continued erosion removes enough soil that promotes slope instability.
Any sign of levee slope **instability** such as a scarp	• no slides, sloughs, slope depressions, or bulges present within levee prism • minor slope stability issues will likely not diminish proper levee performance • major slope stability issues (ie deep seated sliding, cracks that exhibit vertical movement) that will affect levee performance.	Slope instability	Unstable levee slopes increase the probability of slope failure during hydraulic loading.
Cracking of the levee crown or side slopes	• minor longitudinal, transverse, or desiccation cracks with no vertical movement along crack • longitudinal and/or transverse cracks with no vertical movement along the crack but is not expected to affect the integrity of the levee. No cracks extend continuously through the levee crest • numerous longitudinal and transverse cracks that could affect levee integrity. Transverse cracks extend through the entire levee width of the crest.	Slope instability Internal erosion	Cracks in the levee create a plane of weakness that infiltrating water can exploit to initiate a slope failure event.

Table 5.7 Various inspection observations and associated failure mechanisms (contd)

Burrowing animal: holes encountered in such a density that it is an indication of an infestation that could threaten levee integrity	• minimal number of burrowing holes observed and the levee manager/local sponsor has an animal control program in place • burrows are present but are not considered an immediate threat to seepage or slope stability. Existing animal burrow control program needs to be improved • significant maintenance is required to fill numerous existing burrows, and extreme intervention will likely be required to prevent inundation of leveed area until maintenance is complete. Animal burrow control program is not effective or is non-existent.	Internal erosion due to: • collapse/lowering of levee crown • seepage. Slope instability	Animal burrows can be extensive and affect a levee embankment at several elevations and provide more than one entry or exit point through which internal erosion can be initiated with eventual slope failure.
Active seepage: observe unrepaired boils (sandbag rings still in place), wet areas on the landside due to ponding of water on the riverside and the movement of soils	• no evidence of historical seepage that may have transported material. Evidence shows that previous seepage has been repaired or mitigated. Little to no saturation of landward slope or levee toe • evidence or history of active seepage that may have transported minor amounts (limited area of small boils) of material from levee or foundation, saturated landward levee slope or toe not exceeding 25 per cent of levee height with no evidence of instability • evidence or history of active seepage that has transported a significant amount of material from the levee and/or its foundation, such as large boils/cones and/or a large are of small boils. Evidence of progression internal erosion, such as sinkholes in levee, berms or foundation. Seepage from the landward slope of levee with evidence of transporting material; saturated landward slope exceeding 25 per cent of levee height with evidence of instability.	Internal erosion Slope instability	Seepage can lead to piping of fine soil and development of a roofed void that can progress until the riverside is connected with the landside. Collapse of the roofed void could then lead to levee overtopping.
Components of a levee			
Revetment, placed stones: observe missing or displaced stones of the revetment system, irregular settlement of individual stones, presence of woody vegetation, disappearance of fill material between the stones or filter material from underneath	• in general if there are no missing or displaced stones, the condition grade is very good. Once there is an increase in the number of visually observable missing stones, the number of woody stems through the revetment and missing fill material the situation is judged worse.	External erosion	Damage of the revetment can lead to external erosion of the waterside slope of the levee, which endangers its integrity/safety.

Table 5.7 Various inspection observations and associated failure mechanisms (contd)

Revetment, asphalt: observe discontinuities of the surface, loss of a superficial sheet, fissures/cracks	• no damaged surface and absence of fissures indicates good condition. Irregular settlement, fissures and of damage indicates poor condition.	External erosion	Damage of the revetment can lead to external erosion of the waterside slope of the levee, thus endangering its integrity/safety.
Blockage or other evidence of lack of capacity of relief wells or toe drainage systems	• these systems have functioned properly during the last flood event with no sediment movement observed. Wells have been pump tested within the last five years and documentation available • no apparent damage to toe drainage or relief wells systems. No evidence of sediment movement in wells or toe drains and still function. Maintenance records are incomplete but not expected to affect integrity of levee • these systems are in need of repair or have become clogged. No maintenance records and could affect the integrity of the levee.	Internal erosion	Malfunction of landside relief wells/toe drainage systems may allow critical seepage gradients to develop and result in under-seepage and piping failure.
Walls: displacement, settlement or cracking of the wall as evidence of levee movement	• no observable movement indicates good condition. Cracking, deviations in alignment or crest level of the wall indicate poorer condition.	Instability – possibly leading to external erosion	Movement of the levee causes local differential settlement and tilting opening up gaps, which permits leakage and external erosion.
Structures associated with levees			
Culvert/discharge pipes: observe degree of corrosion of metal pipes of fracturing of concrete and clay pipes. Look for misalignments, open joints, crushed roofs, eroded or filled inverts, penetrating roots, seepage outside of pipe at headwall, loss of bituminous coating	• no breaks, holes, cracks in the culvert/discharge pipes that would result in significant water leakage. Original pipe is essentially in its original shape, joints appear to be closed, original coating in place (ie asphalt or galvanising) or are relined with appropriate material – all in good condition. These conditions have been verified using CCTV or visual inspection • small number of corrosion pinholes or cracks that could leak water but overall pipe is structurally sound and not in danger of collapsing. Pipe shape may be deformed in some locations but does not appear to be approaching a curvature reversal. Limited number of joints may have opened and soil loss may be beginning. No complete section loss in metal pipes. These conditions have been verified using CCTV or visual inspection • pipe had deterioration and/or significant leakage, in danger of collapsing or has already begun to collapse, 100 per cent section loss in invert in metal pipe.	Internal erosion	Any opening in a discharge pipe provides an opportunity for the initiation of internal erosion, enlargement of an exterior piping zone, collapse of the crest and so possible levee overtopping.

Table 5.7 Various inspection observations and associated failure mechanisms (contd)

Encroachments: trash, debris, unauthorised farming activity, structures, excavations or other obstructions that are not part of the FRMS	• minimal authorised encroachments, which do not diminish proper functioning of the levee • unauthorised (unpermitted by regulatory authority) encroachments that do not inhibit O&M, emergency operations or levee functionality • unauthorised (unpermitted by the regulatory authority) encroachments that likely inhibit O&M, emergency operations, or negatively affect the levee integrity.	Internal erosion Slope instability	Aspects of the encroachments invade the foundation soils and can provide a shortened seepage path, compromises existing seepage control measures, prevents the observation of detrimental seepage, destabilises levee slopes, or hinder flood fighting efforts.
Levee transitions with flood walls			
Erosion at transition between levee and flood wall	• flood wall is higher than levee crown • levee crown higher than flood wall.	External erosion caused by concentration of overtopping or overflowing	Flow along levee would be longitudinal until a low area at the top of the system was found and then the flows would turn concentrating high flows in the transverse direction eroding the levee and causing the flood wall to overturn due to lack of support.

5.4.4.2 Reporting the results of an inspection

A report of the inspection findings should be sent to the levee managers/local sponsors who maintain a levee system and other agencies related to the levees. Communication of findings is typically by letter to these groups with a copy of the inspection report as an attachment (see Section 5.6). Conclusions of an inspection can require subsequent actions for higher level assessments to be performed, including possibly investigations, installing instrumentation and initiating monitoring (see Sections 5.2, 5.3 and 5.5).

Archived copies of inspection reports should be maintained in perpetuity for future reference. Inspectors on follow-up inspections should have access to prior inspection and diagnosis reports to evaluate potential areas that may need to be closely scrutinised.

In the USA, an inspection checklist report is provided as an attachment to a letter to a levee manager/local sponsor. Figure 5.49 shows a page contained in this inspection report.

Box 5.32 Inspection report used in USA

In the USA, a multi-page checklist is used to perform and report the levee system inspection. The checklist includes sections of general items for all flood control works, levee embankments, concrete flood walls, sheet piles and concrete I-walls, interior drainage systems, pump stations, and flood damage reduction system channels. Each item is assigned a rating of acceptable (A), marginal (M), or unacceptable (U), and remarks may be added to explain the locations and recommendations. Copies of the completed checklist can be obtained from USACE.

4. Levee Embankments

For use during Initial, Routine (Continuing Eligibility), and Periodic Inspections of levee segments / systems

Rated Item	Rating	Rating Guidelines		Location/Remarks/Recommendations
4-1 Vegetation [7]		A	The width of the vegetation-free zone (VFZ) is as-designed, but not less than 15 feet (or extent of project real estate interest, if less) from both the landside and riverside toes of the levee embankment, at ground-level, measured to the plant stem/trunk centerline. The VFZ includes a zone that extends 8 feet vertically from the levee slopes and toes within the lateral extent of the VFZ. Within the VFZ there is little or no non-compliant (non-grass) vegetation. Approved vegetation variances are appropriately documented and the vegetation is maintained within the parameters and boundaries of the variance.	
		M	Within the VFZ there is minimal non-compliant vegetation, consisting of brush, or woody vegetation. This vegetation must be removed but does not currently threaten the operation or integrity of the levee.	
		U	Within the VFZ there is significant woody vegetation that prohibits thorough inspection of the embankment or obstructs the operations or integrity of the levee. This vegetation must be removed to reestablish or ascertain levee integrity. **AND/OR** Approved vegetation variances are not appropriately documented, or the vegetation is not maintained within the parameters and boundaries of the variance.	
4-2 Sod Cover. (If no sod cover is required for erosion protection, rate this item as N/A. See other erosion protection items below.)		A	There is very good coverage (> 90%) of sod cover over any 500 linear feet of levee. Minimal damage due to grazing or vehicular traffic	
		M	There is good coverage, approximately 75% to 90% of sod cover, over any 500 linear feet of levee. Damaged sod as a result of over-grazing or feeding on the levee, vehicular traffic, chemical or insect problems, burning during inappropriate seasons, or other determinal activity.	
		U	There is less than 75% of sod cover over any 500 linear feet of levee.	
		N/A	Surface protection is provided by other means.	

Figure 5.49 A page from the inter-coastal waterway checklist used for inspecting levees in the USA that are within the USACE portfolio (courtesy USACE)

5.5 INVESTIGATIONS, INSTRUMENTATION AND MONITORING

This section reviews and addresses the use of investigations, instrumentation and monitoring in the context of levee performance assessments. It will review existing information, uses of remote sensing technologies and assessment tools, listings of intrusive methods, along with various instrumentation and generalisation of monitoring data analysis. Specific descriptions of these techniques are provided in Chapter 7, however, Chapter 5 provides guidance regarding when and how to apply investigations techniques for use during levee performance assessments.

The discussion of investigations will address:

- what data can be acquired with each investigation method
- what failure modes are linked to the available data
- when a given investigation should be planned during an assessment process.

The main goal of levee investigation is to obtain specific data (topographic, geometric, hydraulic, morphological, geotechnical/geological etc) that can be used in the assessment of condition, performance, or in design of a levee system (new levees, or modification of existing levees). In the framework of this chapter, emphasis is placed on investigation on existing levees, mainly for assessment. In terms of investigations for levee performance assessments, there are two main issues related to the objective of the assessment, according to if it is the first one for an old levee system globally unknown, or a regular assessment for a well-managed system. In the first instance a full investigation program will have to help retrace the geotechnical composition of the levee and its foundation, while in the second some limited investigation will help check that the current condition (including possible evolution) of the levee and its foundation is compatible with its wanted performance. See Section 7.5 for detailed discussions on the various approaches to conducting investigations for existing levees.

These different types of information allow for the assessment of hydraulic structures in terms of relating to the various failure modes or mechanisms (see Chapter 3). This assessment includes the detection of weak points, including their location and quantification that can induce internal and external damage to the levee (eg internal erosion, heterogeneity, cavity formation, settlement zones, and fractures).

In addition to assessing current levee condition (see Section 5.3), a critical use of findings from levee investigations is in assessing future performance. This is principally informed by the assessment of current condition, performance models and the application of likely deterioration or damage processes. Performance and deterioration models are approximations of reality based on historical data and scientific models. Predictions of future performance require various assumptions to be made (eg asset loading, environmental conditions, third party interference). See Chapter 8 for detailed discussion on development of predictive models and analysis of future performance.

Continuous monitoring of levees through the installation of electronic and mechanical sensing equipment can also be employed as part of an intrusive investigation. The two main advantages are that the parameters can be continuously monitored and that the requirement for manual inspections by inspector or vehicle can be reduced. Continuous monitoring will produce a more accurate assessment of change over time than inspection records gathered on an irregular (eg six, 12, 24-monthly) basis. It can also record asset parameters at times of both high and low loading conditions, which can be highly significant data in modelling levee performance (see Chapter 7).

5.5.1 Investigation planning

It is important to adequately plan investigation programs. Investigative data needed to perform levee performance assessments includes:

- detailed topographic elevation data
- morphologic data as well as geotechnical data related to the levee or foundation and hydraulic conditions to help anticipate subsurface conditions
- geotechnical/geologic data for further evaluation of subsurface conditions and development of failure mode analyses parameters.

An often forgotten need of an investigation is preparing a record of the data found, determined, or observed as part of the investigation being accomplished. Protocols for preparing records should be established by the respective country, agency, and/or flood control district so that data recorded can be available for future investigations and design efforts (see Section 5.6).

As a first step of investigation planning, it should be determined what data is needed to satisfy the project objectives, and then select investigation methods that are appropriate for site conditions and meet the project data requirements. In determining what tools and techniques are to be employed in an investigation it is important to recognise what the data requirements are for the issue at hand. For example are ‘absolute’ or ‘relative’ measures required, and how accurate and precise does the required data need to be (see Section 7.9).

For any levee investigation an initial search should be conducted to ascertain what data is already available, and what data is needed to perform the required analyses. A common challenge with the assessment and investigation of existing levees are the gaps in knowledge and documentation regarding levee construction or embankment and foundation conditions. Investigators may need to ‘rebuild’ as-built documentation if it cannot be provided for assessment, which requires more extensive investigation and surveying than would be necessary if existing as-built information was available.

Investigation can be useful during an assessment of a rather well known levee system, in order to check its evolution (deterioration), or an old and poorly known levee system, with a long history of changes leading to an heterogeneous structure (see Box 5.33).

Box 5.33 ***Historical record of phased levee construction***

This idealised cross-section identifies several construction phases performed to raise this levee from 1875 through 1956. The internal geometry of these inclined layers provides valuable insight as to the continuity of soils encountered during recent investigations of the levee materials performed along the centerline of the embankment.

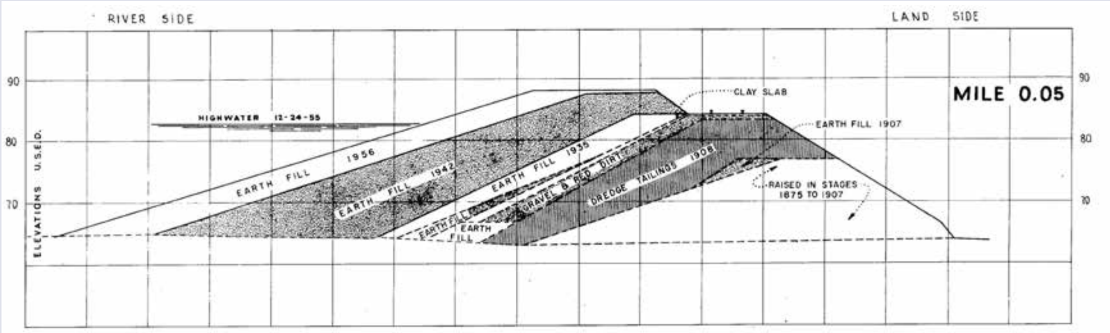


Figure 5.50 ***Typical section of Marysville, California Levee (URS, 2010a)***

The required analyses will depend on the known or suspected uncertainties about the condition or performance of the structure or flood defence system. These concerns may arise from visual inspections, past performance, or from expert judgement. There may also be a requirement to fill in 'data gaps', for example where a dataset of crest height elevations for a levee system is not complete or where the levee foundation geology is not fully known. A typical approach to conducting levee condition assessments, termed the 'tiered' approach is described further in the following section.

5.5.1.1 Investigation planning methods

It is important that methodologies applied to ascertain a levee's condition are consistent with those used for broader assessment. Both these techniques should collect data that is relevant to a risk-based analysis of their condition. Such data gathering methodologies range from visual inspections and surveys using simple techniques, through to complex testing, sampling, and remote sensing methods. Table 5.8 summarises failure modes and mechanisms and associated testing programs and investigation methods.

Many countries adopt a phased approach to levee condition assessment (also see Chapter 2 and Section 5.1) as shown on Figure 5.51.

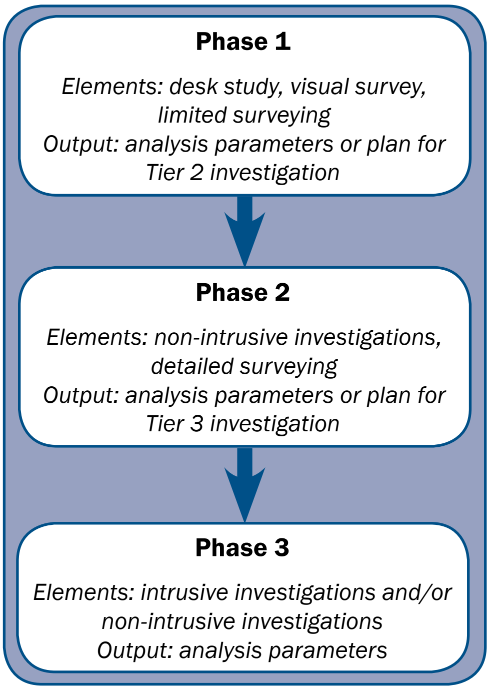


Figure 5.51 ***Example of phased approach to levee investigation (courtesy G Brandner, GEI Consultants, USA)***

Phase 1

The first phase of the classic phased approach to levee investigation consists of a desk study, visual inspection or application of limited survey methods, which is aimed at providing basic information about levees or flood control structures to inform managers about potential weak spots and deteriorating or damaged areas (see Section 7.1). Historic data can encompass a wide spectrum of information sources and will allow proper planning of ground investigations that may be needed. Gathering, collating, and interpreting historic data will help to identify gaps in intelligence and also help to identify areas of uncertainty that need further work. During this step, it is also important to identify and, if possible, locate old breaches or past performance issues in the levee system, mentioned in the historic documents. Performance during past high water events and analysis of available knowledge leads to anticipated modes of failure, which may define a focus for the types of data or density of data needed to analyse the levee and predict future performance.

The office study begins with a search of available information, such as topographic, soil, and geological maps, old river bed maps, and aerial photographs. Pertinent information on existing construction in the area should be obtained. This includes design, construction, and performance data on utilities, highways, railroads, and hydraulic structures. Available boring logs should be secured. Federal, state, county, and local agencies and private organisations should be contacted for information. Visual information, which can be gathered in the field or through aerial photography, will help to complete a bibliographic study to define the initial state (state zero) of the levee. Chapter 7.1 discusses the means and methods of gathering this data. Proper interpretation of historic data can provide a snapshot in time of performance of the levee system and its components. Using historic data will also save money by avoiding duplicative efforts and creates a baseline for subsequent exploratory efforts. The results of the literature search and visual inspection may show that information on levee geometry, topography and geology is lacking and requires further specific field investigation(s).

Phase 2

A second phase of ‘non-intrusive’ investigation aims to assess the potential damage or deterioration of levees identified in Phase 1 in more detail and to ‘trigger’ either further analysis or remedial action. A second phase investigation typically requires some investment and effort to collect new information, such as geophysical data, updated aerial photography, morphologic mapping, or topographic survey data.

Once it has been identified what data is needed, then it should be determined how it is obtained. There are many methods for collecting such data on levees. These are briefly summarised here and described in more detail in Section 7.9.5. The second step uses what are also often referred to as non-destructive testing (NDT) techniques, and include methods such as visual inspection, geophysics or remote sensing. The first step in employing non-intrusive investigations is to define which method, in relation to a specific levee problem, should be applied. Non-intrusive techniques are often considered as an economical and efficient means of collecting information regarding the condition of the levee or foundation, but may lack the refinement or detail offered by physically sampling, inspecting, and testing material collected in the field. The results of non-intrusive investigations combined with the potential risk associated with a given levee, will determine the need for additional, more detailed non-intrusive or intrusive techniques.

Phase 3

A third phase of investigation (often using ‘intrusive’ as well as ‘non-intrusive’ techniques) is driven by the need to ensure that a full understanding of their condition and level of performance can be ascertained.

Intrusive techniques required in this phase are often referred to as destructive testing (DT) techniques. In this context destructive (intrusive) testing is taken to mean any techniques that require the penetration into or removal of material from the levee for examination or analysis, or for the installation of monitoring instruments (Figure 5.52).

Figure 5.52 Destructive investigation, Rhone River, France (courtesy R Tourment, Irstea)

Table 5.8 Summary of mechanism/failure modes and potential testing and sampling programs

Mechanism(s) and/or failure mode(s)	Test/parameter	Investigation or sampling method
Mechanism: scouring, overtopping, or overflowing **Failure mode:** erosion	• embankment geometry (Section 7.9.1) • channel geometry (Section 7.9.2) • index properties (Section 7.8.3.1) • erodibility (Section 7.8.3.6).	• terrestrial surveys for embankment geometry (Section 7.9.1) • bathymetric surveys for channel geometry (Section 7.9.2) • intrusive investigations for soil characterisation and laboratory testing (Sections 7.9.6).
Mechanism: backward erosion, concentrated erosion (piping), contact erosion, dissolution **Failure mode:** internal erosion	• embankment geometry (Section 7.9.1) • channel geometry (Section 7.9.2) • index properties (Section 7.8.3.1) • permeability (Section 7.8.3.5).	• terrestrial surveys for embankment geometry (Section 7.9.1) • bathymetric surveys for channel geometry (Section 7.9.2) • non-intrusive investigations for soil characterisation and parameter correlations (Section 7.9.5) • intrusive investigations for soil characterisation, *in situ* testing, laboratory testing, and parameter correlations (Sections 7.9.6).
Mechanism: shallow sliding, deep rotational sliding, translational sliding, consolidation, creeping, settlement, liquefaction, exceedance of bearing capacity **Failure mode:** instability	• index properties (Section 7.8.3.1) • shear strength (Section 7.8.3.3) • compressibility (Section 7.8.3.4) • embankment geometry (Section 7.9.1) • channel geometry (Section 7.9.2).	• terrestrial surveys for embankment geometry (Section 7.9.1) • bathymetric surveys for channel geometry (Section 7.9.2) • non-intrusive investigations for soil characterisation and parameter correlations (Section 7.9.5) • intrusive investigations for soil characterisation, *in situ* testing, laboratory testing, and parameter correlations (Sections 7.9.6).

5.5.1.2 Structural assessment tools

It is important to perform an accurate and detailed assessment of the structural dimensions and condition of older or undocumented structural members such as floodgates, pipes and reinforced concrete flood walls. As structural steel components age, protective paint systems can fail and the resulting corrosion can reduce the integrity of the structure. Repeated cycles of loading and unloading

1 2 3 4 5 6 7 8 9 10

can result in fatigue cracking, particularly for poorly detailed or fabricated assemblies. Similarly, as pipes age the corrosion protection for corrugated steel pipes can wear away, and corrosion is likely to occur. For pipes comprised of multiple sections of either precast concrete or clay pipe, unexpected levee settlement can result in pipe separation.

Structural assessments are routinely used to gather highly accurate data via detailed measurements carried out by a qualified structural engineer, inspection consultant or specialty contractor. Typical structural assessment tools for pipes/conduits are CCTV, and sonar and laser profiling. The condition of concrete structures is typically assessed using non-destructive assessment tools such as surface hardness (Schmidt Hammer), penetration resistance (Windsor Probe), ultrasonic pulse-velocity, and impact-echo. Ultrasonic tools are available for measuring remaining metal thickness and evaluating both base metal and welds for the presence of cracks or other discontinuities on metallic elements or structures. Hand-held portable units have been in use for many years that can provide metal thickness to a high degree of accuracy, provided the unit is frequently calibrated using a metal block of known thickness. These tools are less useful if the surface is highly variable, deeply corroded or if the metal to be assessed is a casting. More recently multi-echo ultrasonic measuring devices have been in use, which can work on metal with a painted surface, accounting for the paint thickness.

In the US multiple phased array ultrasonic devices are currently being developed for commercial use in the field of steel bridge infrastructure assessment. Multiple phased array ultrasonic sensors can identify tiny cracks located within complex welds, which could easily be missed using more conventional non-destructive testing methods. The challenge until recently has been in the size of the processing equipment, which limited the technology's use to factory applications rather than the structural members of bridges. This application appears to be well suited to the assessment of floodgates fabricated from structural steel or aluminium.

5.5.2 Instrumentation and monitoring for levees

The design of new levees typically incorporates data into various forms of models to verify that the levee will perform at an adequate level without failure. For existing levees, design and construction information may be unavailable or limited, making it difficult to assess the ability of a levee system to perform at an adequate level. Risk assessments or observation of levee performance during flood fighting may reveal potential deficiencies in a levee that warrant investigation either immediately or within a given time period. All of these risk analyses and performance assessments may benefit from data that can be obtained from a variety of instrumentation systems.

Monitoring is conducted through technical measurements repeated and recorded on a regular basis at planned intervals. These measurements can use instrumentation that is installed in the levee, and/or some equipment placed on the levee for the purpose of measurement. Visual observations made during inspections, if repeated and recorded on a regular basis and then analysed, may also be considered as monitoring. The main difference between monitoring and other types of investigation (initial investigations related to design, or specific investigations related to a particular assessment process) is that the measurements or observations can be repeated in order to analyse their evolution. The analysis of this evolution (see Section 5.5.3) can give information on the occurrence of deterioration or on the actual level of this deterioration.

Information obtained from instrumentation can be beneficial for all phases in the levee life cycle, from planning, design, construction through O&M, rehabilitation and repair. The principle objectives of a levee monitoring programme, directly linked to the assessment process, are to:

- contribute to or inform an assessment of the current condition of the asset
- assist in predicting the likely performance of the asset and its contribution to reducing flood risk
- assist in ensuring that the flood defence infrastructure system as a whole functions as required
- assist in reducing or optimising maintenance procedures.

Monitoring of embankment levees is somewhat similar to dam instrumentation and monitoring, which is now common practice. The main types of monitoring measurements are:

- visual observations
- topography and displacements
- hydraulic parameters (piezometric heads, pore pressures, seepage and drain discharges).

Other characteristics can be measured and used as a basis for monitoring analysis, as they may relate, directly or indirectly, to the main properties of the levee in terms of structural or hydraulic behaviour. Techniques for such monitoring include:

- LiDAR
- geophysics
- temperature or displacement analysis through use of fibre optics
- remote sensing.

Table 5.9 details how instrumentation observations are linked to failure mechanisms, and how they are used in a diagnosis and related performance assessment. Additional information on instrumentation can be found in Section 7.9.8.

Table 5.9 Instrumentation observations and associated failure mechanisms

Instrumentation type	Observation/data	Associated mechanisms and failure modes	Description of Issue(s)
1 **Piezometer** Pore pressure cell	Elevated total head Pore water pressure	Seepage issues, potentially leading to stability failure (uplift) and/or internal erosion	Increased pore water pressure decreases effective stresses and shearing resistance along potential failure planes. Decreasing shearing resistance may ultimately lead to slope failure
2 **Survey** Ground anchors Extensometer Settlement cells/plates Fibre optic (strain)	Loss of freeboard due to settlement	Overtopping/overflowing	Long-term settlement should be monitored for new levees to confirm predicted settlement used to define required overbuild for adequate freeboard
3 **Survey**	Deformation	Slope failure/sliding	Initial deformations may be noticeable through comparison of surveys results
4 **Inclinometer** Fibre optic (strain) MEMS (GeoBeads or similar commercial application)	Internal deformation	Slope failure/sliding	Internal deformations may initiate prior to any manifestation of surface indicators that can be noticed during inspections
5 **Piezometer** Fibre optic (temperature) MEMS (temperature)	Elevated total head Changes in temperature (reduction)	Under-seepage/piping	Under-seepage can increase hydrostatic forces at the landside toe causing uplift failure or initiate piping. This could lead to progressive localised collapse
6 **Piezometer** Fibre optic (temperature) MEMS (temperature)	Elevated total head Changes in temperature (reduction)	Through-seepage/piping	Internal erosion can initiate piping, which could cause progressive localised collapse
7 **Weir box** Volumetric methods	Seepage discharge	Seepage/piping	Water from through seepage or drains can be collected into a weir box or in a fixed volume container to measure flow rate and monitor turbidity. Changes in flow rate/ turbidity can indicate development of a potential seepage failure

A programme for instrumentation and monitoring should be adapted to the structure, its foreseeable behaviour in terms of deformation or displacements and evolution of hydraulic behaviour (piezometric heads, pore pressures, seepage and drain discharges). The differences between levee monitoring and dam monitoring are related to the differences in physical characteristics of the structures and also in hydraulic conditions. Levees are often old, poorly defined heterogeneous structures, so planning for a monitoring scheme can be difficult. Designing instrumentation and monitoring schemes are easier for new levees or for improvements of an existing levee.

Another difficulty in designing instrumentation and monitoring schemes arises from the fact that many levees are not retaining water on a regular basis, which makes the response to the loading in terms of hydraulic properties difficult to foresee. Levees that are always partially loaded or frequently loaded have a more regular hydraulic behaviour, which can be monitored and analysed.

5.5.3 Analysis of monitoring data

Data gathered from an instrumented levee are meaningless unless interpreted in a rational method to evaluate what that data reveals about levee performance. Data gives indications of the performance of a levee and need to be correlated to the stresses placed on the levee (hydraulic, wave, and other loads) due to a particular flooding event. Data should also be evaluated and correlated with past performance and combined with interpretation of other types of more subjective information (eg visual observations made during inspections and flood fighting events).

Monitoring data analysis is common practice for dams and reservoirs, which is not the case for flood defence levees, mainly because of the infrequent hydraulic loading of many levees. This infrequent loading makes it difficult to establish a link between loading and structural changes, and to explain these evolutions (Mériaux *et al*, 2012). Classical analysis of dam monitoring data involves taking into account the reservoir water level, the time of the year, the rainfall, and finally the age of the dam as factors that can explain changes in the measurements. The same factors can be used in a first approach for levee monitoring data analysis, particularly for levees that are subject to water level variations.

A first level of monitoring data analysis can be done quite easily by plotting the value of the data against time, with the possible contributing factors (eg events) on the plot with a similar horizontal axis scale. In a first step, the mere existence of some structural changes is an indicator that some unwanted phenomenon may be happening. The change can be compared to limit states associated with the levee, either resulting from the design (eg settlement) or from a previous assessment. In a second level analysis, the change can be explained by expert judgement, associating all available knowledge about the structure in order to give an explanation of the change. In a third level analysis, mathematical modelling can be used to (a) find the level of correlation between the data and the causative factors, (b) remove the effects of factors such as water level, time of the year, and rainfall, leading to (c) identify other factors within the residual change in the data.

5.6 LEVEE KNOWLEDGE AND DATA MANAGEMENT

The data produced during the levee life cycle can and/or should be managed and/or used by the levee manager/local sponsor and/or a regulatory authority to assist in making decisions in regards to maintenance, inspections, flood fighting, and repairs. This data can also be used by higher level experts to perform levee performance assessments and risk analyses (see Sections 5.2 and 5.3). Well-maintained and thorough records, archives, and documents helps those with no previous knowledge of the levee system to be able to understand the system historical performance. This can aid not only in future O&M (see Chapter 4), but also any emergency response during a flood load event (see Chapter 6), flood risk analyses and levee performance assessments (see Sections 5.2 and 5.3), and potential future rehabilitation efforts (see Chapter 9). Historical data such as design documentation (see Chapters 7 and 8), and construction information (see Chapter 10) are also important to understanding the as-built condition of the levee system. Data should also be archived due to possible agency/regulatory requirements and future legal actions.

Table 5.10 presents sources, nature and types of data as well as the need for obtaining the data relating to the life cycle of a levee system. This table is not meant to be exhaustive but to provide a sample of the many types of data that can be available. The levee data can also be input into a GIS (see Section 5.6.3).

Table 5.10 *Source, nature and types of data and their purpose*

Source of data	Nature and types of data (what the source data provides you)		Need for data (link between data and their purpose)
	Nature → *How do you obtain the data?*	← *Types* *Why do you need the data?*	
As-built drawings (Section 9.3, 10.1.5)	• location of levee components • construction details • structure geometry • limits of operation and maintenance jurisdiction.	• spatial location and extent of structure • relative position of components • mapping indicating the project boundaries • levee geometry and configuration.	• to inform analyses to determine potential performance of the levee system during a flood loading condition.
Approvals (Section 9.2.4)	• sign-offs and approvals by responsible persons.	• records of approved information and actions.	• to record confirmed and approved records of processes and procedures undertaken.
Construction (Chapters 9 and 10)	• understanding how/when levee was constructed.	• construction drawings and specs • daily field reports.	• to inform analyses to determine potential performance of the levee system during a flood loading condition • to understand the vulnerabilities of the levee system based on date of construction.
Design (Chapter 9)	• designers intent and purpose (design criteria).	• calculations • memorandums • operation and maintenance manuals.	• to inform analyses to determine potential performance of the levee system during a flood loading condition.
Flood response plans (see Section 6.4)	• community emergency preparedness • existing non-structural measures that may reduce loss of life/injuries.	• flood response plans • evacuation plans • security plans.	• to determine extent of risk reduction measures.
Historical Information (see Sections 4.1)	• records of system performance during flood events.	• flood loads • specific data collected during floods • photos taken during floods • eyewitness accounts – interviews • modifications to the original levee system.	• to inform analyses to determine potential performance of the levee system during a flood loading condition • to inform revisions of system performance requirements.
Inspection reports (see Sections 5.4.4)	• condition grading of levee features • record levee defects • confirmation of proper O&M.	• digital photographs • aerial photographs • visual – arm's length inspection • remote (digital) inspection, ie pipe video.	• to determine the condition of levee components and prioritise the need for repair/rehabilitation/replacement.
Instrumentation (see Section 7.9.9)	• real time data on geotechnical condition of a levee system.	• pore-water pressure transducers and piezometers • inclinometer records • settlement plates • seismograph readings • other.	• continue monitoring • to inform analyses to determine potential performance of the levee system during a flood loading condition.

Table 5.10 Source, nature and types of data and their purpose (contd)

Investigations (raw and interpreted data)	• topography. See Section 7.9.1	• levee geometry • boundary of leveed area • riverbed geometry including bathymetry.	• to determine the condition of levee components and prioritise the need for repair/ rehabilitation/replacement • to be used in analyses to determine potential performance of the levee during a flood loading condition.
	• geotechnical investigation. See Section 7.8)	• geology – subsurface profiling • sediment type/size • permeability • shear strength • compressibility • density • stratigraphy • drill boring/borehole records • ground penetrating radar (GPR).	• to be used in analyses to determine potential performance of the levee during a flood loading condition.
	• hydraulics and hydrological. See Sections 7.2 to 7.5)	• historical flood levels/loading of levee • frequency of flood loading • wave run up • rainfall.	• to be used in analyses to determine potential performance of the levee during a flood loading condition • to understand the impact on the levee system due to interior development and increased runoff.
	• morphodynamics how the river or sea is evolving, particularly close to the levee. See Sections 7.2 to 7.5)	• sediment deposition/erosion.	• to be used in analyses to determine potential performance of the levee during a flood loading condition.
	• assets in the leveed area. See Section 2.1.5.2	• people • houses • public service • crisis management • industrial – financial, economical, pollution • agriculture • environmental • patrimonial (historic, architecture etc) • roads.	• to understand the value of assets in leveed area and justify value of levee system • evaluate of potential economic damages • evaluate of likely loss of life • assess of other potential impacts.
	• climate change. See Section 2.1.4)	• rainfall patterns • loading event frequency • water levels • freeze thaw cycles • wetting/drying cycles.	• to understand potential changes to the incumbent environmental factors affecting the levee.
Leveed area (see Chapter 2 and Section 5.2)	• potential consequences of levee failure.	• remote sensing data • residential property type and location • population data • inundation maps • designated environmental areas • critical infrastructure • commercial property • agricultural land.	• to determine and analyse receptors potentially at risk from flooding.

Table 5.10 Source, nature and types of data and their purpose (contd)

Operation and maintenance manuals (see Sections 4.1 and 9.3.6)	• instructions and standards of care for the levee system • gage operations guide • vegetation management guidance.	• seeding mixtures • manpower/resources to install closures • lubrication regime of pumps and pipe closure devices • testing to measure the remaining resistance of electrical insulation • vegetation management • perform trial closure installations.	• to assess the adequacy of O&M activities in providing continued level of monitoring, maintenance and performance.
Previous assessment results (see Sections 5.2 and 5.3)	• previous risk analysis and condition assessments to see if conditions have changed.	• records of previous assessments and the criteria used • previous periodic inspections.	• understand levee performance • funding purposes for repair/ rehabilitation/replacements • assess adequacy of previous assessments and analyses.

5.6.1 Need for records, archives, documentation

The levee manager/local sponsor and/or regulatory authority are tasked with making decisions regarding maintenance, modifications, inspections, flood incident response and repairs. Having as much data as possible that can be accessed in the decision making process will enable all involved parties to better evaluate the full historical and potential future trends of the levee system. Combining historical data such as as-built or flood performance information with more recently recorded data (paper or electronic inspection reports), levee performance assessments or monitoring data, will give the involved parties the information needed to more accurately make decisions and carry out levee performance assessments and risk analyses.

There are three stages that a typical document evolves through over the course of its life:

1. **Documentation:** the active recording of data, or living documents continuously being updated.
2. **Records:** data that was recorded previously and referenced on a continuous basis.
3. **Archives:** the permanent storage of records not meant for alterations but serving as a permanent duplication of data in the event primary records are lost.

A data management system should be implemented to record and store large amounts of information that are documented throughout the life cycle of a levee. This system should be capable of storing all data types while maintaining easy access to the data. A classic paper-based system, a digital computer file system, or a smart data system in the form of geospatial information housed on a GIS digital platform could be used. The management system could also be a combination of these three systems that can be used to more effectively serve the levee manager/local sponsor and/or regulatory authority.

It is prudent to continuously be mindful of the overarching purpose of recording data. Historical data is meant to contribute to the overall understanding of a levee system while providing a preserved record to all parties that may require access to it in the event of an emergency or even changing of personnel within the levee management and/or regulatory authority responsible for the levee. This will help while determining how information is organised, where information is stored, and on what medium (material holding data, such as paper, disk, or server) the information is published.

There is no single medium that will guarantee the integrity of recorded information. For example, buildings can be destroyed losing all copies of hard data, computer systems can crash, hard drives can fail, and digital data warehoused in buildings may also be destroyed in catastrophic events. The potential to completely lose valuable information should be pre-empted by properly backing up all living and referenced documents while archiving all permanent records in a separate remote location.

Many levees were built before the advent of computers and most records are in paper form. If a digital file has yet to be produced of a paper document, it should be scanned into a digital format, and saved electronically to prevent any further loss of data due to the deterioration of paper over the years. It is ideal to have published data records accessible by multiple staff members, however this carries the potential for unwanted alterations and security concerns. If the file structure and storage method is not consistent with industry standards, added work would be required to foster beneficial collaboration. Other points of concern include accessibility during emergencies. Having only one medium or one storage location could limit access to necessary documents during a crisis situation (during a flood event) where there are network or power outages. So, levee managers/local sponsors and/or regulatory authorities should develop a reasonable crisis management plan should such an event occur.

The ability to alter documents should be left to specified personnel with the technical capacity to make revisions and read only access for all others. Read only access will allow information to be assessable by large numbers of individuals while protecting the integrity of the documents. It is also important for there to be some type of quality assurance in place to ensure there is accuracy when revisions to levee records are made. These revisions could be to a single type of data or to data contained within a database. Regardless, quality assurance should be performed to limit errors in the data.

5.6.2 Levees and information systems

A levee information system should have the capacity to house multiple file types that may be used for different activities related to the levee. This system could be capable of holding maintenance reports produced by a transactional system used by a levee manager/local sponsor, risk analyses ran by the regulatory authority, links to written reports etc. Each file type would have the potential to be different, but all file types need to be stored in an information system. Boxes 5.34 and 5.35 illustrate specific types of information system used in the USA and France by levee managers/local sponsors and/or regulatory authorities, respectively.

The goal of a levee information system is to have a 'single version of the truth' for the overall levee system and its components. Information technology is always evolving and information systems need to evolve accordingly in order to benefit from the new technology, but also in order to stay usable in the long-term.

As documents progress through their stages of life, from a living document to reference data, and finally archived to permanent records, a database system should be established that is capable of tracking and linking to multiple file types along this progression.

Basic qualities of an information system that will benefit the transfer of information most effectively would include:

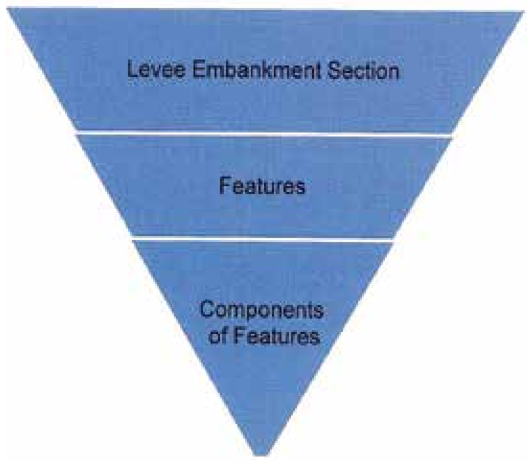
- managing paper and digital file locations
- linking transactional systems used by levee managers/local sponsors and/or regulatory authorities that automatically generate maintenance work orders to the information system database
- capability of populating information to a GIS-based application (see Section 5.6.3).

Information systems are ideally built to transfer information to large numbers of people. Having unrestricted access poses a potential security concern. The possibility of deterioration, unwanted modification, or unwanted access to a levee management system can be mitigated by establishing a system with multiple levels of user privileges. Establishing user profiles and general public access platforms is a potential method that may be used to manage user participation. It is also good practice to implement a program to track changes made to documents, and by whom they were made to reduce the risk of unwanted changes from occurring.

Box 5.34 Information management system in a tiered approach format, USA

Some levee managers/local sponsors in the USA use computerised maintenance management systems for the proper operation and maintenance of their levee. One such system is the Hansen Information Management System. This system is used by the Metropolitan Sewer District (MSD), which is the levee manager for the Metro Louisville, Kentucky Levee System. This database management system automatically generates work orders for staff based on a predetermined maintenance schedule by the levee manager/local sponsor. Figure 5.53 provides a view of how the levee embankment as a complex asset is divided down into the lowest level for managing work related to the components of a feature within a levee section. Figure 5.54 is a screen shot from within the digital Hansen system.

Types of activities performed on the **levee embankment** and **other components** of complex assets. The appropriate level to which the levee embankment or other complex asset should be broken down for tracking are the following:



First tier = levee embankment section (station to station, closure to closure, closure to PS etc). Features would be length, height, width, geotechnical make-up, cover etc. Section would be broken at a physical break point such as a road closure, pump station, or other feature

Second tier = features would be multi-use path, signage, encroached features, toe drain system, mowing areas, crossings, penetration, gate etc

Third tier = components of features would be toe drain manhole, toe drain pipe segment, gate components etc

Figure 5.53 Tiered levee management system – levee components (courtesy MSD – Metro Louisville, Kentucky levee local sponsor)

Figure 5.54 Screen shot of the Hansen Information Management System (courtesy MSD – Metro Louisville, Kentucky levee local sponsor)

Box 5.35 ***Information management system, France***

In France, using a structured approached for analysing and designing information systems, a first study – called strategic diagnosis – was carried out in 1998 by Cemagref (now Irstea) by interviewing levee stakeholders on their current practices and their expectations in terms of information systems. At the time of this study, control of the safety of levees by the State was not organised. Currently, a regulation involves activities of some State services. Table 5.11 is a systemic view of leveed floodplain management and planning activities (for example in Loire, France). Each cell of this table corresponds to specific management and/or planning activities based on a distinct information system, computerised or not.

Table 5.11 ***Systemic view of leveed floodplains management and planning activities in Loire, France, 1999***

Scale (actor)	Levee and river bed management and planning	Floodplain management and planning	Flood risk management and planning
National/regional (ministry/regional authorities)	Works programming Maintenance scheduling	Hazard management plan Prioritisation of objectives of protection	Rescue plan Flood warning system
Levee system (levee manager)	Levee and vegetation management	Zoning plans (land-use policy)	Real time levee inspection plan
Public works (civil engineering units)	Topography and levee visual inspection Maintenance works	Public river domain management	Emergency works on levee weak points

5.6.3 GIS for levee management

Many levees are long linear structures with a significant amount of available data. Because of this, the levee related descriptive data should be linked and be accessible through geo-referenced/spatial data platforms. Compared to conventional information systems, computer aided design (CAD) systems or digital mapping systems, a GIS is a more powerful tool to handle spatially-referenced data. One of its main advantages is to be able to manage, combine and analyse a great variety of spatial data, using topological and spatial analysis functionalities.

The information in a GIS system can be interconnected between multiple levee stakeholders/actors (including technical staff, levee managers/local sponsors, emergency response staff, local/state/regional/national government and/or regulatory authorities, and various other groups), which would provide an easy way for data that is collected and stored to be shared. Accessing this data through a GPS enabled mobile computer while running a GIS digital platform helps inspectors quickly determine where they are, what component they are inspecting, the component's history, previous actions performed at a particular location, or if an observed issue has recently materialised. This will ensure data is accurately collected and can be used with reliability by someone with no prior knowledge of the levee to perform any sort of analysis (risk, slope stability, seepage or other types of analyses) on the levee. A GIS platform enables all data recorded with spatial referencing to it to be linked to each location of interest.

Computer-based levee management systems (LMS) that use a GIS platform have the potential to include data treatment models to analyse flood risk assessments through the use of geospatial layer comparative analysis. Creating an LMS to include spatial elements that are then fed directly into a modelling system enables the consequences of inspection results or scenarios of levee maintenance strategies to be explored. These analyses provide quick assessments for levee managers/local sponsors and higher level authorities to predict potential flood hazards and the ability to make timely decisions to appropriately warn parties in a leveed area of the risks due to potential breach and/or flooding.

Within a GIS/database LMS tool, there should be a spatio-temporal reference system created to include both surficial (X, Y, Z co-ordinates) and linear reference points (stationing points and distance to landmarks or benchmarks) to geo-reference all of the data. Being able to follow the evolution of a levee in order to establish a reliable diagnosis of its performance is a major concern. What this means is that GIS stores information about all events that occurred on or near the levees. Past events remain stored in the database even when they are over because the data model allows recording their beginning and ending validity dates. Such a tool helps safeguard the levee informational capital in an efficient way.

It provides a solution to the traditional problem of forgetting past situations, in particular when levee inspectors retire or leave their jobs. Boxes 5.36 to 5.39 are all examples of country specific GIS/database LMS tools.

Box 5.36 National GIS/database LMS tool, USA

In the USA, the National Levee Database (NLD) was created in 2006 to be an "authoritative database that describes the location and condition of the nation's levees, and the potential consequences behind those levees". This geospatial levee database has over 450 attributes, business related data, and media files of any type. The NLD is also used as a way to communicate this information in a transparent way using standard web services to the public/levee managers/local sponsors and the nation's leaders.

Figure 5.55 Screen shot of the USACE NLD, 2013 (courtesy USACE)

Box 5.37 GIS/database LMS tool, State of California, USA

The State of California initiated the development of a state wide geospatial database of all of the river levees in 2005. The project was a joint effort between FEMA and Department of Water Resources (DWR), USACE, with assistance from a few USA based consulting firms. The framework for the California DWR levee database was guided by FEMA LIS and the USACE NLD. Data gathering for input into the database required information be obtained from all counties in California since levee jurisdiction is divided among federal, state, regional, municipal and local entities.

Figures 5.56 Screen shot of the California DWR levee database, 2013 (courtesy T Schweitzer, Atkins Global)

Box 5.38 *GIS/database LMS tool, UK*

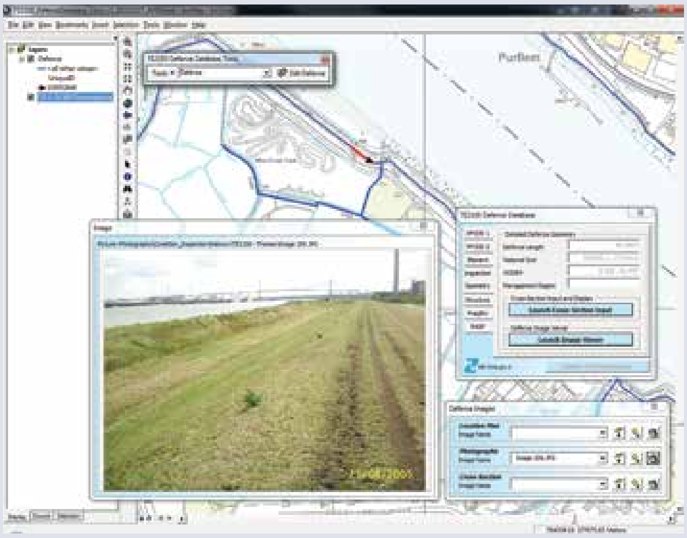
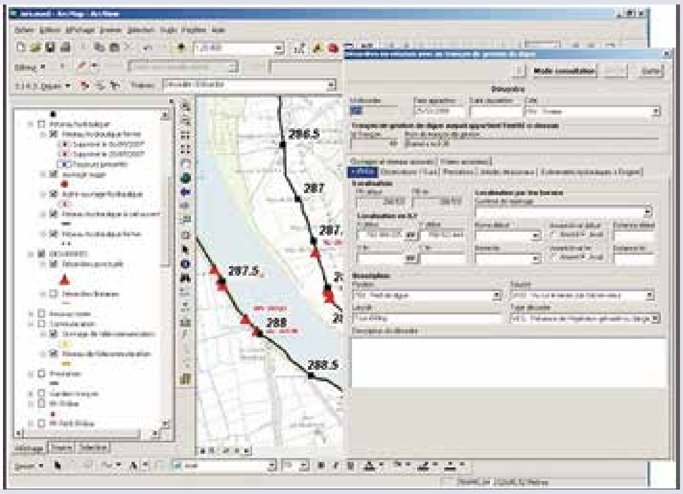


Figure 5.57 shows an example from theThames Estuary 2100 flood defence database. This system was built within ArcGIS to store and maintain all data related to the river defences along the Thames Estuary, London. The system was developed to manage and share data among a number of sub-consultants who were working on data gathering and modelling of the defence systems. The system uses customised forms and tools to display data that is housed in an ESRI personal GeoDatabase (a spatially enabled Microsoft Access database for ArcGIS) and a logical folder structure (for cross-sectional drawings and photos).

Figure 5.57

Screen shot of the Thames Estuary 2100 project defence database (courtesy Environment Agency)

Box 5.39 *GIS/database LMS tool, France*



Another example of GIS-based LMS is the SIRS Digues used in France. Three levee managers that operate more than 1000 km of levees (about 15 per cent of the total length of river levees in France) have used the SIRS Digues in an operational way since 2005. This management system is linked to outside database features. For example, pictures or AutoCAD files stored outside the SIRS Digues database can be linked to an object in order to refine its description. A secondary application was also developed as an independent module using MS Access run-time for data capture and update, eventually outside the office by, for example, the levees inspectors.

Figure 5.58

Screen shot of SIRS Digues, GIS-based LMS developed in France (courtesy SYMADREM)

Developing an LMS in pursuit of a complex relational database that is displayed on a GIS platform will require a defined component hierarchy. Collecting large amounts of information to be housed within the information system database and attaching geospatial reference data to it will be a long a tedious process. The potential benefits of doing this will far outweigh the time and effort to do so.

A geospatial database for levee management can give levee managers/local sponsors a view of the levee system from a large area perspective to review a single levee system component. This can be done within this type of database by constructing data models to aid levee managers/local sponsors in automating maintenance needs and tracking management issues (such as vegetation and nearby landowner encroachments) all of which provides ease in making decisions related to their levee. One approach would be to construct a data model that considers a levee as a pathway component in a source-pathway-receptor (SPR) type data model (see Figure 2.2). In such a system, the levee condition and inspection data can feed directly into modelling systems that enable the consequences of inspection results or scenarios of levee maintenance strategies to be explored.

Building a data model for such a system is very involved and therefore is a costly task involving a number of different experts from different themes. It also requires knowledge of the existing data that is held and the input requirements for the anticipated modelling tasks. In a system such as this, the data models for each individual theme can be treated separately provided the terms used to coupling these themes are agreed.

An example overview of such a complex LMS is presented in Figure 5.59. From this high level overview it can be seen just how complicated the task of data modelling is for a theme cross-cutting levee management system such as the one based on the SPR model. However, given that the simple LMS should include much of the pathway components, the benefits of developing the source and receptor components are very tangible.

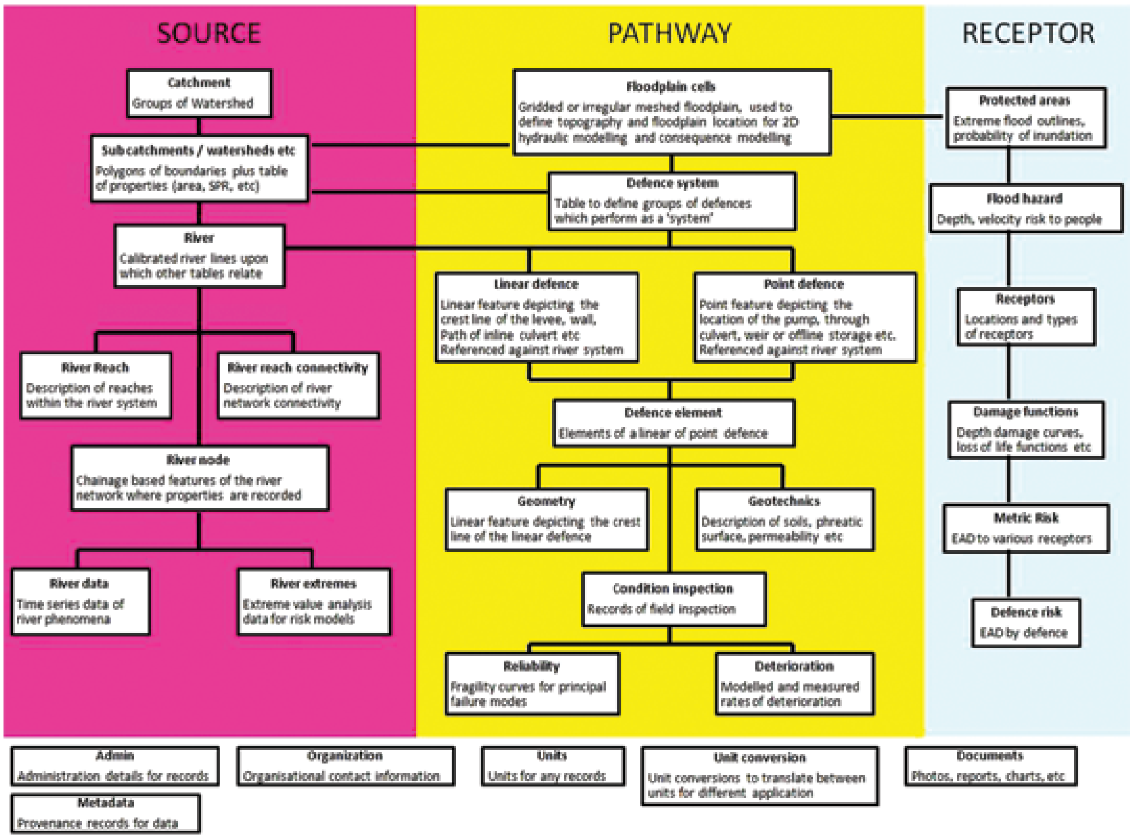


Figure 5.59 ***Overview of a levee management system based on the source-pathway-receptor (SPR) model (courtsey Michael Panzeri, HR Wallingford)***

Some other data models, which can be constructed to assist a levee manager/local sponsor and/or regulatory authority, could be to:

- estimate possible failure points and probabilities of failure with various scenarios
- estimate possible flooding due to high levels of the water body (rivers or coastal) that is external to the levee
- understand conditions that can result in leveed area flooding due to excess interior rainfall
- track the location, and possible conflict of modifications/actions, by third parties in, under and/or through a levee system. Some modifications could include such items as elevation changes to the crown of the levee (related to settlement or other changes), levee alignment changes, extending or shortening a levee, and/or adding new features such as closure structures or drainage features
- model the effects of structural or management changes on interior environmental features such as wetlands and conservation areas
- model the effects of adding or removing features that alter interior runoff characteristics as these changes may affect runoff or pumping control structures.

An LMS can also support flood risk modelling. One of the key components in flood risk models is the defence systems and how they may perform under load, so it follows that developing an LMS that is directly compatible with a flood risk model offers significant benefits to the levee manager/local sponsor and/or regulatory authority.

One of the key benefits of a risk model is evaluation of levee system risk. Using the risk model, it is possible to estimate the probability, and then the expected annual damage (EAD) associated with flooding of the leveed areas of the floodplain. The outputs from such models can feed directly into budgeting programmes to help to distribute funds to the most 'at risk' regions of the floodplain.

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6 Emergency management and operations

Courtesy Árpád Szentiványi

CHAPTER 6 CONTENTS

6 EMERGENCY MANAGEMENT AND OPERATIONS

Chapter 6 explains the management of levees and the role of levee managers in flood emergencies.

Key inputs from other chapters:

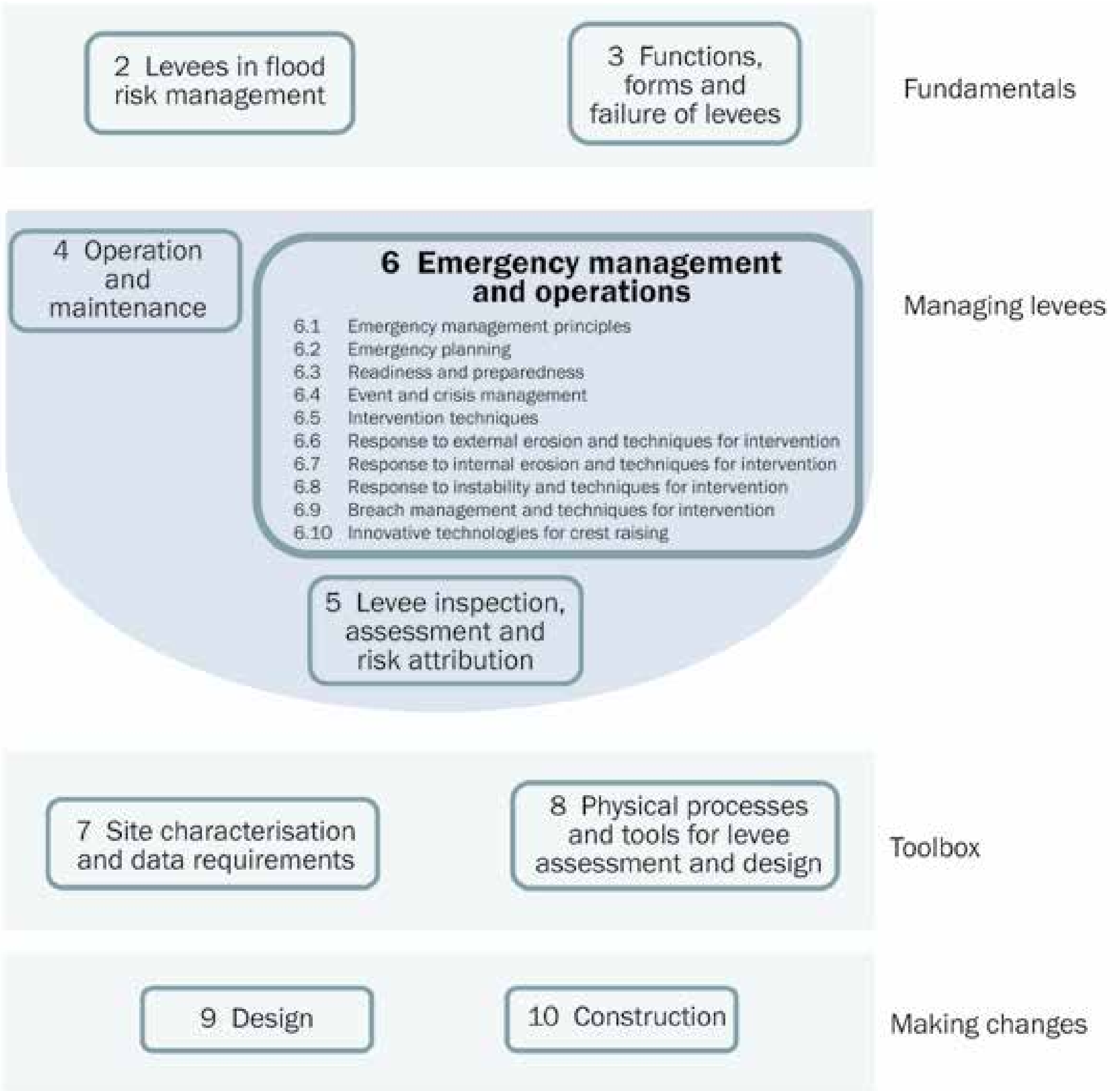
- Chapter 3 ⇨ **functions, forms and failure mechanisms**
- Chapter 4 ⇨ **integration with operations and maintenance**
- Chapter 5 ⇨ **levee performance assessments and flood risk analysis**
- Chapter 8 ⇨ **breach and inundation modelling**

Key outputs to other chapters:

- **post event data and analysis** ⇨ Chapters 4, 5, 9 and 10

Note: The reader should revisit Chapters 2 and 3 throughout the levee life cycle for a reminder of important issues.

This flow chart shows where to find information in the chapter and how it relates to other chapters. Use it in combination with the contents page to navigate the handbook.



CHAPTER CONTENTS AND TARGET USERS

This chapter is divided into ten sections, providing an overview of the emergency management life cycle, which consists of preparedness, response, recovery and mitigation. The chapter focuses mostly on preparedness and response with minor information regarding recovery and mitigation, pointing the reader to other chapters of the handbook for more information on these two topics.

Emergency management principles

Section 6.1 introduces the principles of emergency management and sets out the scope of the chapter. The emergency management life cycle is introduced with details given for the four phases of the cycle.

Emergency planning

Section 6.2 presents the actions necessary to be undertaken prior to an event. Details are given related to required inputs for emergency planning, emergency action planning, flood response plans for levees, staff and security actions in planning, maintenance and update of plans, and data management and use in emergency operations.

Readiness and preparedness

Section 6.3 describes those tasks and activities that are necessary to build, sustain, and improve operational capabilities. Details are given related to training and exercises as well as public awareness.

Event and crisis management

Section 6.4 describes the varied role of levee managers, emergency managers, and responsible officials and their areas of responsibility. This section also details preliminary response activities, full response activities, and post response activities.

Intervention techniques

Section 6.5 introduces the concepts of intervention techniques in light of potential failure mechanisms. Details are given related to flood response equipment, supplies, and activities.

Response to external erosion and techniques for intervention

Section 6.6 presents techniques for intervention in response to external erosion. Details are given related to measures used for levee raising, external erosion protection, and protection from overtopping or overflow.

Response to internal erosion and techniques for intervention

Section 6.7 presents techniques for intervention in response to internal erosion. Details are given related to measures used for reducing infiltration, increasing seepage path, and reducing hydraulic gradient.

Response to instability and techniques for intervention

Section 6.8 presents techniques for intervention in response to instability. Details are given related to measures used for reducing steepness and inclination of slope, reducing uplift pressure, and reducing saturation of the levee.

Breach management and technologies for intervention

Section 6.9 presents techniques for intervention in response to breaching. Details are given related to the stages of breach development and measures for management of each stage.

Innovative technologies for flood crest raising

Section 6.10 presents innovative techniques useful for crest raising. Several representative methodologies are highlighted in this section.

6.1 EMERGENCY MANAGEMENT PRINCIPLES

An emergency is an incident, caused by natural or manmade hazards, that requires the levee manager (owner/operator) and other response partners to take action. A disaster is an event associated with severe property damage, deaths, and/or multiple injuries. Emergency management is the interdisciplinary field dealing with the strategic processes used to avoid or reduce the impacts from such events. The primary goals of emergency management are to save lives, prevent injuries, and reduce property damage. The main objectives for a levee manager, during a flood or storm, to attain these general primary goals are to avoid levee failure and to inform other response partners of the possibility of failure of the levee system or exceedance of the water level.

All risk management actors need to be prepared for emergencies in order to properly respond to such events. The actors are widespread and encompass the levee manager, emergency manager, response personnel, public safety and health organisations, utilities, government agencies and officials, private companies, and citizens. Although multiple actors should be involved in emergency management activities the level of involvement differs for each. Not every actor participates in every phase of an emergency event. However, every actor should apply the general principles of emergency management presented in this chapter as well as participating in communication between the actors during all phases of the event.

As demonstrated throughout history, breaching and overtopping of levees and subsequent flooding of adjacent areas always remain a threat, regardless of the height of a levee system and safety programmes. The burden for this flood risk mitigation is not borne solely by the levee manager, but is shared by all stakeholders for developing emergency management mechanisms to achieve and maintain an effective response. Levee managers play a key role in preventing flooding disasters by planning and preparing for emergency response and assisting in flood response activities.

Emergency preparedness and management is a process that is a necessary component of an effective risk reduction programme aimed at reducing the loss of human life and reducing property and critical infrastructure damage. Response activities during a flood event are intended to reduce the chances of levee failure, whereas planning and mitigation activities before and after a flood event are intended to reduce the consequences resulting from inundation. This chapter addresses basic principles and general guidelines that support effective emergency management practices.

Scope

Figure 6.1 shows the emergency management life cycle, represented as a continuous process that can be grouped in four general areas: preparedness, response, recovery, and mitigation. Each phase of the life cycle is discussed as follows:

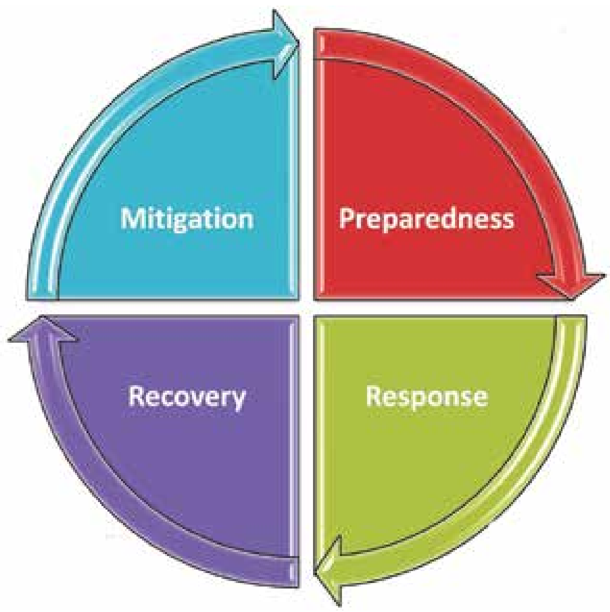


Figure 6.1 Emergency management life cycle

- **preparedness:** includes efforts focused on planning, organising, training, equipping, exercising, evaluating, and implementing corrective actions to ensure effective co-ordination during incident response and overall enhancement of all emergency management capabilities. Levee managers are generally responsible for assuring that the flood defence performs as intended, but co-ordination and planning with all affected stakeholders is essential to achieve the optimal risk reduction for the public. Preparedness activities and measures include the levee inspection process as well as development of communication plans, training of emergency response teams, development of emergency action plans, exercises, procurement and management of disaster preparedness supplies and equipment as well as levee inspection tools. Emergency preparedness plans are intended to provide a user-friendly protocol for managing emergencies in a co-ordinated and effective manner
- **response:** includes the mobilisation of emergency services and first responders to the disaster area and mobilising levee monitoring teams. Response activities include efforts conducted by core emergency services and first responders (eg firefighters, law enforcement, and emergency medical personnel), as well as special emergency operations (eg search and rescue, water rescue), emergency support teams, and emergency contract support. For the levee manager, the efforts range from actions taken to strengthen or raise (if not harmful to some other area) the levee as well as efforts (eg evacuation) by other emergency responders within the leveed area. Activities in the leveed area often involve co-ordinated action between the levee manager, flood risk management authorities and other stakeholders
- **recovery:** includes efforts aimed at returning the levee to full readiness and restoring the affected area to a normal state. Recovery activities are concerned with issues and decisions that should be made after immediate response needs have been addressed. Recovery typically begins after the emergency has subsided, but some recovery activities may be concurrent with response efforts. In this phase, permanent repairs to any damage to the levee or integral parts of the flood defence system would be made. In general, recovery involves individual, private sector, non-governmental, and public assistance programmes that focus on restoring economic activity, rebuilding community facilities and housing, addressing long-term relocation and care of the affected population, re-employment, repair of critical infrastructure, and other measures for community restoration and economic recovery
- **mitigation:** focuses on preventing hazards from developing into disasters in the future, as well as reducing the effects of disasters when they occur. Mitigation efforts are commonly associated with long-term measures for reducing or eliminating risks and/or improving the levee to a better performance level. Mitigation measures can be structural or non-structural. Structural measures involve technological solutions, such as levees or upstream flood discharge reduction. Non-structural measures include land-use planning, legislation, insurance, regulation, and risk communication to the public. Mitigation activities should be informed by risk assessment efforts. Mitigation measures can be influenced by lessons learnt from previous events. For example, areas of weakness identified during a prior flood event or during an inspection or assessment may be strengthened in preparation for the next event, thereby reducing the reliance on flood response efforts for defence. Measures may be implemented before, during, or after a flood or other incident. Several actions can be implemented to control or reduce existing risks. For example, measures dealing with improving the functionality of the levee by increasing the height, width, or bank protection, or curtailing new developments within the community (eg road construction, zoning or building code changes). Prevention measures can be very effective in areas that have not been developed or are in early phases of development. By implementing prevention measures, such as open space preservation and floodplain management, future development can be directed to minimise the risk from known hazards, while maintaining other community goals.

The emergency management life cycle applies to both general flood risk preparedness and management issues and also specific issues that can be useful to place the role of the levee manager in perspective. The emergency management life cycle is presented again in Figure 6.2 with specific information regarding the role of the levee manager and development of material in this chapter and the rest of the handbook.

The information presented for the rest of this chapter, detailing the role of the levee manager, is focused on the preparedness and response phases of the emergency management life cycle. Preparedness is detailed in Sections 6.3 while response is detailed in Sections 6.4 to 6.10.

This chapter does not include information about:

- **recovery activities:** these are mostly focused on final repairs of the flood response system and addressing consequences of the event, which involve the levee manager but to a lesser extent. These topics are covered in Chapters 9 and 10
- **mitigation activities:** these are mainly focused on improving the resilience of the flood response system or the community, which also involves the levee manager to a lesser extent. These topics are covered Chapters 4 and 5 with subsequent links to other chapters.

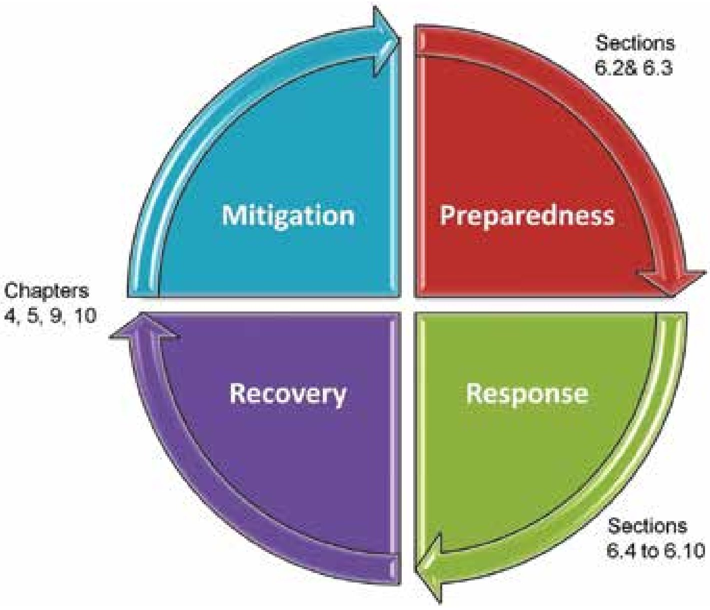


Figure 6.2 Topics addressed in Chapter 6 and other chapters of the handbook on the emergency management life cycle

6.2 EMERGENCY PLANNING

The actions taken in the initial stages of an emergency are critical to ensuring the function of the flood response system, saving lives, and reducing resulting consequences. Only through emergency planning in advance of an event can effective strategies for prevention/deterrence of risks be developed (FEMA, 2010).

6.2.1 Inputs for emergency planning

6.2.1.1 Emergency management and levee failure

Understanding how levees fail is important in the development of emergency preparedness and planning efforts. These concepts are equally useful in the development of effective strategies to facilitate the implementation of potential repair solutions addressing levee failures. It is the levee managers responsibility to understand failure modes to better mitigate disasters. The most common modes of failure are seepage, overtopping, scour/erosion and slope instability.

Emergency preparedness planning for levees focuses on actions that may be considered before and during a flooding event that would supplement normal operating procedures undertaken by the levee manager (Chapter 4). Ideally, planning efforts should also address issues related to delegation of authority and responsibility, with updates incorporated as necessary to account for changes in personnel.

Emergency planning will cover other key aspects such as potential assembly or staging areas for flooding events, location of earth borrow sites, and procedures for maintaining records of equipment, manpower, and supplies. However, as no plan can guarantee that a levee system will not fail under all circumstances, levee managers are encouraged to work with local public safety officials and interact with them during their planning, for example, by providing information about the risk associated with levees. Emergency planning may also consider short-term situations that could arise during the life of the flood defence system. For example, if a culvert that runs through a levee is being replaced, a preparedness plan may be needed in case flooding arises during construction, when levee integrity is compromised.

Traditional failure mitigation strategies are discussed in more depth in Section 6.5. Chapter 3 contains detailed information regarding potential levee failure mechanisms.

6.2.1.2 Risk identification

Levee managers should be aware of the risks associated with the functioning of the levee. Identifying where a levee is likely to overtop or where the levee is weak is key to planning how to employ resources during an event (details of approaches to levee performance assessment and flood risk analysis are given in Chapter 5). This will reduce, but not eliminate the risk to the leveed area. To further reduce the risk, the consequences of failure need to be analysed and minimised as well (discussion on risk analysis and risk attribution in Chapter 5). This risk reduction can be achieved through a variety of measures and instruments (Section 2.2). The levee manager can participate in these activities and assist local officials with reducing the consequences of inundation by understanding and operating the levee correctly.

If a threat to a levee is identified but a failure cannot be averted due to lack of resources, accessibility to expedient methods, or lack of time for their implementation, then the most likely action for the levee manager would be to co-ordinate with the appropriate individuals and organisations to evacuate the population at risk. If the levee manager is capable of addressing the vulnerability of the levee, the corresponding actions will be implemented to strengthen it and reduce the chances of failure. This may or may not be done in conjunction with the evacuations.

Understanding the flood risk, including inundation dynamics (Section 6.4) and the consequences of a levee failure or limitation is essential for decision making, particularly in time of a crisis. One of the most difficult decisions to make during an event includes intentionally flooding one area to save another area of higher consequence. Difficult decisions can be co-ordinated before the flood by designing levees with overflow sections or floodways, or pre-chosen levee segments to breach. Implementing them is still a complex decision, but can be made more acceptable by communicating and co-ordinating with all stakeholders involved before, during, and after flood events. See Box 6.12 for a case example, where the decision was made to intentionally breach a levee to activate a floodway as a measure to minimise flood risks. Deciding such measures during the actual flood without prior design and analysis is generally a bad idea, as consequences on other parts of the system can be worse than the locally avoided ones. The analysis of the flood (may include modelling) should be conducted at a larger scale than the local levee system, because of the influence of the levee system on a large part of the river, upstream, downstream and on the other bank or on the coast because of spatial variability in the wave and water level loadings.

6.2.1.3 Flood inundation mapping

Flood inundation maps are a valuable resource primarily to emergency managers, however levee managers can also benefit from this information. For more information about how to develop flood inundation maps, see Chapter 8. The levee manager may or may not participate in the development of these maps, but may use them as a key resource during a flood event.

The following information describes how flood inundation maps are useful for each cycle:

Preparedness

- threat and type of risk
- population within risk area for evacuation or monitoring
- evacuation procedures (determine routes to be taken)
- shelters for evacuees

- population groups with special needs (eg schools, hospitals, nursing homes, jails, prisons, physical handicaps, senior citizens, foreign language speakers)
- agricultural areas with livestock
- facilities with hazardous materials and nuclear power facilities
- infrastructure within risk areas such as power, communication, transportation
- how the notification and warning process will be addressed
- community outreach and who will be responsible for public information
- command and control system used to co-ordinate response and location
- scope of damage, property destruction, deaths/injuries in an area.

Response

- serve as one of the primary documents to implement the plans
- determine actions to be taken based on current circumstances
- anticipate resources needed and stage them as appropriate.

Recovery

- compare inundation map with actual map post-flooding event
- incorporate inundation map in after action reports
- identify challenges and successes with information identified on inundation maps
- recommend any changes, additions or deletions to inundation maps for future planning and response.

Mitigation

- determine land use before the emergency
- identify areas of flooding risk during levee failure
- share with other entities that oversee land use issues.

So, inundation maps should present, for different scenarios, the sequence of inundation with the extent of the flooded area, water depth, water speed and raise rate. The choice for the inundation scenarios to be modelled and presented depends on a risk analysis of the levee system, presented in Section 5.2.

6.2.2 Emergency action planning

Levee managers are an essential participant in the emergency action planning process and may be responsible for both emergency management as well as levee operation and maintenance (O&M). Each actor involved in emergency response should have their own emergency action plan, which should be co-ordinated with the other actors. Emergency action planning and the resultant emergency action plan assists the levee manager in making decisions before an event (in contrast, 'flood response plans' are more focused on the activities of the levee manager on the levees than emergency action plans, which have a broader scope of activities).

Effective plans tell those with operational responsibilities what to do and why to do it, and they instruct those outside the jurisdiction how to provide support and what to expect. Emergency action plans should address a variety of issues, including:

- communication protocols/back-up plans with local emergency operations centres and the general public
- individual roles and responsibilities during an emergency
- supplies and materials that may be needed to support emergency operations
- co-ordination with evacuation plans.

It is important that the levee manager co-ordinates extensively with the appropriate emergency managers and other stakeholders to ensure a successful emergency response. There should be no ambiguity regarding who is responsible for major tasks. This enables personnel to operate as a productive team more effectively, reducing duplication of effort and enhancing the benefits of collaboration. Successful plans are simple, flexible, are frequently updated and exercised often.

Specific emergency action plans are derived from the aspects of the disaster life cycle: prepare, respond, recover, and mitigate. Integrating these key areas as part of a co-ordinated planning effort will lead to improved overall preparedness for the levee manager and other relevant parties.

- **prepare:** preparedness planning enhances the effectiveness of the flood defences beyond the normal operations and maintenance carried out by the levee manager. Routine operations and maintenance (Chapter 4) are needed for a flood defence to achieve a fully functional state. These routine actions are included in the operations and maintenance (O&M) manual, and do not need to be duplicated in an emergency action plan
- **respond:** response planning provides rapid and disciplined incident assessment to ensure a quickly scalable, adaptable, and flexible response to a threatened or damaged levee (Section 6.4 and Chapter 9). This assessment is based on observations made during event-related inspections (Section 6.4 and Chapter 4)
- **recover:** recovery planning (not discussed in detail in this Chapter) provides for a near-seamless transition from response activities to short-term recovery operations. This may include restoration of interrupted utility services, re-establishment of transportation routes, and the provision of food and shelter to displaced people as well as temporary and permanent levee repairs
- **mitigate:** mitigation planning (not discussed in detail in this Chapter) focuses on reducing disaster impacts through sustained actions that can reduce long-term risk. All mitigation planning efforts should be integrated into a co-ordinated strategy, linked to all of the other emergency planning aspects.

6.2.2.1 Emergency operations centre(s)

The severity of an incident (eg flood, levee failure) primarily determines the extent to which the response stays entirely under the purview of the levee manager or gets escalated to progressively higher levels. This escalation often results in some form of a centrally-led and controlled entity such as an emergency operations centre (EOC). The levee manager needs to be aware of this potential in order to provide or retain information, support and co-ordination.

An EOC serves as a central command and control facility with lead responsibility for carrying out the principles of emergency preparedness and emergency management – or disaster management functions – at a strategic level during a disaster. An EOC is responsible for the strategic overview of the disaster, and does not normally directly control field assets, instead making operational decisions and leaving tactical decisions to lower commands. A common function of all EOCs is to collect, gather and analyse data, make decisions that reduce life loss and property damage, maintain continuity of the organisation, within the scope of applicable laws, and disseminate those decisions to all concerned agencies and individuals. In large emergencies and disasters, the EOC also acts as a liaison between local responders and multiple government jurisdictions.

The key function of the EOC is to ensure that those who are responding at the scene have the resources (eg personnel, tools, and equipment) they need for their response efforts. EOCs may be organised by major functional disciplines (eg fire, law enforcement, medical services), by jurisdiction (eg federal, state, regional, city, county), or by some combination thereof. Boxes 6.1 and 6.2 show examples of typical emergency response command structure. The key feature is that the response is based on a tiered system (in this example three tiers: operational, tactical, and strategic). Note that the levee manager would be involved at the lowest tier (operational) but not necessarily with the other two tiers.

Box 6.1 *Example response command structure in the UK*

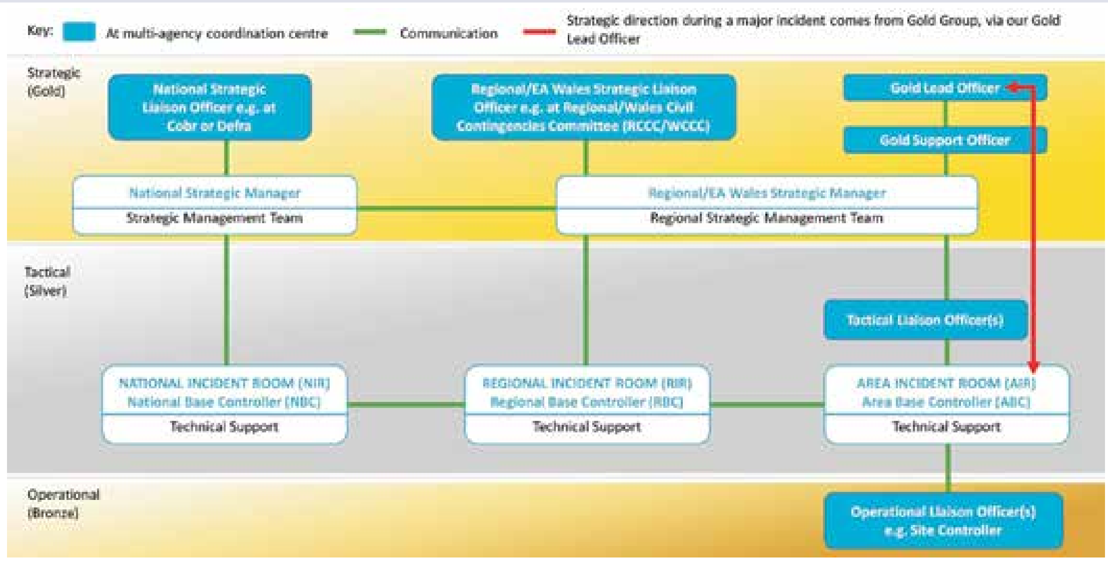


Figure 6.3 *Response command organisation in the UK*

Box 6.2 *Example response command structure in France*

In France, management of protection structures against floods is regulated by several texts:

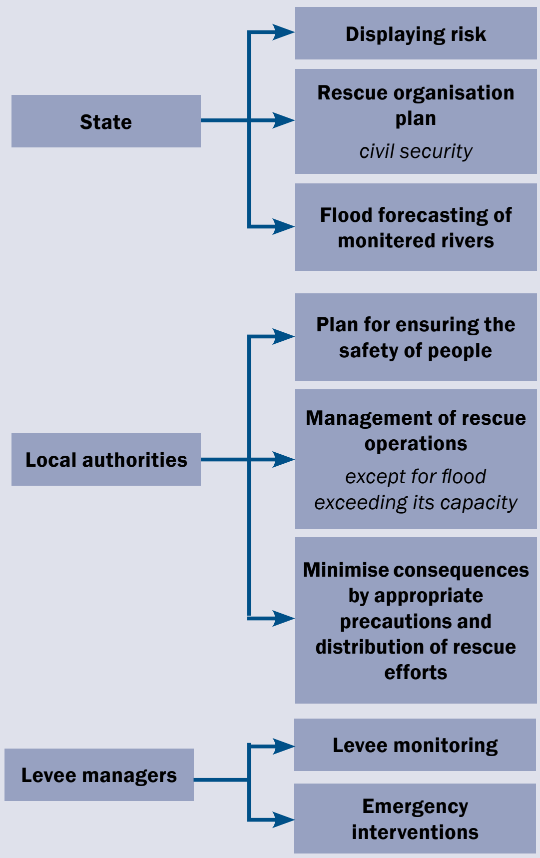
Decree No. 2005-1157 of 13 September 2005 made under the Act of August 13, 2004 for the modernisation of civil security, which provides that "any public or private person contributing to the general protection of populations, is able to ensure permanent missions assigned to it ... prepares its own management organisation of the event...." Furthermore the law prescribes the obligations of the mayor of a local authority (population safety plan) and prefects (rescue organisation plan, forecasting for monitored rivers...)

Decree No. 2007-1735 of 11 December 2007 on the safety of hydraulic structures, which requires managers of levees, the development "of the written instructions in which actions are set to monitor the levees in all circumstances and those relating to its operations during floods.... "

Art. L2212-2 of General Code for local authorities which gives police power to the mayor for:

- preventing by appropriate precautions, and for ceasing by the distribution of rescue needed, accidents and disastrous plagues and pollution of any kind, such as fires, floods, levee failures
- providing all emergency assistance measures, rescue and, if necessary, to cause the intervention of higher authority (prefect).

Sharing of responsibilities between State, local authorities and levees managers during flood periods is summarised in the flow chart.



6.2.2.2 Developing evacuation plans

While levee managers may or may not be directly responsible for developing an evacuation plan, they are encouraged to participate in the planning process and maintain close contact with appropriate governmental agencies during emergencies. They should provide timely and accurate information on levee conditions and the consequences of any failure, given previous studies of inundation and the associated mapping. This co-ordination will assist government agencies in making the appropriate decisions regarding evacuations (see the case study in Box 6.3).

Government agencies and local authorities are generally responsible for declaring the need to evacuate a given area and will, in many instances, already have evacuation plans in place. However, if levee managers are unsure about this, they should verify with local authorities whether a plan exists. Development of evacuation plans should be done in co-ordination with levee managers. This will ensure levee monitoring protocols during emergency conditions and notification procedures for communicating levee status to emergency response personnel are conducted effectively. A case example of how levee monitoring techniques can provide early warning in managing flood emergencies is provided in Box 6.4.

Box 6.3 A case study on the importance of evacuation planning, Storm Xynthia, France (2010)

Summary

Having an evacuation plan with a clear command structure is essential to an effective response. Failure to communicate threats and quickly evacute people can be fatal. Political support and land use planning are also important to reduce flood damages and risk.

On 28 February 2010 at 02.00, the storm Xynthia hit the French Atlantic coast. The storm surge combined with the high tide and large waves caused flood defences to fail along the coastline from the Gironde (Bordeaux) to the Loire Estuary. A significant amount of land (>50 000 ha) was consequently flooded and 47 people died as a result of the storm. Most people died due to the flooding. A number of people died as a result of the storm itself (storm debris). The French departments of Vendée and Charente Maritime suffered the most. Some parts of the departments Gironde and Loire Atlantique were also flooded. The damage figure was about €2.5 bn (EUR). The estimated return period of this flood is around 100 years (based on historical records). It is impossible to give a precise return period for the storm. There is a lack of historical water level measurements available to give a more accurate estimation, furthermore the water level is not the only governing factor, which also includes wind direction.

Disaster management

The most important part of disaster management failed, the storm surge warning was not understood by the disaster management authorities and the public. Meteo France (French national weather service) had clearly provided a warning for the storm on all the TV networks and also given storm surge warnings. But the weather maps of Meteo France that were shown on TV provided no information on the risk for flooding. A small symbol may have been enough to alert the local population to the risk of flooding. Meteo France is not responsible for flood warnings. Local water levels have to be calculated by the local (department and municipal) authorities. Local authorities have to give the warning to the public. As the population prepared for high winds and not for flooding, this proved fatal. They closed windows and (electric) shutters. Electric shutters cannot be opened during a power blackout or flooding.

France has a number of laws restricting construction on the coastline (1985) and in areas prone to flooding (1995). However, since 1990 many houses were built along the coast in flood prone areas. These areas were protected by flood defences that are adequate for agricultural land but not for new housing areas. Maintenance costs for flood defences are covered by local organisations or private owners. A flood may hit a larger area than the parties concerned covering the maintenance costs. So maintenance costs and reconstruction costs are not covered by all beneficiaries.

A number of reasons contributed to the fatal aspects of the floods:

- building licenses for flood prone land were given by local government, elected officials (mayors), contrary to spatial planning laws
- buildings constructed since 1990 are usually only on the ground floor. Buildings from 1960 up till 1980 usually had the living quarters about two metres above soil level
- insurance companies often give a rebate (related to theft) if shutters or steel bars are installed on property owners' windows.

Lessons learnt

Flood warnings should be given in such a way that disaster management services and the general public can understand them and can evaluate which action they can take. This means a simple and explicit message (ie no technical jargon). It is important that both professionals and the public understand the same message. It has to be clear who gives which warning, who analyses flood risk and who is responsible for the communication with the public.

During storms some equipment can fail including water level gauges. Also communication links can fail. Contingency planning for failure of equipment is necessary. Redundancy in measuring devices is needed. Managing flood defences asks for strong, independent, local organisations with a very clear mandate for maintenance, new construction, financing and the inspection. All beneficiaries have to pay for the maintenance. Costs and benefits need to be proportional for all interested parties to reduce the risk of overdue maintenance.

1 2 3 4 5 6 7 8 9 10

Box 6.4 ***A case study on the use of 'smart dikes' and visualisation surfaces as part of managing flood emergencies, UrbanFlood, European Union***

UrbanFlood, a European project funded under the EU seventh framework programme has investigated the use of sensors within levees to support online early warning systems and real time emergency management. Including such 'smart dikes', as part of early warning systems (EWS), can play a crucial role in mitigating flood risk by detecting potentially unsafe conditions and predicting the onset of a catastrophe before the event occurs. Also, it provides real time information on the behaviour and strength of a flood defence structure during an event.

UrbanFlood has investigated and demonstrated at pilot sites in Germany, the Netherlands and the UK the feasibility of remotely monitoring dikes and floods, whether from nearby offices or from other countries and continents through the secure use of internet-based technology. The systems that process and use the sensor data, such as models of the levee structure, failure mechanisms, breach development and the resulting flood inundation, along with the necessary visualisation software, have all been linked using internet technologies. Discussions and demonstrations of the visualisation of the results on multi-touch surfaces (Figure 6.5) has indicated that the approach, which allows co-ordination of all relevant information, may be particularly attractive to emergency management organisations.

Figure 6.4 ***Fibre optic cable in geotextile fabric being laid in levee to measure ground strains at a pilot site (courtesy Victoria Bennett, RPI)***

Figure 6.5 ***Multi-touch surface in use to bring together levee sensor information and flood and emergency management simulations (courtesy UrbanFlood)***

6.2.3 Flood response plans for levees

As mentioned in Section 6.2.2, *flood response plans* are more focused on the activities of the levee manager on the levees than emergency action plans, which have a broader scope of activities. Flood response plans constitute a critical element to address necessary specific actions to be taken in order to help ensure that a levee provides the design height of a system during flood events. Flood response plans may encompass a variety of areas including not only emergency operations conducted on the levee, but also maintenance, engineering, and key support operations such as contracting, equipment, facilities, and communications, and the levee manager is advised to co-ordinate extensively with local and national authorities and other members of the community.

A key component of a flood response plan is the development of an emergency notification flow chart to establish who will be notified by whom and in what priority. The flow chart should include names and essential contact information (eg home, office, mobile). The flowc hart should also include emergency management agencies that need to be notified at various flood stages. Staffing of flood response is also an issue to be addressed in the plan and ensuring adequate personnel are available to operate 24 hours a day should be considered.

In order to get help from other actors and to inform them, it is strongly recommended that levee managers compile a directory including telephone numbers for the area's emergency operations centre, local contractors, flood response supply and equipment vendors, hospitals, railroad/highway departments, police and fire departments, and any other critical numbers.

As a basic planning recommendation, the flood response plan should include annotated drawings describing flood defence system features and potential areas of concern during a flood event. The list should clearly note:

- low areas
- areas subject to boils
- areas of known seepage
- areas of recent rodent activity
- alternate access points to the levee (should the primary become impassable)
- locations of drains that should be checked for closure
- available sources and locations of sandbags, pumps, and other supplies.

Along with this information, levee managers should include a detailed table of all of the locations of flood defence system features that may need to be closed such as floodgates, flap gates, and other closure structures and the organisation that is responsible for these closures. This table should denote the river level or other indicators that would signal that each of these flood defence system features needs to be closed. Also, the flood response plan should clearly describe protocols to notify the corresponding highway or railroad agencies responsible for closing roads or railroad tracks running through levees and flood walls.

Flood response plans should be published on paper and also can be published on the internet, or could be a communication of both for the widest distribution (see Box 6.5 for an example of a web-based flood response plan, and Figure 6.6 for a sample flood response plan outline).

Box 6.5 *Example of web-based communication of flood response plan to the public*

Figure 6.6 Example of web-based flood response plan (courtesy City of Coquitlam, Canada)

6.2.4 Including staff and levee security in planning

Security issues should be addressed in the context of the overall threats facing the levee.

To ensure all personnel are aware of established security measures and protocols, including proper procedures for reporting suspicious activities, levee managers may choose to develop a security plan. The availability of a security plan can contribute greatly to the improvement of security as it details roles and responsibilities, physical and cyber security requirements, co-ordination with law enforcement officials, and procedures for reporting suspicious activities.

The scope and content of a security plan should be commensurate with the size and complexity of the levee system. The plan should clearly outline employee responsibilities as they relate to security, and should also address co-ordination mechanisms with the corresponding law enforcement jurisdictions.

Basic elements of the security plan may include:

- co-ordination with law enforcement agencies
- reporting and managing security incidents
- physical security
- communications and cyber-security
- employee and contractor identification
- security contracting
- co-ordination with emergency/operational plans.

In addition, the plan could address any security issues associated with visitor access, recreation areas, tours or other routine activities. The plan should also address the linkage between security activities and the levee's overall emergency preparedness plan.

6.2.5 Maintenance and update of plans

Emergency preparedness plans should be reviewed periodically, and after actual events or exercises, levee personnel should closely examine actions taken to determine whether they were effective and efficient. It is also beneficial to review the plans after events triggering its activation, after unusual or unexpected incidents, or after review of the plan by another actor.

Periodic plan updates should include verification that sources of emergency equipment and supplies, contact names, and telephone numbers are current. Updates should also incorporate a review of evacuation routes and emeregency shelter locations since these may change over time.

Given that floods may occur decades apart, it is important that any pertinent information be recorded for use in future planning efforts. Debriefing sessions should include all participants, to allow valuable feedback on lessons learnt, bearing in mind that the organisational arrangements and division of tasks between actors may change before the next flood. Information obtained during these sessions may include actions that worked well, areas for improvement, and recommendations to resolve any outstanding issues or concerns.

6.2.6 Data management and use in emergency operations

Information about a levee system is gathered over time to support the objectives of asset management. Managing data and keeping organised records about points where emergency action, previous breaches have occurred is important because these may indicate points of weakness in the levee.

These weak points will need to be more closely monitored during routine inspections and subsequent high water events. This information may also be used to inform decisions about permitting encroachments on the levee and for prioritising corrective actions on the levee.

Box 6.6 Example flood response plan template, Ireland

Flood emergency plan structure

Cover page

Plan revision list

Executive summary

Table of contents

1) Introduction to the flood emergency plan.
2) Area of operation and flood history – risk assessment.
3) Roles and responsibilities.
4) Flood warning stages and action plan.
5) Information management and the media.
6) Appendices.
    - a List of contacts.
    - b Maps.
    - c Field Equipment, facility resource list, main buildings.
    - d Sandbag/Flood proofing policy and procedures.
    - e Evacuation and vulnerability registers.
    - f Incident report form and flood records.
    - g Traffic management.
    - h Recovery and clean-up operations.
    - i Flood forecasting and warning (system details).
    - j Safety, health, and welfare considerations.
    - k Training and testing of flood emergency plans.
    - l Flood emergency plan distribution list.
    - m List of definitions.
    - n Public information.
    - o Mutual aid agreements with other local authorities.
    - p Details of instructions for temporary flood defences.
7) Agency specific procedures.
    - a Gardai.
    - b Local authority (fire, civil defence, engineering).
    - c Permanent defence forces.
    - d HSE.
    - e Support services.

Figure 6.7 Example flood response plan template from Ireland

Design of emergency repairs should take into consideration the levee composition and history (see Chapter 3 and, for data management, Section 5.6).

Table 6.1 provides a sample of all the types of data that can be used during emergency operations as well as indicating links to other relevant portions of the handbook where more information can be found. See Chapter 5 for how data will be managed and accessed.

Table 6.1 Data useful during emergency operations

Type of data	Description of data and use in emergency operations	Links
Operations records	Records may include, but are not limited to: • dates and notes on operation of pump stations and gates • dates and notes on trial installations of closure structures • preparation for floods and other emergency events • protocols for stockpiling materials and carrying out emergency drills. Use: • information to reference when patrolling the levee.	Chapter 4
Past performance data	Past performance data (also called points of distress), loading at the time distress was noted, and any effect on the levee will be important information for the maintainer to be familiar with. Include points where the levee required emergency action to prevent breach, reinforcement, as well as documentation regarding any permanent fixes. Use: • information to reference when patrolling the levee.	Chapters 4 and 5
Data/results from inspections, safety assessments, surveys	Information collected from previous work provides: • levee and foundation stratigraphy • geology and geomorphology • material properties. Use: • knowledge of likely failure modes • areas that do or do not need patrol/survey • selection of emergency measure.	Chapter 5
Flood response plans	Flood response plans include reference information for how to react during an emergency situation and contingency plans. Details on these plans can be found in Section 6.2. Use: • reference when completing trial closures of closure structures/ emergency preparedness drills • reference when acting in emergency response capacity • reference key points to be checked during flood related inspections • reference solutions to be used when a problem is detected during a flood related inspection (Section 6.2).	Section 6.2, Chapter 4
Contact information	The levee manager should have to hand contact information for: • any contractors working on the levee, past or present • flood response personnel • personnel required for emergency drills or trial closures (including those with access to stockpiled materials or required to install closure) • local authorities and emergency managers • local news media • stakeholders (particularly those affecting large populations like office buildings, hospitals, prisons etc) and community leaders affected by levee failure. Use: • manager knows who to contact if there is a problem • manager knows who to contact to complete emergency preparedness exercises.	Section 6.2, Chapter 4

6.3 READINESS AND PREPAREDNESS

Emergency preparedness efforts that include the entire spectrum of stakeholders lead to more resilient communities. Flood risk preparedness responsibilities are usually shared in some way between national and local agencies, private sector stakeholders, and the general public. Although it is not possible to completely prevent or mitigate every hazard that poses a risk, emergency preparedness efforts can help to reduce potential impacts of disasters by taking certain actions before an event occurs (US Department of Homeland Security, 2012).

Preparedness includes undertaking the tasks and activities that are necessary to build, sustain, and improve operational capabilities to prevent, protect against, respond to, and recover from an incident. Preparedness is a continuous process that involves:

- development of emergency plans
- assigning and training staff who can assist in key areas of response operations
- identifying resources and supplies that may be required in an emergency
- designating facilities and equipment for emergency use.

Levee managers should develop the appropriate level of preparedness regarding potential security issues, and also general risk management, particularly under conditions that stress the system. Facility staff need to be fully trained in assessing, observing, and reporting potential security vulnerabilities and suspicious incidents. Flood events can increase the attractiveness of the levee system as a potential target for security incidents. This is also the case for levee managers to be aware of general risk management.

6.3.1 Training and exercises

Important practical considerations and physical limitations that could be easily overlooked in a plan should be identified during exercises in which staff of the levee management organisation are trained to operate, maintain, and patrol the levee system. Periodic training and/or exercises are necessary in order to demonstrate how to operate the closure structures, patrol and inspect the area during a flood, and respond to sand boils and other deterioration and damage processes. Training and exercises also inform staff how much time and resources are necessary to complete certain tasks.

As general guidance, basic training and exercise activities should include:

- physical operation of features specific to the protection system (eg sluice gates, pumping stations, closure structures)
- notification of emergency response personnel
- test of communications and back-up communications system
- mobilisation of monitoring teams and monitoring flood defence system features
- basic flood response techniques
- co-ordination and control (eg between volunteers, patrols, operators, nearby levee districts, highway department, state emergency operations centre)
- dissemination of information to the public.

6.3.1.1 Training

During an emergency or disaster response, it may be necessary to assign staff to jobs other than those that they normally perform, or use personnel or volunteers normally not working for the levee manager organisation. So, it is critical that adequate training be provided in order to enable them to respond effectively to a levee emergency. Some personnel may already be employed within the community, but others may be recruited specifically for service in emergencies. Regardless of employment status, they should be recruited, assigned, and trained for their levee related tasks before an emergency event occurs. When possible, they should be included in exercises that enable them to practice these tasks under simulated emergency conditions so that when an actual emergency occurs, they can seamlessly transition into their new role. Training of all personnel should also include building awareness of health and safety issues.

6.3.1.2 Exercises

Emergency response on a levee or levee system can be complex and challenging and may require exercises (see Box 6.7 for an example of a full-scale exercise) for personnel in order to mount an effective response. They are typically categorised in two main types: **discussion-based exercises**, and **operations-based exercises**.

Discussion-based exercises are normally used as a starting point in the building-block approach of escalating exercise complexity. These types of exercises typically highlight existing plans, interagency/inter-jurisdictional agreements, procedures, and assist in developing new ones. Discussion-based exercises are valuable tools for familiarising personnel with expected capabilities and responsibilities. Discussion-based exercises may include seminars, workshops, tabletop exercises, and games.

Operations-based exercises represent an elevated level of complexity. They are used to validate plans, policies, agreements, and procedures reviewed and/or developed through discussion-based exercises. They can clarify roles and responsibilities, identify gaps in resources needed to implement plans and procedures, and improve individual and team performance. Operations-based exercises are characterised by actual reaction to simulated events, response to emergency conditions, mobilisation of resources, and commitment of personnel, usually over an extended period of time. Operations-based exercises may include drills, functional exercises, and full-scale exercises.

Documentation of exercise activities is important to identify shortfalls that may exist in planning and co-ordination, training, personnel, equipment, and facilities. A structured review or de-brief process can provide valuable feedback on the effectiveness of policies and procedures, identify areas for improvement, and give suggestions to correct deficiencies. Lessons learnt during exercises should be incorporated into subsequent training sessions, the emergency preparedness plan, flood response plans, or security plans.

Box 6.7 Full-scale exercise example, Exercise watermark, UK (2011)

Exercise watermark (2011) was the largest and most successful civil defence preparedness event ever held within England and Wales. It was a flood exercise with live play, took place between 7–10 March 2011, and involved more than 20 000 individual players across the resilience community. It provided a solid test of the nation's flood readiness.

The exercise involved ministers at the Cabinet Office Briefing Room, the Welsh Government, more than 10 government departments, 14 local resilience forums, and over 40 separate playing locations while locally delivered exercises involved a futher 34 groups. It demonstrated the capability to manage the response to a national flooding emergency.

Managed by Defra, supported by the Welsh Government, and delivered through the Environment Agency, *Exercise watermark* set out to test new arrangements against a severe flood scenario within England and Wales. These included the national flood rescue arrangements, Multi-Agency Flood Plans, new flood warning codes, and the Flood Forecast Centre as part of a range of initiatives implemented since the severe floods of 2007. The scenario included surface water, fluvial, reservoir and coastal flooding with breaches of flood defences, and a reservoir dam as part of the exercise.

The interim report was produced in June 2011 and contains 28 recommendations for planning, delivery, and review, and 31 proposed recommendations for learning outcomes from the exercise. These were high level recommendations and were relevant to exercise planning, emergency preparedness and incident response. There were also recommendations for community, local, and national levels.

6.3.2 Public awareness

The best examples of well-supported levee systems are in jurisdictions where levee managers and emergency managers have ensured that local businesses and citizens understand the flood hazard and the importance of the flood control system. While the levee manager may not be required to carry out public awareness activities, they are an important part of the community involvement process. Levee managers may be able to promote a greater awareness of key issues through publications and planned public meetings, as described here:

- **provide public materials:** many individuals, especially those living within a leveed area, will benefit from reading brochures detailing local flood protection. Levee managers might also release annual newsletters or newspaper articles. Information should be presented on the following topics:
  - how the levee system functions to defend the leveed area and its limitations

- consequences resulting from levee failure and exceedance of their limits
- historical overview of past floods and experiences, emphasising that historic scenarios may not represent what will happen in the future
- flood response plans and procedures – how the community can contribute
- local flood evacuation plans.

- **schedule public meetings:** unless there is an actual flood, attendance at public meetings on flood control may be minimal, especially if the public is not properly informed on flood control. Levee managers may find it useful to combine such meetings with discussions on local industry or other issues, or to raise certain issues during community events, such as annual or special public awareness programs
- **awareness of adjacent systems:** levee managers need to understand and communicate to potentially affected stakeholders and the community how adjacent sections of levees or components on private property impact the larger system. Even though these components might not be situated within the area of responsibility, the community could still be flooded if adjacent systems do not operate properly.

See Box 6.8 and Box 6.9 for examples of stakeholder engagement and awareness.

Box 6.8 Example of flood mapping and stakeholder engagement, Risk MAP Program, USA

The Federal Emergency Management Agency (FEMA), an agency within the United States Department of Homeland Security, co-ordinates the federal government's role in preparing for, preventing, mitigating the effects of, responding to, and recovering from all domestic disasters, whether natural or manmade, including acts of terror. FEMA has recognised that a critical element of reducing flood risk involves the identification of the flood hazard. However, hazard identification and hazard mapping alone does not necessarily reduce flood risks.

In an effort to fullfil its role and objectives, FEMA created the Risk Mapping, Assessment, and Planning (Risk MAP) program (FEMA, 2008). The vision of Risk MAP is to collaborate with state, local, and tribal entities to deliver quality data that increases public awareness and to lead actions to reduce risks to loss of life and property. To achieve this vision, FEMA will evolve its focus from traditional flood identification and mapping to a more integrated process of identifying, assessing, communicating, and mitigating flood-related risks.

This vision is being intensely applied in those communities with levees. FEMA has identified new procedures that are improving the USA's ability to map and mitigate flood hazards related to levees. This process is based on two primary hallmark principles – an interactive stakeholder engagement process, and more robust engineering and mapping approaches – that establish the framework for a more credible, technically sound, and cost-effective approach.

Figure 6.8 Vision for the Risk MAP life cycle (courtesy FEMA)

Interactive stakeholder engagement process: the levee analysis and mapping procedures include a highly interactive co-ordination process with key stakeholders, including community officials and levee owners. This process may include the formation of a local levee working group, members of which will include community officials and levee owners. FEMA will work with stakeholders to select the appropriate engineering and mapping approach based on a variety of factors, such as available data, levee system and flooding characteristics, potential level of risk landward of levee, levee owner willingness to contribute data or analyses, and available FEMA funding for the study.

More robust levee flood risk engineering and mapping approaches: previously, FEMA used only one analysis and mapping approach to assess the flood risk associated with any levees that did not provide a minimum of flood protection for an event that had a 1 per cent chance of occurring in any given year. Currently, levee-impacted communities are able to apply a variety of analyses and mapping procedures that better reflect their unique circumstances and better characterise their flood risk. These include overtopping, breaching, and a natural valley (without levee) analysis.

Box 6.9 Public flood awareness over time – a study, Erzgebirge Region, Germany

The interest of the public in flood prevention always depends on the level of awareness and the corresponding perception of the flood danger. After a flood event this public flood awareness peaks and unfortunately decreases relatively quickly as other events or problems develop over time.

Flood awareness curves corresponding to the regional awareness level, local awareness level, and technical expert community are shown in Figure 6.9. These curves, which correspond to a region near Dresden, are based on general assumptions and could be refined and quantified through empirical sociological investigations. However, some tendencies can be illustrated with this simplified representation:

- flood awareness at the regional and local level decreases relatively fast. The curves indicate that general regional flood awareness fell to half of its value in less than 10 years after the flood event
- the peak magnitude of the short-term regional perception increases over time, probably due to the expansion of the media and public communication mechanisms
- due to the apparent extreme flood recurrence period of 30 years (1897, 1927 and 1957) in the Erzgebirge region, many people expected a comparable extreme flood in 1987, which did not occur
- a number of planning, legislative, organisational and structural initiatives for flood protection were undertaken after each flood event, as shown in the figure (red bars). There is a time lag between the flood event and the corresponding flood protection initiatives, due to the associated planning, design, and construction efforts.

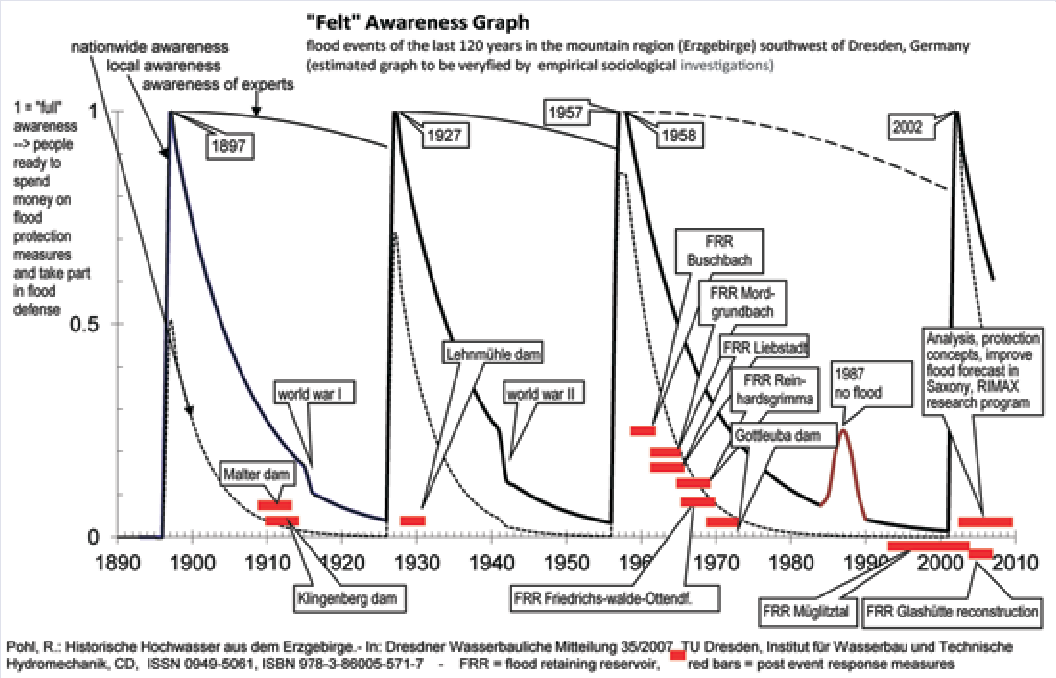


Figure 6.9 Public awareness over time

6.4 EVENT AND CRISIS MANAGEMENT

Levee managers, emergency managers, and responsible officials need to be aware of flood potential in their areas of responsibility. The use of available resources during a flood needs to be evaluated based on risks to personnel and equipment. Resources should be targeted to areas where levee performance concerns and/or potential failure consequences are the greatest.

Levee managers need to be aware of the following stages as the flood/storm event unfolds:

1. A meteorological forecast may predict the probability of high rainfall or a storm.
2. Based on the forecast the possibility of a flood may be predicted.
3. Based on this prediction a flood warning may be issued.
4. Levee managers move into **preliminary response activities**.
5. As the event unfolds, levee managers adopt **full response activities**.
6. **Post-response activities** after the peak of the event allow a return to more normal conditions.
7. Finally, after the event, recovery and mitigation may be implemented.

As the threat increases, the response needs to be adjusted accordingly. If the flood predictions allow time for emergency preparation or strengthening measures, then those measures should be implemented. On the coast, it is important to be aware that accessibility onto the levee for action may be limited by the severity of storm conditions (wind speed, wave overtopping etc). If the predictions are so severe that available resources or time constraints will not allow for adequate response, then evacuations may be undertaken. In some cases, emergency strengthening and evacuation might be used together.

As a storm approaches or as the flood potential increases, other responsible authorities may activate their emergency operation centres and their full emergency action plans in parallel with the activities of the levee manager.

6.4.1 Preliminary response activities

Upon receipt of official information forecasting potential high water or storm, levee managers should prepare for response activities. Levee managers are strongly encouraged to contact local emergency managers and activate their operations centre. Emergency managers and emergency operations centres should be informed of the condition of levee systems and given ongoing updates on the situation updates during a flood response.

As part of preliminary response activities, levee management staff should:

- divide the levee into geographic sections and assign the sectors to teams or individuals
- verify that staff have access to gate keys, current rosters, listings of flood defence system features and closures, plans, and other critical items
- co-ordinate efforts with communities upstream and downstream and eventually on the other bank of the river
- alert the community (leveed area) to the potential for flooding, giving it advance warning to take action and minimise potential damage to businesses and homes
- ensure local emergency operations centres are informed of the situation
- begin documenting the situation and send situation reports to the local emergency operations centre, as necessary
- provide safety briefing to response teams.

These activities should be part of the flood response plan (see Section 6.2).

As floodwaters approach the levee system, an initial high water/storm inspection should be carried out, in which the levee manager should ensure that special attention is given to the following items:

- water conditions, including waves, and any accumulation of trash, debris, ice etc
- condition of road, rail, and water access
- reconfirming the location, quantity, and conditions of all necessary tools and materials (eg sacks, sandbags, lumber, lights) and distribute and store them at points where maintenance is anticipated
- communication systems check
- inspection of drainage structures by the levee manager or other responsible authority (most drainage structures are situated to convey interior drainage from low points of the leveed area through the levee by gravity flow. Due to the location, drainage structures are generally subject to inundation at lower stages than most other flood defence system features, and special attention should be given to flap gates and other drainage structures that might not be accessible later)
- condition of any recent repairs to the levee system
- identification of any boil or seepage areas or other type of deterioration or damage, or water level getting close to the levee crest level.

Additional preliminary response activities could include:

- review assignments for patrols, closings etc

- obtain lists of all equipment, including motorboats, cars, construction and earthmoving equipment, and trucks that can be made available
- assess needed equipment support (vehicles, radios etc)
- verify serviceability of flood management equipment
- record guage readings and continue to monitor river stages
- close public access
- install levee or flood wall closures as necessary, all road closings should be co-ordinated with the corresponding transportation or railroad authorities before limiting access through the levee
- remove any hazardous materials from the vicinity of the levee system.

6.4.1.1 Patrols and inspections

A critical activity that occurs in an event response (in both preliminary and full response) is patrols, also called in other parts of this handbook as 'inspections'. To minimise damage and prevent levee system failure, problems need to be detected as early as possible and resolved accordingly.

Information presented in this section related to patrols is based on optimum conditions where there are no constraints in resources (personnel, equipment, funding, or time). Certainly, this will not always be the case and conditions dictate the level of patrol activity that can occur. If resources are a limiting factor then the levee manager should focus patrols on those sections that pose the highest risk (such as urban versus agricultural, areas with history of poor performance, areas with known susceptible material, areas with critical infrastructure, and other).

Ideally, the levee system should be patrolled at a frequent interval during preliminary and full response activities. During preliminary response a frequency of once per day is recommended, increasing that frequency as the event worsens. Patrols need to be conducted by teams rather than by individuals. Typical responsibilities of levee patrols are listed in the following:

- **general activities:**
  - record gauge readings
  - inspect fences on the waterside of the levee frequently to ensure they are free from debris (collected debris should be cleared immediately or the fence should be cut to free the debris and decrease the possibility of damage to the levee)
  - verify that all necessary access roads and ramps along the levee are usable or will be satisfactorily conditioned
  - take photographs of all significant issues (use date/time stamp and GPS co-ordinates, if available). Note gauge readings on photos if possible
- **specific levee activities** – look for the following signs of distress:
  - sand boils or unusual wet areas landward of the landside toe
  - slides or sloughs in levee embankment and signs of embankment instability (rotational or slip failures)
  - overtopping (predicted increases of water level exceeds observed freeboard)
  - seepage (water observed exiting the levee embankment or landside toe)
  - wave wash or scouring of the waterside (vertical scarps appearing along the embankment)
  - low areas in levee crown
  - check relief wells (flowing/non-flowing)
  - check flap/sluice gates for proper closure
  - check gap closures
- **specific flood wall activities** – look for the following:
  - saturated areas, seeps, or sand boils landward of the flood wall and sinkholes on both sides
  - settlement (vertical movement) of the flood wall
  - bank caving that may affect the structural stability of the flood wall

  - inspect toe-drain risers (discharging/non-discharging)
  - inspect the monolith joints for signs of failure or material deterioration
  - check gap closures
  - tilting of the structure (where water pressure forced the structure landward)
  - overtopping (predicted increases of water level exceeds observed freeboard)
  - concrete cracking and other deterioration
- **specific pump station activities:**
  - verify proper ventilation (eg fans, vents) of the pumping plant to prevent overheating of pump motors
  - look for sink holes or wet areas around the perimeter of the pumping plant and/or settlement of the pump house, all of which could potentially be the result of separation in the conduits. If this condition is suspected, pumps and motors should be shut down until an engineering review can be conducted to analyse the condition
  - verify that assigned operators are on duty 24 hours daily (24/7).

The patrol (or inspection) observations and actions should be described in a formal report that can be used, either as a way to pass information to management staff that can decide for emergency maintenance or repairs, or subsequently as a way to facilitate feedback and improvement of the plan. It can also help the management in difficult legal situations to justify its actions. See Section 5.6 for future use in any assessment or decision making.

6.4.1.2 Safety and security precautions

To reduce the risk of injury to patrol team members, the best method for conducting a patrol is to have a three person team walk in a line across the levee with one person on the waterside of the levee near the water surface, one at the top of the levee, and one on the landside toe of the levee. The team should move slowly enough to enable the waterside member to probe below the surface with a rod in order to discover any erosion that may be taking place. All personnel need to have appropriate safety gear such as a safety line and flotation vest.

The waterside member also needs to be especially observant of floating objects. The limbs and roots of an uprooted, floating tree can strike anyone walking along the edge of the water. To increase the chance of identifying floating objects, walk in the upstream direction when patrolling the waterside of the levee. When patrolling flood walls, the patrol should not walk the top of the wall, but rather concentrate on potential problem areas on the landside of the wall.

Each person on the patrol should be thoroughly familiar with the community evacuation plan and signals. If evacuation is necessary, the patrolling organisation should move to a predetermined location and keep the team intact. If evacuated, when returning to the levee system, physical conditions may be considerably different from those observed before the evacuation, especially if the levee was overtopped. If overtopping occurs during nightfall, it is recommended that the patrols do not resume until daylight, although there may be cases where this recommendation cannot be followed.

Patrols need to look out for anyone that seems out of place, or any activity that seems suspicious. Individuals may try to take advantage of the already dangerous situations on levees or flood walls for their own purposes. Any suspicious activities should be reported immediately to law enforcement authorities.

6.4.1.3 Emergency maintenance and repairs

Once an inspection has been completed, urgent maintenance issues should be addressed before the floodwaters rise further, a breach starts to develop, or a new flood occurs. Emergency maintenance activities are no substitute for normal annual maintenance (see Chapter 4), and owners/operators should not defer the activities listed as follows:

- immediate attention should be given to the crest level of each levee section or profile by comparison of existing crest levels with those shown in 'as-built' record drawings. Fill any settlement, holes, gullies, and washes in the levee crown, embankments, and landside berms with compacted fill material. Materials used to fill depressions should be obtained from distant sources (not adjacent to a levee system) unless it has been determined that borrowing in areas adjacent to the levee will not adversely affect its stability or the control of under seepage. Use adjacent material only under direct supervision of the section leader and with the advice of appropriate staff. The fill material should be compacted and protected from wave wash and other erosion as necessary. Use sandbags, if adequate fill is not available

- examine all drainage ditches on the landside of the levee and remove any obstructions. Be prepared to construct seepage drainage ditches, including appropriate filter arrangements, but not until actual seepage appears. Excavation of ditches near the levee, or in the long berm area, is hazardous and should not be undertaken except under direct supervision of appropriate staff
- drainage structures are generally subject to inundation at lower stages than most other features of the flood control flood defence system, and any maintenance problems should be corrected as quickly as possible. All flap and sluice gates that are in critical condition should be manually checked and repaired as needed before the outlet end of the structure becomes submerged. Remove debris or other potential obstructions. If the gate system on a drainage structure fails to operate and cannot be repaired, consider blocking the structure opening by other means.

6.4.1.4 Interaction with the community

The patrol team may see observers on the levees or at flood walls. In densely populated areas, an additional person should be assigned to each patrol team to act as a safety officer, explaining the dangers that are present. Teams may distribute instruction cards describing the community evacuation plan. It is important for the members of the public to be warned of the danger in the vicinity of the levee itself and evacuated or kept away.

6.4.2 Full response activities

Depending on the predicted severity of the flood event, a levee manager and/or emergency manager will stage their responses, specifically inspection and intervention actions, according to the threat. The decisions about the level of response would generally be made in co-ordination with local authorities.

Some flood plans prescribe specific sets of actions at specific river stages (or floodwater levels), as described in Box 6.10. If the plans do not detail specific actions at defined water surface elevations, the level of response will depend on the decisions of the local authorities. At lower threat levels, less frequent patrols may be needed. At higher threat levels, however, additional teams will be called in and a higher degree of activity will be warranted. Boxes 6.11 and 6.12 present case examples of adjusting response level to changing flood conditions.

During full response activities, patrols are continuously conducted. In addition to preliminary response activities, levee staff should:

- keep a record/inventory of flood management equipment, materials and supplies as they are used
- keep the public informed of the current situation through the media, if warranted
- carry out any intervention actions (see Sections 6.5 to 6.10), including repairing any erosion, seepage, or stability problems
- keep pumping station operators on duty whenever it appears that operation is imminent during flood periods, even when station operation has been automated. Operators should thoroughly understand the manner in which the pumping station was designed to operate and be capable of manual operation should automated equipment or sensors fail
- use portable pumps to pump water over the levee if water ponds in undesirable areas or is rising too quickly in ponding areas
- patrol ponding areas
- monitor debris basins and trash racks for sediment and accumulated debris.

Box 6.10 Staged response to flood fighting example, France

SYMADREM is a public institution responsible for monitoring, management and maintenance in all circumstances of levees located in the Delta of the Rhone. Its scope of management covers 210 km of river levees and 30 km of sea levees, which were erected during the 19th century. 115 000 people are protected by levees. SYMADREM has 24 permanent agents (eight engineers, eight levees guards and eight administrative staff). Main decisions are made by a board of 29 elected people.

During flood periods a graduated monitoring and emergency response is employed.

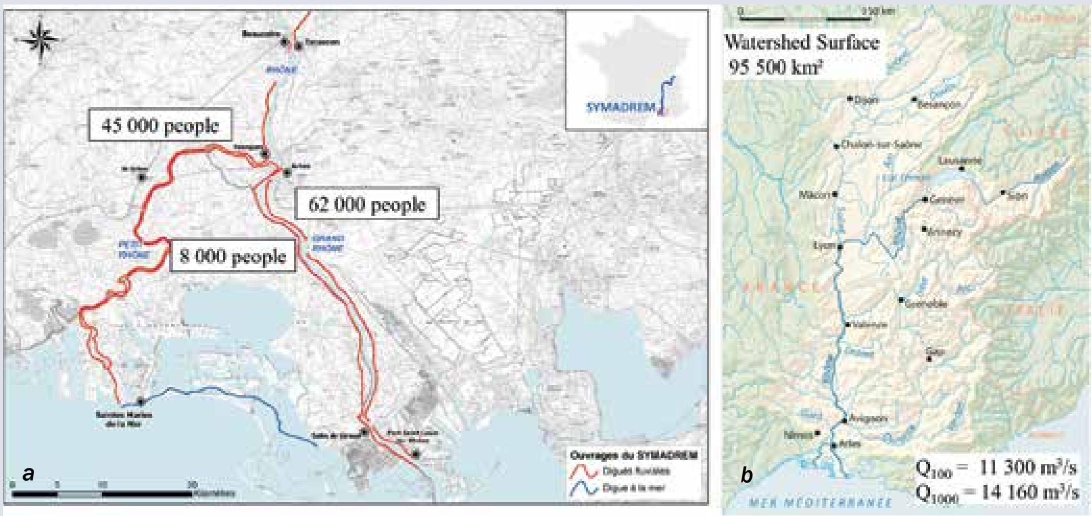


Figure 6.10 Delta of the Rhone river (a) and watershed (b) (courtesy SYMADREM)

Issues during flood periods: Camargue levees were built in the second half of the 19th century after the great floods of 1840 and 1856, whose return periods were respectively 400 and 250 years. The structures were erected on other older levees. Given their form of construction (compaction with manual tamping devices of 15 kg) and their heterogeneous composition (alternating silt/sand) due to successive stages of building, levees are very exposed to failures by internal erosion. The probability of structural damages exists from the early loading from the river and increases with the importance and duration of the flood. Floods of 1993, 1994, 2002 and 2003 showed that breaches can occur in levees before the water reaches the crest. Under these conditions, any early deterioration or damage that is not detected quickly and treated immediately, can worsen rapidly and lead to formation of a breach and the flooding of the leveed area.

Principles of levees monitoring and emergency interventions plan: given respective obligations of SYMADREM and local authority mayors (see French Policy of framework in Figures 6.11 and 6.12) and in response to the important length of levees to monitor and insufficient staff to implement effective monitoring, SYMADREM has set up a plan, based on the provision by municipalities of municipal officers and citizen volunteers.

Five alert thresholds are defined, according to the flow upstream of the delta, geometry of the structures and safety levels of levees:

Alert threshold	Decisions	Discharge threshold per group		
Pre-alert	Plan activation: monitoring of specific points	4200		7500
Alert 1	Reinforced monitoring (closing of crossing hydraulics works)	5500		8400
Alert 2	Linerar monitoring (day only)	6750	7500	9000
Alert 3	Linear monitoring (day and night)	8400	9000	10 500
Alert 4	Safety level – evacuation of monitoring team	XX	XX	XX

For each alert threshold, an action plan corresponding to the risks caused by the flood, is defined. The stakeholders involved in this plan are:

- SYMADREM for levees management
- local authorities for ensuring safety of population and supply monitoring team
- companies of public works for emergency interventions
- flood forecasting governmental agency
- prefectures and civil security (army and rescue department) for crisis management and organisation of rescue.

Entities of levees monitoring and emergency interventions plan are:

- command post for levee monitoring, which is composed of engineers, levee guards and administrative staff

Box 6.10 Staged response to flood fighting example, France (contd)

- **44 monitoring teams:** municipal officers and volunteers from communal reserve of civil security (under direct authority of the mayor and functional authority of SYMADREM, when monitoring levees)
- **eight municipal correspondents (appointed by the mayor):** responsive for composition and management of monitoring teams (equipment, transport)
- companies (three) of public works for emergency works.

The command post for levee monitoring is based in the headquarters of SYMADREM. For the first two levels of warning (early warning and alert no. 1), levees are not directly influenced by the river, with the exception of specific areas of weakness such as lack of freeboard or presence of former breaches. Monitoring of levees is carried out by the levee guards. Repair of the disorders (burrows of badgers) are executed by the companies in charge of maintenance works. It is also during these stages that the floodgates of the hydraulic structure crossings are closed by owners and the gates closing the access at the levees opened. During these two phases, the command post for the levee monitoring is reduced (only directors). The synopsis of the organisation setup for these two stages is:

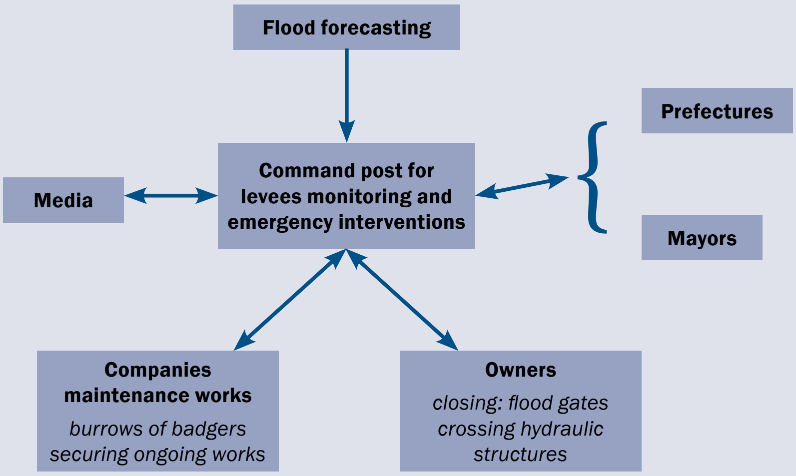


Figure 6.11 Early alert and alert no.1 – SYMADREM management during flood, links with others

For alerts no.2 and no.3, levees are influenced by the river. Linear monitoring is in place. On alert no.2, monitoring is performed during the day. On alert no.3, monitoring is performed 24 hours a day. The 44 linear monitoring teams, composed of municipal employees and volunteers from communal reserves of civil security, are under the direct authority of the mayor and functional authority of SYMADREM, as soon as they are on the levees. The command post for the levee monitoring is widened (all staff mobilised). The synopsis of the organisation setup for these two stages is:

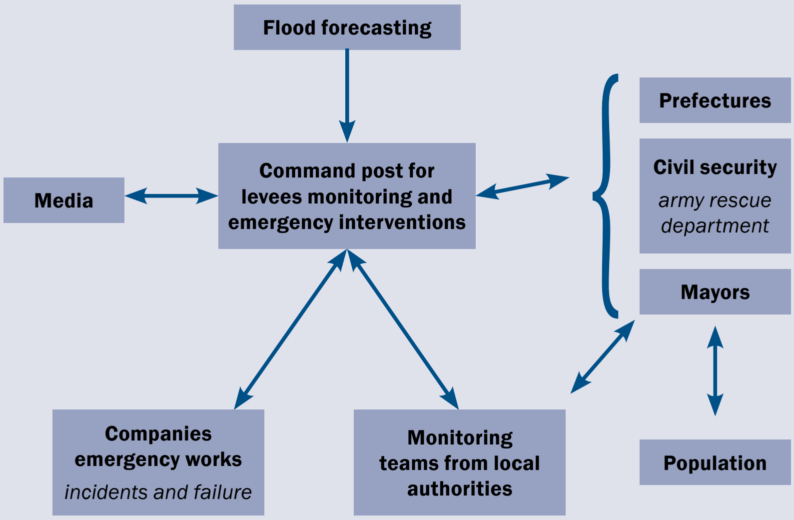


Figure 6.12 Alert no.2 and alert no.3 – SYMADREM management plan during floods and links with others

For alert no.4: safety or design level is reached.

Monitoring teams are evacuated. Information is made to mayor and prefect to organise the security setting of population. Beyond the level of safety, levee monitoring is limited to the vigilance of the levee guards. Monitoring by helicopter is requested to the representative of state (prefect).

Box 6.10 ***Staged response to flood fighting example, France (contd)***

In addition, the management plan includes the following provisions:

- stopping, in case of danger, the monitoring of levees and interventions
- information from the authorities in case of imminent danger and levee failures
- assessment of post-event
- training and simulation exercises
- provision of monitoring equipment (life jackets, lamps)
- establishment of a system of benchmarks
- signage of access to levees.

The improvements planned in the three coming years are:

- additional storage areas for materials
- development of additional access points at the levees
- securing communications (digital radio)
- securing power supplies
- setting up an automatic phone call
- geolocation.

Note
Exercise on the SYMADREM levees, guard is equipped with a parka with life jacket integrated. Municipal officers are equipped with classic life jackets. Communications are made with a digital radio.

Figure 6.13 ***Levee guard (a) and one monitoring team during a flood simulation exercise (b) (courtesy SYMADREM)***

Box 6.11 *Flood response – early detection example, Gowdall Barrier Bank, UK (2000)*

Summary

Even if a failure cannot be prevented by emergency actions, the flooding might be delayed long enough to provide time for full evacuations.

In November 2000, flooding of 150 properties in Gowdall village, major roads, the East Coast railway line and the surrounding countryside occurred following the failure of a washland barrier bank at Heck Ings. However, due to emergency works undertaken by the local Environment Agency team and their professional partners, flooding of the village was delayed, giving the residents an additional three days to evacuate their homes and move their possessions to safety.

Controlled washlands (areas for flood storage) provide flood risk management in the area and are bounded by the riverbank, a higher level barrier bank to the landward side, and then subdivided into compartments by lower level cross banks. The upstream compartments fill then cascade, via the cross-banks, into the next compartment down. On the morning of 2 November, a site patrol noticed slips on the outer face (dry side) of a barrier bank in the Heck Ings compartment. An engineer visited the site, decided that a breach was inevitable and that it was too dangerous to carry out repairs.

The railway line providing the transport link into the Drax power station lies between the breach site and the village, and was identified as a potential secondary line of defence. Environment Agency staff worked closely with the railway authority, the power station, the Internal Drainage Board, highways authority, emergency services and the public to ensure this could be achieved. Where the local road passed beneath the railway a temporary dam was constructed to a height of three metres. A second minor access track crossing was blocked with locally found clay. Culverts were also plugged with clay. A fish farm, a house, and a residential caravan on the ‘wrong’ side of the secondary defence were evacuated.

The bank finally breached at 3.00 am on 3 November 2000. The temporary dams successfully held a two metre head of water, however once the water built up behind the new defence line some leakage occurred and some culverts blew. The floodwaters started to fill the fields towards Gowdall village. The villagers were warned by loud hailer vans and additional manpower started sandbagging all properties in the village. Once it became apparent that major flooding could not be prevented the village was evacuated, although some residents refused to leave their homes.

Figure 6.14 ***Levee breach in Gowdall (a) and temporary dam under construction on roadway under Drax railway line (b) (courtesy Environment Agency)***

Box 6.12 Flood response – risk-informed example, Birds Point Levee, USA (2011)

Summary

Emergency managers may, at times, be faced with choices about flooding one sparsely populated area to protect a more densely populated area.

Deploying a flood-control tool it had not used in 74 years, the U S Army Corps of Engineers (USACE) detonated explosives to breach part of the Birds Point levee in Missouri's Bootheel region to ease the flooding in the town of Cairo, Illinois, and elsewhere in the region.

The decision to 'activate' the Birds Point–New Madrid Floodway was made by the president of the Mississippi River Commission (MRC). When the first segment of the two mile-long 'fuse plug' levee was breached, darkness prevented journalists from seeing how quickly the swollen Mississippi River rushed into the farmland of the floodway. Another segment of the frontline levee was to be breached by explosion later in the night and a third segment, the next morning.

The controversial decision to activate the floodway for the first time since 1937 set off a wave of complaints from Missouri officials who warned of the damage to fertile farmland on the 130 000 km2 floodway, but stirred praise from Illinois officials who wanted the USACE to use every tool available to ease record flood conditions at Cairo and elsewhere near the confluence of the swollen Ohio and Mississippi rivers.

Across the river, officials inspected sand boils and other evidence that floodwaters were undermining some levees around Cairo, most of whose 2800 residents had been evacuated. The National Weather Service said the Ohio River had crested above 18.9 m at Cairo – the level at which a master plan calls for activation of the Birds Point–New Madrid floodway.

Major General Michael Walsh, the MRC President commented about his decision:

> *"Making this decision is not easy or hard – it's simply grave – because the decision leads to loss of property and livelihood – either in a floodway – or in an area that was not designed to flood. The state of Missouri has done a superb job of helping people escape the ravages of water in the floodway. So, with the tool that has withstood many tests: the test of operation in 1937; decades of challenges that resulted in the 1986 Operation Plan; reviews and numerous unsuccessful court challenges – I have to use this tool. I have to activate this floodway to help capture a significant percentage of the flow. I don't have to like it, but we must use everything we have in our possession in the system to prevent a more catastrophic event. So, today, I give the order to operate the Floodway."*

The hours immediately following the detonation saw the Ohio River at Cairo fall more than 15 cm, to 18.7 m. That still surpassed the former record of 18.1 m, set in 1937.

The success of the comprehensive Mississippi River and Tributaries (MR&T) system to date is rooted in the lessons learnt. Perhaps the most important lesson was the necessity to accommodate the Mississippi River by not attempting to exclude it entirely from its natural floodplain. The floodway and backwater features of the MR&T system was implemented for this very purpose – to accommodate the natural tendencies of the river during times of flood, and to help relieve the enormous stress on the levee system and the danger to people, their homes, and the businesses that support the economy.

Damages prevented in the flood of 2011 are in the tens of billions of dollars to date. The levees, floodways, spillways, and backwater areas of the MR&T project are preserving lives, communities, and industry from the impacts of catastrophic flooding. Channel improvements on the Mississippi River are serving as a critical part of the flood control system in this historic event. Without river bend cut-offs, dikes and revetments, the flood would overwhelm the MR&T project and the communities it protects.

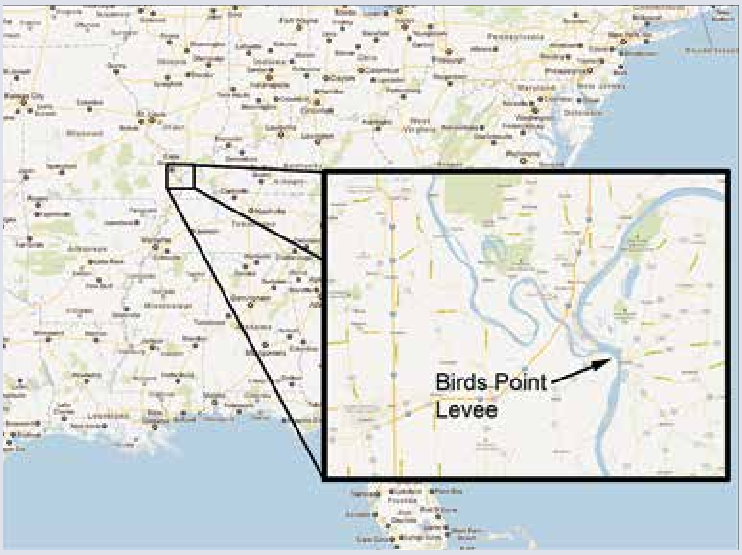


Figure 6.15 Location map for Birds Point Levee (courtesy USACE)

Box 6.12 *Flood response – risk-informed example, Birds Point Levee, USA (2011) (contd)*

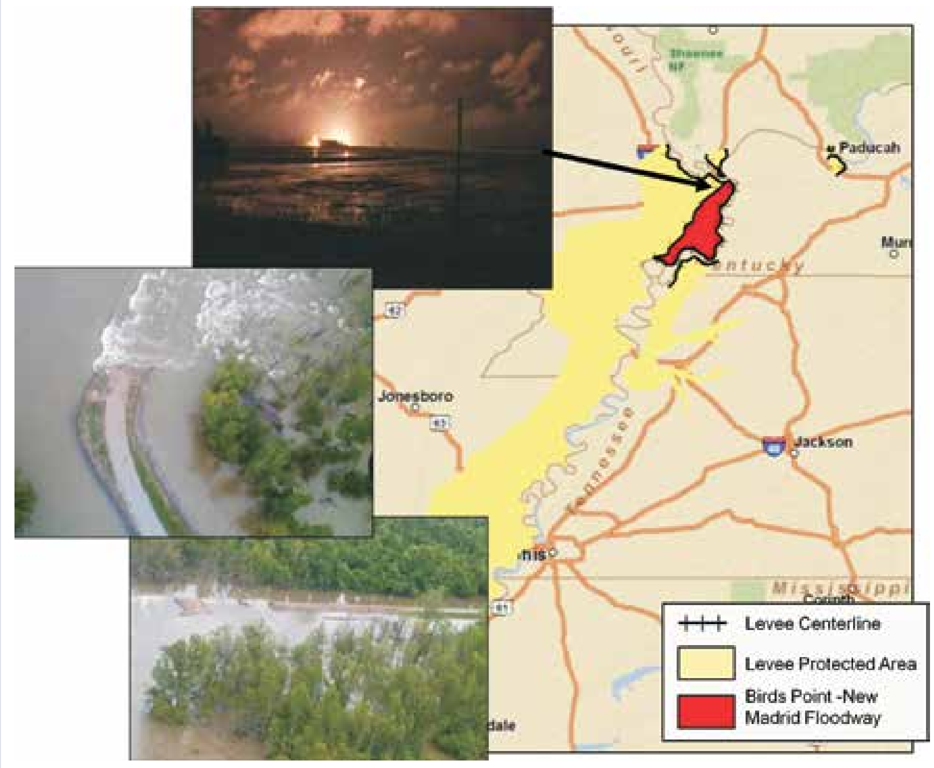


Figure 6.16 ***Birds Point Levee controlled breaching (courtesy USACE)***

6.4.2.1 Evacuation plan activation

At some point it may become necessary to initiate an evacuation of the threatened area. It is unlikely that the levee manager will have the authority or resources to perform this activity. An evacuation will most likely be ordered by the emergency manager (local, state or federal authority). The levee managers role in such a case will be to abide by instructions given by the proper authority. See Box 6.13 for an example of evacuation. Levee managers may also provide the necessary information to the emergency manager to be able to decide the evacuation.

Box 6.13 *Evacuation issues in developing countries*

The levee system encroachment issues faced by developing countries, such as India, are significantly different from those faced by Europe and the USA. Figure 6.17 illustrates how one group of people living in an Indian floodplain responded to a levee breach by moving their community onto the embankment itself. Though this can be an obvious challenge to flood-fighting, these people simply had no other dry place to go.

Figure 6.17 ***India evacuees on a levee***

6.4.3 Post response activities

After the response activities are complete, the levee manager should take care to ensure that the pre-event level of protection provided by the levee is quickly restored.

6.4.3.1 Short-term operational activities

Review of the operations and maintenance (O&M) manual (see Chapter 4) will provide specific details about some of the needed actions post-flood event. Other actions that may be needed to return the flood defence system area to the pre-flood condition can be found in the emergency action plan. Among these are:

- immediate operational activities (which may not be the direct responsibility of the levee manager):
  - return sluice gates in the levees to the normal position
  - open all closure structures in the levees and properly clean and store all components.
- inspection and repair:
  - conduct a post-event inspection, noting high-water marks, locations and extent of damage
  - make repairs to the levee as soon as possible in preparation for the next flood event
  - all temporary protection measures (eg sandbags and material placed during temporary levee raises) should be removed and disposed of properly
  - restore any damaged access routes, staging areas and collateral damage to the pre-flood condition.
- equipment:
  - make an inventory of all remaining flood response equipment, sandbags, and other supplies
  - repair or replace damaged equipment, and restock supplies in preparation for the next flood event
  - salvage any reusable or recyclable materials and supplies (eg wood from flashboards).
- other meetings and activities:
  - meet with key personnel, volunteer representatives, and community partners to debrief, share remaining concerns, and discuss lessons learnt during the event
  - leverage community interest and success stories to increase community awareness about the importance of the levee system
  - revise local emergency preparedness plans to account for lessons learnt and changes to recommended procedures
  - beginning planning for any long-term needs, such as studies or improvements (including improvement of level of protection).

Deployment of temporary solutions during flood emergencies should not be considered to be a permanent solution. Alternative mitigation strategies should be considered after the flood event and, where necessary, appropriate actions should be initiated to install permanent structures.

6.4.3.2 After action report

Following an event, actions and results should be properly documented in a report. It is likely that the levee manager and emergency manager will both produce documentation related to the event (and indeed may also be possible/useful/necessary for a national scale analysis, incorporating all activities). The levee manager should co-ordinate any specific activities that were conducted into the larger report on the levee system to ensure complete and thorough reporting. After-action or feedback reports are generally made available to all interested public safety and emergency management organisations and serve the following important functions:

- a source for documentation of response activities
- identifies problems/successes during emergency operations
- analysis of the effectiveness of the components of the emergency action

1 2 3 4 5 6 7 8 9 10

- identify any potential needed improvements to the levee or the level of protection provided
- describes and defines a plan of action for implementing improvements
- captures key lessons learnt.

The key components of after-action reports are:

- overview
- goals and objectives
- analysis of the outcomes including levee performance and response actions
- analysis of the capacity to perform critical tasks
- summary
- recommendations (including specific improvements for each stakeholder).

If consideration is given to making an intervention (semi-) permanent, specific complete performance assessments of the applied intervention techniques (such as seepage berms) should be carried out.

6.4.3.3 Long-term mitigation

Lessons learnt can provide emergency management officials and levee managers valuable information about how to plan for the future and provide improved responses. This information can also be used to inform public officials and residents about flood risk and to assist in public policy discussions concerning land use and building codes. Other outcomes of an after-action review could produce either a more effective response during an emergency or improvements to the flood control defences to reduce the need for flood response. See Box 6.14 for example of lessons learnt from an after-action report.

Box 6.14 After-action items examples, Hurricane Agnes (1972) and Tropical Storm Lee (2011), USA

In 1972, a large hurricane struck the USA eastern seaboard causing major flooding in the state of Pennsylvania. The levees protecting a major town were insufficient and losses were very large. As a result of this, the flood protection system around the town was improved and when another large storm hit the same area in September 2011, the levees held and protected the town.

A significant flooding event happened in June 1972, when a hurricane-turned-tropical storm struck Pennsylvania and stalled over the central part of the state for nearly 24 hours. Hurricane Agnes dropped a minimum of five, and in some areas as much as 18 inches of precipitation on Pennsylvania, inundating streams, rivers, and towns. On the evening of 23 June 1972, Agnes moved north across western New York and into Canada, dissipating along the way. In its wake, the storm left a persistent drizzle and one of the most devastating natural disasters in the history of the United States.

Federal flood control structures constructed by the USACE, Philadelphia District successfully accomplished their intended purpose during the storm. Elsewhere, however, floodwaters topped non-federal flood works and inundated towns, leading the Philadelphia District to mobilise in response. Starting around the clock, the Philadelphia District activated staff before the arrival of Hurricane Agnes for field monitoring, maintaining a watch on storm advance, river stages, readiness of reservoirs to store floodwaters, and availability of sandbags. On 23 June, as the storm hovered over Pennsylvania, Philadelphia District officials directed that an emergency operations center be activated. Shortly afterwards, the District staff deployed to Wilkes-Barre to assist in sandbagging, although their efforts were halted when floodwaters overflowed existing dikes and deluged the town. In other areas closer to Philadelphia, the District assisted in the removal of debris from the Schuylkill and Delaware rivers.

In the aftermath of Hurricane Agnes, Wilkes-Barre's levees were strengthened and improved to protect against future flooding. They were tested several times by substaintial flooding and tested to the extreme in 2011 by Tropical Storm Lee. The levee system had undergone a rehabilitation in the early 2000s and it was decided to construct a roadway flood closure on Market Street as opposed to relying on sandbagging for protection at that location.

Wilkes-Barre's $175m upgraded system of dikes and flood walls prevented tens of millions of dollars in property damage from Tropical Storm Lee in 2011.

When a flood event causes extensive damage, the lessons learnt from that event should be used to plan for future events. Hurricane Agnes demonstrated where flood defences were weak and the response was to strengthen and improve those defences. As shown by the storm in 2011, the efforts were successful and well executed.

6.5 INTERVENTION TECHNIQUES

Even though there are many types of failure modes and origins, the emergency responses to these signs of distress can be similar (Environment Agency, 2009, Ogunyoye *et al*, 2011, and State of California 2010). There is no absolute method that can be applied to guarantee successful operation of every levee system. However, failure to react in a timely manner and apply proven flood response techniques greatly increases the risk of failure. Although each flood is unique, there are many common elements from one flood to the next, and proper planning will improve response time and chances of success.

Levee managers and owners are responsible for their levee systems O&M, and are also key stakeholders in flood response activities during high water/storm events. To be ready for these tasks, they are responsible for establishing flood response plans (Section 6.2.3), conducting training, stockpiling needed materials, and for other flood preparations (Section 6.5.1). This section outlines some basic activities that will help ensure that flood responses will be timely and effective.

The choice of intervention techniques will depend on the threat posed by the flood. River systems vary greatly in the flood duration, size of river and velocity of flows. Coastal flooding is significantly different in that the wave action and tidal influence make intervention during a storm very difficult, so the best response for a coastal levee is to make preparations before the storm and take action once a storm is predicted.

Typical circumstances during a flood may differ strongly among levees along different types of water systems, as will the required flood response activities. So, the requirements for intervention techniques differs to a certain extent among levees. Table 6.2 presents some differences. The consequent impact on flood response and techniques is that levees along the coast, estuaries, lakes and some rivers:

- are hard to access during the flood, especially the waterside slope and crest
- need focus for patrolling to detect signs of distress or external erosion, eg the waterside slope (wave attack) and crest (overtopping)
- requires a fast response.

Table 6.2 Differences in flooding characteristics

Feature	Wind driven events *Levees along coast, estuary, lake*	High water driven events *Levees along river, canals*
Weather condition	Storm, high wind speeds and high waves are likely	Can be any, but high wind speed (and waves) are not likely
Response/ preparedness	Forecast approximately days/weeks	Forecast likely in terms of weeks (although in small catchment areas also days, sometimes hours): flash floods
Duration of the event	Short: approximately one day (peak: hours)	Long (likely): varies from days to weeks
Repair	Hours/days: depending on the damage, before the next tide or at least before the next storm (which may be a matter of days)	Weeks: most likely a second flood will next follow soon, time repair may be in terms of weeks/months (except small catchments)
Failure mode	Focus on external erosion	All failure modes equally likely

6.5.1 Flood response equipment and supplies

Levee managers should maintain a stockpile of necessary supplies and equipment used to respond to typical high water/storm events. In a flood, stockpiled materials will provide the capability to quickly initiate a preliminary response while additional materials and equipment are being delivered. The specific requirement for supplies varies depending on the size of the levee system and on past flood events. The best way to determine the necessary quantities for the current stockpile is to inventory the type and quantity of supplies that were used during previous floods if such information is available.

Flood risk management materials and equipment may include (in no particular order):

- **sandbags:** levee managers should maintain an adequate supply of sandbags that can be used for a levee raise or sandboil ring diking. Burlap sandbags have a limited shelf life (usually about eight years if stored in a humidity controlled environment) and should be inspected annually and replaced when necessary. The stockpile of sandbags should be stored in a dry, secure location that does not expose the sandbags to sunlight (Figure 6.18). Continued sunlight and weather will rapidly deteriorate the sandbag material

Figure 6.18 Sandbag storage at a levee pumping station (courtesy USACE)

- **plastic sheeting:** there are many applications for plastic sheeting during flood responses. If it is one of the items that typically gets used in a community during a flood response and there is any doubt about its availability during an emergency, it should be stockpiled in preparation for a high water event
- **shovels/sandbag filling machines:** if the levee system contains areas where large quantities of sandbags will be needed, a reliable method for filling them is also needed. Levee managers may want to consider investing in equipment that will assist in completing this process more quickly
- **emergency lighting:** it is strongly recommended that levee managers maintain emergency lighting, permanent or movable, that would be readily available for use during flood responses
- **communication systems:** reliable communications are extremely important for co-ordinating flood response efforts and for calling for assistance when needed. Mobile telephones work very well, but are limited in their capacity for communicating with multiple people at one time. Cellular networks may become inoperable or overloaded during an emergency. Two-way radios may be preferable as they are extremely reliable for short distances and have the capability to broadcast to several people at once. Without a reliable communication system, any flood management effort will be more difficult and may require additional manpower. Advances in communication technology now enable various communication systems to speak with one another without requiring separate radio systems. Many public safety organisations are replacing legacy communication systems with these newer systems. A wide range of facilities, such as nuclear power plants and prisons have incorporated these upgrades. Levee owners may consider updating their systems to help consolidate communications
- **sources of borrow material:** sources of landside borrow material should be located prior to a flood event. Several borrow areas should be identified in advance as wet weather or muddy conditions could unexpectedly limit access to some sites. Careful consideration should be given to access points of the levee when selecting sites for the borrow material
- **rip-rap (rock armour) for erosion:** while it might not always be necessary to stockpile rip-rap, it is important to know the location and telephone numbers of local quarries capable of supplying rip-rap during an emergency. Some levee systems maintain a supply of gravel on site to ensure levee access routes during an emergency
- **flotation vests:** the health, safety and welfare of workers and volunteers should always be the highest priority during a flood response. Flotation coats or vests should be worn at all times when

working near the waterside of the levee or near fast moving water. Floodwaters can quickly sweep a person downstream, and hypothermia can set in quickly in cold water conditions

- **pumps:** pumps are a critical part of any flood management effort. Pumps are used to control interior drainage and seepage through the levee. Levee managers or local authorities that experience frequent flooding should consider purchasing one or more high capacity pumps
- **temporary flood wall systems:** temporary flood wall systems can be installed to provide a flood defence barrier in areas where the predicted height of the rising river is higher than the height of the levee system or in areas where no flood defence system exists. Several products of this type are available and discussed in Section 6.10.

6.5.2 Flood response activities

It is critical for levee managers to understand the impacts of flood response activities. Where and how to employ flood response efforts involves:

- knowledge of the local area
- the condition of the flood defences
- the state of preparedness of the locals
- the predicted storm severity.

Special considerations should be given to the nature of the flood threat (ie whether the flood is due to river flooding, a large rain storm, or coastal flooding associated with large waves or storm surge). As in the example in Box 6.15, thought should be given to unintended consequences of a temporary measure.

Caution

Many of the intervention techniques in this section may appear to represent a good course of action as a predicted flood approaches, but doing so may have unintended hydraulic and structural consequences such as:

- **more rapid inundation of the leveed area in the event of overtopping or breach**
- **damage to another jurisdiction downstream or across the river**
- **damage to the levee itself.**

An overall flood management effort should address these types of issues and co-ordinate the impacts of all activities.

Box 6.15 Coastal flood response options, USA

Figure 6.19 Consideration of unintended consequences (courtesy USACE)

A large storm was predicted to affect the west coast of the State of Washington (USA) in 2010 when emergency planners determined the existing levee system was inadequate to provide the necessary protection. The map (Figure 6.19) shows where the storm surge was expected to flow into the populated area (red arrows). Access to these areas was not possible, so an alternate location was proposed. However, after further evaluation, the setback berm was determined to be in such a location as to hold water in that made it through the first line of defence and was not constructed. Although the berm would have prevented flooding from a direct assault, the negative impact of the structure preventing water on the landward side from returning to the sea made this solution impractical. Careful consideration of the whole flood response effort should be given so as to avoid unintended consequences.

6.5.2.1 Response activities and levee failure mechanisms

A well-designed and constructed levee, that is properly maintained, and does not experience overflow/overtopping should be able to hold throughout a flood event or a flood event up to its design level. However, it is important to recognise that whenever there is water holding against the levee, the potential for an emergency condition to develop exists. The danger increases with the height of water, the duration of the flood stage, the intensity of the current, and the wave action against the levee face. See Box 6.16 for an example of a typical flood response. There are three main categories that can be identified associated with mechanisms that could potentially lead to a levee failure:

- **external erosion** – includes erosion triggers such as overflowing, overtopping, wave wash conditions, and scour
- **internal erosion** – includes erosion triggers such as seepage and sand boils
- **instability –** includes soil movement and landslides.

Potential levee failures may be prevented if prompt action is taken and proper response methods are employed. The following sections describe some of the general actions that should be taken to raise the crown of a levee or to respond to sand boils, seepage problems, or wave wash if these problems are identified during a patrol. Table 6.3 gives an overview of the response measures that can be used as intervention for the three categories of levee failure mechanism previously discussed. This list is by no means exhaustive but represents either a response measure that is predominantly used or a response measure that is representative of a class of measures. Each of these measures will be discussed in some detail in the remainder of this section. It is important to note that all the measures presented in Table 6.3 are meant to be used as a means to prevent the levee from progressing from deterioration to damage to breach. In the event that a breach is initiated or fully developed, then response activities are unique and will be detailed later in Section 6.9.

The methods described here represent good practices developed because of many years of experience in dealing with problems resulting from high water/storm events. There is a wide range of intervention techniques that can be implemented depending on the conditions of the flood event. A levee manager should evaluate the event situation and identify the requirements in order to respond given available materials, equipment, staff, and time.

Table 6.3 Measures to be used as intervention in an emergency response

Failure mechanism	Applicable intervention	Measures
External erosion	Raise the crest (Section 6.6.1) (locally only, or if carefully planned at a larger scale)	Place and compact bulk fill (earth, clay, ash etc) (Section 6.6.1.1)
		Construct sandbag levee (Section 6.6.1.2)
		Use novel material: lightweight concrete blocks, straw bales, tyre bales (Section 6.6.1.3)
		Drive piling (Section 6.6.1.4)
		Construct a flashboard structure (Section 6.6.1.5)
		Portable cofferdam structure (Section 6.10)
		Portable dam system (Section 6.10)
		Water inflated barrier (Section 6.10)
		Water filled tubes (Section 6.10)
		Open celled plastic grid wall (Section 6.10)
		Filled permeable container (Section 6.10)
		Demountable barriers (Section 6.10)
	Provide erosion protection (Section 6.6.2)	Construct rock/rip-rap berm (Section 6.6.2.1)
		Place asphalt/bitumen layer (Section 6.6.2.2)
		Construct small groyne (Section 6.6.2.3)
	Provide protection against overflowing/overtopping erosion (Section 6.6.3)	Place plastic sheeting on the land side and the crest (Section 6.6.3.1)
		Construct an emergency spillway (Section 6.6.3.2)
Internal erosion	Reduce infiltration to reduce through-seepage (Section 6.7.1)	Place plastic sheeting on the water side (Section 6.7.1.1)
	Increase seepage path to reduce through-seepage (Section 6.7.2)	Construct seepage berm (Section 6.7.2.1)
	Reduce hydraulic gradient to reduce under-seepage (Section 6.7.3)	Ringing sand boils (Section 6.7.3.1)
		Increase landside water level (Section 6.7.3.2)
Instability	Reduce slope inclination and steepness (Section 6.8.1)	
	Reduce pressure underneath levee (Section 6.8.2)	
	Reduce saturation of levee (Section 6.8.3)	

Box 6.16 *Flood response example in the Netherlands*

The combination of heavy rainfall and high water levels at sea (due to two large north-western storms) resulted in unusual situations in the Northern Provinces of the Netherlands in January 2012. Water from the inland water transport system could no longer be discharged at sea and many levees were overflowing and risking instability due to saturation.

Several precautionary measures were taken by the water boards Noorderzijlvest and Fryslân:

- opened special flooding areas to reduce water levels in other parts of the system or to prevent the water from rising further
- stopped draining water to the main water system (causing shallow inundation in the polders)
- set water pumps to full capacity to drain water to the sea
- placed sandbags on landside slope and toe of levee to prevent micro and macro instability and piping
- placed impermeable geotextiles on the waterside slope of the levee
- placed sandbags and bare soil on top of the levee to prevent from overtopping
- the village of Woltersum was evacuated, including the cattle of dairy farms
- extra inspections executed to detect cracks, deformations, water and sand coming through the levee.

During normal situations the water boards lead and are responsible for operations and maintenance of the levees. During critical situations the Regional Policy Team and Regional Operational Team are responsible and make decisions. The central government is then in charge instead of the water boards, because integral decisions need to be made. The role of the water board is solely to give advice. During evacuations the mayor of the regarding village is in charge.

Stakeholders that are involved during emergency preparedness are mainly employees of the water boards, boroughs and provinces, constructors, medical aid organisations, fire department, police departments and the army. Engineering companies, Directorate-General for Public Works and Water Management and volunteers are hardly involved in these situations.

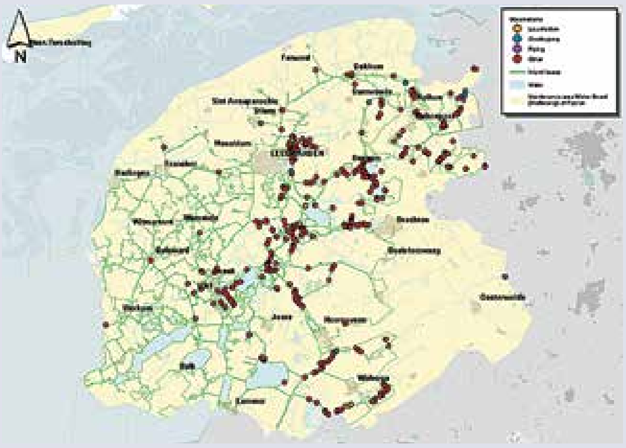


Figure 6.20 *Flooding in the Netherlands, January 2012 (courtesy Wetterskip Fryslân)*

Difficulties that were faced during flooding in January 2012, and lessons learnt:

- massive media attention (water board at Fryslân gave 15 interviews a day with five employees working 15 hours a day to answer press questions)
- changes of roles and responsibilities need to be clear and communicated with people in the field
- lack of capacity to make the calculations and write decision papers
- large effort was required for registration of the movements of the cattle (as required by rules regarding animal diseases), and thorough decontamination of the trucks after each visit of a farm.

6.6 RESPONSE TO EXTERNAL EROSION AND TECHNIQUES FOR INTERVENTION

External erosion is the wearing away of a surface (bank, streambed, embankment, or other surface) by floods, waves, wind, or any other natural process. See Chapter 3 for a detailed description of the external erosion process.

There are three main conditions that can trigger external deterioration and potential failure mechanisms on levees. These are:

- **overflowing** refers to the steady flow of water over the levee crest
- **overtopping** refers to the intermittent water flow over the levee crest, including wave action
- **wave wash** refers to the erosion of the levee slope on the water side as a result of wave action.

It is important to note that wave action effects can be triggered by passing boats, onshore winds, or storms. In either case, wave action may seriously damage a levee, particularly if the water surface is near the levee crown, if the levee is newly constructed, or if the levee is constructed of sandy soil. In many cases, the necessity for wave wash protection cannot be foreseen, and construction often becomes an emergency operation. The fluctuation of river stages as well as the uncertainty of weather conditions often make it impracticable to anticipate wave wash damage, except for the assembling of necessary materials and supplies at convenient supply bases. A severe storm of a general nature may cause serious damage to the levee line. As it is impossible to predict the severity and duration of such storms, it is the duty of field forces to recommend the construction of protective works that can be reasonably justified and to hold themselves in constant readiness to support emergency response as they ensue.

Techniques to mitigate external erosion generally focus on:

- raising the crest elevation
- providing erosion protection on the levee slopes.

6.6.1 Levee raising measures

The impacts of raising a levee should be considered when formulating a plan of action. Co-ordination of the levee manager with the local authorities should be ongoing during the flood event and plans for raising a levee need to include the impacts to other flood districts or communities within the flood basin. When a flood is predicted far enough in advance that it allows for construction of temporary levees in areas of high consequence, the appropriate equipment and material should be located and mobilised immediately. Flood forecasts can be used to determine how high to build a temporary levee or to raise an existing one. The alignment for a temporary levee is generally determined by practical considerations about land use and ownership, as well as physical constraints related to the logistics of construction.

There are a number of ways that the levee crown can be raised (Table 6.3). Care should be given to ensure that the levee raise does not jeopardise the stability of the existing levee by adding excessive weight and flood loading to the levee. Excess weight could cause slope failure. Heavy equipment should not be used on a levee whenever the water surface level is near the top of the crown, as the vibration may cause a failure. In no case should such equipment be allowed on an earthen levee after the levee has started to seep. It should also be checked that raising the levee does not flood previously leveed areas, without properly informing the local authorities and considering possible evacuation of the local population.

6.6.1.1 Place bulk fill

Provided the work is carried out well in advance of the high water event, in areas where there is sufficient space for construction and with the proper equipment, the most efficient means of raising low stretches of the levee is to use bulk fill (see Figure 6.21). The fill material could be soil (local or hauled), gravel fill, ash from coal fired stations, black furnace slag, or other suitable material. This measure can be used under these conditions:

- there is good access to the embankment and slope
- embankment can withstand heavy equipment.

This measure should be avoided if there are substantial flows over the crest that can wash away the material being placed or if there are safety concerns for personnel and equipment.

Before placement of the fill material the existing embankment surface should be scarified and clear of any debris. The material should be placed on the crest in lifts, ideally with each lift properly compacted. This may not always be possible depending on the situation. Figure 6.22 shows an increase in levee height by simply placing bare soil (clay) in the crest. The height is determined by predicted levels of water/wave.

1 2 3 4 5 6 7 8 9 10

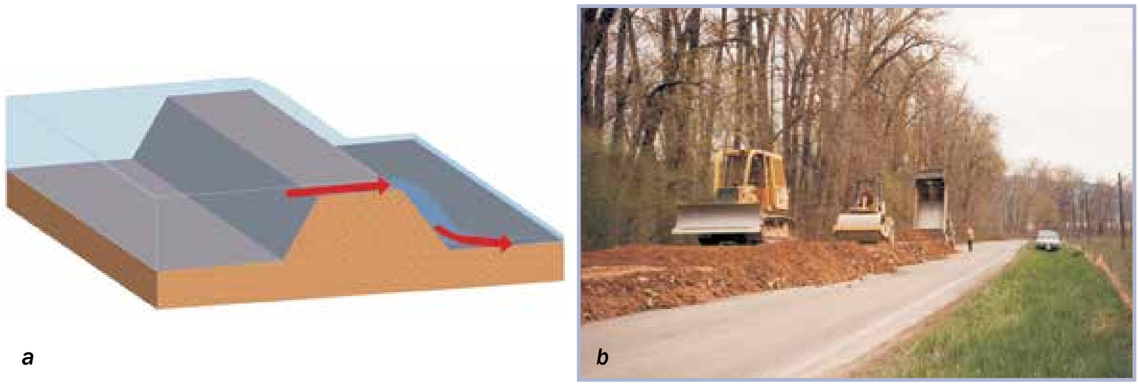


Figure 6.21 ***Levee overflow (a) and levee raise with earthfill (b) (courtesy ASCE)***

Figure 6.22 ***Increase crest height with bare soil (clay) (courtesy Wetterskip Fryslân)***

6.6.1.2 Construct sandbag levee

Sandbags (or alternatively big bags) can be used to raise the height of an existing levee (Figure 6.23), or they can be used over open ground to protect an area with no levee at all. Any time a sandbag levee will be constructed over one layer high, the bags should be stacked in a pyramid structure to ensure stability. The basic rules of thumb in constructing these structures is that they should be approximately three times as wide as they are high, and the sandbags should be staggered within each layer just as they are staggered from one layer to the next. The directions of the bags (transverse or longitudinal) may be alternated, as long as no loose ends are left exposed. The base area available limits the height of a sandbag capping, so a levee can usually be raised only a few feet by this method. The measure can be used under these conditions:

- good access to embankment crest and slope
- embankment can withstand heavy loads and equipment
- ample supply of sandbags, sand, and staging area.

This measure should be avoided if there are substantial flows over the crest that can wash away the sandbags being placed or if there are safety concerns for personnel and equipment.

The following is a description of the proper steps required to construct a sandbag levee:

- clear the foundation where sandbags are to be placed. This will provide for a good boundary between the ground surface and the sandbags, and reduce the amount of seepage that could occur along the boundary
  - sandbags should be filled one-half to two-thirds full

- refrain from tying bags if they are filled at the site of placement. If the bags are to be filled off site and transported to the placement site, then tying the bags will prevent losses due to spillage
- place the filled bags length-wise and parallel to the direction of flow
    - lay the unfilled portion of the bag flat on the ground
    - place the succeeding bags on the unfilled or tied portion of the previously laid bag and stamp into place to eliminate voids and form a tight seal
    - stagger the joint connections when multiple layers are necessary and stack the sandbags in pyramid fashion.

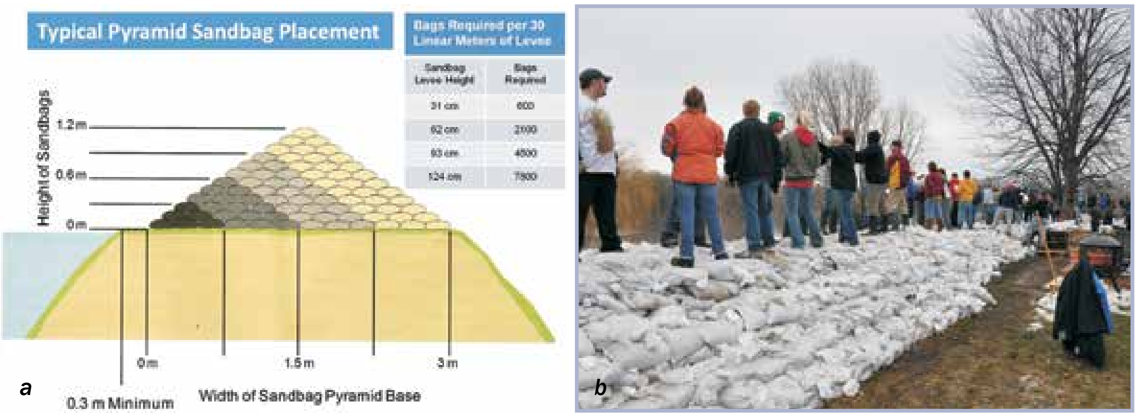


Figure 6.23 ***Typical sandbag levee (a) and levee raise with sandbags (b) (courtesy USACE)***

6.6.1.3 Use novel materials

Although bulk fill and sandbags are the most common material used to effect a levee raise, other novel materials can be used under certain conditions. These materials include lightweight concrete blocks, straw bales and tyre bales (Figure 6.24). The benefit of using this material is in situations where the levee can not support any heavy material or the crest width is too narrow to facilitate use of fill or sandbags.

Figure 6.24 ***Lightweight concrete block (a), straw bales (b), and tyre bales (c) (courtesy Environment Agency)***

All three material types require the use of plastic sheeting or geotextile to cover and act as an impermeable layer. Also, these materials will require stabilising with wire, ropes, or some type of ballast.

6.6.1.4 Sheet piling

Where river flood levels will be elevated for a considerable period of time, installing sheet piles is a good option to consider if there is a need to raise the levee (Figure 6.25). Piling can be driven either waterside of the levee (near toe, toe, midslope) or through the levee crest. If the piling is driven on the waterside, depending on the height above grade and expected water levels, then support may need to be provided by filling between the piling and levee with some type of fill material.

If permitted, consideration can be given to extracting the piles after the event and returning the levee to its original condition. This can be done where:

- there is good access to embankment crest and slope
- the levee can withstand large heavy equipment
- necessary repairs are made to any damage to the levee by the activity.

This measure should be avoided if water levels are high (generally placed at low water levels) or if there are safety concerns for personnel and equipment.

Piling would generally be placed when water flows are low or subsided. The piles are driven into the embankment to a depth determined for proper stability and/or cut-off of flow. Any fill material that is used should be of sufficient size or compaction to not be affected by erosion.

Figure 6.25 Use of piling for levee raise (courtesy Environment Agency)

6.6.1.5 Flashboard structures

This type of levee raise can be very useful in certain conditions, Figure 6.26. Generally, this measure takes time to construct and is material/labour intensive. The measure is straight forward to construct and can accommodate additional raises to a certain extent. The primary materials used in this measure are wooden stakes and panels, and sandbags or other suitable fill material. The measure can be used under these conditions:

- good access to embankment crest and slope
- embankment can withstand large heavy equipment.

This measure should be avoided if there is limited time available for placement or there are safety concerns for personnel and equipment.

To construct a flashboard, wooden panels are driven vertically into the crest on the waterside. The panels are supported with sandbags or fill material on the landside. Also, wooden stakes are used to prop the panels. An impermeable material (plastic sheeting or geomembrane) can be placed over the structure.

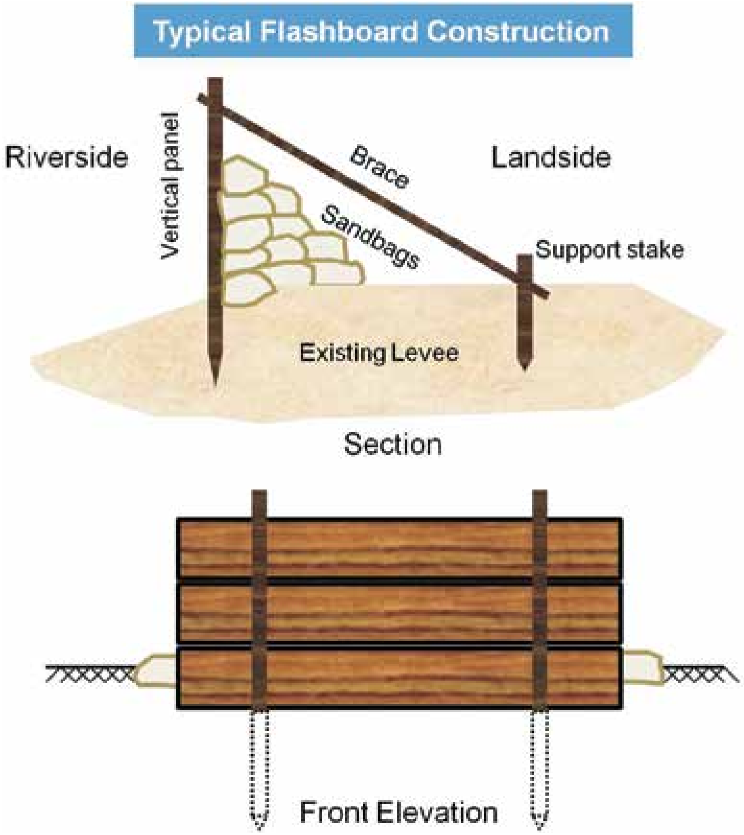


Figure 6.26 Use of flashboard for levee raise

6.6.2 External erosion protection measures

Scour is the erosion of the riverside slope of the levee by abnormally high water velocities or wave action, Figure 6.27. Physical conditions that cause scour are outside angles in the levee, waterway gaps that have been cut through abandoned levees, secondary levees, and topographical features that may create relatively deep channels adjacent to the levee during high water/storm event. Levees constructed across points of land are often subject to current scour as a result of the concentration of flow. Scours are particularly dangerous due to the treacherous manner in which they develop and the difficulty of detection until almost irreparable damage has been done. However, the chief danger, is that the scour will work into the levee slope. This type of scour resembles the caving bank of a river in action and appearance, in that it erodes under water and has a vertical caving face. When the water is near the top of the levee, and by the time the vertical caving face appears above the water surface, a large portion of the levee is gone. Flood response personnel should make careful observations of the riverside of the levee in all reaches where an unusually fast current is apparent. For flood protection projects that have been designed using a hydraulic model, careful observations of the riverside of the levee should be made where the profiles show a steep high water slope. Turbulence in areas where the water is shallow is a good sign of no scour, but should be monitored. If the turbulence unexpectedly becomes still, scour may be suspected and soundings should be made immediately. Conversely, in deep water, scour may be indicated by turbulence and eddies. Field personnel should be particularly watchful for such conditions. If erosion is evident, immediate steps should be taken to protect the levee.

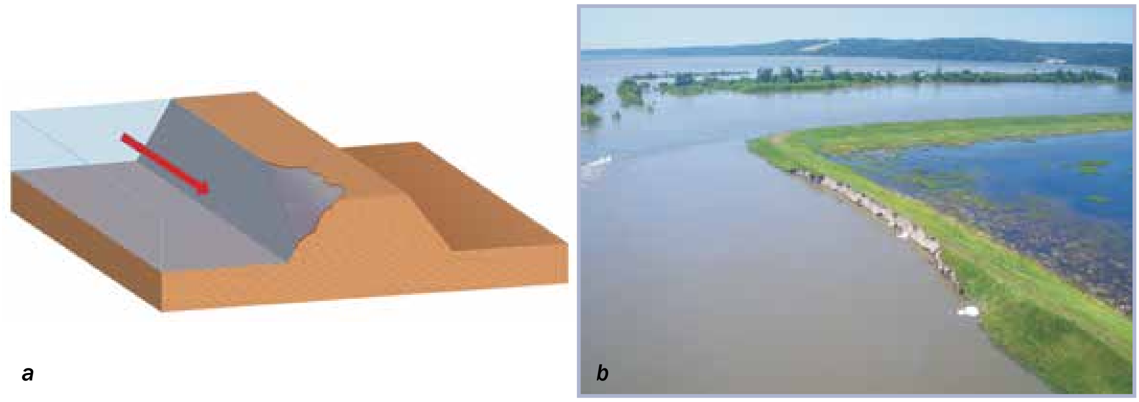


Figure 6.27 Scour on a levee graphic (a) and field image (b) (courtesy ASCE)

The methods used in protecting a levee against current scour depend entirely upon local conditions. In some cases, the current attack is so severe, and the scour is of such serious nature, that it requires specially designed structures that cannot be constructed with the ordinary high water response equipment and personnel. Usually, however, current scour can be prevented or stopped by relatively simple techniques. In cases where minor current attacks are evident on the levee slopes, especially on newly constructed levees, the riverside slope should be protected from current action. The current protection work should extend as far under water as practicable in an attempt to restore the original levee cross-section.

6.6.2.1 Rock berm

Construction of a rock berm is a positive means of providing slope protection and has been used in instances where erosive forces (carried by current, waves, or debris) were too large to efficiently be controlled by other means (see Figure 6.28). The rock can be from any source but should be sufficiently large and angular to resist movement once placed. This method has the advantage that it can be placed under adverse conditions where other measures could not, such as high water, strong currents or waves. The disadvantage of this measure is that the material is usually high in cost and not local to the needed area, requiring potentially long haul times. This measure can be used under these conditions:

- there is good access to the embankment and slope
- embankment can withstand heavy equipment
- adequate supply of rock.

This measure should be avoided if there are safety concerns for personnel and equipment.

For construction of a rock berm the material is first placed at the bottom of the levee waterside toe or bottom of the scour hole, then continued up the waterside levee face (some locations prefer to construct the berm on the landside slope of the levee, see Box 6.17). The width of the berm is primarily determined by the nature of the scour and water forces. Material is placed until the berm is stable and will sustain an additional lift. The length of the berm should be sufficient to fully cover the scour area and overlap the undamaged levee on both sides. Where possible, consideration should be given to first placing a geotextile or other filter to limit wash out of fine material, but it should be appreciated that this may be impractical under the prevailing conditions.

Figure 6.28 Construction of a rock revetment (courtesy USACE)

6.6.2.2 Asphalt/bitumen surface

Many levees particularly in coastal settings are covered with a layer of asphalt, concrete blocks or stones (see Figure 6.29). This revetment material may be damaged during a storm in the form of cracks, fissures, or missing material (Pullen *et al*, 2007). In these cases the levee manager should try to make an emergency repair as soon as possible to minimise the damage. There are basically two options for making these repairs. One would be to replace missing material with stones that are held in place with asphalt that has been mixed and heated such that it can be 'poured' into the material. The second option related to cracks and fissures is to fill those damaged areas with the same pourable asphalt. The asphalt will harden as it cools down. The additional use of a geotextile can be considered if the wave and wind conditions allow. Care should be given to the use of geotextile to prevent the material from interfering with the penetration of asphalt between and around the repair material.

Figure 6.29 Examples of levee with asphalt (a) and stone revetment (b) (courtesy STOWA)

6.6.2.3 Construct small groyne

This measure consists of constructing a small groyne from the levee extending a short distance into the channel or sea to deflect the current away from the levee or add protection from wave attacks to the levee. Groynes of this type are known as deflecting groynes since they change the direction of flow without repelling it. They are generally short and used for limited, local protection.

This emergency groyne can be constructed of several types of material including rock, sandbags, earthfill, timber, or any other available substantial material. Preferably, groynes would be placed in the dry at locations where severe scour may be anticipated. However, they can be constructed in the water (using suitable material) under certain current and wave conditions. Consideration should be given to the resulting hydraulics since haphazard placement of a groyne may have detrimental consequences. Some examples of groynes are given in Figure 6.30.

Figure 6.30 ***Examples of rock groyne (a), timber groyne (b), and earthfill groyne (c) (courtesy Wikimedia Commons)***

Box 6.17 ***Levee scour intervention example, France***

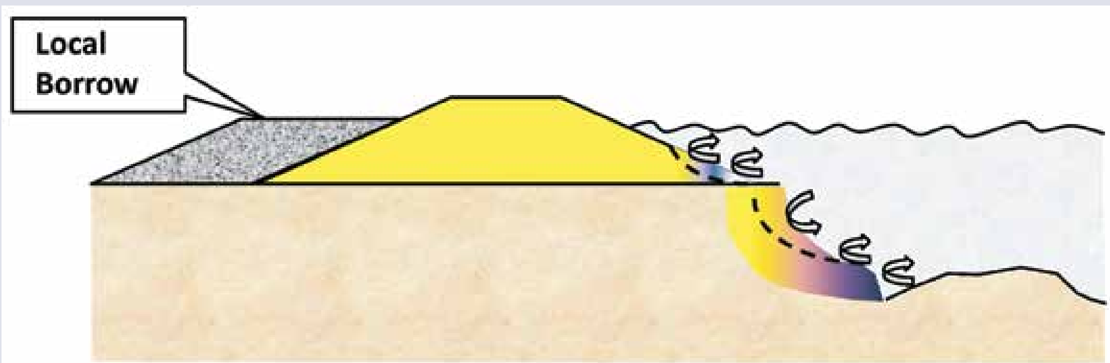


Figure 6.31 ***Scour intervention example, France (courtesy SYMADREM after Irstea)***

In France preference is given to landward interventions for delaying external erosion consequences. As shown in Figure 6.31 local borrow is placed on the landward side of the levee in response to toe erosion on the riverward side.

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6.6.3 Protection from overtopping/overflow erosion

6.6.3.1 Plastic sheeting

The use of an impermeable sheeting (plastic or geotextile) that is properly anchored can be an efficient measure to protect a levee from erosion as a result of overtopping or overflow (see Figure 6.32). The material needed consists of impermeable sheeting, sandbags and rope, all of which are normally in ample supply. The amount of equipment is minimal and may consist of a tractor or bulldozer to assist in unrolling the sheeting. This measure can be placed both in the dry as well as in high water/storm events, although it is most commonly placed before the event. This measure is used under these conditions:

- access to the embankment and slope
- adequate supply of sheeting, sandbags, sand, and rope.

This measure should be avoided if there are safety concerns for personnel and equipment.

The following procedures should be followed depending upon whether installation is being done before or during the event.

- placement in the dry (during low tide or before water levels rise)
  - dig a trench parallel with the embankment at the toe
  - place the leading edge of the geotextile inside the trench and backfill
  - unroll the geotextile up the waterside slope
  - ensure there is ~1m overlap between sheets and anchor down with sandbags
  - drive stakes into the ground just above the area to be protected (the stakes are 1.3 m apart with a 0.3 m stagger)
  - secure the tarp to the stakes with tie-down buttons
  - use a criss-cross method to place the sandbags on all the edges of the tarp
  - ensure that the landside termination of the material is properly secured inside a trench and backfilled or anchored with sandbags
- placement in flood/storm
  - using sandbags as bottom weights, anchor the geotextile at the waterside toe
  - using sandbags, counter-weight the textile against the embankment slope. When impermeable geotextile is used, this will prevent air from being trapped between the embankment slope and the geotextile.

Figure 6.32 Use of plastic sheeting to prevent overtopping/overflow erosion, placing plastic sheeting on levee waterside (a) and finished placement (b) (courtesy USACE)

1 2 3 4 5 6 7 8 9 10

6.6.3.2 Emergency spillway

In the event that one of the temporary levee raise measures detailed previously cannot be implemented or have been unsuccessful then consideration should be given to construction of an emergency spillway, Figure 6.33. This method can be used to accommodate water flowing over the levee while preventing erosion or potential breach of the levee. This measure could also be used if only a short segment of the levee is not at required grade or there is a need to reduce the water level. The materials needed for this measure are sandbags and plastic sheeting. The measure can be used under these conditions:

- good access to embankment crest and slope
- good supply of sandbags, sand, plastic sheeting.

This measure should be avoided if there are substantial flows over the crest that can wash away the sandbags being placed or if there are safety concerns for personnel and equipment.

To construct an emergency spillway, remove the landside slope of all material or debris that would impede placement of the plastic sheeting. Place the plastic sheeting up the landside slope over the crest onto the waterside slope. Link all sides of the sheeting with at least one row of sandbags. Further sandbags may be required to accommodate level of flowing water or to provide additional weight. Sandbags placed on the crest should tie into high ground or other levee raise measures.

Figure 6.33 Use of emergency spillway (State of California, 2010)

6.7 RESPONSE TO INTERNAL EROSION AND TECHNIQUES FOR INTERVENTION

As a river, stream, lake, canal, or sea rises, the hydrostatic pressure against a levee slope increases significantly and can force water into the levee embankment and its foundation (see Figure 6.34). This seepage will generally follow paths of least resistance. According to the geotechnical properties of the soil, internal erosion may then happen. Internal erosion (as seen in Section 3.5.2.2) is indeed a family of different mechanisms:

- **backward erosion:** detachment of soil particles when the seepage exits to an unfiltered surface leading to retrogressively growing pipes and sand boils
- **concentrated leak erosion:** detachment of soil particles through a pre-existing path in the embankment or foundation
- **suffusion:** selective erosion of the fine particles from the matrix of coarse particles
- **contact erosion:** selective erosion of the fine particles from the contact with a coarser layer.

Note

The term 'piping' can be confusing because it is often used to describe either of these internal erosion mechanisms or a combination of these. Internal erosion, once it has started, may lead to a breach in the levee, either alone, or in combination with other mechanisms (instability, settlement and overtopping).

Seepage is generally not a problem unless:

- the landward levee slope becomes unstable due to saturation
- internal erosion happens as a consequence of seepage
- pumping capacity or runoff limit for the levied area is exceeded.

Techniques to mitigate internal erosion generally focus on:

- reducing the seepage flow by:
  - reducing the infiltration of floodwater in or under the levee
  - increasing the seepage path
  - reducing the hydraulic gradient
- accommodate the seepage, but prevent internal erosion from developing, by improving filtration (see also Figure 6.41 in Box 6.18).

Warning

Fighting seepage in an improper manner can cause internal erosion. Pumping of seepage should be held to a minimum, and ponding should be allowed during high water to the extent that it does not cause damage. Levees have been endangered during past floods by attempts to keep low areas pumped dry, and additional time and effort were expended in controlling sand boils caused by pumping.So, seepage should be permitted if no apparent ill effects are observed and if adequate pumping capacity or tolerance for runoff is available in the levied area.

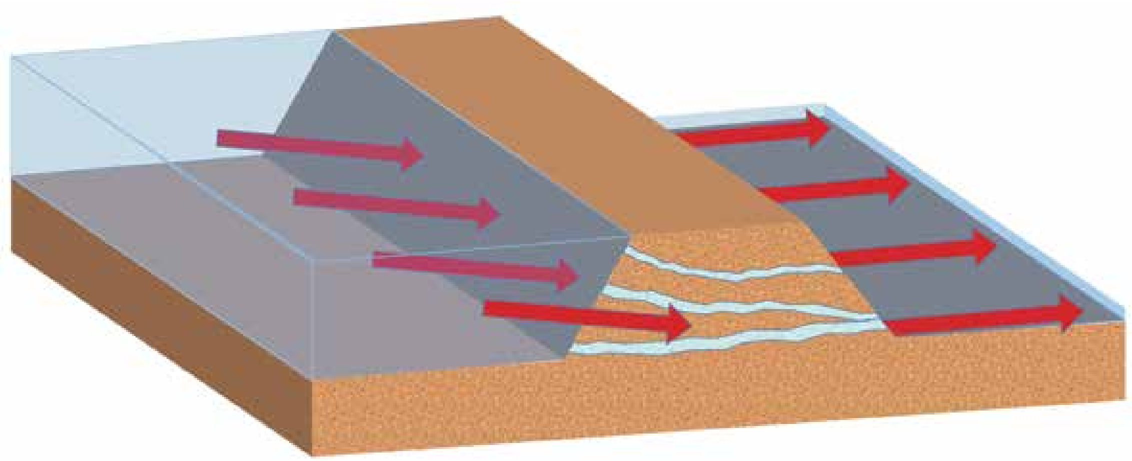

Figure 6.34 Internal erosion process (courtesy ASCE)

6.7.1 Reduce infiltration measures

If a section of levee is known or suspected to be susceptible to damage induced from seepage then measures can be taken to reduce the amount of water that is flowing through the levee. These measures provide a means to lessen the movement of water into the soil thereby reducing the hydrostatic pressures and reducing the potential for internal erosion. These measures can be implemented under wet or dry conditions and are generally straightforward.

6.7.1.1 Impermeable sheeting

The use of an impermeable sheeting (plastic or geotextile) that is properly anchored can be an efficient measure to reduce infiltration (see Figure 6.35). This measure can be used to reduce infiltration over very long stretches of the levee or it may be used at discrete points where a waterside boil inlet has been detected (see Figure 6.36). This measure has been previously detailed in Section 6.6.3.1.

Figure 6.35 ***Use of impermeable sheeting to reduce infiltration over large area (a) and field application (b) (from DWR, 2010 and courtesy USACE)***

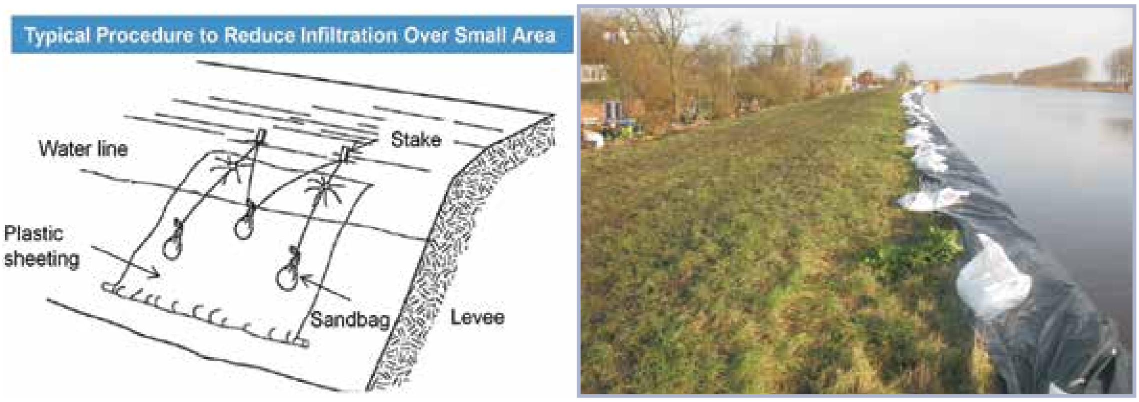

Figure 6.36 ***Use of impermeable sheeting to reduce infiltration (from DWR, 2010, and courtesy Henk van Hemert)***

6.7.2 Increase seepage path measures

If a levee has continuous seepage or under-seepage problems then one of the most common solutions is to construct a riverside blanket of relatively fine-grained impervious to semipervious soils. If these blankets are continuous and extend riverward for a considerable distance, they can effectively reduce seepage flow and seepage pressures landside of the levee. The effectiveness of the blanket depends on its thickness, length, distance to the levee riverside toe, and permeability. However, these measures need to be implemented in dry conditions and so might be most appropriate for preliminary measures. In a high water/storm event when seepage flow and seepage pressures are a concern then the only alternative is to construct a seepage berm on the landside of the levee (see Figures 6.37 and 6.40).

6.7.2.1 Seepage berm

If uplift pressures landward of a levee become greater than the effective stress of the foundation top stratum, heaving and rupturing of the top stratum may occur, resulting in sand boils. The construction of landside berms can eliminate this hazard by providing additional length required to reduce uplift pressures at the toe of the berm to tolerable values and additional weight needed to counteract these upward seepage forces. This measure can be used effectively under these conditions:

- adequate space on the landside for construction
- ample supply of material (soil, rock etc)
- consideration has been given to requirements for any necessary filter layers or geotextiles, given practical installation constraints (time, operating environment etc).

Four types of seepage berms have been used, with selection based on available fill materials, space landside, and relative costs:

1 **Impervious berms:** a berm constructed of impervious soils restricts the pressure relief that would otherwise occur from seepage flow through the top stratum and consequently increases uplift pressures beneath the top stratum. However, the berm can be constructed to the thickness necessary to provide an adequate safety margin against uplift.

2 **Semipervious berms:** semipervious material used in constructing this type of berm should have an in-place permeability equal to or greater than that of the top stratum. In this type of berm, some seepage will pass through the berm and emerge on its surface. However, since the presence of this berm creates more resistance to flow, subsurface pressures at the levee toe will be increased.

3 **Sand berms:** while a sand berm will offer less resistance to flow than a semipervious berm, it may also cause an increase in substratum pressures at the levee toe if it does not have the capacity to conduct seepage flow landward without excessive internal head losses. Material used in a sand berm should be as pervious as possible, with a minimum permeability of 100×10^{-4} cm per sec. Sand berms require less material and occupy less space than impervious or semipervious berms providing the same degree of protection.

4 **Free-draining berms:** a free-draining berm is one composed of random fill overlying horizontal sand and gravel drainage layers (with a terminal perforated collector pipe system), designed by the same methods used for drainage layers. Although the free-draining berm can afford protection against under-seepage pressures with less length and thickness than the other of seepage berms, its cost is generally much greater than the other types, and so it is rarely specified.

Response personnel should start by dumping material against the back of the levee. The material should be shaped with a bulldozer (if possible), being extremely careful not to nick the levee itself. Any gashes in the levee could allow water to start flowing. These berms can be constructed rapidly and be used to great advantage if the materials and personnel are available to implement them.

Figure 6.37 Seepage berm constructed landside of levee (courtesy USACE)

6.7.3 Reduce hydraulic gradient measures

Technically, hydraulic gradient is the difference in head measurements over the length of the flow path. Practically, this means that as the water level increases on a levee due to a high water/storm event the difference in elevation of water from the waterside to landside of the levee produces a pressure on the levee and foundation. This pressure is an uplift pressure that can either reduce the weight of the overlying soils or puncture the foundation in the form of sand boils. Both of these conditions are hazardous to the levee and can lead to failure if not properly addressed. There are three measures that are generally used to address these two concerns:

- landside berm
- ringing of sand boils with sandbags or other suitable means
- increasing landside water level to lessen hydraulic head difference across the levee.

Landside berms will not be discussed in this section since the previous discussion presented on seepage berms is directly applicable. As stated, seepage berms serve to increase the seepage path and reduce hydraulic gradient. Some general considerations for construction of a berm to reduce hydraulic gradients include the permeability of the berm material and the size of the berm. Generally a pervious or semi-pervious material would be used in this case as opposed to an impervious material.

6.7.3.1 Ringing sand boils

Water that issues from sand boils flows through pervious strata under the levee and then breaks through the surface cover, washing with it material from beneath the levee base. A sand boil may gradually undermine a levee and result in a failure by causing sudden subsidence of the levee. It is difficult to evaluate the seriousness of sand boils. Consequently, all sand boils should be watched closely. Any boil that enlarges and increases its discharge of material, especially if located within 60 m of the levee toe, is considered to be a threat to the levee and should be controlled. Treatment of boils, however, is not limited to those within 60 m of the levee toe. Incipient boils should be marked conspicuously so that patrols can locate them without difficulty and observe changes in their conditions. A boil, which discharges clear water in a steady flow, is usually not a serious menace to the safety of the levee. However, as the hydraulic head is increased to stem soil particle movement, the pore pressures within the levee are also increased and slope stability may be affected. The only action necessary in this case is to make careful and frequent observations of the boil and to drain the excess water to prevent its impoundment near the levee. However, if the flow increases and also then carries a material load of sand and silt, corrective action should be taken immediately to prevent levee failure. This measure can be used under these conditions:

- there is a visible sign of sand boil
- the sand boil is of a size to warrant concern
- the sand boil is transporting fines.

This measure should be avoided if there are safety concerns for personnel and equipment.

An accepted method of ringing or sacking a sand boil is shown in Figures 6.38 and 6.39. The base of the sack ring is prepared by clearing the adjacent ground of debris, vegetation or other objectionable material, to a width sufficient for the base of the ring. The base should then be thoroughly scarified to provide a watertight bond between the natural ground and the sack ring (a very important step). The sacks are laid in a general ring around the boil, with joints staggered and with loose earth as mortar between all sacks. In general, it has been found that the best results can be obtained by starting construction of the sack ring at its outer edge and working toward the centre. The ring is carried to a sufficient height to stop the flow of soil from the boil. Work is stopped when clear water only is being discharged. A v-shaped drain constructed of two boards or a piece of sheet metal should be inserted near the top of the ring to carry off the water. A spillway made of sandbags can also be used to discharge water from the sandbag ring.

It is impossible to establish exact dimensions for a sack ring because varying field conditions will govern each specific situation. The diameter of the ring, as well as its height, depends upon the size of the boil and the flow of water from it. Response personnel should determine the size of the ring upon consideration of the following:

- the sack ring should have sufficient base width to prevent side failure
- the width should be determined by the contemplated height of the ring, and should be no less than 1.5 times the height
- the enclosed basin should be of sufficient size to permit the sacking operations to keep ahead of the flow of water. If there are any weak areas close to the sand boil, it is recommended to include them within the ring, and so avoiding the possibility of a subsequent breakthrough.

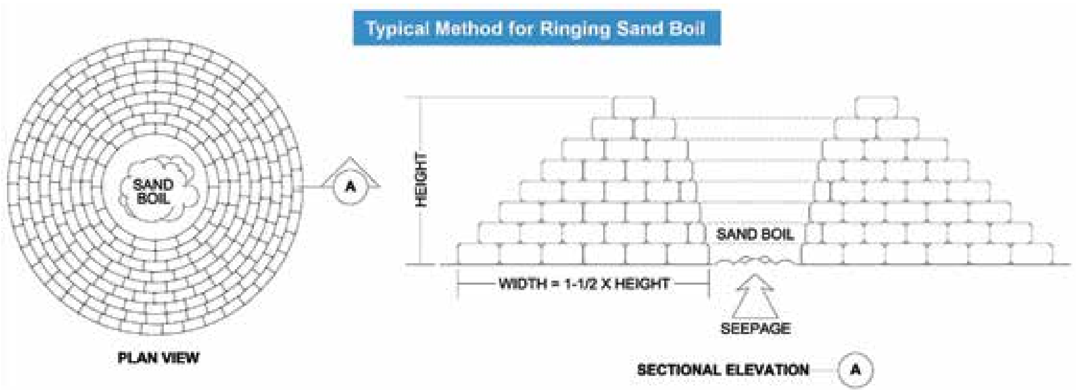

Figure 6.38 *Recommended method for ringing sand boils (courtesy USACE)*

6.7.3.2 Increase landside water level

Another method to reduce the hydraulic gradient through the levee is to raise the water level in landside ditches (if present) or creating small dams on the landside close behind the levee in which the seepage water may be caught, creating ponds on the landside. For this measure it should be ensured that stability will not be threatened by saturation of the levee.

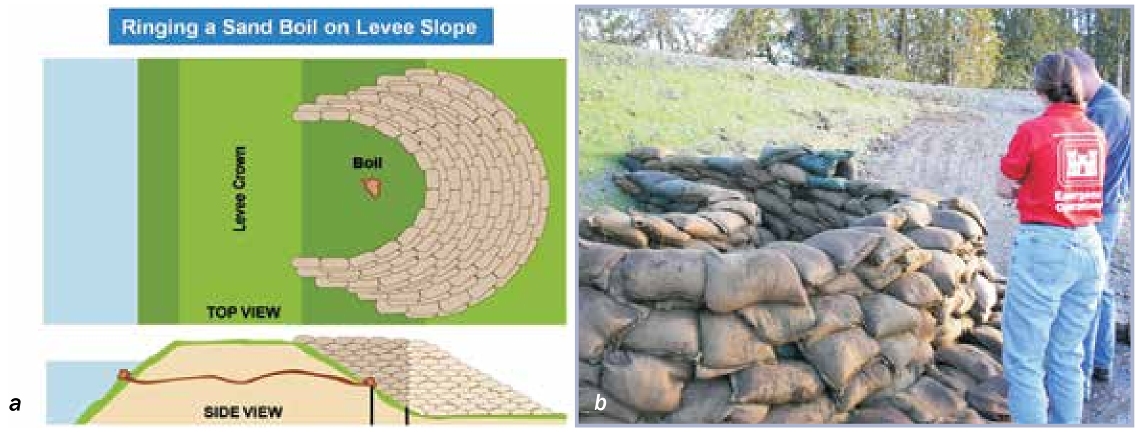

Figure 6.39 *Schematic of ringing a sand boil on levee slope (a) and field implementation (b) (courtesy USACE)*

6.8 RESPONSE TO INSTABILITY AND TECHNIQUES FOR INTERVENTION

Previous discussions related to internal erosion have highlighted many of the concerns related to instability. As the water level on the levee increases the uplift pressures reduce the weight of the soil producing a potentially unstable levee. Also, as the saturation of the levee increases this can reduce the strength of the levee material and create local slope failure on either the landside or waterside. Normally, the local slope failures are small and isolated, which can be relatively quick and easy to repair. These generally do not threaten the overall stability of the levee however they do represent a loss of material that creates a shorter flow path and exposes material subject to erosion or scour. In any event, these local slope failures should either be repaired by placing and compacting soil or covered with plastic/geotextile and monitored until repair can be completed. The techniques that will be presented will address those concerns that are likely to lead to breach of the levee if not addressed. Techniques to mitigate instability generally focus on:

1. Reducing the steepness and inclination of the slope.
2. Reducing water pressures underneath the levee.
3. Reducing groundwater table/saturation of the levee.

6.8.1 Reduce steepness and inclination of slope

Steep slopes, on the order of 1V:2H or steeper, have a higher potential of failure than gentle slopes, on the order of 1V:3H or more gentle. If the stability of the slope is a concern an effective intervention measure is to simply reduce the steepness of the slope. In an emergency response situation this is accomplished by the addition of a landside berm. How flat the resulting slope becomes is dependent mostly on amount of space for constructing the berm and amount of material. All of the previous discussion and examples related to a seepage berm apply for this case. Adding a landside berm serves to not only reduce the steepness of the slope but also adds additional weight to counter the uplift pressures. The use of a landside berm for both instability and internal erosion is presented in Box 6.18.

6.8.2 Reduce uplift pressure

The measures that can be used to reduce uplift pressures in the levee or landside foundation is placement of a landside berm, which has already been discussed in great detail.

6.8.3 Reduce saturation of levee

Since saturation can lead to instability in the levee, reducing the saturation is an intervention measure to improve stability of the levee. This topic has been detailed previously in the section discussing measures to reduce infiltration by use of an impermeable sheeting and sandbags.

Box 6.18 Instability and internal erosion mitigation examples, France

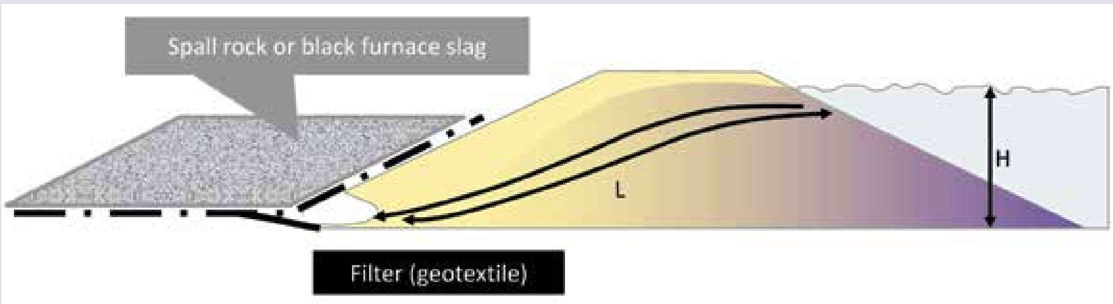

Figure 6.40 ***Through seepage intervention (courtesy SYMADREM after Irstea)***

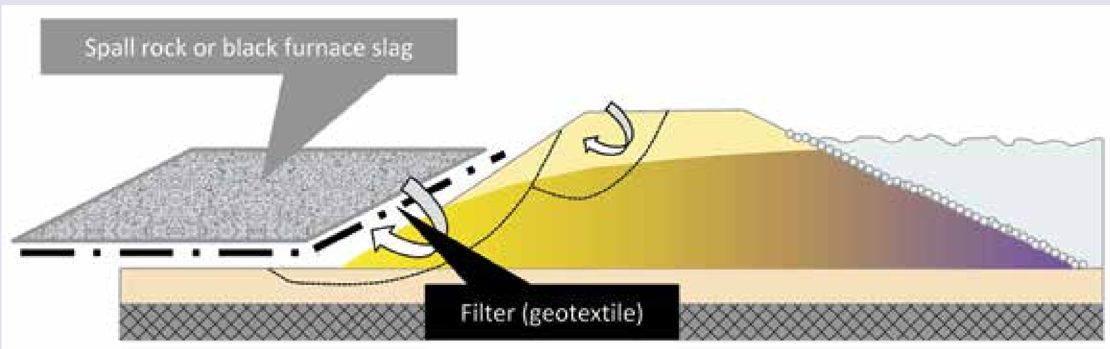

Figure 6.41 ***Stability intervention (courtesy SYMADREM after Irstea)***

Placement of fill on the landward side of the levee along with a geotextile can improve both the resistance to internal erosion as well as global stability of the levee slope.

6.9 BREACH MANAGEMENT AND TECHNIQUES FOR INTERVENTION

Levees can breach before overflow/overtopping (see Figure 6.42) if there are structural issues with the levee making it unstable under a hydraulic load. Seepage, stability, and scour can all lead to a breach if not successfully mitigated during a flood response (see previous section).

Whatever the initiating mechanism, once the levee has breached, the velocities through the breach can be quite high and access to the breach to attempt a closure can be challenging. Appropriate materials to close a breach can be hard to procure and deliver to the site. Large rock is generally the best material, but many alternative materials have been tried in the past ranging from large sandbags (0.75 m^3), to large trees, to railroad cars. Generally, access to a breach will be using trucks along the top of the levee, or in some cases only possible via helicopter, since both the riverward and landward side of the levee will be inundated.

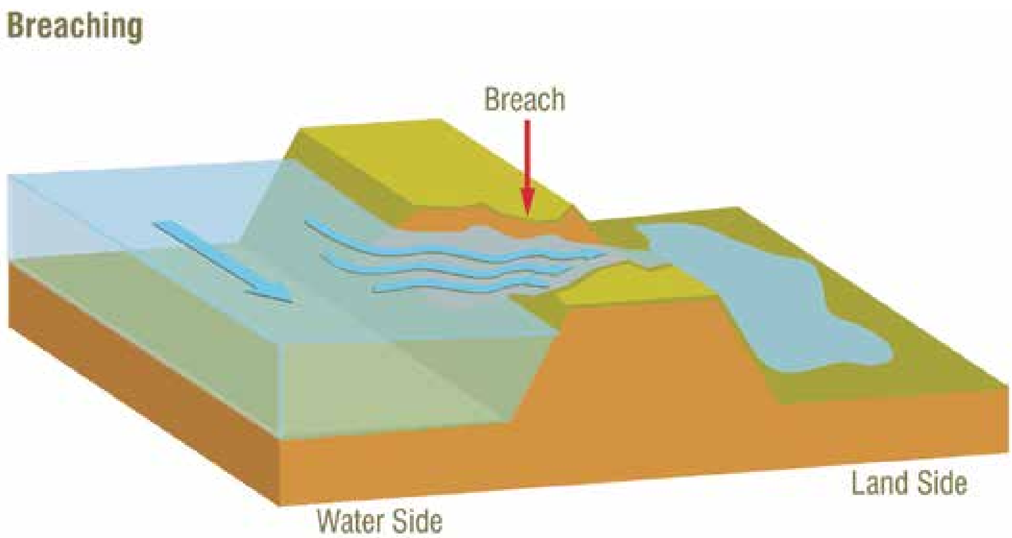

Figure 6.42 Levee breach before overflowing (courtesy ASCE)

Breach growth affects both the levee body and its foundation. There are several stages of a breach (Figure 6.43 presents one type of breach development), which will determine the type of emergency response that can be implemented. The response personnel should be familiar with these stages to assist in arranging for a proper response.

For each stage of breach, the following information provides guidance toward action that should be taken.

Stage 1 and 2: intervention measures should be employed

- use intervention measures as described under external erosion

Stage 3 and 4: still a good chance to prevent full breach and limit breach growth

- use intervention measures as described under external erosion and/or place material in the breach

Stage 5: unlikely to prevent full breach, focus on limiting breach growth

- place material in the breach

Stage 6: repair during the event is only possible for small embankments or with major resources. Focus on limiting breach growth

- if the breach is small and the flow is not dramatic then place material in breach.

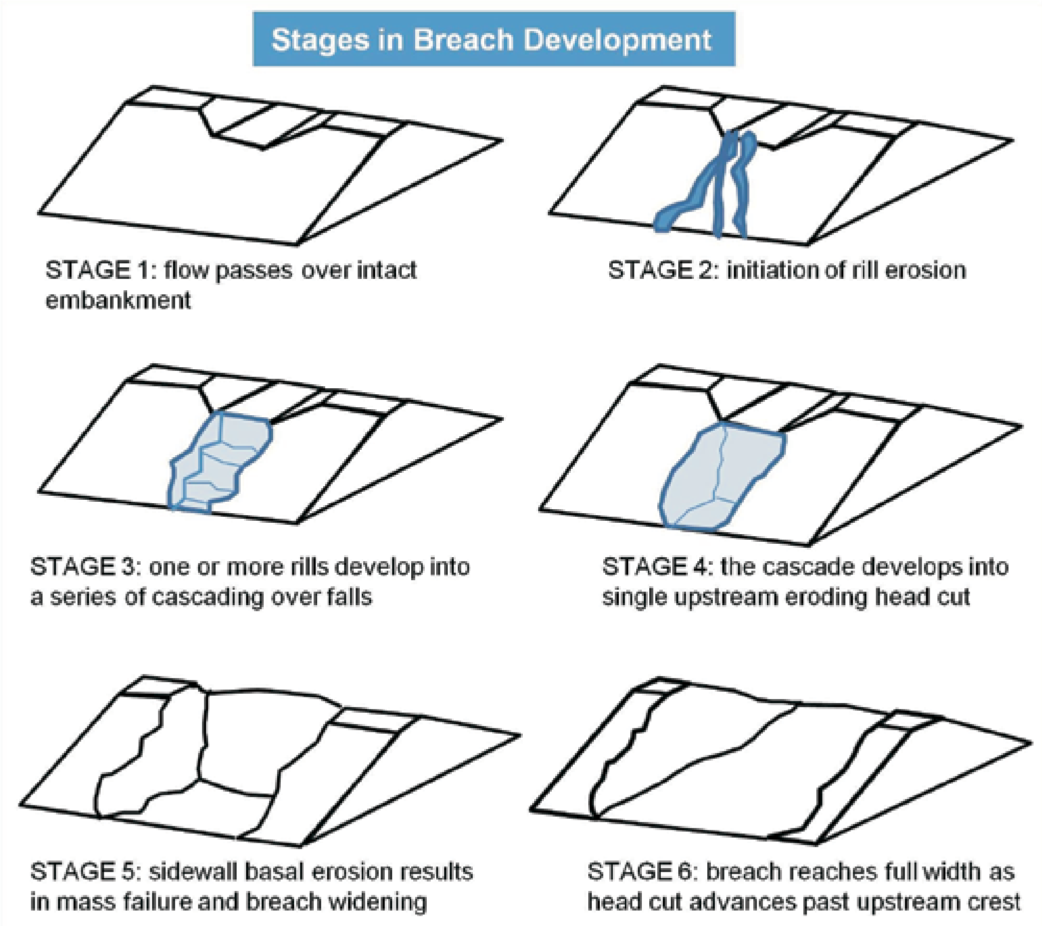

Figure 6.43 Stages of breach (Hahn et al, 2000)

The material necessary for closing a breach with high velocities flowing through it will need to be large and heavy in order to stay in place against the flows (see Figure 6.44). Large rock is typically used for these efforts that will work well to stem the flow, but as a consequence, the closure berm will generally leak substantially. For this reason it is advisable to construct the closure berm either landward or riverward of the breach so that the porous material will not be in the final levee footprint (see Figure 6.46). Material selection should be given considerations such as presented in Table 6.4. Figure 6.45 shows the use of sandbags for closure of a levee breach.

Table 6.4 Material considerations for filling large breaches or under high flow velocities

Considerations	Suggested solution method
• measures with small units (bulk, sandbags) are not likely to seal a large breach and such lighter elements will be washed away before flows are staunched • placement by plant (non-manual) is faster and can be safer. Check if it is safe to intervene in this case	Consider use of more stable larger elements placed inside the breach and placement by machine such as bulk fill, gabions, large sandbags (0.8 m^3), concrete blocks, barge sinking, old cars or similar

Figure 6.44 ***Response to levee breach, limiting breach growth using big bags (a) and gabions (b) (courtesy Environment Agency)***

Figure 6.45 ***Emergency repair of coastal levee Nam Dinh, Vietnam after Storm no.7 (2005) (courtesy VNICZM project, Nam Dinh pilot office)***

Also, consideration should be given to the alignment of a breach closure. It is important to remember that emergency responses to breach closure are temporary measures that may impede the activity of permanent repair once the crisis has passed. Table 6.5 presents three breach closure alignment options with advantages and disadvantages of each.

Table 6.5 ***The following are breach closure alignment options (shown in Figure 6.46)***

Option	Advantages	Disadvantages
A	• shallower water depths • any lost material from closure will add to permanent fix or fill scour hole • can be used as cofferdam for permanent fix.	• longer path and time • more material • trees or obstacles in the way.
B	• shallower water depths • shortest closure path • less time.	• higher velocities • large rock under alignment could interfere with permanent fix.
C	• shielded from river currents • lower velocities.	• longer path and time • more material (could have large scour hole).

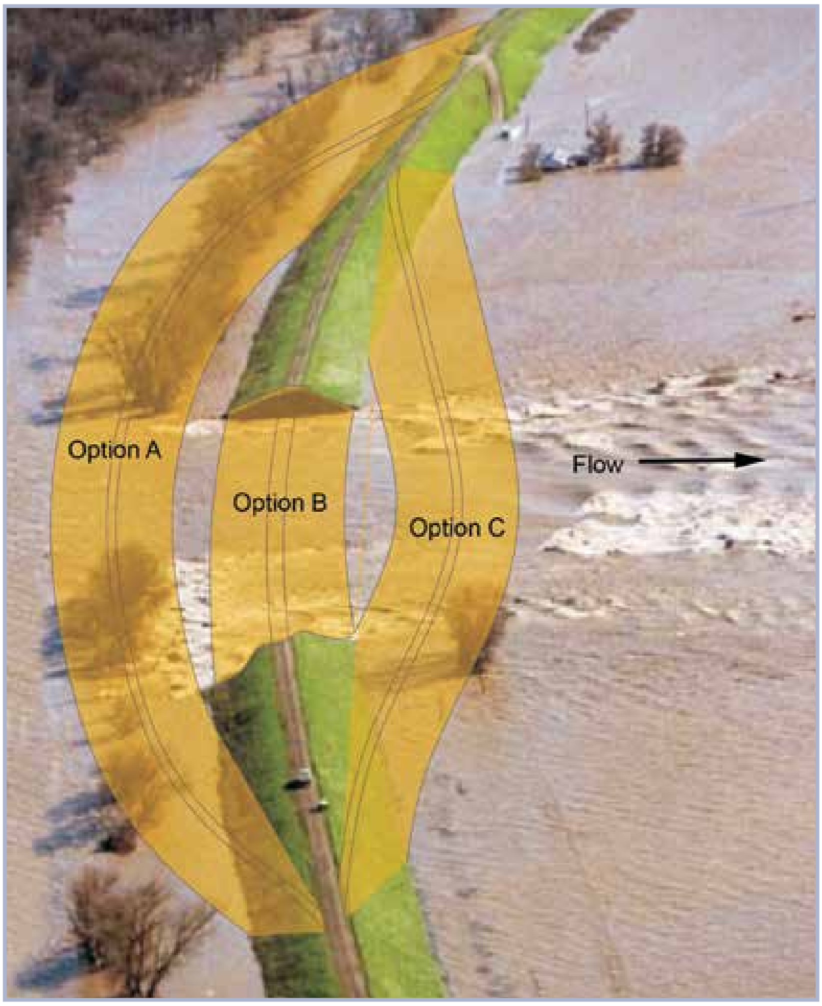

Figure 6.46 *Solution: breach closure alignment options (courtesy USACE)*

6.10 INNOVATIVE TECHNOLOGIES FOR CREST RAISING

Sections 6.4.1 and 6.4.2 presented information concerning material and methodologies that are traditional common means of responding in a flood emergency. However, these are not the only materials and methods available. Boxes 6.19 to 6.25 highlight some common technologies that employ state-of-the art materials and innovative designs. The methodologies presented are not by any means an exhaustive list but those presented are meant to illustrate types of methodologies such as filled tubes, filled containers, freestanding barriers, frame barriers, and sectional barriers. Each methodology has additional features such as being rigid or flexible, air filled or water filled, permeable or impermeable, and automatic or manual. There are a wide variety of other commercial solutions based on the same principles.

Although all of the innovative systems shown can be deployed on a levee crest, each situation needs to be evaluated to ensure that the right solution is chosen for each application. However, the application of these technologies have proved to be successful and in some situations serve as the method of choice for response actitivity.

Box 6.19 Open-celled plastic grid flood wall

Details

- collapsible plastic grid 20.3 cm high
- expands into 1.2 m × 1.2 m or 1.2 m × 0.6 m sections
- interlocks
- filled with sand from the top with a loader, excavator, bottom-dump, or other piece of earthmoving equipment
- light enough to be handled by two people
- small enough to be manageable in the wind
- fits into a pickup truck bed or helicopter
- requires no special tools
- small footprint and cross-section.

Figure 6.47 Plastic cell field setup (courtesy USACE)

Advantages	Disadvantages
Easy and quick to construct	Some breakage of grid if handled roughly
Very stable, even on soft soils	Added weight may decrease slope stability
90 % reusable	Requires machinery to fill
1.8 m wide footprint (1.2 m high structure)	Some difficulty in removing fill from cells after use
Very low seepage	

Box 6.20 Portable cofferdam systems

Details

- uses a steel supporting structure
- continuous reinforced vinyl liner membrane
- means of water diversion, retention, or impoundment
- the support structure is designed to transfer hydraulic loading to a near vertical load, thereby creating a free-standing structure with no back brace to extend to work area
- the liner system is flexible, sealing most irregular contours
- this system can be installed almost anywhere, in any configuration, and to any length
- the equipment is offered as rental item in heights of 1 m, 1.5 m, 2 m and 3 m.

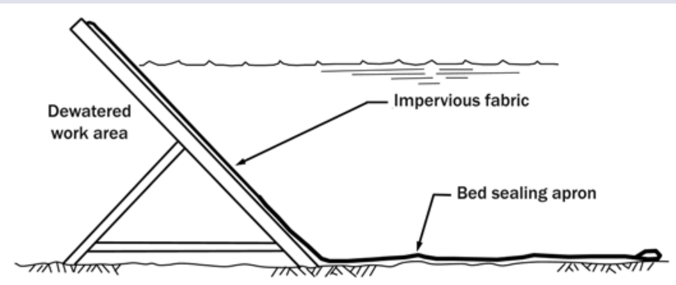

Figure 6.48 Portable cofferdam diagram (courtesy USACE)

Advantages	Disadvantages
Easy and quick to construct and remove	Height cannot be increased if flood worsens
Low seepage once bed seal established	Requires wide levee topwidth to be set up on levee crest
100 % reusable	May be damaged by floating debris
Stable, except on soft soils	Access to levee top limited when installed

Box 6.21 *Portable dam system*

Details

- rapidly deployed and removed
- assembled along pre-installed anchor line and seepage barrier
- a standard element is 2.1 m long
- 0.75 m to 1.2 m level of protection
- custom elements available
- 1.2 m wide foot print (1.2 m high structure)
- 100 per cent reusable and repairable.

Figure 6.49 *AquaFence implementation (courtesy USACE)*

Advantages	Disadvantages
Easy and quick to construct and remove	Should have pre-installed anchor line and barrier
Low seepage	Height cannot be increased if flood worsens
100 % reusable	Requires a large levee crest to setup

Box 6.22 *Water-inflated barrier*

Details

- water inflated property protector – uses any water source
- industrial-grade, vinyl-coated polyester membrane material
- internal baffle system provides role over stability
- 0.3 m to 2.5 m high tubes provide protection to 75 per cent or 0.2 m to 1.8 m high
- rapidly deployed and removed
- 3.8 m wide foot print (1.5 m high protection)
- stores compactly.

Figure 6.50 *Water inflated barrier field setup (courtesy USACE)*

Advantages	Disadvantages
Uses readily available water as fill material	Height cannot be increased if flood worsens
Can be positioned very quickly	Requires wide levee topwidth to setup on levee crest
Very stable, even on soft soils	Added weight may decrease levee stability
100 % reusable	Access to levee top limited when installed
	Can be punctured by equipment or vandals

Box 6.23 Water-filled tubes

Details

- uses any water source
- flexible interlocking tubes
- single tube 0.5 m diameter, 15.25 m long
- stack like a pyramid up to 6 m held together by straps
- rapidly deployed: 1.5 minutes from fire hydrant or three minutes by pump
- height easily added by strapping next row to current structure
- easily removed, using water for washing
- three tubes high: 1.5 m wide foot print (1.2 m high protection)
- stores compactly, 15.25 m delivered in 200 litre drum
- can be filled with concrete.

Figure 6.51 Tiger dam system implementation (courtesy USACE)

Advantages	Disadvantages
Easy and quick to construct and remove	Can be punctured by equipment or vandals
Can adapt installation to situation	Requires wide levee topwidth to setup on levee crest
100 % reusable	May be damaged by floating debris
Height of protection can be increased if flood worsens	Access to levee top limited when installed
Very stable even on soft soils	

Box 6.24 Filled permeable container

Details

- cellular barriers of permeable material
- lined with geotextile or geosynthetic fabrics
- filled with aggregates to form barrier
- containers strengthened and held in place by wire meshes, pins, frames
- impermeability controlled by fill material
- stackable, flexible, conform to foundation.

Figure 6.52 Example of filled permeable container (courtesy USACE)

Advantages	Disadvantages
Height of some systems can be increased by stacking	Clogging of material/effluents within the fabric can make cleaning difficult or impossible
Can be installed by relatively unskilled labour	Stacked defences require significant width, which may not always be available
Small storage space required	Some steel supports and pins may buckle or deform beyond reuse under stacking and service loading
Adapts to uneven terrain	Need to dispose of large volumes of probably contaminated material after flood event
Can use readily available fill material	Seepage can be a problem, but this can be minimised by using a suitable choice of geotextiles and fill
	High bearing pressure on bedding surface when stacked
	Some can be reused, but only a limited number of times

Box 6.25 Demountable barriers

Details

- rigid panels placed horizontally between stanchions
- permanent foundation with stanchion guides
- lined with seals to ensure water tightness
- stanchions can be permanently installed or attached to installed connections.

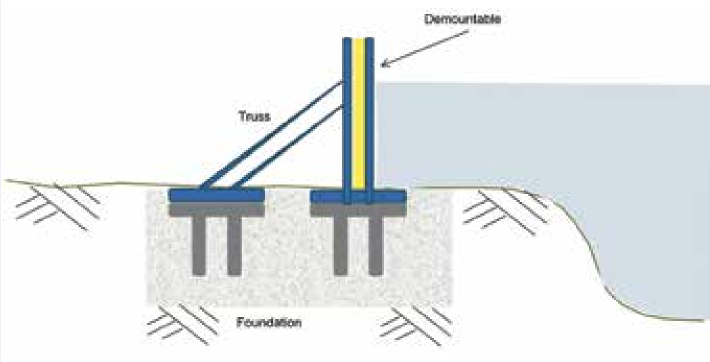

Figure 6.53 Schematic of demountable barrier (see Chapter 4, Table 4.3 for field application)

Advantages	Disadvantages
Generally robust and well engineered	Large storage area required
Good resistance to loading and impact	Heavy transportation and lifting requirements
Very durable	Long installation and mobilisation period
Can be increased in height by adding panels up to the height of the frame	Permanent parts susceptible to damage and vandalism
Very low seepage through and under the structure	

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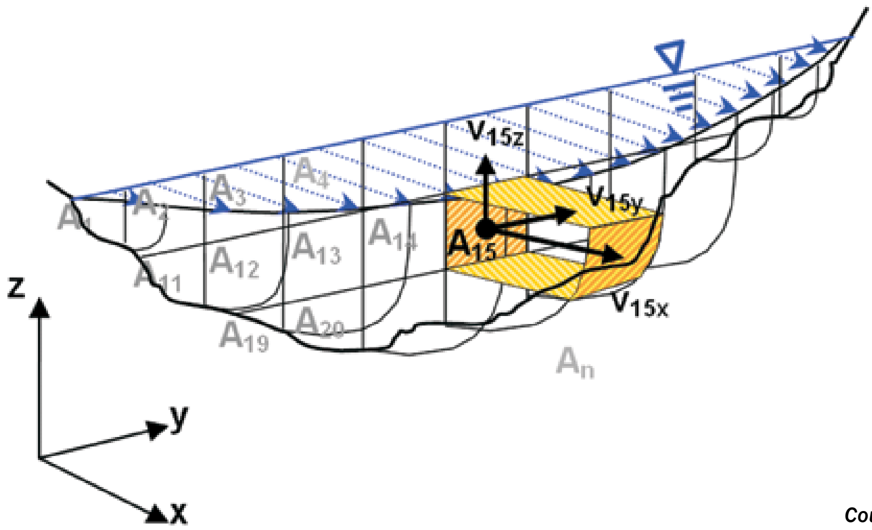
7 Site characterisation and data requirements

Courtesy S N Wersching



Courtesy R Pohl

CHAPTER 7 CONTENTS

7 SITE CHARACTERISATION AND DATA REQUIREMENTS

Chapter 7 details approaches to hydraulic, morphological and geotechnical site characterisation and data collection.

Key inputs from other chapters

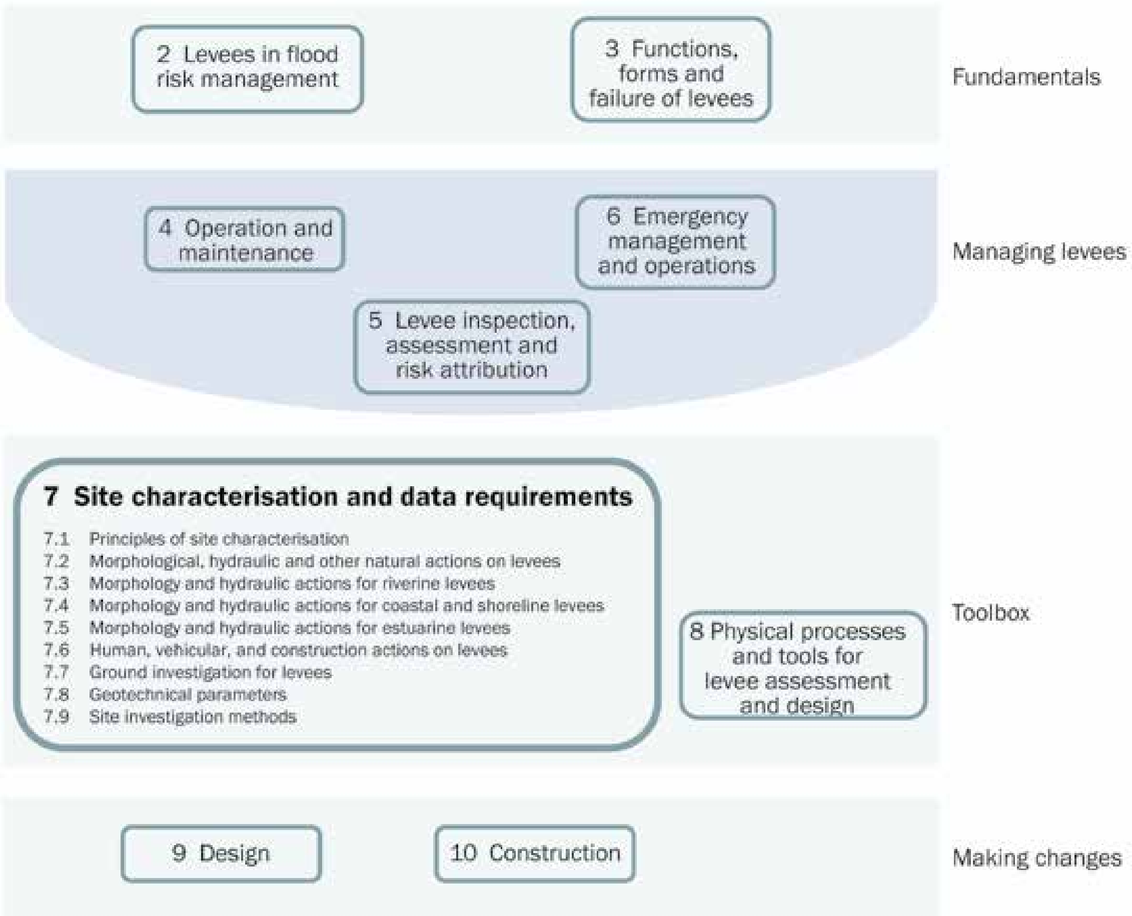
- Chapter 5 ⇨ **data requirements for analytical evaluation**
- Chapter 9 ⇨ **evaluation of data requirements**
- Chapter 10 ⇨ **field data for analytical evaluation**

Key outputs to other chapters

- **data defining site characteristics** ⇨ Chapters 4, 5, 8, 9 and 10

Note: The reader should revisit **Chapters 2 and 3** throughout the levee life cycle for a reminder of important issues.

This flow chart shows where to find information in the chapter and how it relates to other chapters. Use it in combination with the contents page to navigate the handbook.



CHAPTER CONTENTS AND TARGET USERS

This chapter is divided into nine sections, providing information relative to characterising both the loading and actions applied to levees, and characterising the nature of the soils from which levees are formed and founded.

Principles of site characterisation

Section 7.1 discusses the need for characterising a site. It maps at a high level the process by which this is done through a phased approach to suit the requirements of the project at a given stage of development. The approach is aimed at ensuring that the process is carried out efficiently and economically, minimising expenditure and time input while maximising the information gained. As part of this process the development of a conceptual site model (CSM) is considered. It outlines the steps in the development of a CSM from an initial desk study to the development of a more detailed model obtained through site specific investigations. It reinforces the idea that the CSM is a working document and should be revisited and revised as more information becomes available during the course of the project.

Morphological, hydraulic and other natural actions on levees

Section 7.2 provides information related to the interaction between levees and environmental processes, the influence of seasonal change and extreme events, the influence of long-term changes, and actions from other natural processes.

Morphology and hydraulic actions for riverine levees

Section 7.3 considers all the actions and loads necessary for evaluation or design of riverine levees. This includes fluvial morphology, hydraulic actions, stream flow data, river geometry, river flow data, basic energy and flow states, flow velocities, water levels and depths, sediment transport, effects of wind and waves, ship induced currents, ice effects. Additional material is provided to address modelling of flaluvial loads and actions as well as uncertainty in data and analysis.

Morphology and hydraulic actions for coastal levees and shoreline levees

Section 7.4 considers all the actions and loads necessary for evaluation or design of coastal and shoreline levees. This includes coastal morphology, water levels, waves, tsunamis, marine currents, extreme water levels and waves, and joint probability of waves and water level. Also, information is provided to address sources of global and local data as well as uncertainty in data and analysis.

Morphology and hydraulic loads for estuarine levees

Section 7.5 considers all the actions and loads necessary for evaluation or design of estuarine levees. This includes estuarine morphology and hydraulic actions.

Human, vehicular and construction actions on levees

Section 7.6 considers other additional actions on levees from human, vehicular and construction activities.

Ground investigation for levees

Section 7.7 discusses the processes by which the techniques of geotechnical investigation and instrumentation can be combined to good effect to understand the nature, properties and performance of the geoenvironment within which the levee has to operate. This includes the ground on which the levee is founded, and the materials that form it.

Geotechnical parameters

Section 7.8 provides a summary of the techniques available to evaluate the geotechnical parameters required for the analysis of levees. These include indicative values, correlations with index properties, and laboratory and field tests. The evaluation of a characteristic value from the data is discussed.

Site investigation methods

Section 7.9 provides a summary of the techniques available to obtain the data to characterise the environment within which the levee operates.

7.1 PRINCIPLES OF SITE CHARACTERISATION

Levees are structures formed from naturally occurring, usually locally sourced, materials that interact with the environment in which they are placed. They are usually only periodically subjected to the extreme hydraulic loads for which they have been designed, although some may impound water permanently. In order to understand these interactions and to achieve a cost effective solution with the appropriate level of resilience, there is a need to quantify the characteristics of the environment where the levee is located. This chapter outlines the factors that may be considered when characterising a site and its environs to assess the condition of or undertake improvements to existing levees and to facilitate the design of new ones.

At a high level this chapter contains two main themes:

1 The loads that act on levees be they hydraulic or from another source.

2 The nature of the soils from which the levee is formed or founded on.

This information is combined to form a conceptual site model (CSM). The model may be used as a basis on which to assess the performance of a levee whether existing, improved or new.

This chapter is considered to be a 'tool box' chapter within the handbook. It aims to provide the designer with the tools to obtain the information to characterise the site and to develop a robust CSM containing an appropriate level of detail that is sufficient to undertake a performance assessment or design of a levee. As such the chapter is targeted at those involved with the condition assessment and design of levees. It also provides the levee operator with an understanding of the processes that the designer needs to undertake in order to obtain information on the site to an appropriate level to understand its characteristics, both hydraulically and geotechnically.

The chapter is divided into nine subsections, each dealing with a discrete element of the site characterisation process. A flow chart mapping the outline structure and contents of the chapter is presented in Figure 7.1.

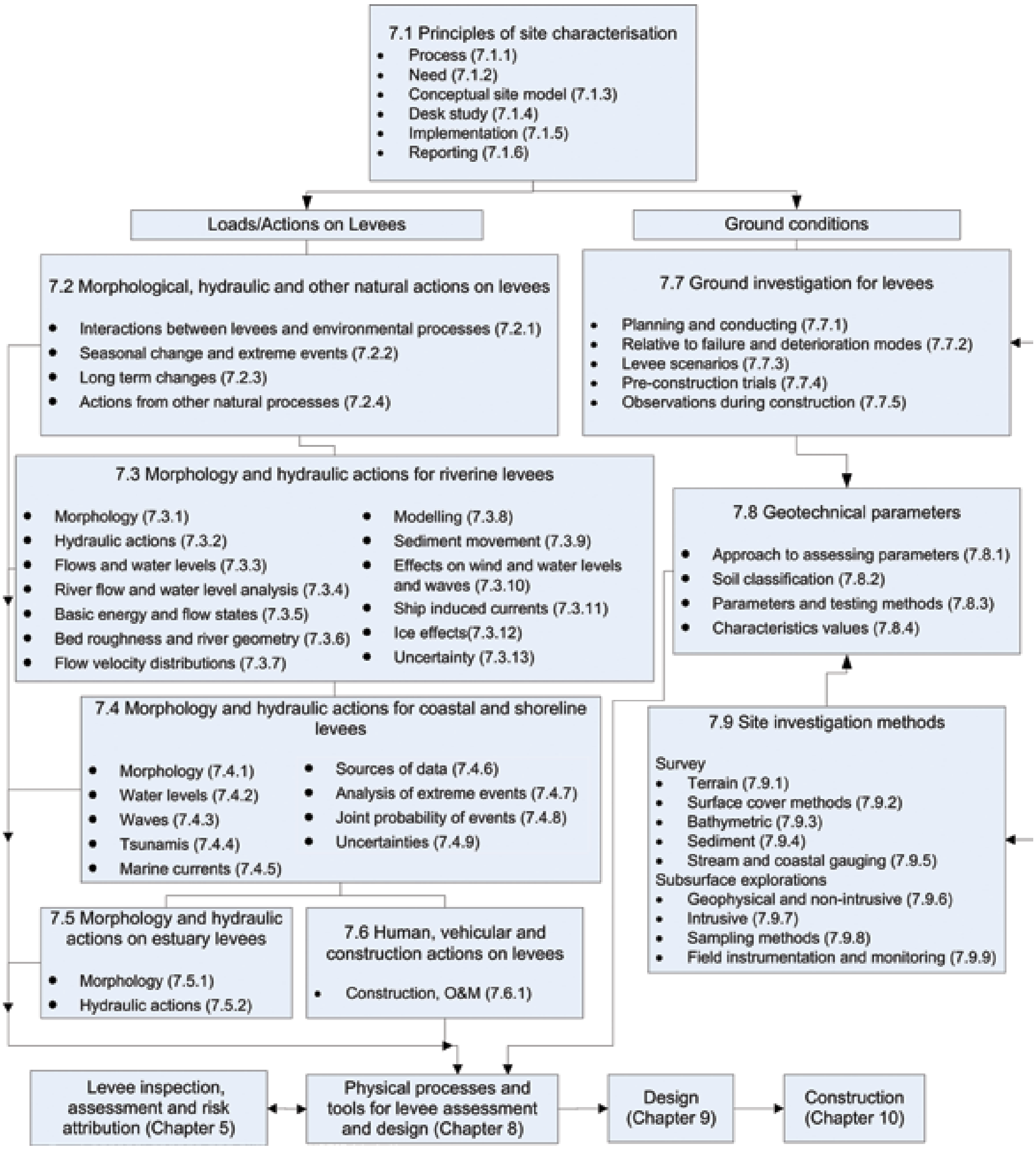


Figure 7.1 ***Chapter structure and contents***

Site characterisation is the process by which information on the physical aspects and time dependent processes at a site are gathered and interpreted. The information can be collated into a working document, referred to as the CSM (Section 7.1.3), which contains the fullest understanding of site conditions at the current time. This information may include data on bathymetry, morphology, hydraulic and other forms of actions that interact with the levee, including its internal composition and associated structures, ground conditions and land form. An understanding of these elements, together with the ways in which they interact and how they may be affected by climate change is critical to the successful assessment of the performance and design of a levee system with sufficient resilience to withstand a future design event. A flow chart mapping the outline structure and contents for the rest of Section 7.1 is presented in Figure 7.2.

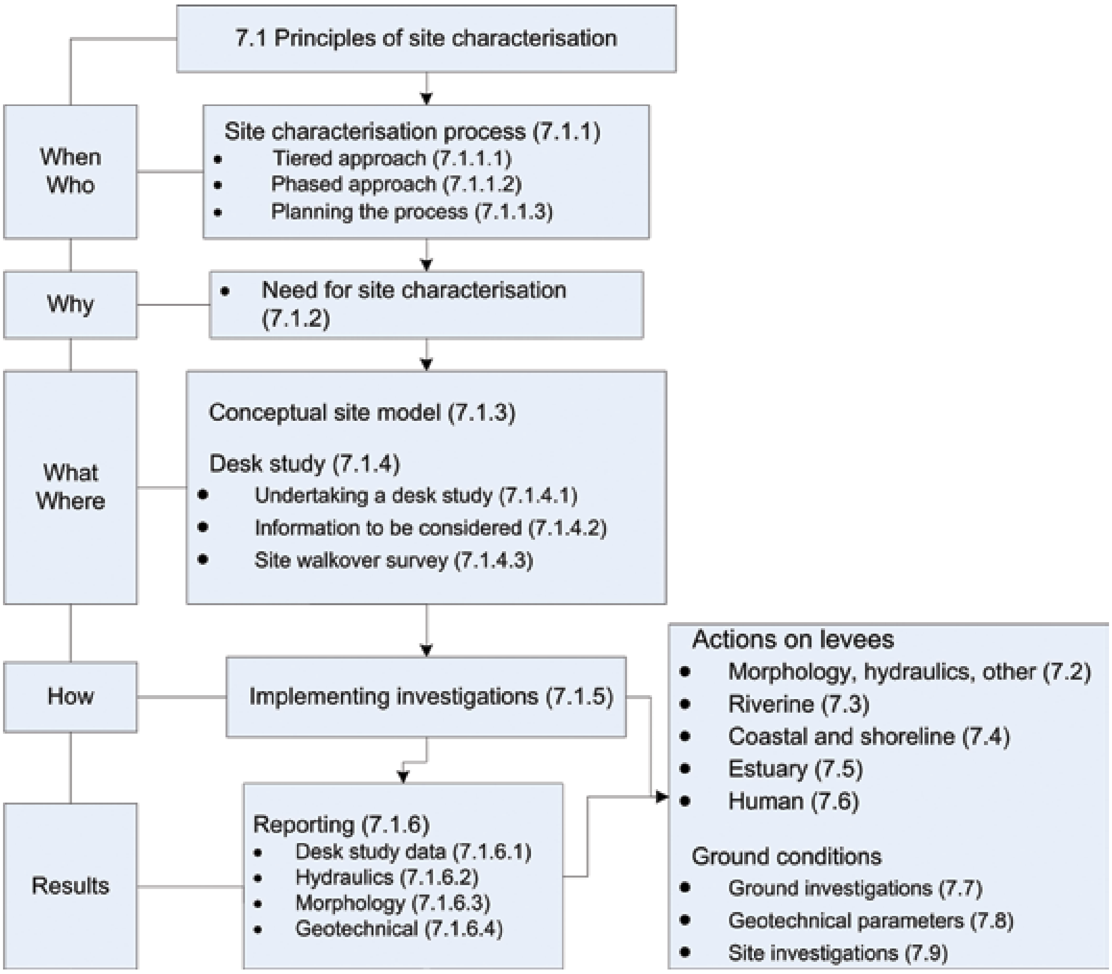


Figure 7.2 ***Structure and content of Section 7.1 and interaction with other subsections***

7.1.1 Site characterisation process

When embarking on a programme of site characterisation consideration needs to be given to the processes by which it will be undertaken, the timescale over which the activities will span, the resources required and the interaction of the technical disciplines throughout the course of the characterisation process. The starting point for the process is the assumption that a need for action has been identified.

7.1.1.1 Tiered approach to characterisation

Some countries allow a tiered approach to be adopted for characterisation so that the effort in characterisation is proportionate to risk factors such as the height of the levee and the damage potential in the event of a breach. There is further discussion on the geotechnical categorisation of levees in Section 9.2.4.

An example of the application of a tiered approach to geotechnical site investigation is presented in Box 7.1.

Box 7.1 Application of a tiered approach to geotechnical site investigation (from DWA, 2013)

For low levees, which may comprise a small levee less than 1.5 m high with a low damage potential, on ground that does not include any adverse geotechnical features, it may be sufficient to evaluate ground conditions from:

- local knowledge based on construction experience in the immediate area
- undertaking simple forms of intrusive investigation (trial pits, window samples and probing)
- inspection of material exposed in the landside drainage channel.

For higher levees with a high damage potential a more rigorous approach to geotechnical site investigation is appropriate to assess the ground profiles and internal structure of the levee, and to evaluate geotechnical parameters relevant to the required analyses to be performed.

7.1.1.2 Phased approach to characterisation

The characterisation process generally involves several phases. Each phase leads to the successive collection of more data leading to a better appreciation of the site characteristics, a more robust understanding of design and serviceability issues, and a refinement of the condition assessment and/or analyses. This section addresses the broader approach to characterisation, such as might be required to design a new levee. Some aspects are also applicable to assessing the condition of an existing levee and the need for and nature of improvement works to an existing levee. The site characterisation approach for assessing the condition of an existing levee is detailed in Sections 5.5 and 7.7.3.

Each phase of work should have specific objectives aimed at providing information on the site to a level of detail appropriate to the requirements of the project at that time, and allowing subsequent phases of work to be scoped to target the known issues. Failure to adopt a phased approach to characterisation could result in inappropriate characterisation that does not provide the level of information required by the project. This could lead to delays as supplementary studies may be required late in the process and higher costs may be incurred through abortive work started too early and later found to be unnecessary. Typically the characterisation process may involve three sequential phases or tiers as presented in Table 7.1.

Table 7.1 Typical phases of the characterisation process

Phase	Activity
Reconnaissance	Initial assessment based on available information collected through a desk study. Some investigation may be required to support the hydraulic characterisation of the site. The information is sufficient to allow rudimentary modelling and calculations to be undertaken for a few options, which can be used to assess the viability of proceeding with some form of flood risk management project.
Feasibility	More detailed information gathered and more extensive modelling and calculations undertaken for a broader array of options. Information is sufficient to allow the options to be worked up to a level of detail that allows them to be costed so that 'decision makers' can determine which option is to be taken forward to construction. This may include developing a better understanding of the interaction of the levee, ground and hydraulics and the issues that will affect the levee performance. Findings allow the scoping of additional data collection (quantity and nature) required for the detailed evaluation. At this stage the hydraulics is typically well defined but additional field data may be required to support the more sophisticated analytical/numerical models. Where no or very limited quantitative data on ground conditions is available, limited investigations may be required to inform the geotechnical assessment.
Detailed	The primary objective of this stage is usually the characterisation of the ground through a rigorous programme of investigation. Some additional hydraulic data may be required but this is usually well defined by this stage. There may be a need to update or refine the data, specifically if there is a need for advanced hydraulic modelling. The development of the final project schema and preparation of construction information (plans and specifications). This may be for the design of a new levee or improvement works to an existing levee.

A summary of three potential phases of site characterisation for hydraulics and morphology, and geotechnics, together with an indication of the information required and the uses to which the information may be put is summarised in Table 7.2 and 7.3. A more detailed consideration of these processes is considered in Sections 7.3 to 7.7.

Table 7.2 Phasing of investigations – hydraulics and morphology

<table>
<tr><th colspan="2">Investigation phase</th><th>Typical information</th><th>Use of information</th></tr>
<tr><td rowspan="2">Reconnaissance</td><td>Office based collection of data</td><td>• bathymetry
• topography, LiDAR
• aerial photographs
• rudimentary models
• approximate estimates of stage-discharge relationships
• statistics for available gauge data (discharge, water levels, waves, winds, currents)
• information from existing schemes.</td><td rowspan="2">• provide a general understanding of the river, coastal or estuarine system. The level of understanding will vary with the extent of information available
• provide information to inform scoping of subsequent phases of investigation
• provide information to support preliminary project/design decisions.</td></tr>
<tr><td>Field based collection of data</td><td>• observation of stream channel: sediments, vegetation, floodplain, existing infrastructure and existing flood control features
• identify potential impact areas
• morphologic assessment of river or coastal system
• water levels
• waves and currents system
• topography and levels.</td></tr>
<tr><td colspan="2">Feasibility</td><td>• additional bathymetric and topographic surveys
• develop detailed hydrologic and hydraulic models for river, coastal or estuarine system
• morphological studies
• perform risk-based analysis.</td><td>• support development of higher resolution models
• assess system performance
• evaluate alternative solutions
• set top of levee elevation
• determine spillway requirements.</td></tr>
<tr><td colspan="2">Detailed</td><td>• develop operation and maintenance (O&M) manual
• design of spillways/overflow sections
• detail calculations/modelling for erosion protection.</td><td>• provide owner information and instructions to operate and maintain project
• provide sufficient information to allow the detailed design to be developed.</td></tr>
</table>

Where the investigations include other ground related issues such as archaeology, contamination and unexploded ordnance (UXO), an assessment of these factors should be undertaken by appropriate specialists. The findings should be included within the CSM as they present risks to the project (safety, cost, programme) and need to be taken into account by subsequent phases of the work.

Table 7.3 Phasing of investigations – geotechnics

<table>
<tr><th colspan="2">Investigation phase</th><th>Typical Information</th><th>Use of information</th></tr>
<tr><td rowspan="2">Reconnaissance</td><td>Office based collection of data</td><td>• topography, LiDAR, geological maps
• aerial photographs
• existing exploratory hole logs
• information from existing schemes.</td><td rowspan="2">• provide a general understanding of the ground conditions. The level of understanding will vary with the extent of information available
• preliminary identification of failure and deterioration modes
• provide information to inform scoping of subsequent phases of investigation
• provide information to support preliminary project/design decisions and, if required, identify likely sources and availability of local borrow materials.</td></tr>
<tr><td>Field based collection of data</td><td>• observation of surface soils, rock (where exposed), landforms and other site features
• changes in vegetation
• areas of poor drainage
• evidence of instability
• seepage.</td></tr>
<tr><td colspan="2">Feasibility</td><td>• geophysical survey
• widely spaced exploration holes with laboratory and in situ testing
• cone penetration tests (CPT) at and between exploration holes
• trial pits with laboratory and in situ testing.</td><td>• confirmatory data to support the desk study and improve the interpretation of non-intrusive investigations
• evaluate suitable and likely quantities of local borrow materials
• provide site specific quantitative data to support outline design, assessment of risks and to inform early project decisions.</td></tr>
<tr><td colspan="2">Detailed</td><td>• additional more closely spaced exploration holes with laboratory and in situ testing
• groundwater observations, in situ testing (slug tests, pump tests), and models if warranted.</td><td>• provide sufficient information to allow design to be fully developed, including the detailed assessment of borrow material, if a source has been identified.</td></tr>
</table>

7.1.1.3 Planning the process

The site characterisation process should adhere to a credible time schedule defining the project activities, milestones, resources (staff, equipment, funding), and deliverables. The schedule needs to allow sufficient time to undertake the interdependent activities in a logical sequence with appropriate resources. It should be driven by the scale of the project, allowing sufficient time to undertake the data collection and technical evaluation of the site to the required level.

Other factors that may need to be allowed for in the time schedule are:

- environmental aspects, including flora and fauna, and socially sensitive areas may impose constraints on site working hours during the day, limit site work to specific times of the year, require longer term constraints on the project and impose restrictions on the method of work
- consultation may be required with stakeholders, third parties or the public
- legal action may be required to gain access or defend legal challenges to undertaking the works.

Staff resources need to be identified at the start to suit the scale of the project and the programme. A full team may include a geotechnical engineer, hydraulic engineer, geologist, geomorphologist, hydrogeologist, biologist, environmental/regulatory specialist, archaeologist, historian, as well as stakeholders (municipalities, levee owners and operators, neighbouring landowners, and members of the population) served by the levee.

There are advantages to making team members aware of larger project issues outside their immediate technical disciplines. They will be able to identify any information they encounter that has a bearing on these issues and raise them with the team leader for dissemination.

Examples of such issues might include:

- borrow material source areas

1 2 3 4 5 6 7 8 9 10

- access issues
- construction logistics
- adjacent flood control projects
- interior drainage.

Managing and keeping records from site characterisation activities is crucial as they document the baseline data, the processes, analysis and assumptions made in characterising the site, and as such form part of the CSM. Over time, with the collection of more data from other and subsequent phases of investigations, along with monitoring and performance observations, the interpretations within the CSM will need to be re-evaluated and updated or augmented, in-line with it being a working document. The records provide a valuable first source of information when researching clues to explain any issues identified during later inspections and condition assessments. They also provide a source of information for future maintenance and improvement works.

It is good practice to collect and collate relevant and required records, and to make them available to the owner/operator of the levee. This includes both physical hard copies and electronic files. Where data is stored electronically both the storage medium and data format need to be kept up-to-date. Metadata ie 'data about the data', should also be recorded as methodologies can change over time and can affect how the data are used.

Table 7.4 summarises where the types of data collected during site characterisation are discussed and presented in text and tables. Section 5.6 and Table 4.1 contains further information on data management.

Table 7.4 Data collected during site characterisation

Type of data	Section	Table
Desk study information	7.1.4.1 and 7.1.4.2	7.7
Site walkover survey	7.1.4.3	7.8
Hydraulics	7.1.6.2	7.9
Morphology	7.1.6.3	–
Geotechnical	7.1.6.4	7.10

7.1.2 Need for site characterisation

In simple terms if a site is not characterised there will be no criterion against which the performance of the levee can be assessed or a design undertaken. From a hydraulics, hydrology, and morphological perspective, the principal purpose of site characterisation is to collect information about the movement of surface water in and through the catchment or coastal zone with the aim of developing models that represent the physical processes involved. Geotechnically the characterisation process aims to establish the spatial distribution and engineering properties of the soils forming the foundation and the levee, evaluate the existing groundwater regime, and identify and assess potential sources of borrow material, if necessary.

The characterisation process may also be required to address other ground related issues such as archaeology, contamination and UXO. Specialist advice should be sought where these aspects are to be investigated as they are outside the scope of the handbook and the requirements may vary with local codes, standards and regulations.

At the same time consideration should be given to the non-engineering environmental and ecological aspects of the site and the implications of their interaction with any works. (Chapter 2, Section 3.1.2 and Chapter 10).

Some influences and their effects on the levee and its environs that are typically considered in the site characterisation process are summarised in Table 7.5.

Table 7.5 Some influences on site characteristics of the levee and its environs

Influence	Effects on site characteristics
Hydraulics of river, coastal and estuarine environment	• interaction of levee and hydraulics is a dynamic system and its behaviour is complex • levee construction will change stage and discharge probability relationships • levee construction or modification may alter the sediment transport regime causing additional change in water levels over time • wave action will influence levee height and requirements for erosion protection • existing flood management projects alter system behaviour and will interact with proposed levees • new or improved levees may impact on existing stabilisation/protection measures, which were installed to mitigate instability problems • changed water depth and duration resulting from levee construction may adversely affect wetland function and habitat • changes may be required to internal drainage systems.
Morphology of river, coast and estuarine environment	• levee construction will change system dynamics that may impact on levee function through the alteration of hydraulics and habitat, leading to changes in flows, scour and deposition patterns, and vegetation • morphological processes are a function of timescale. Unstable conditions can result in under cutting of the levee through channel migration or beach lowering • morphological thresholds exist that when exceeded can result in abrupt changes.
Geotechnical	***Natural geological deposits and manmade ground***
	• natural deposits can display a complex spatial distribution, heterogeneity and variability in geotechnical properties over short distances, complicating characterisation • natural localised features may be present on the ground that could adversely affect the performance of the levee • manmade ground can be heterogeneous and geotechnical properties may be difficult to evaluate • manmade ground could be contaminated. Contaminants could be mobilised through changes to the surface and groundwater regime caused by the levee • identification of usable sources of locally won borrow material. The economics of levee construction is significantly influenced by the cost of borrow materials, which is reflected by the transportation costs.
	Groundwater-surface water interaction
	• where a groundwater cut-off is required to limit seepage during a flood event it can disrupt the natural groundwater flow toward the water body during normal conditions. This may result in an elevated groundwater level and the potential to mobilise contamination.
	Existing levee
	• the levee can have a complex internal structure due to successive phases of historical raising or repair, or through design by the inclusion of drains, filters, relief wells, conduits, sheet piling or other penetrations, which can influence the performance of the levee • the levee can include or may requires surface feature such as armouring, vegetation and a crest wall to improve resilience and the flood protection level.
	Existing instrumentation
	• the interaction of an existing levee with the foundation soils and hydraulics may be defined by data from existing instrumentation.

Table 7.5 Some influences on site characteristics of the levee and its environs (contd)

Land use (past, current, future)	***Past land use***
	• the presence of archaeology, ordnance and contamination may affect project acceptance/approval, design, cost and programme, and pose issues during construction • historic infrastructure may create preferential seepage pathways or hindrances to construction, and may require design changes or removal/mitigation during construction.
	Current land use
	• may impose constraints on the design and construction activities • influence runoff characteristics, resulting still water levels and flow volume for rivers • influence shoreline behaviour in coastal area • as a secondary function the levee may be used as a route for infrastructure, which will impose a load on the levee and may constrain future improvement works.
	Future land use
	• long-term regional development plans for the area may constrain design/layout • future development may affect runoff characteristics and resulting still water levels and flow volume for rivers • future infrastructure may add unexpected loads to levee • future development increases the population and infrastructure to be protected, and so may affect selection of flood protection level and overall risk management.
Local climate	***Current climate***

	• **precipitation:** annual total and seasonal intensity (arid regions have <400 mm/year) influences the:
	  ◦ role and type of vegetation used as surface protection
	  ◦ runoff characteristics, which affect still water levels and flow volume for rivers
	  ◦ performance of fill material in terms of desiccation, fissuring and cracking
	• **temperature:** daily and seasonal variations influences the:
	  ◦ role and type of vegetation used as surface protection
	  ◦ formation of ice and whether its actions on the levee need to be addressed
	• **wind:** mean speeds (wind run), gust speeds, direction and return periods influence the:
	  ◦ wave and storm surge prediction for coastal and estuarine levees
	  ◦ potential for air blown sediments to choke vegetation/accumulate on a levee.

	Future climate

	• potential for change in precipitation, temperature, wind and sea level influences the:
	  ◦ hydraulic loads on levee, which may increase
	  ◦ growth and nature of vegetation, which may no longer be effective as erosion protection
	  ◦ degree of desiccation of levee material resulting in cracking and fissuring
	  ◦ changes in sea level, storm severity and frequency, complicating the prediction of future storms.

Regional/global factors	• the long-term flood protection level can be affected by regional changes in ground level due to:
	  ◦ tectonic movement
	  ◦ post glacial isostatic rebound or settlement
	  ◦ mineral extraction
	  ◦ consolidation of deltaic sediment
	• ground surface elevation changes in combination with sea level rise resulting from climate change can lead to difficulty in maintaining fixed regional/national benchmarks.

7.1.3 The conceptual site model (CSM)

The CSM presents the fullest current understanding of the site conditions and dynamics and as such it is a working document which needs to be updated periodically. There are several guidance documents on how to develop a CSM for environmental and ordnance sites (ASTM E1689-95, 2008, and USACE, 2003). Although none exist specifically for flood control projects, the principles can be readily adapted to levees. The CSM for levees may contain, but not be limited to, the information outlined in Table 7.6.

The CSM serves the following purposes:

- describes and defines the nature of the flooding problem through topography, hydraulics, subsurface conditions, land use, infrastructure, stakeholder concerns, economic assets at risk, population at risk, desired level of protection and risk tolerance

- presents the current geotechnical understanding of site conditions
- serves as a basis for evaluating levee alignments and assessing impacts of site conditions on scheme alternatives
- identifies knowledge gaps and areas of uncertainty
- identifies key assumptions to be confirmed or disproved
- identifies any site constraints that will logistically or technically impact the overall scheme approach
- identifies the nature and quality of the data required by the technical disciplines
- defines data quality requirements for the planned analyses
- provides a basis for scoping and sequencing future investigations.

Table 7.6 Principal information that may be included in the CSM

Information		Chapter/Section
Conceptual site model	Environmental and ecological aspects	2, 9, 10
	Topography, bathymetry and morphology	7.3, 7.4, 7.5, 7.9
	Evaluation of protection level	9.5.1
	Evaluation of hydraulic loads	7.3, 7.4, 7.5
	Condition assessment of existing levee, where appropriate	5, 7.7.3
	Other actions on the levee	7.2.4, 7.6
	Constraints to the works	9.1, 9.2, 9.3
	Desk study data	7.1.4
	Site specific data on ground conditions	7.7, 7.8, 7.9
	Evaluation of borrow material	7.7.3, 9.13.1
	Evaluation of credible failure and deterioration modes	7.7.2
	Surface cover	7.9.2
	Contamination, archaeology and UXO	Seek specialist input

At the start of a project, information for the CSM may be obtained through a desk study of published or previously obtained data, supported by a site walkover, and that is updated as new information becomes available. The information it contains can be systematically refined to a level appropriate to that stage of the project's development through a phased approach of information gathering, which reflects the requirements of the current phase of the project.

On completion of project it may be necessary to further update the CSM with any new data as it can be used to complement the O&M manual (Chapter 4), or in support of any future condition assessment (Chapter 5) or emergency works (Chapter 6).

7.1.4 The desk study

The desk study is the first step in site characterisation, whether it is for the condition assessment or improvement of an existing levee or the design of a new levee. The desk study is the collection, collation, interpretation and integration of available information describing the site, its environs, and the factors that will affect the proposed or existing levee (hydraulics, morphology, topography, geology, ecology, and past, present, and future land use). It encompasses a broad body of information sources and represents the first phase in the development of the CSM, and may be prepared at the reconnaissance phase of the project. A site reconnaissance or walkover survey can be made during the desk study to cross check the information gathered with the physical evidence in the field. A desk study is specifically recommended by some country codes and standards (see Box 7.2), represents good practice, and should be prepared before undertaking a subsurface investigation.

Box 7.2 Example of requirement to undertake a desk study

BS EN 1997-2:2007, clause 2.1.1(3)P requires the "careful collection, recording and interpretation of geotechnical information". Clause 2.1.1(6) suggests that "before designing a site investigation, the available information and documents should be evaluated in a desk study."

7.1.4.1 Undertaking a desk study

The amount of information available for the desk study will vary from project to project. The time and resources required to develop the desk study will be influenced by the:

- location of the data and how readily it can be obtained
- amount, quality and the purpose for which the data was previously obtained
- amount of pre-processing required to get the data into a usable format.

Desk studies for new and existing levees face different challenges. For new levees the study relies entirely on available data that were generally collected for purposes other than levee design, and so some of the information may not be applicable or missing. Existing levee systems may have a large body of data to draw from but the very presence of the levee adds to the complexity of the study. Some of the challenges existing levees present include:

- the levee may have been designed and/or built by local or private parties without regard to design standards, or in accordance with standards current at that time
- the nature and adequacy of both the foundation soils and levee material need to be investigated, and evaluated
- levee systems often have complex construction histories. The original construction may have experienced many phases of improvement, including raising and widening. So its internal structure is likely to be heterogeneous (see Section 3.3.3 and Figure 5.50). It may incorporate infrastructure, structures, utilities, and other penetrations, with various construction methods, materials and standards adopted over time, often with no or incomplete documentation
- levee systems have similarly complex repair histories, often performed piecemeal, with interim repairs in lieu of systemic repairs
- the interaction between the levee system and the river/coastal/estuarine hydraulics need to be assessed
- there is usually performance information (settlement, seepage, piezometer readings, visual observations, maintenance records etc) that also requires collation, analysis, and interpretation
- the original site characterisation investigations may have been done in accordance with standards at the time, but may not conform to current day practice, resulting in insufficient characterisation and significant uncertainty. Examples include widely spaced boring data, outdated or unknown soil classification systems, missing test data, use of a deterministic flood level to set crest elevations, and defined freeboard heights
- all phases (investigation, condition assessment, design, construction, repair, improvement works) may not be fully documented, and records may be incomplete or have been destroyed due to the age of the project, poor record management practices, or property transfers
- levee systems often include a variety of ancillary structures (culverts, siphons, pump stations) necessary for drainage of interior areas
- the basis for setting levee height may not be well established
- levee performance from a stability and/or overtopping perspective may be unknown.

The desk study will primarily cover subsurface conditions and site hydraulics but will include other areas of information on the site.

7.1.4.2 Types of information to be considered in preparing a desk study

Typical information to be consulted in the development of a desk study is outlined in Table 7.7, together with an indication of how the information might be used.

Table 7.7 Typical information to be considered when preparing a desk study

Topic	Source of information	Use of information
General – information needed by all	• talk with existing project operations staff, land owners and older local residents regarding operations, performance, logistics, history, and any issues noted • local firms and agencies may have knowledge of site or region-specific issues, and history of performance and design practice • review existing documents detailing past work, studies, investigations, including information on past performance of existing defences and identify any earlier known problems, historic or anecdotal (crest settlement, seepage, breaches and repairs, ie unexpected deviations in bank alignment, areas of excessive maintenance) • topography – historic and current site survey (conventional or LiDAR) • aerial photographs – historic and current. Also obtain current for different times of year to capture seasonal changes in vegetation cover, and if possible pre and post-flood conditions • historic and current maps/photographs showing roads, property, and other historical/cultural features, including existing levee and associated structures • GIS data coverage published by various bodies showing physical features and that often includes geology.	• gain understanding of flooding problems and levee performance issues by seeking first-hand accounts and observations from those who have witnessed flooding and deal with levee operations • gain familiarity with project background • put current observations into historic context • gain understanding of site and catchment topography and features affecting drainage • identify changes over time in site use and activities, including construction on and near a levee system.
Hydraulics of river, coastal and estuarine environment	• bathymetry/morphology: underwater elevations of bed and likely changes with time • topography: profile of levees and land form • tide and current data • coastal wave and water level data (emphasis on storm climate) • meteorological records, including precipitation data • stream gauge records, showing river elevations and flows, if available • historic flood/storm event information, including maps, photographs, videos, anecdotal accounts, newspaper articles, town/county/national records, and high water mark surveys (if available) • catchment-wide topography.	• gather and plot meteorological data • gather and plot river levels and flows relative to precipitation events (system response) • develop stage and discharge hydrographs for existing conditions • compute statistics of gauge data, estimate uncertainty relationships for flows/stages • develop flow/stage duration curves • compile and plot historic coastal wave and water level data • evaluate historic and predicted coastal data – tides, currents, surge, waves.

Morphology of river, coastal and estuarine environment	• bathymetry – underwater elevations of bed and likely changes with time
	• topography – profile of existing levees and land form
	• aerial photography over several decades
	• bed and bank material size, gradation and type (cohesive, non-cohesive)
	• sediment transport: rate and mode
	• identify sediment sources and sinks
	• presence and type of vegetation.
		• influence of bathymetry and topography on waves development (coastal)
		• interaction of channel and floodplain flows
		• develop a sediment budget
		• long-term trends in:
		  • channel position
		  • channel profile
		  • channel dimensions
		  • beach position
		• stream type (braided, meandering etc).

Table 7.7 ***Typical information to be considered when preparing a desk study (contd)***

Geotechnical	• geological maps: solid and surficial geologic maps, and associated reports • reports and maps from special studies related to mineral resources, water supply, large-scale remote sensing geophysical investigations, and other investigations • boring logs and geotechnical data related to the project or drilled in the vicinity of the project for other purposes (ie bridges, buildings, other infrastructure foundation, environmental characterisation, remediation), and archived samples • well records, including boring logs, well construction diagrams, pumping data, including pumping rates and water levels, and water quality data • groundwater maps showing aquifers, water supply wells, predicted yield rates etc • evaluation of groundwater and surface water interactions • historic field instrumentation monitoring data, including piezometer readings, and settlement gauge readings.	• assess the spatial distribution of geological features through interpretation of aerial photographs, LiDAR data, existing data form geophysical and intrusive investigations • create summary plan of data by overlaying data from historic maps and plans, topography, aerial photographs and geological mapping to show how features relate to each other and to establish any correlations between the location of known past and present performance issues with the levee and ground features • develop preliminary geological cross-sections, ensuring that all data are reported to a common co-ordinate system and vertical datum, and preliminary geotechnical parameters based on existing information • assess likely sources of borrow material and evaluate whether site won borrow can be used • develop an understanding of potential failure and deterioration modes to aid decision making and the formulation of a scope of investigations.
Land use	• property maps and identify landowners • land development programmes • potential/known archaeological sites • registries of historic sites and landmarks, such as the landmark survey (UK) or national/state/county equivalent • determine potential sources of contamination • national or state environmental agency register or inventory of contaminated waste sites (USA environmental protection agency, other international equivalents) • UXO desk study information from specialist contractors/consultants • identify potential environmental constraints, such as endangered species, critical habitat, invasive species and water levels and water quality values having bearing on flora and fauna • mining records • local and regional government records • universities.	• review all sources of information regarding past, current, and future proposed land use • land use studies will face different challenges for urban levees compared to rural levees. Urban levees will be dominated by issues involving utilities, old foundations, adjacent structures and loads, and fill zones. Archaeological sites, contamination and ordinance may still underlie developed urban sites. Rural levees will offer greater opportunities to recognise the distribution of natural deposits as they have not been disturbed or covered by development • evaluate the risks of encountering ordnance in areas subject to wartime bombing or military training grounds • identify historic land uses that may result in contamination, eg tanneries, petroleum suppliers, degreasing activities, dry cleaners, and other industries/activities known for releasing contamination to the environment.

Surface data (maps, aerial and terrestrial photographs, and LiDAR or terrestrial surveys) can be particularly useful sources of information. Surface data covering a range of time, new and old, can be a permanent record of conditions at the time the data was recorded. It can be referenced with other data in GIS. Scrutinising surface data can provide a broader understanding of the water shed, stream and coastal dynamics. Old maps and photographs can aid in the understanding and assessment of morphology, channel adjustment over time (Figure 7.3) and changes in land use. Aerial photographs can provide an overview of the nature and extent of vegetation cover within the water shed.

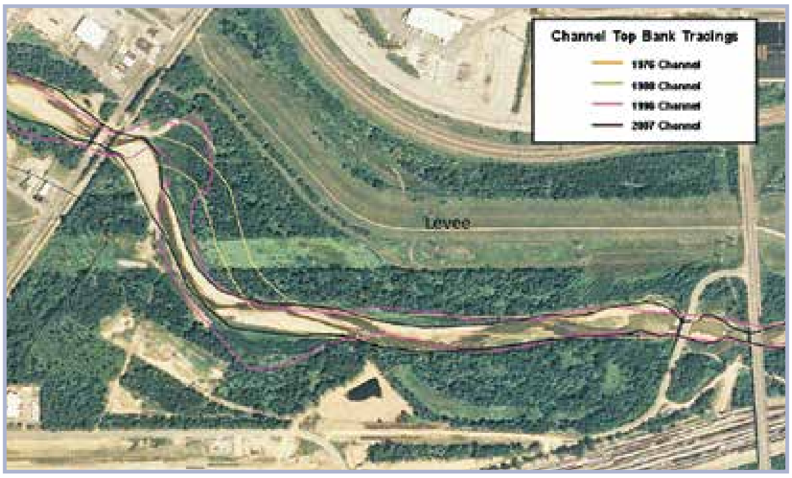


Figure 7.3 *Channel tracing comparisons from aerial photography, note levee above channel (courtesy Charles Little, USACE)*

Surface data can also provide clues that may help to identify and explain localised changes in ground conditions. Features evident on aerial photographs, such as changes in soil colour and vegetation cover, or in its vitality within a single crop, may be indicators of a variation in subsurface conditions. Surface data can also be used to interpret shoreline change at the coast (seasonal and long-term). Similarly, topography can be a reflection of subsurface conditions, specifically in the identification of paleochannels, which can be a point of weakness within the levee foundation. An illustration of the use of LiDAR in the identification of paleochannels is presented in Box 7.3.

Box 7.3 ***Example of the use of LiDAR in identifying the location of paleochannels***

Within the Fenlands of eastern England there are many rivers and manmade drains confined by levees. A geologically recent transgression of the sea resulted in the silting up of the ancient creek system. Subsequent drainage of the fens has resulted in settlement of the land leaving the ancient silt infill creeks slightly higher than the adjacent more clayey deposits so that they form low level sinuous ridge features across the flat landscape, called roddons. Where the roddons are pronounced they can be seen from a visual inspection of the site. However, they are better seen from above through aerial photographs. There are cases where the roddons are not immediately visible from a site walkover or aerial photographs but LiDAR can be used to identify the slight relative differences in elevation across the site.

Figure 7.4 shows one such case where areas of seepage had been reported by the levee owner. A site walkover and aerial photographs showed no evidence of the roddons. However, when the LiDAR was analysed their locations and extent became clear (evident as orange and yellow sinuous features and the broarder c1km feature of the main channel evident across the bottom left-hand side of the figure) and the areas of seepage were found to correlate precisely with the locations where the levee crossed the roddons.

Figure 7.4 *Paleochannels identified using LiDAR (courtesy Environment Agency)*

This technique can also be used to identify areas where old creek systems have become infilled with softer deposits. In this case the creeks are defined by sinuous features that are at a slightly lower level then the adjacent deposits. Areas of localised excessive settlement or slope failure are likely to correlate with the infilled creeks due to the lower strength/higher compressibility of the infill soils and the lower relative ground level, requiring a higher levee to achieve the required standard of protection.

7.1.4.3 Site walkover survey

The site walkover survey is critical for team members to gain a physical sense of the site. To gain maximum benefit from the visit it is preferable to undertake it partway through the desk study (information gathering) phase so that some background knowledge will have been gained beforehand and the team will have a general understanding of the challenges the site poses to their discipline. The walkover survey aims to identify visual/physical clues to site issues. Some issues will be of interest to several disciplines. Indications of some of the observations that can be made during a site walkover are listed in Table 7.8.

Table 7.8 Some observations that may be recorded during site walkover

Topic	Site observations
Hydraulics of river, coastal and estuarine environment	• land use • springs • evidence of buried utilities, adjacent infrastructure (roads, bridges, buildings, power plants etc) • tie-ins with natural high ground • location of stream, tide, or wind gauges • high-tide lines • evidence of past flooding and flow paths • existing encroachments to channel and floodplain • woody debris • navigation issues • diversions or discharges • vegetation type(s) and position along channel and in floodplain • estimates of channel and floodplain roughness.
Morphology of river, coastal and estuarine environment	• scour and bed material • stability of stream • type of channel pattern • diversions or discharges • woody debris • navigation issues • existing armour and channel training features • sources of sediments • types of sediments and grain sizes • existing encroachments to channel and floodplain • vegetation type(s) and position along channel and in floodplain or at the coastline • presence of coarse bed materials, especially boulders/large rocks • evidence of past flooding and flow paths • shoreline orientations and beach slopes.
Geotechnical	• settlement • seepage • instability • cracks/fissuring • animal burrows (materials excavated) • changes in vegetation type and colour • changes in soil type and colour • patterns within any surface undulations across adjacent fields • exposures in drainage channels • springs • evidence of buried utilities or other penetrations, adjacent infrastructure (roads, bridges, buildings, power plants etc) • tie-ins with natural high ground • channel bed materials • visit nearby quarries and potential borrow sites to see native materials *in situ*.
Land use	• current land use and evidence of past land use • potential adjacent sources of contamination • native species • endangered species • invasive species • vegetation.

For an existing levee system the site walkover survey affords an opportunity to examine and understand how all the components of the system operate and interact, and to visually identify any potential localised 'weak' locations. Section 5.4, contains a more detailed critique on the inspection of existing levees for the purpose of condition assessment. Box 7.4 provides an example of specific information obtained during a site walkover.

In addition, the team may wish to consider the following:

- tour the area from a boat or by helicopter to provide a different perspective on the site
- bring details of current and/or future planned projects, if available
- bring an overlay of site maps and aerial photographs
- bring up-to-date land ownership documents showing boundaries and rights of entry, specifically where the land is privately owned
- bring a camera with a global positioning system (GPS) to assist with documenting site observations
- if seepage is a possibility, then bring a temperature-conductivity meter to assess flows
- talk with the local population, particularly older members.

Box 7.4 ***Examples of hydraulic and morphologic information obtained during site walkover***

Figure 7.5

Bed material characteristics in stream channel (courtesy Andy Gaines, USACE)

Photographs can document sediment characteristics of the stream or coastal area (see Figure 7.5). If the photographs are taken at the time of construction, they may provide information of construction details that are now hidden. They can help to identify ways in which floodwaters could short circuit the levee, for example via canals. Records taken before, during and after a flood can provide clues as to how the floodwater behaved the extent of any breaches, ie area flooded, and allows an assessment to be made of the quantity of floodwater. This information can be used to calibrate the flood model. Observations of stream characteristics provide information about sediment sources, sediment movement, and stream profile and bank and bed stability (see Figures 7.6 and 7.7).

Figure 7.6

Moderate to severe bank erosion, bank height is approximately 10 m (courtesy Andy Gaines, USACE)

Figure 7.7

Stream bed instability and use of armour to stabilise stream profile (courtesy Andy Gaines, USACE)

1 2 3 4 5 6 7 8 9 10

7.1.5 Implementing investigations

Implementing a field investigation requires an appropriate team to ensure all key data are collected and reported, and the investigation is done in a safe manner. Work and safety plans typically are prepared before field mobilisation, converting the scoped objectives into field tactics.

The day to day on-site management of work is done by the field operations leader, who may be the lead investigation contractor on site, responsible for the overall orchestration of the work, directing other staff, overseeing subcontractors, and co-ordinating with design engineers. There may be multiple activities taking place at the same time, with different crews, contractors and equipment. Responsibilities include ensuring that work is performed in accordance with the work plans, field checking methods and records, and documenting the field work, typically through daily reports. The efforts in co-ordinating logistics, overseeing subcontractors, and minding the budget and schedule should not overshadow the most important role of the field operations leader, which is to ensure the necessary data are obtained by the field program.

The field operations leader should perform some real time technical analysis of the data collected to verify completeness and identify any unexpected findings that may warrant further investigation, ideally while the investigations are in progress and the appropriate equipment and crews are still on site. The field operations leader will be the first to verify the CSM against field conditions, and note any conditions that challenge the assumptions and understanding embodied by the CSM. Depending on the arrangement between parties, these latter roles may lie with the investigation designer or their representative on site.

It is incumbent on the investigation designer to visit the site at the start of the field programme. There may also be a representative of the investigation designer on site either full or part time, ie a supervising engineer. The investigation designer may be required to clarify locations or confirm modifications to the investigations with the field operations leader and verify that correct procedures are being adopted. It is important that the investigation designer observe a representative selection of the materials encountered in the field first-hand, to have a true physical understanding of site conditions. Other aspects of the role of the investigation designer during the investigations are presented in Section 7.7.1.

In addition to documenting field activities and progress, daily reports can highlight any deviations to the work plans, so that changes in methods are recorded and explained, along with any unexpected findings in the investigations. Results should be constantly compared to what was expected or predicted so that any significant differences can be relayed promptly to the design engineer.

All site staff and visitors have responsibility for their safety and that of each other. If any unsafe operations are observed they should be made known. An individual may typically be designated as the site safety officer who is responsible for ensuring compliance with the safety plan, and can stop work to correct unsafe practices or conditions. Other responsibilities may include utility location and avoidance before intrusive investigations take place, co-ordination with local navigation officials for marine work, and maintaining safety equipment and supplies on site. Common safety issues include:

- trips and falls from height
- lifting
- machinery
- trenches
- poisonous plants, reptiles and insects
- weather
- potential contamination
- marine work
- ordnance.

7.1.6 Reporting

Reporting is the collation and presentation of data in a form that can rapidly be assimilated by the end user. The way in which the data are presented and the level of detail included will depend on the end user and the intended use. The data could be existing information obtained for the desk study along with information from project specific investigations. A report can comprise all, or a combination of text, tables, plots, and figures in hard or electronic format. It provides an auditable trail through the data assessment process and the output can be used to update the CSM.

7.1.6.1 Desk study data

The report prepared at the end of the desk study may form part of a reconnaissance report, consisting of both text and graphics, and provides a summary of the available information used to develop the preliminary CSM. It may include key findings, preliminary parameters, and site walkover survey observations, critical data gaps with recommendations, uncertainties, and key assumptions.

Tables 7.7 and 7.8 give an indication of the nature of the information to be considered during the preparation of a desk study. While the amount and quality of the data may be limited and less site focused than any obtained later through project specific investigations, the process of compiling the report could follow the general guidance in this section, or other such guidance provided by country codes and standards.

7.1.6.2 Hydraulics

A separate formal written report is not typically prepared as an output from a hydraulic study. Hydraulic analyses and results are frequently contained in planning level study reports and other detailed design reports. Reporting typically consists of the collation of existing information gathered during the desk study, new field data collected during site investigations, and analytical calculations and/or numerical model results used to calculate inputs required to assess the elevation of the levee and the effects of hydraulic loads on a levee and on the morphology. This information may include topics presented in Table 7.9.

Table 7.9 Possible output information from a hydraulics study

Water system	Possible output information
All	• historic aerial photographs and maps showing river, coastal and estuarine genesis, evolution, and dynamics • historic storm event data • overflow peak discharge estimates and hydrographs (for unsteady modelling) • statistical estimates of target values for water levels (ie one per cent, or other event) • calculations • models.

Rivers	• present and future hydraulic conditions with and without project (including option scenarios)
	  ◦ discharge-probability curves with confidence limits shown
	  ◦ stage-probability curves with uncertainty indicated
	  ◦ stage-discharge relationships
	  ◦ water surface profiles
	  ◦ currents/velocity estimates
	  ◦ expected annual exceedance and conditional non-exceedance probability water levels.

Coasts and estuaries	• probability distribution of surge, wave height and associated period • probability distributions of overtopping rates • currents/velocity estimates • tides.

7.1.6.3 Morphology

Morphological assessments involve collecting historic data so that past behaviours can be described, at least qualitatively. These behaviours can then be used as a model, or projection, of future behaviour. The data may be derived from hard copies or printed material (older aerial photography and topographic/bathymetric maps), scanned images, and digital data. The dates of when each piece of information was originally obtained need to be recorded so that the changes over time can be evaluated and documented. The type of information assessed may include:

- projected channel adjustment without project and impacts on stage-probability relationship
- projected channel adjustment with project and impacts on stage-probability relationship
- projected shoreline stability with and without project, and the impact of waves and water levels on the stability of the levee.

There may be no specific format that is used to present the morphological assessment but it should aim to capture the data, the interpretation of the data with assumptions, and the conclusions. The level of detail recorded will depend on the phase of study: reconnaissance, feasibility, or detailed.

7.1.6.4 Geotechnical

Geotechnical data from the investigation can be collected by handwriting on paper, entering directly into a field computer or other electronic hand-held devices, or collected digitally. The submission of factual data can be in a variety of formats. Table 7.10 summarises some of the data that could be available from an investigation and the issues associated with them. However, all data should comply with a common framework recorded as metadata. This could include:

- names of staff and affiliation
- make and model of plant/craft
- investigation methods and sampling tools
- serial numbers and calibration dates
- survey vertical datum and horizontal co-ordinate system (Section 7.9.1)
- soil classification systems (Section 7.8.2).

Table 7.10 Possible forms of factual data and associated format issues

Possible forms of factual data	Data format issues
• field books • daily reports (activities, technical issues, equipment, staffing, measurable items relating to the Bill of Quantities) • logs of explorations • *in situ* and laboratory test records • instrumentation installation diagrams • electronic data deliverables (survey, geophysics, CPT etc) • maps, LiDAR and sketches • photographs/videos.	• both paper and electronic media are vulnerable to field physical hazards – weather, being lost etc • paper forms need to be filled out by hand for later transcription. Both actions could introduce data errors • directly recording to electronic media reduces the risk of transcription errors but this format often lacks the flexibility of paper, with fixed data entry fields and menus, and may have no option for including visual data (sketches or photographs) • future formats may involve tablets that capture handwritten notes and visual data in log form, and convert to typed text in data fields.

Ground investigation field logs

Information from intrusive investigations is recorded on field logs. Non-intrusive methods typically involve electronic data collection, although the field effort is also typically chronicled in a field diary. Field logs often include:

- names of crew members
- equipment (make, model)

- exploration locations – sketch, co-ordinates, and elevation
- observations made while advancing the exploration hole
- *in situ* testing methods and results
- sampling interval, method and recovery
- soil descriptions to a recognised standard (Section 7.8.2).

Issues that may be encountered in preparing field logs include:

- **site staffing levels and experience:**
  - field duties are often assigned to the most junior staff and senior staff may have limited availability and/or budget to spend time on site, and so the most critical first-hand observations are made by the least experienced staff
  - sometimes contract administration takes priority over technical duties
  - field staff members need the opportunity to calibrate their visual classifications by checking their field descriptions against laboratory test results
  - intrusive investigations for levees are sometimes shallow and involve continuous sampling, as the depth range with the greatest variability in soil type and properties usually occurs at shallow depths. As a result, logging requires the most amount of time when the site staff has the least amount of time available due to the frequency and variability of samples recovered at the start of an exploration, increasing the potential for errors or missing information
- **contractual and technical issues:**
  - exploration contractors are often paid by the length of exploration hole completed. This can encourage 'hole making' rather than the collection of good samples and performing good quality *in situ* tests
  - where samples are not recovered, drilling observations may provide the only clues to material types being penetrated. It can be difficult to log samples while also tracking drilling observations (loss in return drill fluid, rough drilling, sudden change in rate of advance etc)
  - fine details of soil, such as stratification, are difficult to preserve, and can easily be lost or destroyed in the processing of samples and if not described in the field when they are fresh then these details are lost from the record
  - field descriptions on logs should be updated with actual gradation/strength data from laboratory analysis when available
  - the comparison of field descriptions with laboratory classifications can be used to identify biases by field crews. The typical tendency is to overestimate the fines fraction and underestimate the coarse fraction, but this can vary by individual.

Data reporting formats

Information included on the field logs, together with the results of laboratory and *in situ* tests, is usually submitted in paper and/or electronic format, with appropriate metadata. Data entered for the word processing of field logs is generally used to produce the electronic data. It is preferable to have the same person that prepared the field logs to prepare the data report. This can improve the quality of data entry and reduces transcription errors. If someone else transcribes data from field logs, it is helpful if they are knowledgeable about investigations so that they are able to understand and interpret the notes on the logs. Data reports can be prepared to a national recognised format and accompanied by metadata, which details the format in which the data are presented, project information, data versions and date. Paper deliverables should include appropriate comparable information. Some benefits and disadvantages of data reporting formats are presented in Table 7.11.

Table 7.11 Benefits and disadvantages of data reporting formats during interpretation

Benefits	Disadvantages
Paper	
• a log and test result sheet is more legible and provides more intimacy with the data when reviewing and interpreting the information • the process of transcribing data from paper to electronic format aids in the mental assimilation of the data by the transcribers.	• potential for transcription errors from paper to electronic format to produce plots or sections • increased interpretation time.
Electronic data	
• received data can be filtered for correct format and gross errors in reported values • database can be queried to draw graphical plots, cross-sections and 3D ground models • database provides potential long-term storage of data, provided that storage media is updated and metadata retained.	• may not be available in time for use in design • robust quality assurance/data management plan needed to ensure up-to-date and correct data is used • original field records need to be preserved • user may feel removed from the data.

Spreadsheets are often the simplest method to manipulate or migrate data between software platforms. However, spreadsheets do not provide the robustness of a database for long-term data storage, nor do they provide data security. A database ensures the integrity of the data and mitigates against user error by preventing accidental deletion or overwriting of records.

Data visualisation

The fundamental objective is to layer, integrate, and synthesise the various types of data into comprehensive figures that depict the most significant subsurface conditions to the designer. There are issues associated with using data from intrusive investigations undertaken at a limited number of discrete locations and depths, such as SPT *N* values, and integrating it with data collected continuously at ground surface (geophysics) or with depth (CPTs), and interpreting conditions three dimensionally. Some of the other considerations for preparing 3D visualisations and presenting graphical plots of numerical data are presented in Table 7.12.

While levees are linear structures, it is considered to be good practice and recommended by many country standards and codes, to undertake investigations to define the ground profile not only along the alignment of the levee but on transverse sections (Section 7.9.7.2). Many software programmes used to create cross-sections are also capable of creating fence diagrams (an irregular section connecting points of intrusive investigation) and 3D visualisations. 3D representations may be particularly useful for levees that are founded on potentially highly variable alluvial and coastal deposits that can transition or pinch out over small distances. Numerical data values, such as CPT tip resistance and SPT *N* values can also be viewed and contoured in 3D.

Table 7.12 Some considerations when presenting data visually

Form of visualisation	Consideration
2D sections and 3D ground models	Deciding whether soil units are continuous or discontinuous between investigation points: • the unit may be continuous in plan but there could be localised areas ('windows') where it is absent (not deposited or subsequently eroded). On a single 2D cross-section the unit may appear to be discontinuous. Deciding whether to consider horizons as a collective unit, or split them into discrete units: • discrete units having the highest permeability may be 'split out' and retained in the stratigraphic sequence, especially if overlain by material of much lower permeability. However, when working at a larger scale, such as a regional groundwater model, it may be more appropriate to 'lump' the units into a single unit, where an average permeability might be representative of the composite unit. ***Interpolation of unit boundaries*** • interpolation may be most difficult where there is greatest variability in the units, and uncertainty. Judgement should be applied based on an understanding of the geology. It is preferable to join known data points with straight lines unless there is other supporting data to suggest otherwise, such as geophysics. However, the straight-line interpretation could misrepresent other data trends that are recognisable in 3D, but not apparent in a single 2D section. Any unit boundary should honour and preserve the factual data. Introducing dummy points or using mathematical algorithms to smooth the unit boundaries should be done only with great caution, when scientifically supported by site geology • viewing data in 3D can be extremely informative, especially in complex geologic depositional environments where changes occur very quickly both laterally and vertically as it may allow patterns to be recognised that would not have been otherwise apparent in simple 2D section. However, sufficient data will be required to develop a 3D model. This means that an area of potentially complex geology needs to be recognised early on in the investigations so that sufficient exploration is undertaken to develop a 3D model.
Graphical presentation of numerical data	Selection of scales and display of data fields: • keep same scales were possible to allow direct visual comparison of common datasets • data integration can be facilitated by plotting single or multiple data fields relative to depth below ground surface, depth below top of strata and elevation along the y-axis to evaluate which approach afford the strongest correlation.

Report preparation

At the end of an investigation there is typically a comprehensive report prepared documenting the field work performed and characterising the subsurface materials encountered, using text, tables, plots, figures, and cross-sections. A typical structure for a geotechnical report is summarised in Table 7.13.

The investigations may have included elements to explore the concentration of contamination, archaeology and UXO. Documentation of these can be included in the report and the principles set out in Table 7.13 may apply, but appropriate specialist advice should be obtained.

Table 7.13 Typical content structure of a geotechnical report

Section Heading	Content
Aims and limitations of report	• objective of report • intended use of report and limitations of liability.
Introduction	• site location, setting, geology.

Objectives of investigations	• reason for investigation:
	  ◦ phase – reconnaissance, feasibility, design (new levees)
	  ◦ condition assessment (existing levees)
	  ◦ modification/Improvement (existing levees).

Table 7.13 Typical content structure of a geotechnical report (contd)

Methods of investigation	• sequencing of field work • scope of non-intrusive and intrusive investigations, including equipment used • sampling frequency • *in situ* testing methods and frequency • laboratory testing and quantities • instrumentation and installation records.
Deviations from planned scope of work	• reasons for deviations.
Interpretation of subsurface conditions	• strata and units encountered: material descriptions, distribution and thickness • range of values and characteristic values of geotechnical parameters for each unit • validation of geotechnical parameters through empirical correlations • site specific correlations between field and laboratory or other field data • interpretation of instrumentation data and comparison with calculated performance. Modifications to geotechnical parameters based on back analysis of instrumentation data, if required • groundwater observations and existing flow regime.
Other characteristics	• seismicity – peak ground accelerations, return frequency.
Conclusions	• update CSM • identify findings and uncertainties relevant to design.

Figures	• plan views:
	  • show all locations of intrusive and non-intrusive investigations, and instrumentation together with levee alignment and background mapping
	  • show other information:
	    ○ superficial geology
	    ○ surface geophysics results
	    ○ surface and subsurface features from LiDAR, aerial photographs
	    ○ groundwater elevation contours
	    ○ locations of cross-sections
	• vertical sections through the subsurface showing:
	  • ground profile along alignment of levee crest
	  • additional profiles parallel to levee
	  • cross-sections cutting across levee, from waterside to landside
	  • oblique angle to levee (not perpendicular to levee alignment)
	  • information as appropriate that is also shown (legend)
	  • display data fields along log (SPT *N* values, shear strength etc)
	  • select data fields pertinent to project issues
	  • installation details (piezometer tip elevation and response zone)
	  • depth at which water was first encountered and range of water level readings
	• 3D ground models:
	  • oblique representations of model if developed and appropriate
	• numerical data:
	  • data plots supporting assessment of geotechnical parameters
	  • data plots supporting assessment of instrument data.

References	• standards used • published empirical correlations.
Appendices	• results of intrusive investigations, including exploration hole logs, *in situ* and laboratory test results, and instrumentation data • results of non-intrusive investigations, including interpretation by specialists • other stand-alone reports.

Further reading related to the topics discussed in Section 7.1 is presented in the following box.

Further reading

There are a number of books and publications, which will provide useful further reading:

- Steeds, J E, Slade, H J and Reed, W M (2000) *Technical aspects of ste investigation*
- BRE (1987) *Site investigation for low-rise building desk studies*
- BS 5930:1999+A2:2010 *Code of practice for site investigations*
- Dumbleton and West (1976) *Preliminary sources of information for site investigations in Britain*
- Weltman and Head (1983) *Site investigation manual*

There are no explicit requirements to undertake a separate desk study for hydrologic, hydraulic, morphologic, or sedimentation investigations but various guidelines describe the types and sources of data that need to be characterised and documented as part of the site characterisation process.

- USACE (1995a):
  - Chapter 1, Section 7 outlines studies necessary for flood damage reduction studies.
  - Chapter 2 provides common hydrologic engineering requirements for study and design efforts.
  - Chapter 7 specifically addresses requirements for design and analysis related to levees and includes provisions for interior drainage
- USACE (1995b)
  - this regulation describes the procedure and rationale for conducting sedimentation invesgtigations in support of the hydrologic analysis and hydraulic design of civil works projects, including levees
- USACE (1994a):
  - Chapter 1, Section 7 describes a systematic approach to assessing channel stability for flood control projects, including levees
  - Chapter 2 describes principles for assessing site characteristics of channels and for identifying potential stability problems
  - Chapter 4 provides information about potential data sources and how to assemble site information needed to assess site conditions
- USACE (1989):
  - describes data necessary for performing sedimentation investigations in rivers
  - Chapter 1 identifies the need for and level of detail required of investigations
  - Chapter 2 defines reporting requirements for different study phases through design documents
- Biedenharn *et al* (1997)
  - identifies specific data needed for developing the geomorphic assessment of a stream system and gives recommendations for documenting the data and its interpretation
- Sayers *et al* (2003):
  - provides a framework, guidance and tools to support objective-led data management for flood risk management. In particular, it gives information on establishing the design objectives and data, in assessing the adequacy of available data, and the value of obtaining additional data.

7.2 MORPHOLOGICAL, HYDRAULIC AND OTHER NATURAL ACTIONS ON LEVEES

Hydraulic loadings are controlled by the nature and profile of the surface over which the water moves and the forces that cause it to move. The former is determined by the bathymetry while the latter is controlled by elevation, precipitation, runoff, tide, wind, and other factors. As the design of a levee is based on providing a standard of defence against a predicted future event, the effects of time (morphological change) on the bathymetry and topography and the drivers to water movement should be taken into account.

The intent of Sections 7.3 to 7.5 is to present morphological, hydrologic, hydraulic, waves and sedimentation information needed to identify the hydraulic actions on a levee in river, coastal and estuarine systems (note that the handbook does not attempt to address these aspects in their entirety and where appropriate the reader is guided to some of the many text books on these subjects). In all cases, the assessment of the hydraulic actions in these sections represents **the still water level, wave and current environment developed immediately to the waterside of the levee.**

Sections 7.3 to 7.5 serve as a general reference that may be used as required in combination with tools in Chapter 8 to support the condition assessment of existing levees (Chapter 5) or the design of improvement works to existing levees or design of new levees (Chapter 9). It is recommended that specialists in these individual disciplines are engaged to perform the necessary field investigations and analyses, and are co-ordinated by the investigation designer.

A flow chart mapping the outline structure and contents of Sections 7.3 to 7.5 is presented in Figure 7.8. The physical phenomena (see Table 7.14) are discussed within the field where they are predominant:

- hydrological and morphological impacts on water levels, flows and velocities on riverine levees (Section 7.3)
- astronomical, meteorological, seismic and morphological impacts on water levels and waves on coastal levees (Section 7.4)
- combined effects within the section on estuarine levees (Section 7.5).

Caution

The hydraulics and morphological material described in the subsequent sections of this handbook should not be treated in isolation. For example, there are many environments where:

- **wave action is important other than on the coast (Section 7.4)**
- **current action is important other than in rivers (Section 7.3)**

For each situation, there should be a conceptual analysis of the physical processes and their interactions to evaluate which effects are most significant (see Section 7.2.1)

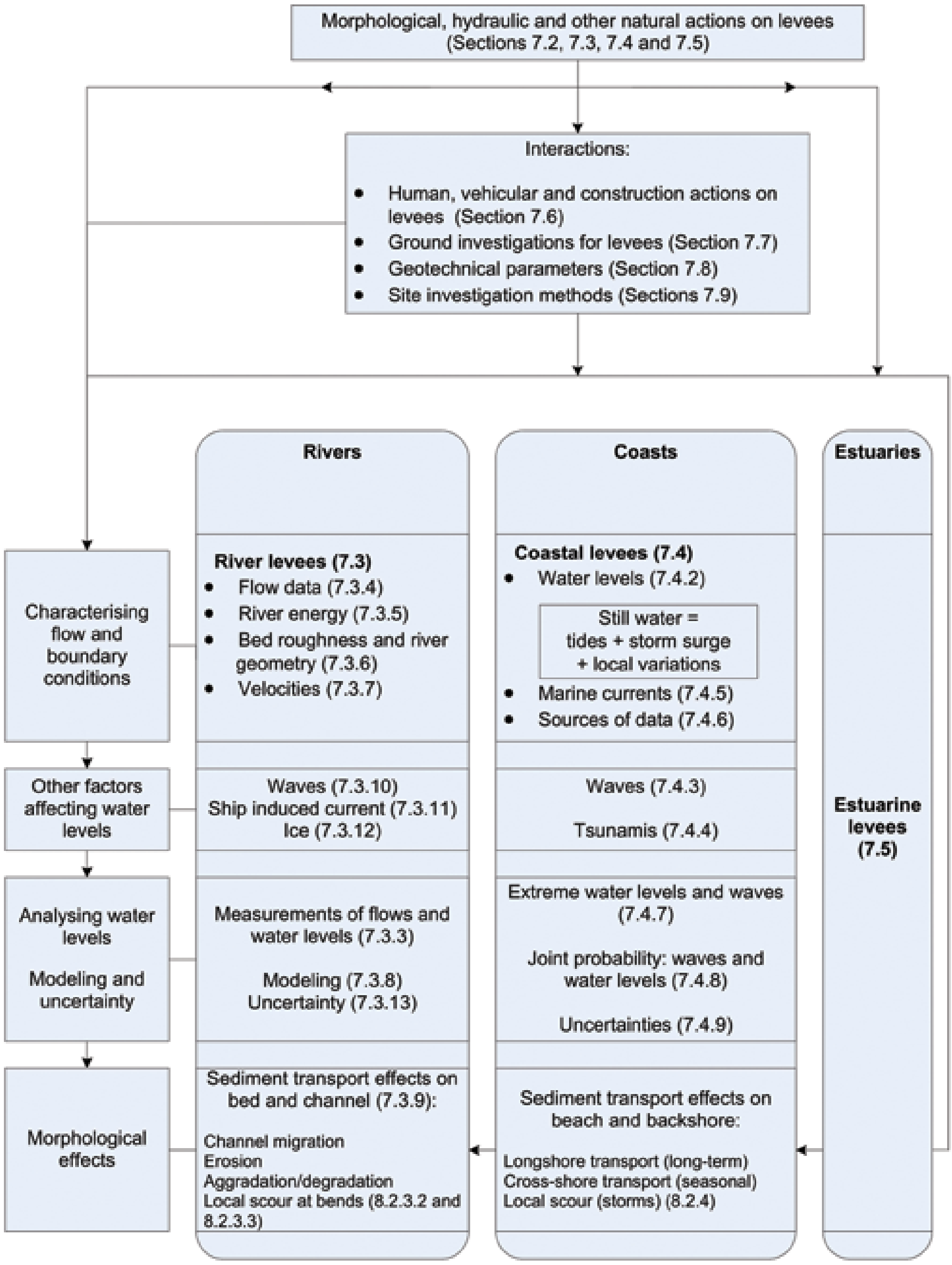


Figure 7.8 ***Structure and content of Section 7.2, and interaction with other sections***

Table 7.14 Influences and interactions to be considered when quantifying hydraulic loads acting on levees

Category/levee environment	Influence	Interactions
Rivers and streams	Catchment hydrology	• basin shape and orientation relative to climate patterns • precipitation, amount and form (rain vs. snow and ice) • topography and soil characteristics • dominant vegetation cover • stream network density • climatic zone (tropical vs. desert).
	Channel and floodplain hydrodynamics	• channel boundary composition • presence of vegetation, roughness • encroachments from infrastructure and/or flood control works • stage-discharge relationships • flow boundary roughness • floodplain characteristics – lowland (low gradient) vs. upland (high gradient) • superimposed events/multiple coincident rare climatic conditions • routing and combining flow hydrographs through stream system.
	Morphology	• sediment mobility and transport • width and depth adjustment trends • discharge and thresholds • longitudinal profile gradient • sinuosity of channel and change over time • channel type • equilibrium state of system • sediment sources and system energy to transport material • anthropogenic induced change.
Coastal and estuary zones	Astronomical tides	• tidal water levels and timing are generally predictable.
	Bathymetry, morphology and landform	• bathymetry, topography, and bed roughness (vegetation) influence hydrodynamics • wave and current-driven sediment transport (across and along the shoreline) • waves and currents change bathymetry with time, which in turn influences the waves and currents • bed deposits and material composition affect behaviour (mean grain size, sorting, layering, cohesiveness) • levee landward slope erosion due to overtopping by waves or water levels.
	Hydrodynamics	• wind generated waves • wave, wind, and tide driven long-shore and cross-shore currents • water levels driven by combined winds, waves and tides • potential levee overtopping by waves and water levels • storm climatology (tropical and 'extra tropical' storms) • some regions experience tsunamis • climate change (sea level rise, altered storminess).

7.2.1 Interaction between levees and environmental processes

As illustrated in Figure 7.8, there are a large number of potential interactions between topography, bathymetry, morphology and hydraulic actions, in Table 7.14, which should be considered in the assessment or design process.

Hydraulic actions result mainly from natural processes, but may be influenced by the presence of the levees and other structures. For example:

- in a river system the water may already be confined by an existing system of levees and structures, which will influence the hydraulics and morphology
- in a coastal system, the interaction of the waves and current with the structures will influence the morphology that in turn, by modification of the bathymetry, may affect the magnitude and direction of the waves incident on the levees

Figure 7.9 presents how, for the flaluvial environment, channel development may be affected by interactions between four key elements: hydrology, morphology, sedimentation, and hydraulics.

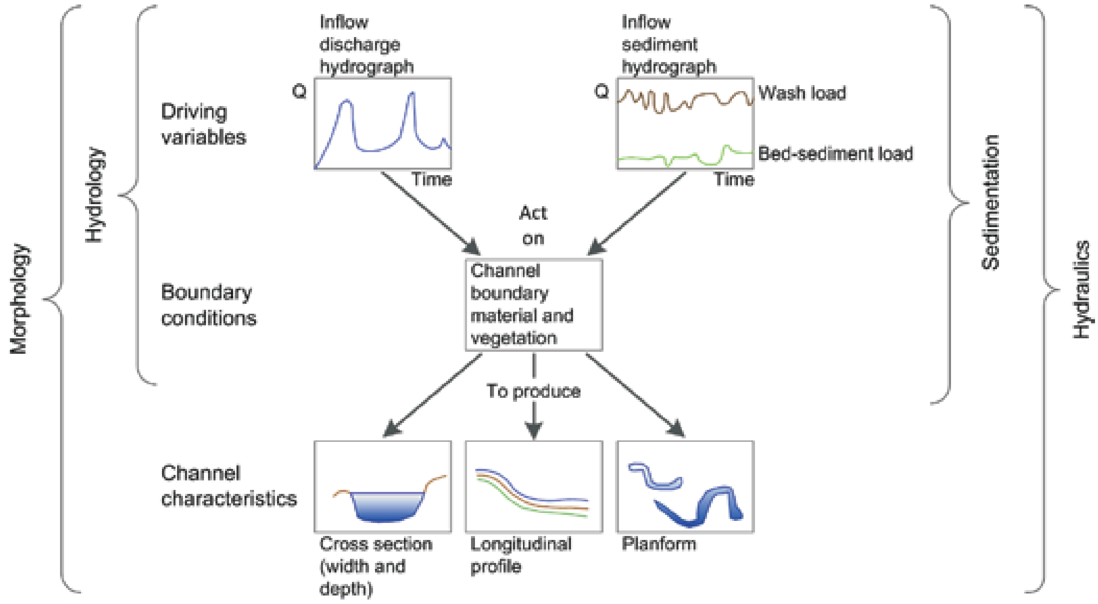


Figure 7.9 ***Interactions between variables affecting the formation and response of river channels***

The levees themselves also modify and interact with the morphology and hydraulics and can be controlled by the engineering design of the levee as discussed in Section 9.2.3 and 9.4.1. The addition of levees in a river floodplain or around coastal margins may change the delicate balance between these interactions. Tools for quantifying the interaction of the engineered levee with the hydraulic loads are presented in Chapter 8.

7.2.2 Influence of seasonal change and extreme events

Bathymetry, morphology, hydraulics, landform and occurrence of extreme weather all influence the hydraulic load.

In rivers, stream bed elevation and its variability has a primary role in determining water surface elevations and river behaviour over time. At coastal sites, the water depth and slope of the foreshore can control wave height. As stream and near-shore currents can produce continuous changes in bed elevation, it is often necessary to consider bathymetric elevations obtained at different times (seasonally or over decades). Typical conditions causing seasonal change are:

- rivers:
  - flood and dry seasons
  - growing season(s) – changes in vegetation that affect roughness
- coastal, including estuaries:
  - monsoons and El Niño/La Niña-Southern oscillation
  - winter–summer storm patterns
  - tropical and extra-tropical cyclones – hurricanes, typhoons, northeasters
  - seasonal water levels – Great Lakes of North America.

7.2.3 Influence of long-term changes

In addition to the morphological processes there should be an awareness of other factors that can contribute to longer-term change, such as climate change and large scale geological factors, both natural and anthropogenic, as introduced in Chapter 2 and Section 9.5. The following list, which is not exhaustive, provides examples of where these processes have affected levees or areas where flood protection may be needed in the future:

- settlement/compaction of deltaic sediments (eg South Louisiana, Bangladesh)
- lack of new sediment input to system from river diversion, upland land use changes and urbanisation, dredging (eg Mississippi River delta, Nile delta)
- groundwater and hydrocarbon extraction leading to settlement (eg Galveston Bay, USA west coast, Manila Bay, Rangoon, Tokyo Bay)
- sea level rise (eg USA east coast, western Pacific Islands) – see Box 7.5
- river mouth switching/migration (eg Mississippi River).

Box 7.5 Sea level change and its impacts

Long-term measurements of global mean sea level (MSL) show a rate of rise of 1–2 mm/year over the last century. The present consensus is that the rate of rise will probably increase to about 5 mm/year, with some regional variations, although as yet there is no definitive evidence that this acceleration has started.

MSL may change relative to the levels of the land and relative MSL can be defined as the difference between local changes in land elevation and global sea level changes. These changes result from a variety of processes, several of which can occur simultaneously. The following six processes can contribute to long-term relative MSL change. However, all processes do not necessarily apply to all geographic locations:

- eustatic rise caused by the melting of land-based ice sheets and glaciers or global change in the oceanic water level due to the expansion of near surface ocean water due to global ocean warming
- crustal subsidence or uplift from tectonic uplifting or down warping of the earth's crust. These changes can result from uplifting or cooling of coastal belts, sediment loading and consolidation, or subsidence due to volcanic eruption loading
- seismic subsidence caused by sudden and irregular incidence of earthquakes
- auto-subsidence due to compaction or consolidation of soft underlying sediments such as mud or peat
- climatic fluctuations may also create changes in sea level. For example, surface changes produced by El Niño due to changes in the size and location of high pressure cells
- sea level rise resulting from climate change.

The impact of climate change and other long-term changes should consider location specific data and documents.

The resulting increases in MSL are likely to cause related increases in all other water levels, including extreme water levels. In many cases sea level rise may become an issue, especially if wave heights that are depth limited, increase due to the higher water levels. Sea level rise would accordingly increase wave attack on coastal levees. (CIRIA; CUR; CETMEF, 2007).

7.2.4 Actions from other natural processes

Actions from external natural processes other than direct hydraulic loads may be subdivided as shown in Table 7.15. The following specific points should be noted:

- seismic actions are normally assumed not to act at the same time as hydraulic actions, unless the levee is impounding a permanent water body, as the chance of the simultaneous occurrence of a flood event and a strong earthquake is very low. However, this risk needs to be assessed on a site by site basis. All the other actions should be considered in combination with the imposed hydraulic load
- the impact of climate change on direct actions, such as hydraulic loads, are considered in outline in Section 7.3, 7.4 and 7.5, and the quantification of indirect effects, such as desiccation are discussed in Sections 9.5 and 9.12.2.
- the management and mitigation of some biological influences on levees such as tree growth, grazing and burrowing animals are considered in Chapter 4 and Section 9.12.3.
- actions from internal (deterioration) processes are considered in Chapter 8.

Table 7.15 Actions arising from external natural processes other than hydraulic

Action category	Action type	Suggested assumptions/operational restrictions/mitigation measures
Windblown sediment	Choke surface vegetation	Manage by specification of appropriate vegetation/surface protection to the levee.
	Accumulations on crest and upper sections of levee	For vulnerable sites include allowance in stability analysis. State what has been allowed for on drawings, with periodic removal of accumulations as part of maintenance regime, if required.
Wind on vegetation	Destabilising force in slope stability	The force depends on the wind speed, the coefficient of aerodynamic drag, Cd, and a factor applied that compensates for extra force due to turbulence. The drag coefficient of trees depends on the type of tree, the density of the foliage and the wind speed (see Box 7.6) The size of the hole formed when the root ball is pulled out of the ground varies with species, and ground and groundwater conditions, but for planning it could be taken as having a width of five times the trunk diameter and a depth of two-thirds of the trunk diameter. In some species it will be more of a plate shaped void than hemispherical.
Seismic	Shaking creates: • increased vertical and horizontal inertial loads from the dead weight of levee • liquefaction of natural saturated, loose soils. Seismic action varies significantly depending on proximity to a tectonic plate boundary and thickness of surface deposits.	• design accelerations are normally defined in National Standards (see Box 7.7). In addition, in some countries there are national technical guides, which can provide useful information on site characterisation (Box 7.8 gives details for the UK) • the vulnerability to damage under seismic action is characterised by the geotechnical parameters of the site (Section 7.8). Tools to carry out this assessment are presented in Chapter 8.8 • in some cases seismic activity is associated with a rise or fall in elevation of a landmass. This will result in a change in the hydraulics and morphology of the area, as well as having a direct effect on the level of integrity and freeboard afforded by the levee. Under these conditions a full re-evaluation of the site characteristics will be required • tsunami can be created by seismic shaking. This is covered under hydraulic loads in Section 7.4.4.
Ice on river	Increases water levels due to downstream blockage at bridge	Increase in hydraulic load is dicussed in Section 7.3.12.
	Ice pileup on water face of levee causing scour damage and ice elevated above design water level	Design methods are given in CIRIA; CUR; CETMEF (2007).
	Horizontal loads on vertical faces, when ice expands as it melts	The load depends on both the thickness of ice (which depends on number of degree days of freezing) and the rate of temperature rise when expanding (ICOLD, 1996, and USACE, 2002).
Rainfall	Saturates levee and reduces shear strength	Allow for elevated pore pressures in effective stress slope stability analysis.
	Surface scour due to concentrated runoff	Provide adequate surface protection, and where impermeable surfaces are unavoidable, such as at structures, detail so runoff does not concentrate on levee face.
Sun	Parches and kills surface vegetation	Specify vegetation appropriate to local climatic region, or if unsuitable for vegetation use gravel/cobbles. This should include consideration of potential future climate change.
	Desiccation of materials used to form levee	Use low plasticity clay and/or capillary break, piling or enhanced freeboard (Frith *et al*, 1997).
Climate change	This may affect both hydraulic actions and local climatic actions onto the structure	• impact on hydraulic actions • design to assess potential impacts on local climate (rainfall, sun, wind etc).

Box 7.6 *Evaluation of force on tree due to wind*

The pressure (p) exerted on a tree can be estimated from:

$$p = 0.5\, \rho_a\, TV^2\, Cd \quad (7.1)$$

(from Deltares, 2013)

where:

ρ_a = is the air density (kg/m^3)
(1.22 kg/m^3 at 20°C and 1013 mb pressure)

T = turbulence factor

(from NEN 6702:2001, but typically 2.5)

V = is the wind velocity (m/s)

Cd = is the dimensionless drag coefficient
(for trees with no leaves it can be taken as approximately 0.2 on the crown of edge trees, reducing to around 0.02 for trees further back in a wooded area, where the distance is over 30 times the height of the edge tree. On the edge this increases to 0.6 if the crown of the trees are in leaf. A value of 1.2 can be used for the tree trunk)

So, for a tree with leaves subject to a wind speed of 21 m/s:

$$p_a = 0.5 \times 1.22 \times 2.5 \times 21^2 \times 0.6 = 404 \text{ kg/ms}^2 = 0.40 \text{ kN/m}^2$$

For a tree with a crown diameter of 5 m this would give a force at the middle of the crown of 7.8 kN. Coppin and Richards (2007) give a similar equation but without the turbulence term.

More detailed calculations can include dynamic effects, with guidance given in Section 3.2 of Wong *et al* (2011).

Box 7.7 *Characterising sites for earthquake, examples of national standards*

Europe

BS EN 1998-1:2004 sets out the principles of seismic design. Section 3 of Part 8 sets out how the site is characterised into ground types A to E, which relate to the depth and type of superficial deposits. The reference peak ground acceleration is defined in national annexes for each member country.

USA

The United States Geological Survey (USGS) ground-motion database and ground motion parameter calculator are tools to calculate the seismic design values for both buildings and bridges, to a variety of US and international design codes (USGS, 2013). For levees the design standard for the annual chance ground motion depends on the levee's category and the agency co-ordinating the evaluation policy, generally being between 1 in 100 and 1 in 500 chance per year.

Box 7.8 *National guides can provide additional information on site characterisation for earthquakes (UK example)*

An engineering guide to seismic risk to dams in the UK (Charles, 1991) and its application note (ICE, 1998) provide useful information on historic earthquakes in the UK and methods of inferring the variation in peak ground acceleration along with annual chance of occurrence at different locations within the UK.

Further reading

Details on hydrology, hydraulics, waves, sedimentation and morphology, which is related to the information in Section 7.2 is presented as follows:

Hydrology

- Bedient *et al* (2002) *Hydrology and floodplain analysis*
- Chow *et al* (2013) *Applied hydrology*
- Maidment (1993) *Handbook of hydrology*

Hydraulics

- Chow (1959) *Open-channel hydraulics*
- Chadwick and Morfett (1998) *Hydraulics in civil and environmental engineering*
- Henderson (1966) *Open channel flow*

Waves

- Abbott and Price (1994) *Coastal, estuarial and harbour engineers' reference book*
- Dean and Dalrymple (1991) *Water wave mechanics for engineers and scientists*
- Goda (1985) *Random seas and design of maritime structures*
- Herbich (ed) (2000) *Handbook of coastal engineering*
- Holthuijsen (2007) *Waves in oceanic and coastal waters*

- Sorensen (1993) *Basic wave mechanics for coastal and ocean engineers*
- Pullen *et al* (2007) *Euro top: Wave overtopping of sea defences and related structures: assessment manual*

Sediment transport mechanics

- Graf (1984) *Hydraulics of sediment transport*
- Julien (2010) *Erosion and sedimentation*
- Simons and Senturk (1976) *Sediment transport technology*

Morphology

- Horikawa (1988) *Nearshore dynamics and coastal processes: theory, measurement, and predictive models*
- Julien (2002) *River mechanics*
- Komar (1997) *Beach processes and sedimentation*
- Leopold *et al* (1964) *Fluvial processes in geomorphology*
- Soulsby (1997) *Dynamics of marine sands*
- Thorne *et al* (1997) *Applied fluvial geomorphology for river engineering and management*

7.3 MORPHOLOGY AND HYDRAULIC ACTIONS FOR RIVERINE LEVEES

The principal driving parameter in the evaluation or design of riverine levees is water level. Water levels establish the conditions under which the levee will perform. Water levels are a direct result from hydrologic and hydraulic conditions that exist in the catchment and their interaction with basin geology and boundary conditions.

Hydraulic loads describe the estimation of flow characteristics that range from large, rare flood events to normal, everyday flows. The information in this section complements that contained in Chapters 2, 5, 8 and 9, and references to those overlaps are cited in the text. There is considerable overlap between morphology and river hydraulics as indicated in Figure 7.10. In a sense, morphology sets the stage upon which the basin runoff acts to form the river system. This does not imply that the process is static, quite the contrary. The interaction between basin runoff and channel boundaries is an ongoing process. The river will continue to adjust over time and levee design should take this into consideration.

Assessing levee design requires an understanding of present as well as future conditions over the life of the project. Hydraulic conditions (water surface elevations and discharges) and morphological conditions must be developed and analysed for:

- **present without project conditions:** forms the baseline against which options are compared to
- **future without project conditions:** provides the framework for assessing change anticipated in the absence of a project, and provides understanding of risk against which options are compared to
- **current with project conditions:** provides the means to compare change resulting from options/plans
- **future with project conditions:** provides the means to assess performance over the project life.

Assessing the state of existing levees should also consider past and anticipated changes in water surface elevations over time such as climate change and regional/global factors.

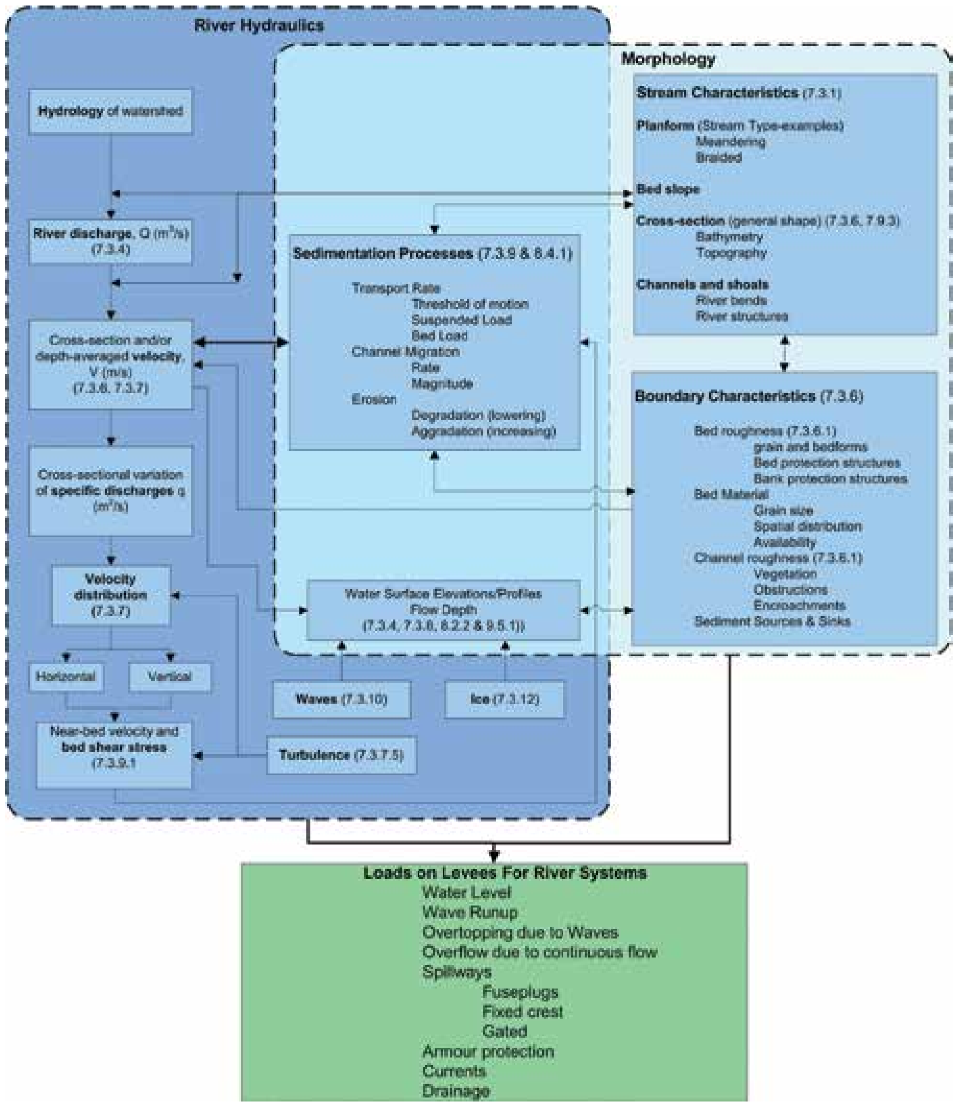


Figure 7.10 *Relevant sections of the handbook for analysis and design considering river currents and discharges*

7.3.1 River morphology

Rivers are dynamic systems that can change in response to variable hydraulic actions. Even a single flood event can produce large adjustments such as degradation or aggradation of the channel bed with associated impacts on water levels, the expansion or narrowing of the channel width, and migration of the main channel or avulsion (adoption of a new course at a different location). Understanding river morphology, the changes of river plan form and cross-sectional shape, provides a basis to assess the mobility of a river that may affect levee stability and the design. In the same way the possible impact of levees on river mobility also needs to be taken into account. The main morphological processes that impact levees are related to:

- **lateral movements:** such as shifts of channel position, development of meanders, avulsion and change of stream pattern
- **vertical movements:** such as degradation and aggradation, scour related to flood events, local, bend, confluence and contraction scour and bed-form migration.

Levees may change the initial hydraulic conditions and so change the dynamic equilibrium of the river. For example, levees that protect areas within a river valley or floodplain may restrict a portion of the natural conveyance area of the river (see Figure 7.11). The confinement of river width is likely to have an influence on longitudinal bed profiles, resulting in scour processes, and increasing water levels (Section A on Figure 7.11). As most river systems with levees have subcritical flow regimes, the increase of water levels may propagate upstream of the leveed reach (Section B on Figure 7.11), which could result in flooding of upstream properties. Hydraulic conditions in the reach downstream of a leveed reach may also be adjusted and have a significant role in changing the morphologic response of the river. An extreme example would be where the reduced hydraulic gradient upstream of a leveed reach changes the river from a meandering system to a braided system. This could result in rapid changes in channel location within the available floodplain. Such changes may lead to flows being redirected toward the levee or along the levee toe. Even when radical shifts in channel response are not anticipated, the channel will try to adjust to the new state of dynamic equilibrium consistent with the modified hydraulic conditions.

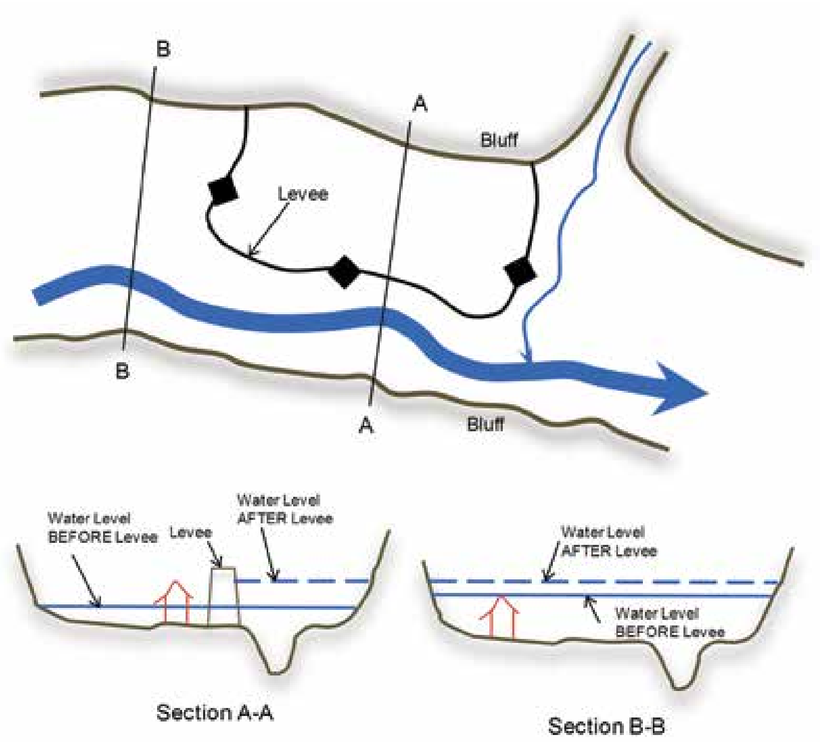


Figure 7.11 Impacts of levee on river water levels (courtesy USACE)

Floodplain development with levees may involve the realignment of the river by suppressing bends, decreasing the length of the river and increasing the channel slope. This may modify the sediment load conditions resulting in channel bed degradation (see Box 7.9). Even small changes in channel profile can cause significant changes in the 'quasi equilibrium' of the system and how sediment is transported.

Table 7.16 summarises 10 different types of profile, the stability problems associated with the channel form and the potential effect this could have on a levee.

Table 7.16 Some stream channel types and their characteristic stability problems and potential impacts on levees (after USACE, 1994a)

Channel type	Typical features	Stability problems	Potential impacts to levees or threats to levee integrity
Mountain torrents	• steep slopes • boulders • drops and chutes.	• bed scour and degradation • potential for debris flow.	• bank instability, shifts in channel alignment.
Alluvial fans	• multiple channels • coarse deposits.	• sudden channel shifts • deposition • degradation.	• rapid change in water profiles, rapid shift in channel location, bank instability, shifts in current direction.
Braided rivers	• interlacing channels • coarse sediments (usually) • high bed load.	• frequent shifts of main channel • scour and deposition.	• bank instability, rapid shifts in channel location, rapid change in water profiles, shifts in current direction.

Table 7.16 Some stream channel types and their characteristic stability problems and potential impacts on levees (after USACE, 1994a) (contd)

Arroyos/ Wadies	• infrequent flows • wide flat channels • flash floods • high sediment loads.	• potential for rapid changes in planform, profile, cross-section.	• bank instability, rapid changes in current pattern and water profiles.
Meandering rivers	• alternating bends • flat slopes • wide floodplains.	• bank erosion • meander migration • scour and deposition.	• bank instability at bends, gradual changes in channel location and current patterns.
Modified streams	• previously channelised • altered base levels.	• meander development • degradation and aggradation • bank erosion.	• bank instability, changes in water profiles.
Regulated rivers	• upstream reservoirs • irrigation diversions.	• reduced activity • degradation below dams • lowered base level for tributaries • aggradation at tributary mouths.	• change in water profiles, increased bank height, bank instability.
Deltas	• multiple channels • fine deposits.	• channel shifts • deposition and extension.	• changes in water profiles, gradual change in channel location, distributary channel development causing deposition.
Underfit streams	• sinuous channels • low slope.	• meander migration.	• bank instability.
Cohesive channels	• irregular or unusual plan form.	• variable.	• variable, but response typically slower due to cohesive boundary materials.

Table 7.17 extracted from USACE (1994a) provides a rating of how some flood protection measures may impact stream stability depending on the channel types as described in Table 7.16.

Table 7.17 Rating of flood control measures for channel stability (after USACE, 1994a)

Flood protection measures	**Channel types**									
	Mountain torrents	***Alluvial fans***	***Braided rivers***	***Arroyos/Wadies***	***Meandering rivers***	***Modified streams***	***Regulated rivers***	***Deltas***	***Underfit streams***	***Cohesive channels***
Levees set beyond stream meander belt	1	2	2	1	1	1	1	2	1	1
Levees set within stream meander belt or along bank line	2	5	5	4	3	3	2	4	2	2
Compound channel – low-flow pilot plus flooding berms	5	8	8	7	7	6	6	7	4	4
Significant channel widening	6	9	9	8	8	6	7	7	5	5
Significant channel widening and deepening	7	9	9	9	9	8	8	8	6	7
Significant channel widening, deepening and straightening	8	10	10	10	10	8	9	9	7	8
Floodway, diversion or bypass channel	4	5	5	5	4	4	4	5	3	3

Notes

Channel stability rating scale:
No stability impacts » 0
Major impacts on stability » 10

7.3.1.1 Approach to undertaking a morphological study in a fluvial setting

A morphological assessment is essential to establish the present state of river behaviour and to support projections of changes in behaviour that may result from levee construction (or alterations where existing levees are modified). Morphological assessments can include estimates of current system stability, changes that may occur over the life of a proposed levee once constructed, as well as projections of changes that may result over that same project life in the absence of the constructed levee.

A morphological assessment can include the review of historical information, present day information collected in the field (Section 7.1.4) and the use of models in order to evaluate:

- channel plan form (width, and appearance/location/size of bars, islands, pools, riffles/rapids)
- channel bathymetry (cross-section data or in-channel information)
- channel location (stable, wandering, avulsing or migrating)
- bank scour (channel widening or channel migration as rate of widening or bank movement, m/yr)
- bed elevation (stable, incising or aggrading including estimated rates of change, m/yr)
- floodplain characteristics (vegetation or land use changes etc)
- bed material (sizes, volume and composition)
- flow obstructions (logs, log jams, bridges, groins etc)
- channel modifications (water operations, mining, levees etc)
- natural changes in catchment (fire, landslides, deforestation)
- anthropogenic changes (agriculture, grazing, houses, dredging, channelisation and straightening, road/bridge crossings)
- sediment budget (sediment sources, sinks and transport).

The detail of the assessment and the processes that should be investigated will depend on the particular conditions of the river and projected levee. For example, a system with high rates of woody debris that may be directed towards the levee in a sharp bend will require a detailed assessment of the processes.

Historical information, such as aerial photographs and topographic data, can be used to determine river pattern, location and width over several decades providing an insight into channel stability. When limited adjustment is observed, future river conditions can be estimated by simple extrapolation of past and current rates of change, provided that no major changes occur in the catchment and river system. For example, channel degradation can be inferred from observations of bed levels at particular locations and at different times. The rates of degradation estimated with historical data can then be used to predict future bed levels. However, a levee within the river channel may change the initial conditions in the system preventing the application of such simple extrapolations.

The lack of change in plan form or floodplains could also be noted, especially if known large flow events have occurred during the comparison timeframe. If there is no historical information available, field assessments of old channels and their sediments, the presence of terraces and other information visible in the floodplain provide data on past valley/river conditions.

Present day information should be collected over areas extending downstream and upstream of the project, relative to the various phases of the project. The information presented in Table 7.18 is also considered to be appropriate and applicable to estuaries. Section 7.9 presents different methods to collect the required data.

1 2 3 4 5 6 7 8 9 10

Table 7.18 Recommendations for morphologic data collection for rivers and estuaries (adapted from CIRIA; CUR; CETMEF, 2007)

Project phase	Data required
Reconnaissance	• bed geometry in the estuary – use available maps and navigation charts, cross-sections every 1 km to 5 km for general survey • bed geometry in a river – cross-sections every 5 km to 10 km along entire river for general survey, adapted to the length of the river • historic aerial photography over different times to determine pattern and general dimensions • topography – use available point and contour elevation maps, additional elevation data using lower resolution photogrammetry or satellite imagery • qualitative assessment of sediment budget • limits of investigation extend well beyond (upstream and downstream / up-current or down-current) the area of primary interest • land use • debris transport • ice jam potential • imposed modifications (engineered features) • in-stream mining • zoning restrictions (or lack thereof).
Feasibility	• bed geometry in the entire estuary – combination of cross-sections every 1 km to 5 km for general coverage and every 10 m to 100 m for details • bed geometry in a river – cross-sections every 10 m to 100 m for detailed survey, adapted to the length and width of the river • floodplain geometry adjacent to estuary or river – use photogrammetry, LiDAR, 3D laser scanning to develop digital terrain model (DTM), or conventional levels continued at same intervals for bathymetry to extend of the expected inundated area • sediment transport in rivers should be measured at one or more locations during low and high river discharges to enable the relationship between water discharge and sediment transport to be determined. This can be used for selecting the appropriate model and equation for sediment transport prediction • transported sediment should be sampled to determine its characteristics • sediment transport in an estuary is difficult to measure. Tide, waves and differences of water density cause rapid changes in transport, so extensive measurement campaigns are needed • along a river, bed material sampling should be done every 1 km to 10 km depending on size of river. The bed should be sampled in at least three positions over each cross-section • in an estuary, bed material should be sampled according to a survey that has a square grid of between 1 km and 5 km, with the grid being adapted to the width of the estuary.
Detailed	• bed geometry of the estuary or river where a project feature exists – cross-sections every 10 m or DTM with a 10 m square grid. Limits of detail survey to be determined using critical zones identified by hydraulic results from alternative appraisal or expected construction limits, whichever is greater • topographic geometry of floodplain adjacent to a project feature – cross-sections every 10 m or DTM with a 10 m square grid.

7.3.1.2 Importance of river characteristics

The characteristics of a river channel change within the catchment. Typically, adjustment of the longitudinal profile of a stream occurs from the headwaters of the catchment to the coastal zone as depicted in Figure 7.12. The longitudinal profile greatly influences the processes of water and sediment transport in a channel. Over long timescales, changes will always occur as a river system attempts to maintain a dynamic equilibrium to balance sediments supplied from the upstream catchment with sediments discharged towards the lower portion of the river course. Depending on the type and size of the river, the timescale to achieve dynamic equilibrium may be large, ie of the order of centuries. While this length of time may appear irrelevant to an engineering project having a life of 50 years or less, the long-term equilibrium is of crucial significance as it will identify trends of river adjustment at medium timescales. The following are some basic concepts to be considered when working with catchments and rivers (Biedenharn *et al*, 1997):

- the river is only part of a system
- the system is dynamic

- the system behaves with complexity
- morphologic thresholds exist, and when exceeded, can result in abrupt changes
- morphologic analyses consider timescales (system response scale versus project life) that:
  - provides an historic perspective
  - gives a view of past and potential channel stability/instability
  - assesses impacts of the proposed improvements.

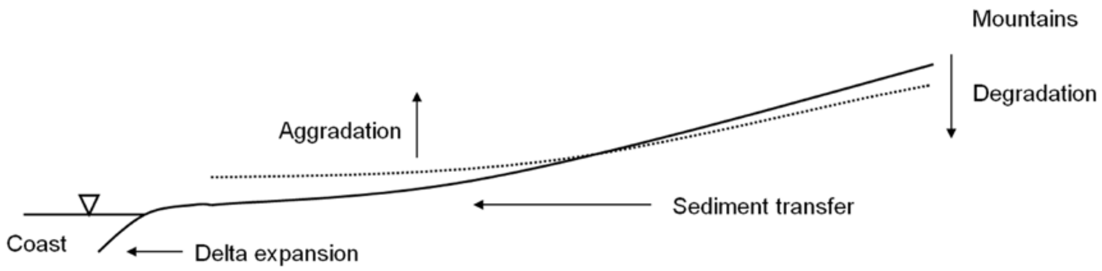


Figure 7.12 Typical longitudinal profile and direction of change through time (courtesy USACE)

Numerous river classification systems can be found in the literature (Bettess *et al*, 2011, Brice, 1964, and Morisawa, 1985). Other river classifications systems such as Schumms' or Downs' (as presented in García, 2006) provide relationships that link observed trends and patterns to fluvial and sediment processes responsible for driving channel change. Table 7.19 summarises the characteristics of four general types of river: straight, meandering, braided and anastomosed. A river classification provides the means to understand evolution processes. For example, in the case of meandering rivers the processes likely to happen are those related to downstream migration, avulsion and cut-offs. In the case of braided rivers, the common evolutionary processes are the shifting or creation of new channels.

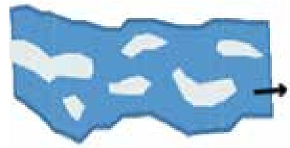
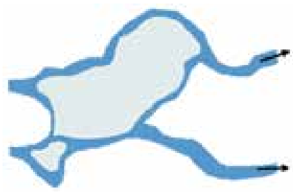
García (2006) states that the concept underpinning river stability is that over time the cross-sectional dimensions and longitudinal slope of the channel adjust so that the channel is able to convey the discharges of water and sediment. So, the essential parameters related to the dynamic equilibrium of a river are:

- water discharge
- energy slope
- sediment load
- sediment type (grain size of its bed and banks).

For a catchment these parameters are determined by climate (rain, temperature), geology (topography, lithology), ground characteristics, vegetation and human activity within the catchment and along the stream.

Variations in sediment load or water discharge affect not only the longitudinal profile but also the stream cross-section and the river alignment. For example, a river may change from a single channel system to one that has multiple, wider channels if the bed load is increased significantly. Along the coast, a long and hydraulically inefficient channel may be replaced by a shorter and more direct channel. So, the design of a levee project in a river environment should assess the river behaviour along a reach that includes areas upstream and downstream of the proposed project.

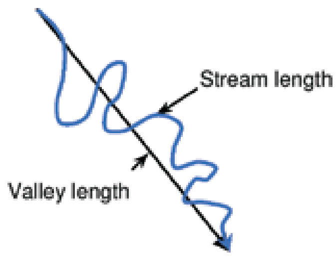
Table 7.19 *River classification (after Morisawa, 1985 and Leopold and Wolman, 1957)*

Type of river	Characteristics				
	Channel pattern	*Sinuosity*[(a)] L_s/L_v	*Principle sediment load type*	*Width to depth ratio*[(b)]	*Sketch*
Straight	Single channel with pools and riffles, meander-ing thalweg	<1.05	Suspension mixed or bedload	<40	
Meandering	Single channel	>1.5	Suspension mixed load	<4	
Braided	Two or more channels with bars and islands	<1.3	Bedload	>40	
Anastomosed	Two or more channels with bars and islands	>1.5	–	–	

Notes

a Sinuosity is the channel length, L_s, between two points of interest divided by the valley length, L_v, between the same two points.

b Width to depth ratio is determined by dividing the channel width at top bank by the mean depth at bankfull conditions (some classification systems may use maximum depth).



Lane (1955) provided a qualitative relationship between the four parameters to assess river equilibrium:

$$Q_s \times D_{50} \propto Q_w \times S \qquad (7.2)$$

where:

Q_s = sediment discharge (m^3/s)

D_{50} = representative sediment particle size (typically the 50th percentile)

Q_w = water discharge (m^3/s)

S = energy slope (m/m)

The relationship is used to assess the change in any one variable given estimated changes in the remaining variables. It is not necessary to know values for the variables and only the expected direction of change, higher/lower is considered. Box 7.9 presents an example of the use of this relationship.

Box 7.9 ***Example of qualitative method to assess morphologic change using method of Lane (1955)***

Situation:

A levee is to be constructed adjacent to a sand bed stream. Approximately two-thirds of the floodplain will be removed from the hydraulic cross-section. There will be no change in catchment runoff characteristics.

Given:

- water discharge, Q_w, from upper catchment remains constant
- stream bed material, D_{50}, does not change as there are no new sediment sources or sinks to alter the bed gradation.

Assessment:

- consider the right hand side of Lane's (1955) relationship (Equation 7.2).

Because Q_w will not change (there are no changes in the catchment), the energy gradient increases with a decrease in flow area. So, S will increase.

Analysis:

Q_w = constant

D_{50} = constant

S = + (increase)

Q_s = ?

Using Lane's relationship (Equation 7.2), predict sediment response (let the top line indicate the relative direction of expected change).

$$\overset{?}{Q_s} \bullet \overset{(c)}{D_{50}} \propto \overset{(c)}{Q_w} \bullet \overset{(+)}{S}$$

This indicates that Qs should be expected to increase to maintain the relationship.

$$\overset{(+)}{Q_s} \bullet \overset{(c)}{D_{50}} \propto \overset{(c)}{Q_w} \bullet \overset{(+)}{S}$$

An increase in Q_s indicates that degradation or lowering of the channel bed should be anticipated. Check stable bank height to see if there is direct effect on bank and levee stability.

7.3.1.3 Threshold condition in river morphology

The concept of a threshold condition in river morphology plays a significant role in evaluating the potential for change. Thresholds, generally evolved from empirical data used to develop morphologic relationships, identify a critical condition where change should be expected to occur. When threshold values are crossed, change can be rapid. Thresholds are typically based on empirical evidence and are defined where a change in river type or characteristics appear in the data. They play a significant role in establishing morphologic characteristics as well as the response to changes in a river system dynamics.

The threshold separating meandering and braided river channels is of particular interest to levee projects. Various definitions have been proposed for separating meandering and braided rivers. Leopold and Wolman (1957), Ackers and Charlton (1970), and Ackers (1982) proposed relationships between longitudinal bed slope, S_0 (m/m), and the bank full or dominant discharge, Q (m^3/s). Bank full discharge occurs at the point just before floodplain inundation. Dominant discharge is the equivalent flow that would create and sustain the river cross-section. Leopold and Wolman (1957) studied a variety of natural streams and produced a best fit of data, which is defined by Equation 7.3.

$$S_0 = 0.0125Q^{-0.44} \qquad (7.3)$$

Meandering streams as classified by Leopold and Wolman (1960) are those with a single channel and sinuosity greater than 1.5. Braided streams are those that have relatively stable alluvial islands and two or more channels.

Richards (1982) provides an alternative method to distinguish between meandering and braided by use of a stream power index Ω (m^3/s) = QS_0 with variables as defined for Equation 7.3. A threshold value Ω_{lim} is developed using the D_{50} size of bed material as shown in Equation 7.4. A braided river has a stream power index that is higher than Ω_{lim}. A meandering river has a stream power index smaller than Ω_{lim}.

$$\Omega_{lim} = 0.011D_{50}^{0.77} \qquad (7.4)$$

These and other simplified methods are based on empirical data. River hydraulics manuals (eg Jansen, 1979, and Bravard and Petit, 2000) can provide further information on particular characteristics and methods of analysis to establish channel type. CIRIA; CUR; CETMEF (2007) also provides descriptions of methods for evaluating these and other morphologic thresholds.

7.3.2 Hydraulic actions on riverine levees

Water levels and discharges are the principal boundary conditions for the analysis and design of levees, where water levels are a function of the discharge (see for example Box 7.15). Determining these parameters is therefore the essence of a levee hydraulics investigation. The ability to predict the water level, discharge and velocity of any point on a river as a function of time is important for the design of levee alignment, geometry, navigation and for determining requirements for bed and bank armour protection and levee surface protection (see Section 9.6) and for analysis of environmental impact or enhancement.

River discharges linked to bed slope are the dominant influence on hydraulic conditions in rivers. Other influences include:

- floodplains and embankments
- structures in the river such as spur-dikes, roadway crossings, and barrages
- roughness of the river bed and floodplains
- confluences, bifurcations, weirs and spillways.

Variations in water level and discharges are caused by meteorological influences. Water levels also depend on the local bathymetry, which is in turn influenced by currents in the flow.

Discharges

Normal river flows are those that occur on a daily basis throughout the year. These are non-flood flow conditions but because they occur most of the time their characteristics are important particularly for normal operations and daily maintenance issues. However, several types of floods exist:

- **transient floods:** these have short durations with a probability of one of occurring during the lifetime of the levee
- **rare floods:** these are short duration events with a high probability, but less than one, of occurring during the lifetime of the levee
- **accidental floods:** these are extreme condition floods that occur in combination with other events (seismic situation) and have a low probability, much less than 1, of occurring during the lifetime of the levee.

Flood discharges that exceed the channel's banks and where water begins to spill onto the floodplain, are typically described in terms of the probability that they may occur in any given year, for example, a two-year flood or a 50 per cent chance event, a 100-year flood or one per cent chance event etc. Levees are typically designed for these larger flood events. So, estimating site conditions to represent these large events is vital to designing or evaluating levees, and is the main discussion for this section (see also Chapters 8 and 9).

Water levels

The principle of conservation of mass and momentum is used to compute flow conditions needed to develop the water level-discharge relationship. Most problems can be solved by combining the conservation laws with simplifying assumptions, a set of boundary conditions and empirical parameters. The basic equations and a discussion of simplifications can be found in Chow (1959) and Henderson (1966).

The discharge-probability and water level-discharge relationships are typically the first information derived. If the water level discharge curve is combined with discharge-probability information, a water level-probability function can be created. The water level-probability function indicates the likelihood that the maximum unregulated or regulated water surface elevation will reach a specified value in any year and subsequently expose people and property to the associated threat.

Other hydraulic boundary conditions are closely related to the discharges and water levels and include velocity and water depth. The term 'current' is often used when describing movement of the water and refers to both velocity and direction of water movement.

The hydraulic conditions also affect the morphology (Section 7.3.1) and rate of sediment transport (Section 7.3.9) through the boundary shear stress and rate of energy dissipation and sediment transport rate. As a result, discharge also affects the river boundary as it interacts with bed and bank materials to shape the channel geometry.

Uncertainty: because the physical processes are not completely known or represented in models, there is uncertainty in estimated values, which should be considered. Flood risk analysis and levee performance assessment (Chapter 5) are the accepted methods for incorporating uncertainty in estimating water level for levee design or for evaluating existing levees.

7.3.2.1 Discharge and water level relationships

Measurements or estimates of discharges or water level can be presented graphically in several ways. Some important diagrammatic/relationships and their relationship to levee analysis and design are detailed as follows:

Hydrographs

Water level or discharge plotted as a function of time (Figure 7.13). This may be used in selecting yearly peak values and evaluating characteristics of floods. It is also useful when sequencing construction and maintenance activities.

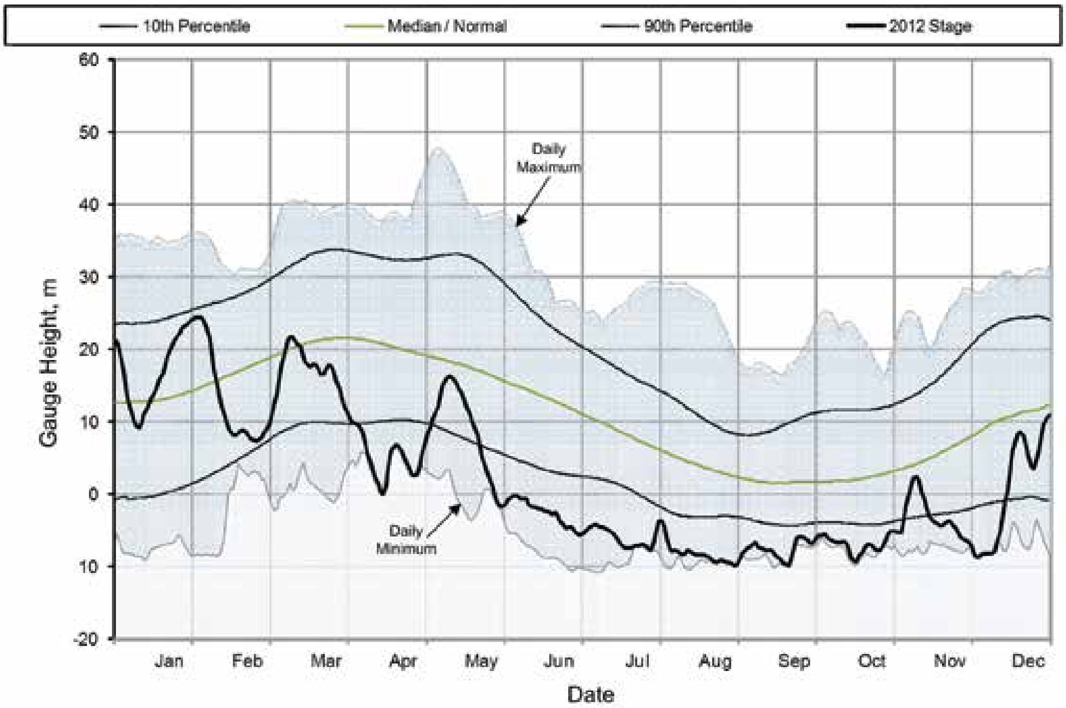


Figure 7.13 Generic example of a water level hydrograph plot

Mass curves

Cumulative discharge as a function of time (Figure 7.14). This may be used in evaluating storage components of the levee system such as phased overtopping to reduce flood crests elsewhere or where a levee affects interior drainage.

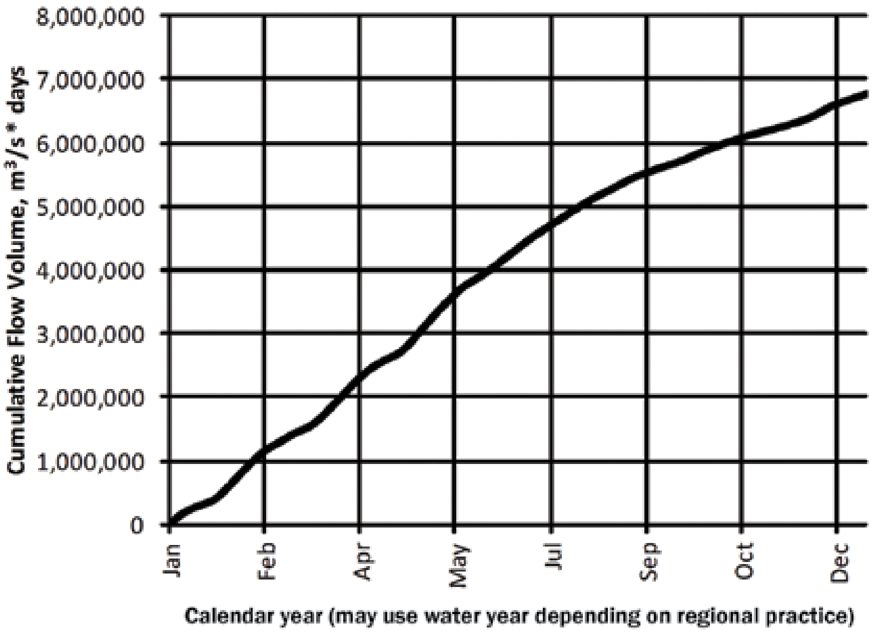


Figure 7.14 *Generic example of a mass curve*

Duration curves

Number of days a certain discharge or water level is (or is not) exceeded (Figure 7.15). It provides an insight into the length of time levees may be exposed to floods.

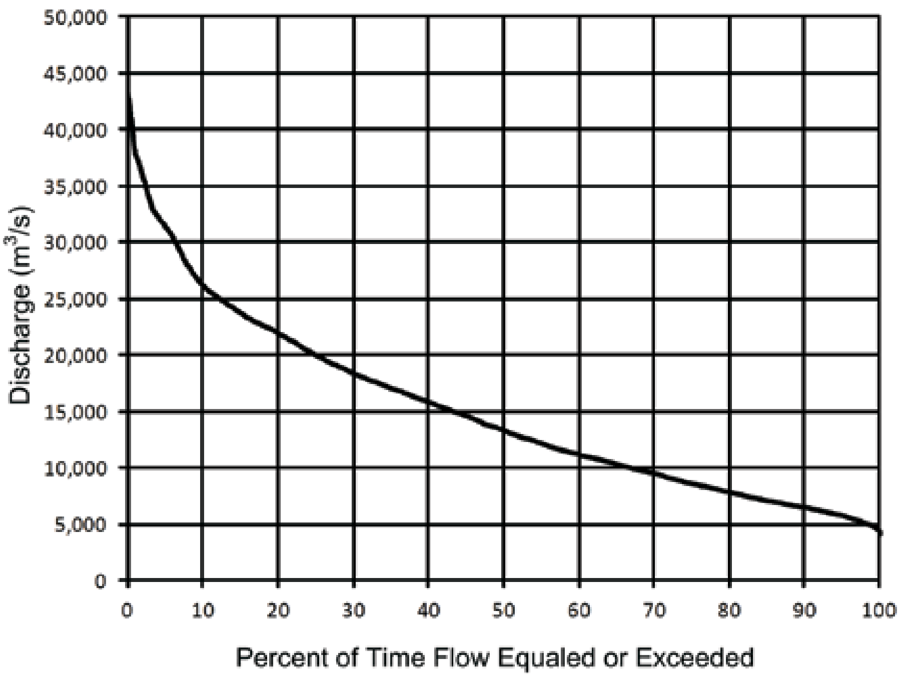


Figure 7.15 *Generic example of a flow duration curve*

Discharge exceedance curves

Number of days a certain discharge or water level is equalled or exceeded. This is similar to the duration curves and may also aid in planning construction works (new or remedial).

Water level relation curve

Water level at various stations as a function of one specific location (Figure 7.16). May be used to evaluate conditions over an extended reach relative to a particular point of interest. Most useful for projects over an extended reach of river.

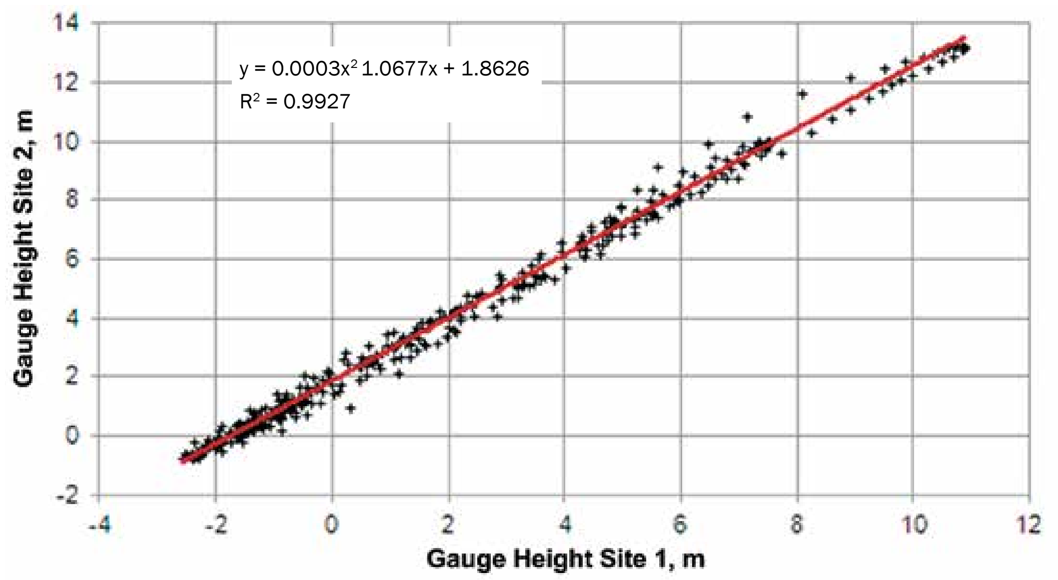


Figure 7.16 ***Generic example of a water level relationship between two locations on the same river (sites 1 and 2)***

Rating curve

Relationship between discharge and water level at one station (Section 7.3.4 and Figure 7.17).

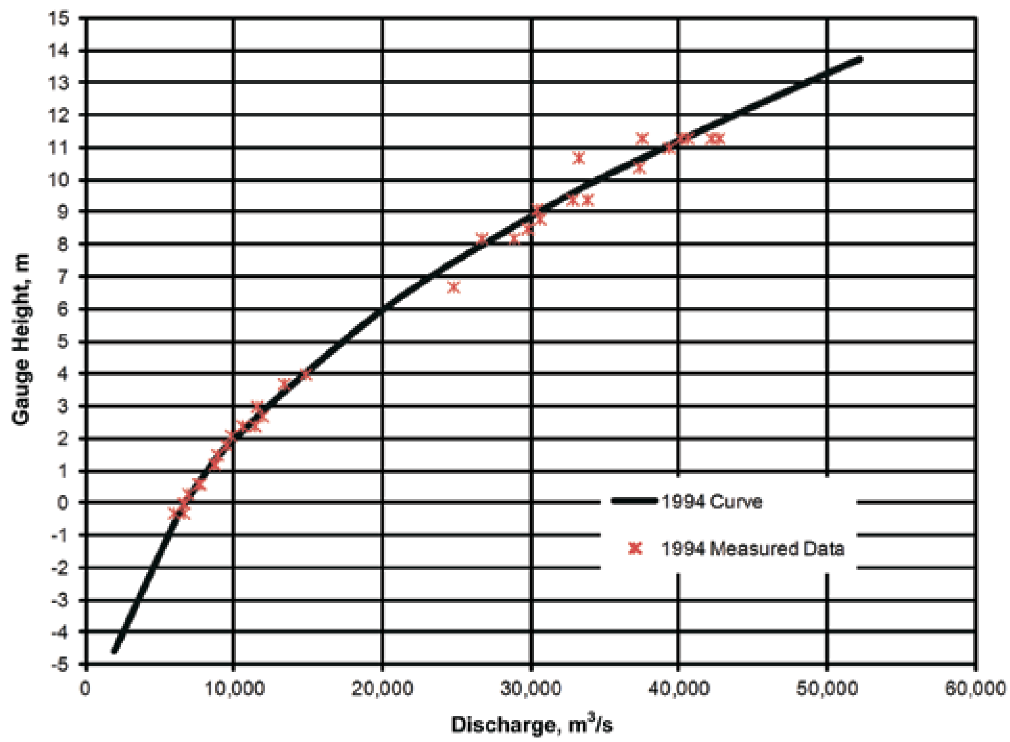


Figure 7.17 ***Generic example of a rating curve showing measured points and best fit line***

Specific gauge curves

Water levels for given discharge(s) versus time (Figure 7.18). This may provide an indication of change in channel capacity over time, ie if the water level trends upward over time for a given discharge the capacity is decreasing and higher flood levels may be expected.

1 2 3 4 5 6 7 8 9 10

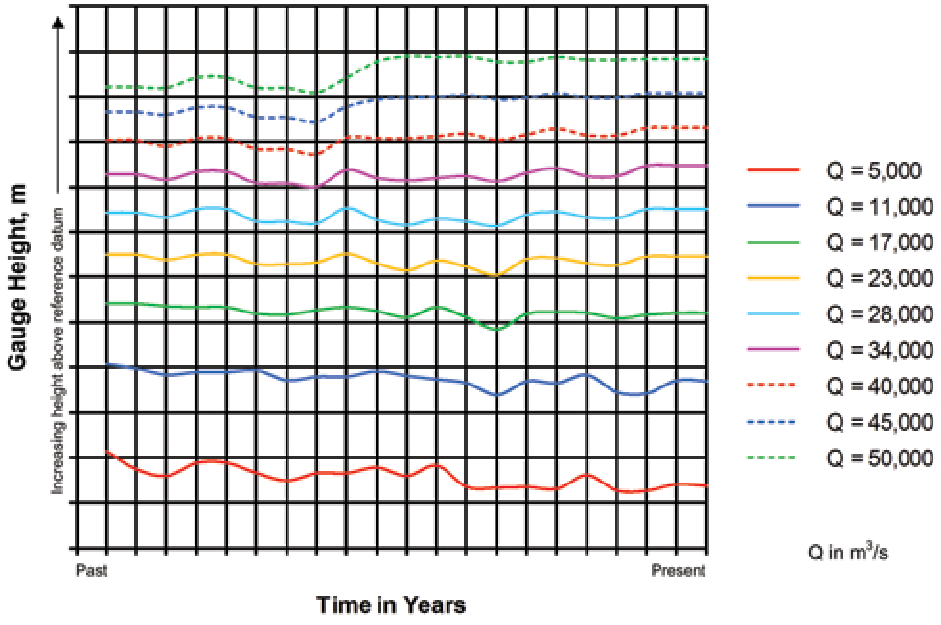


Figure 7.18 Generic example of specific gauge curves

7.3.2.2 River hydrology and flood flows

Basin hydrology governs water levels in a river by producing discharge hydrographs that vary with time. When a flood hydrograph occurs, it can be visualised as a wave front that moves downstream through the stream system. Extreme water levels are associated with flood waves. Propagation of flood waves through the river system is a function of catchment and channel characteristics. Several types of flood can be distinguished:

- **flash floods:** of catchment areas, frequently small in size, with immediate response to rainfall. Generally have a short duration
- **fast floods:** over a short concentrated period of a few hours because of strong rainfall or in catchments having steep slopes
- **plain floods:** with slow kinematics, caused by overflow from the main channel
- **groundwater floods:** combined with overflow from rivers and are very slow to spread in the floodplain, and also slow to subside.

Before being able to predict hydraulic loads on levees, it is necessary to have a basic knowledge and understanding of the meteorology and catchment characteristics for the location in question. Hydrology relates to the estimation of surface runoff and stream flow including both small and large flood events using historic data and estimation tools. Hydrological estimates include both discharge as well as event duration, as needed to analyse flood reduction measures. Table 7.20 presents various hydrological analysis needed for levee projects.

Table 7.20 Hydrological analysis needed for levee projects

Measure	Rainfall-runoff			Frequency and gauge statistics			River hydraulics	
	Reconstitute historic floods	*Develop hypothetical floods*	*Analyse the changed discharge/ water level-frequency*	*Develop historic data*	*Develop from hypothetical events*	*Volume-duration studies*	*Elevation (water level) conversion from discharge*	*Sediment transport/deposition analyses*
Levees	Y	Y	Y	S	Y	N	Y	S
Interior drainage[1]	S	Y	Y	N	Y	N	S	N

Notes

Y = usually done, major part of study

S = done less often, still major part of study

N = not usually done

1 Interior drainage issues that result from levee construction include the interruption of natural flow paths.

Even long-term historic data records may not encompass the full range of flow events needed for hydrologic extrapolation and levee evaluation/design. Estimating large floods (hydrology) can be done statistically using empirical (historical flood, river gauging) information that can be extrapolated from nearby systems, or can be estimated from frequency rainfall information using a rainfall-runoff model, Table 7.21.

Table 7.21 Hydrological analysis for levees

Component	Determinants	Provides/influences
Catchment runoff	Topography (steepness or slope), land use, soil type (infiltration rates), vegetation, climate (precipitation), basin shape, basin orientation relative to predominant weather patterns, stream network development	Rate, duration, and volume of water derived from the catchment
Groundwater interaction	Soil stratigraphy and permeability, presence of aquifers	Base flow, loss of water from surface flow in loosing streams
Flood routing	Channel and floodplain characteristics, change in available volume within floodplain due to levee project, presence of storage or detention features as part of project (eg provisions for overflow of some levees to reduce loadings on other levees)	Changes in rate, duration, and volume of water due to the influence of stream, floodplain, and project components
Statistics	Observed stream data, synthetic data derived from long-term simulation using catchment characteristics and models, transposition of data from similar catchments, statistical method used	Understanding of extreme events through discharge-probability relationship, duration curves, understanding of basin response through plots of water level and flow hydrographs at one or more points along stream of interest

For levee design and analysis, knowledge of the magnitude and probability of large events is crucial. So long-term collection of flood information is frequently available on large river systems, very often at multiple locations.

7.3.3 Measurements of flows and water levels

Prediction of future river discharges and water levels requires data on stream behaviour, both current and past.

So, it is desirable to have a long-term systematic stream gauge network measuring the water surface elevations over time. Water levels should be measured relative to a reference point, which may be arbitrary or adjusted to the local vertical datum. Where an arbitrary point is used, this point (referred to as gauge zero) can be tied to the local vertical datum. The water level above the reference point is known as gauge height. A variety of measurement devices may be used (see Section 7.9.5, and also Figures 7.19 and 7.20). Some locations also include discharge measurement, which may be accomplished by several different techniques. Measurement of flow discharge, the volumetric rate of water flux, provides a more complete understanding of site conditions.

Figure 7.19 ***Direct flow measurements by hydrographers using Acoustic Doppler Current Profiler (ADCP) (a), field data collection platform for stream gauge and precipitation (b) (courtesy USGS)***

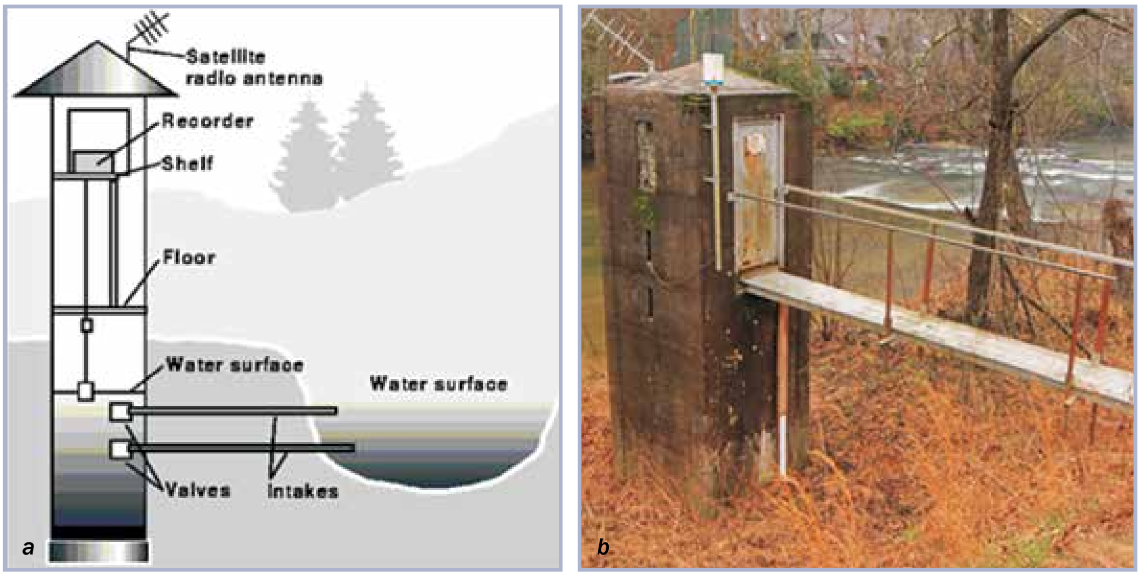


Figure 7.20 ***Schematic (a) and photograph (b) of a typical river gauge installation (courtesy USGS)***

Stream flow data from a stream gauge network are used throughout the design, flood threat detection and forecasting process.

Historical water levels and corresponding flows are used to:

- configure and initially calibrate the catchment runoff models. The parameters of the soil-moisture accounting model and the unit hydrographs are found through a 'trial and error' process in which computed values with trial estimates are compared with observed flow data
- configure and initially calibrate the channel models that are used to determine design water surface profiles.

Observed water levels are used in real time to:

- identify any existing threats due to high water in rivers or streams
- assess the quality of forecast models. If the computations do not reflect the observations well, forecasters adapt the model, adjusting parameters and states to improve the fit before issuing a forecast
- assess the quality of water control system simulation, and to some extent, the efficiency of the operation. If weather forecasters see that simulated values do not match the observed well, they will adjust the models, contact the operators for additional information or in some cases contact the operators and advise that the actual operation is other than what is intended and expected
- quantify the impacts of levee overtopping and breaching. Data about conditions upstream of a breach along with those downstream permit forecasters and emergency responders to infer the properties of the breach, leading to better decisions about an appropriate response.

A number of other tangible benefits are attributable to stream gauge systems, including:

- planning, designing, operating, and maintaining the nation's multipurpose water management systems
- issuing flood warnings to protect lives and reduce property damage
- designing highways and bridges
- mapping floodplains
- monitoring environmental conditions and protecting aquatic habitats
- protecting water quality and regulating pollutant discharges
- managing water rights and trans-boundary water issues
- education and research
- recreational uses.

These direct and indirect, tangible and intangible benefits can easily outweigh the costs of providing and maintaining such a network. Box 7.10 presents examples of cost savings derived as a result of a gauge network.

The value of stream flow records increase over time. Stream gauges with a long period of record are particularly valuable as they form a baseline for information about future changes. Knowledge of flood volumes, peak flows and corresponding water levels, and the timing of the runoff from catchments are basic requirements for planning, designing, and operating cost effective flood management projects.

Box 7.10 ***Benefits of a stream gauging network***

The questions posed for stream data are broad and difficult to answer as the value of the gauge network is not intrinsic. Instead it is a value that is accrued when the network is integrated with appropriate analyses and actions. The benefits attributable to collecting stream flow data cannot be separated from the benefit of analysing and using the data for better decision making. The absence of data will preclude success, even if it cannot be claimed that success is only due to the availability of stream gauge data. However, the value of a stream gauge network can be inferred on the basis of the economic benefit due to prudent flood management at both the local and national level. For example:

- **Folsom Dam, California, USA:** upgrade costs including increasing the height of the dam, expanding the outlet capacity and constructing an auxiliary spillway could have been avoided if a long record of flows was available. The potential savings is equivalent to $63m annually
- **Mecklenburg County, North Carolina, USA:** increased certainty in floodplain mapping for land use regulations could help prevent $330m in potential damages. If that cost is spread over 50 years, with a discount rate of six per cent, the benefit is approximately $20m annually
- **Central Valley, California, USA:** stream gauges with a long record of river stages and flows enabled efficient and economic design of the new flood defence system.

The cost saving for the first two cases alone represents a significant portion of the $114m annual cost of operating the US stream gauge network.

Extrapolating from these examples to a national scale, the USGS found that accurate design of levee improvements, using a long record of flows, can save potentially about $4.4m/km. If just 32 km of levees are repaired nationally, the saving is equivalent to $140m. The potential cost saving for the 16 000+ km of federal project levees in the US exceeds the cost of operating the stream gauge network.

7.3.3.1 Flow measurement and characterisation

Discharge Q (m^3/s) of a river is the volume of water that passes through a cross-section of the river in a given unit of time. Discharge can be constant or it may vary by location and/or time. Discharge that does not change with location or time is the simplest type to analyse using principles of continuity of mass and momentum. Table 7.22 shows different characteristics of discharge and the associated designation.

Table 7.22 Discharge characteristics

Discharge	Designation	Observer sees
Not changing in time	Steady	Constant water level at a single location
Not changing in time or space	Uniform and steady	Constant water level at any location along stream
Changing by location	Non-uniform or spatially varied	Water level changes with location, sometimes rapidly
Changing in time	Unsteady	Water level changes over time at a single location
Changing in time and space	Unsteady	Water level changes over time at multiple locations

River discharges may vary considerably with time. This variability is determined by climate and hydrologic factors.

For the simplest case where discharge does not change in time or space, Equation 7.5 can be written:

$$Q = V_1 \cdot A_{c1} = V_2 \cdot A_{c2} = V_3 \cdot A_{c3} = \cdots = V_4 \cdot A_{c4} \tag{7.5}$$

where:

Q = volumetric flow rate (m^3/s)

V = average flow velocity (m/s)

A_c = cross-sectional flow area (m^2)

The subscripts on V and A_c indicate different cross-section locations along the river.

During flood events in unregulated rivers flow is rarely steady or uniform due to the contribution of runoff along the watercourse and varying floodplain characteristics.

Where river discharge is not measured directly it can be derived. Typically, water level readings, geometry, and velocity measurements are used to calculate river discharge. Several methods of obtaining discharges exist (Herschy, 1998, and Ackers *et al*, 1978) including:

- estimate velocity distribution in the cross-section (combined with measured cross-section area)
- point gauges with propeller or electromagnetic velocity meters
- line averages from ultrasonic transmission
- acoustic doppler current profiler (ADCP)
- artificial controls such as weirs or flumes constructed with a standard design
- electromagnetic gauging at an instrumented site
- sampling to estimate the dilution of a tracer injected into water
- direct volumetric measurements.

Some of these are discussed further in Section 7.9.5.

These methods provide an instantaneous discharge. Additional steps are required to develop a time-series of discharges from water level observations using either an automated recording gauge or manually read staff gauge (a staff gauge is a graduated vertical scale that is referenced to a known vertical datum). To interpret the data from continuous water level readings the relationship between water level and discharge, called a rating curve, should be known. An example is shown in Figure 7.21. Periodic flow measurements, using for example velocity meters, are initially used to define a rating curve and then to define shifts (seasonal, systematic and random) from the rating curve. The shifted rating curve is then used to derive the discharges from a particular river water level during intervals when no instantaneous measurements are available. Seasonal variations exist for a number of reasons including temperature effects on water viscosity, vegetation, different levels of discharge due to wet or dry seasons, and/or changes in the river bed due to seasonal flows.

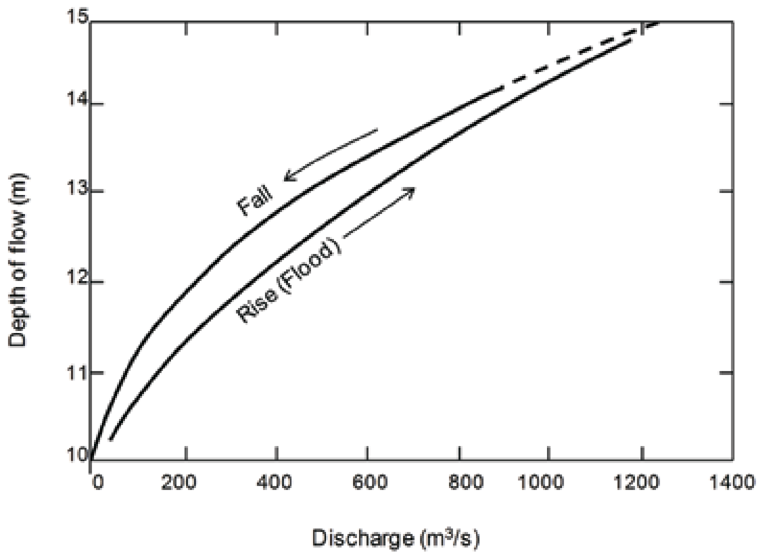


Figure 7.21 ***Example of a water level-discharge rating curve in flood showing rising (flood) and falling limbs of the hydrograph for the Sobat, tributary of the White Nile, Africa (Shahin, 1985, modified by Bravard and Petit, 2000)***

Rating curves can be used to determine flows, but only if the relationship between the depth of flow and the discharge is unique, ie if only one discharge corresponds to a specific depth of flow. This tends to occur only at gauging stations where the morphology of the river bed does not vary significantly or where the slope of the water surface is constant during the rise and fall of a flood. It is also the case for gauging stations at fixed geometry locations, such as weirs or flumes. In certain rivers, the values of discharge obtained during the rise and fall of a flood differ for the same water level. This results in a loop around the average rating curve. The loop effect is called hysteresis, with the discharge for a given water level being lower on the rise of the flood than on the fall (Figure 7.21). The hysteresis effect can be large, for example a spread of ± 1 m about the average rating curve is common in the Mississippi River. The maximum discharge does not necessarily correspond to the maximum depth of flow. Consideration of the hysteresis effect in setting levee loadings is generally included in assessing discharge uncertainty. Accuracy in measuring instantaneous discharge also introduces uncertainty with actual measurements typically within ± five per cent of true discharge values.

Discharge data include measured and/or synthesised flows along with frequency, velocity, duration, and depth information. Measured data at gauges are the preferred source for this category, but there are rarely enough measured data available.

7.3.3.2 Water level data

Water levels are generally the most accurate type of hydraulic data as they can be determined continuously with available equipment. The type of recording device and meteorological factors, such as waves or freezing, can affect measured water level values. Discharge data measured at specific times are used to develop rating curves, which allow the estimation of discharge from measured water levels.

Evaluating water levels begins by using historic gauge data, if available. Where historic data are not available, it is necessary to calculate water level using energy principles as described in Section 7.3.5. Even where historic data exists, it is necessary to estimate water levels that result from proposed project features, such as the construction or raising of the levee. Frequently, numerical models are employed to estimate the necessary hydraulic information required to complete a levee design (Section 7.3.8).

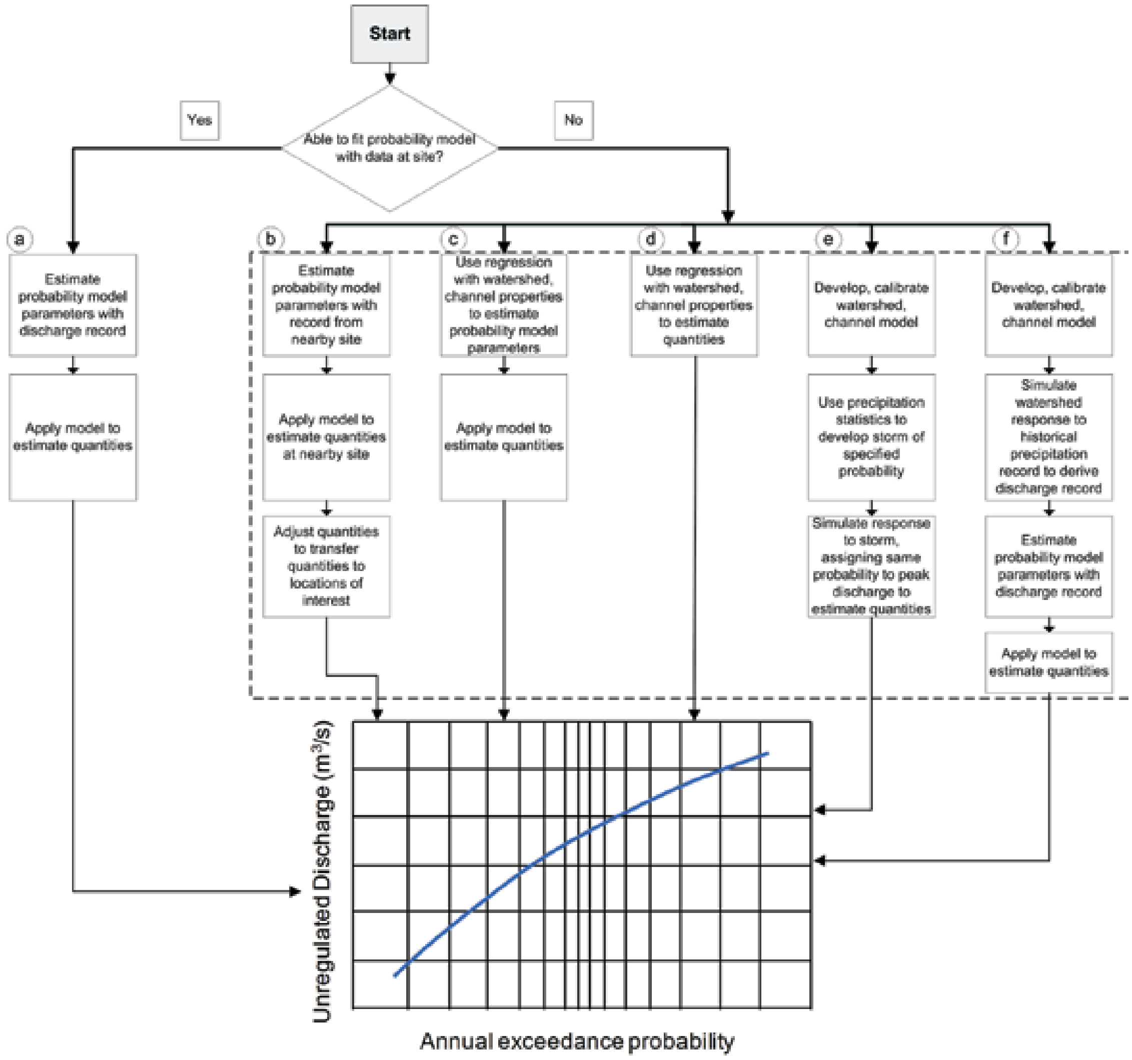
7.3.4 River flow and water level analysis

7.3.4.1 Hydrological analysis

Where stream gauges exist, a statistical analysis (Table 7.23) of historic flow data provides discharge-probability information for establishing required levee heights. If historic data are not available, several methods can be used to create the needed discharge-probability relationship (Figure 7.22).

Table 7.23 Statistical estimation methods for discharge-probability relationship

Statistic extreme-value distribution	Data analysed
• Gumbel distribution (or general extreme value type I) applied to annual maximums • Exponential law (coupled to a Poisson distribution) • Log-Pearson III applied to annual maximums • Weibull distribution.	• annual maximum discharge or peak (may be calendar year or hydrologic year as defined by character of hydrograph) • maximum discharge or peak for specific time of year, eg growing season • peak values over specific threshold.



Notes

a Observed stream discharge data available at point of interest.

b Observed stream discharge data at nearby point of interest.

c Deriving analytical model parameters using regional regression analysis.

d Discharge-probability equations derived from regional regression analysis.

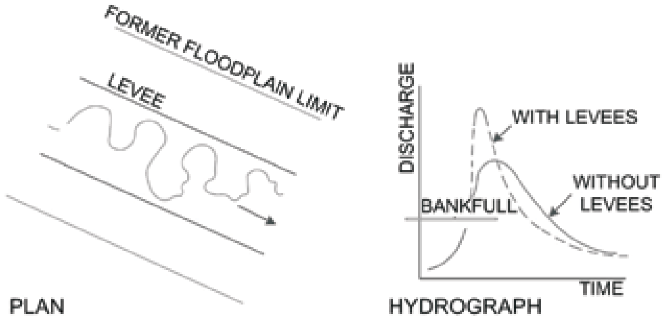
e Rainfall-runoff models and frequency based precipitation.

f Continuous simulation using hydrologic models and historic precipitation records.

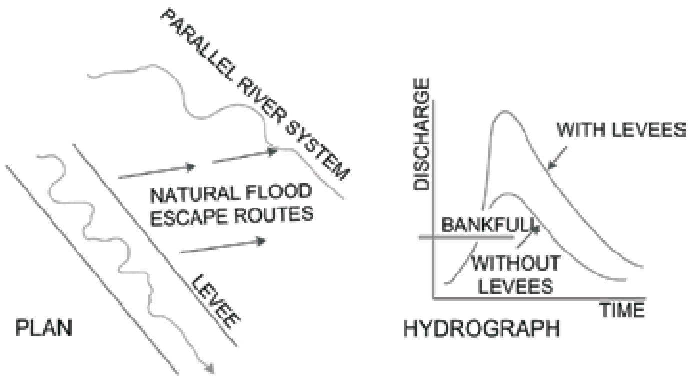
Figure 7.22 Methods for estimating discharge-probability function (from USACE, 1996)

Using historic data (see Sections 7.3.2 and 7.3.3) to predict current and future flood peak magnitudes assumes that the historic data is representative of the present and future conditions at the project site, also known as gauge stationarity. This assumption may not be appropriate if a project site has experienced a shift in climate or runoff patterns due to natural or human influences. Flood frequency analysis is performed through a statistical approach (Table 7.23), which ranks historic discharge data and determines a frequency proportional to the range of the data available. So, the length of data record available has a significant influence on certainty of estimated values. For instance a record of 20 years estimates approximately a five per cent chance event with fair certainty, and estimates the larger one per cent flood with significantly less certainty (Figure 7.22). Confidence limits derived from statistical analysis provide a means to consider certainty of estimated discharge-probabilities. Details of statistical distributions and methods can be found in guidance by, for example, Potter and Colman (2003).

If there are reservoirs, diversions, or other features within the catchment that significantly alter the magnitude or timing of runoff, an alternative approach is needed to establish the discharge-probability function. Naturally occurring phenomena, such as ice dams, may also alter discharges and should be accounted for in developing the discharge probability function. In this case a technique should be used to adjust altered or regulated discharges to remove the effects of alteration or regulation, and to create a set of homogenous data compatible with the pre-alteration condition. This procedure yields a set of pre-condition and presumably random discharges that can be analysed to derive an unregulated or pre-condition discharge-probability function. Adjustments for intended or unintended regulation of discharge can be found in various publications (USACE, 1989, 1994a and b, and 1995), but generally requires the use of simulation models. The presence or addition of a levee in floodplain of a river may also impact on discharges as depicted in Figure 7.23.



Effect of reducing floodplain storage and conveyance (flood volume unchanged)



Effect of blocking escape routes (flood volume increased)

Figure 7.23 ***Effects of levees on flood hydrographs (courtesy USACE)***

7.3.4.2 Bankfull discharge

The bankfull discharge (Q) is the most significant discharge that should be used in the analysis of the river regime (Section 7.3.1), as it is generally close to the dominant discharge for bed load transport and shaping of the stream geometry. Bankfull discharge is regarded as having a return period of one to two years, and can be estimated with a simple empirical relationship between the discharge and the drainage area (Equation 7.6). To develop this regional relationship, several bankfull discharges should be estimated in a region to determine values for coefficients 'a' and 'b' in Equation 7.6 (Table 7.24 gives some typical values for the coefficients). In order for the relationship to be reasonably accurate for extrapolation to similar streams, the streams used for calibration should be similar in climate (storm patterns), topography (mountainous versus plains) and sediment transport (the systems transport relatively the same type and amount of sediment).

$$Q = aA^b \tag{7.6}$$

where:
A = catchment area (m^2)
a, b = dimensionless regional coefficients

Table 7.24 Calibrated values of the coefficients 'a' and 'b' (Bravard and Petit, 2000 and CIRIA; CUR; CETMEF, 2007)

a	*b*	Source
0.277	0.828	Nixon (1959)
1.705	0.774	Hey (1982), and Richards (1982)
0.209	0.791	Andrews (1980)
1.161	0.666	For locations in the UK: Derbyshire (Petts, 1977) Cheshire (Hooke, 1987) Pennine Chain (Carling, 1988)
0.087	1.044	Petit *et al* (1994)

7.3.4.3 Flood wave propagation

The primary assumptions of steady flow analysis are that:

- peak water level nearly coincides with peak flow
- peak flow can be accurately estimated at all points in the river network
- peak water levels occur simultaneously over a reach of channel.

In reality, peak water level does not coincide with peak flow for moderately sloped river channels (S_0 <1%) or for highly transient flows, in the case of a sudden inundation of water from, for example, a breach. The phenomenon where several possible values of discharge may exist for a single water level cause a hysteresis effect that results from changes in the energy slope producing a peak discharge preceding peak water level.

Caution

It is not recommended to assume that peak water level occurs simultaneously at two or more cross-sections along a reach of river. Such an assumption is imprecise since all flow is unsteady and flood waves advance in the downstream direction. The wave-like character of a flood wave results in downstream hydrographs lagging those upstream. Unless significant tributary inflow exists or the channel has no floodplain (eg a canyon) a decrease in the peak value of the flood wave can be expected with distance downstream.

The passage of a flood wave is characterised by a gradual rise and fall of the water surface over a time of a few days to several weeks. Seddon (1900) showed that propagation velocity of a flood wave, also called wave celerity, in a wide rectangular channel can be expressed as:

$$c = 5V/2 \tag{7.7}$$

where:
c = flood wave propagation velocity or celerity (m/s)
V = average flow velocity in the river (m/s)

Corbett (1945) and Wilson (1990) provide a lower estimate for c in natural rivers as:

$$c = 1.3V \tag{7.8}$$

Tang *et al* (2001) provide a method for estimating flood wave celerity at reach scale using typical cross-section geometry: the wave celerity depends strongly on river discharge in a nonlinear fashion, especially near the bankfull capacity of the river channel.

7.3.4.4 Discharges/abstractions from nearby sources into the river (eg outfalls, intakes)

In most river systems the amount of water varies throughout a catchment based on sources/tributaries/outfalls that add water to flows or sinks/diversions/extractions that take water from a system. Estimating the changing amount of water in a river system is often referred to as 'water accounting'. In its more realistic form, complex rules exist in a catchment that also account for the timing of water movements, especially in a river that has large irrigation or other water supply requirements.

7.3.5 Basic river energy and flow states

The energy grade line, H (m), for steady and uniform flow can be expressed in terms of the bed elevation, the flow depth, and the velocity by Equation 7.9:

$$H = z + \frac{h_p}{\cos(i_b)} + \frac{\alpha V^2}{2g} \tag{7.9}$$

where:

i_b = angle of the bed (slope)
z = level of the riverbed compared with the reference level (m)
g = gravity acceleration (m/s^2)
h_p = water depth perpendicular to the river bottom (m)
V = mean flow velocity (m/s)
α = Coriolis (energy) coefficient (-)

Equation 7.9 is the basic energy equation for river hydraulics. The Coriolis coefficient, α, accounts for a non-homogeneous velocity distribution within the cross-section. Carlier (1972) states that α is often assumed equal to 1.0, but it can reach values of 1.35 in natural river channels (Sellin, 1969) or even be in excess of two for compound channel sections with connected floodplains (Henderson, 1966).

Where flow is gradually varied, there is a variation in energy between two cross-sections that are separated by a distance, L (m), along the channel. This introduces an energy loss, h_f (m), into Equation 7.10. Writing the energy between two cross-sections A and B as in Figure 7.24 yields:

$$H_A = H_B + h_f \tag{7.10}$$

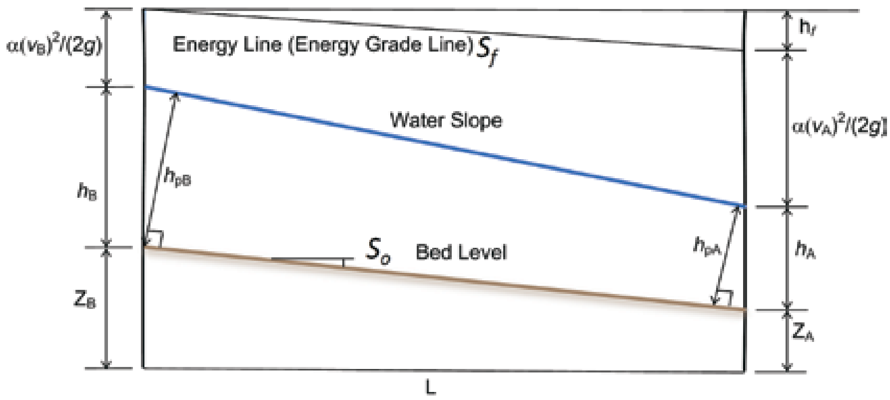


Figure 7.24 Energy in two cross-sections, A and B, separated by a distance of length L in a gradually varied flow (after Bravard and Petit, 2000)

The ratio of h_f/L stands for the slope of the energy line. It reflects the transfer of kinetic and potential energy in the main flow into other forms of energy not accounted for in Equation 7.9. The transfer of energy is caused by:

- internal viscous friction of the flow
- accelerations and decelerations of the current, which changes the turbulent energy
- frictional drag on the banks and the bottom of the bed
- transport of sediment.

A real river system contains confluences and bifurcations that complicate the movement of water and sediment, and so determination of the energy line. At a confluence, backwater effects may occur upstream on both rivers, which may cause loss of equilibrium at the confluence. At a bifurcation, such as bridges, weirs, outfalls and intakes, the local geometry determines the local flow patterns and so may also cause loss of equilibrium conditions.

7.3.5.1 Froude number and assessment of flow states

The Froude number, **Fr**, is a dimensionless ratio between inertial and gravitational forces. It describes different flow states in open channel flow. The flow state indicates a level of energy within the flow and describes its general behaviour. Both are significant factors to levee evaluation and design. In options appraisal or actual design of a levee, it is necessary to identify locations where different flow states may exist. In particular, locations where features exist near critical or supercritical flow conditions may experience rapid changes in water levels. These changes may be the result of natural features or may be caused by the introduction of manmade structures such as bridges, roadway embankments, river training structures or levees.

Equation 7.11 shows the Froude number ratio, **Fr**. The denominator represents the speed of a shallow surface wave relative to the speed of the water, $c=\sqrt{gh}$, which is known as wave celerity.

$$\mathrm{Fr} = \frac{V}{\sqrt{gh}} \tag{7.11}$$

where:

V = the average flow velocity (m/s)
g = gravitational acceleration (m/s^2)
h = flow depth (m)

In irregularly shaped cross-sections flow depth may be represented by the hydraulic depth, h_D (m), which is the flow cross-section area, A_c(m^2), divided by the top width, B (m), of the flow ($h_D = A_c$/B).

Using the Froude number, flow state can be classified into three categories as:

1. **Subcritical (Fr <1):** for a subcritical state, flow is controlled from a downstream point and any disturbance is transmitted upstream. This is the most common state for natural rivers. This condition leads to backwater effects. Subcritical flow is in a lower energy state than critical or supercritical. The normal depth (depth that results under uniform flow in a channel of given slope, geometry and roughness) is greater than the depth determined for critical flow. Normal depth, y_n, is the water depth that occurs when the channel bed slope and water surface slope are parallel.
2. **Critical (Fr = 1):** at a critical state the celerity equals the speed of the water and any disturbance to the surface will be stationary. This is an unstable energy state and standing waves may develop. For levees, critical flow exists at spillway crests or when overtopping occurs. It may also occur at restrictions or transitions such as the entrance to or exit from a culvert or where a steep bed slope is encountered.
3. **Supercritical (Fr > 1):** for a supercritical state, flow is controlled upstream (eg at a weir, spillway crest, or overflowing levee crest) and disturbances are transmitted downstream – disturbances downstream of a point cannot be translated upstream. Supercritical flow is in a higher energy state than critical or subcritical. The normal depth is less than the depth determined for critical flow.

Rapid changes in water levels and turbulence occur when flow transitions from supercritical flow, through critical flow to subcritical flow. This rapid change is known as a hydraulic jump (Section 8.2.2.3). The amount of energy dissipation and degree of turbulence at a hydraulic jump can cause significant erosion. Special attention should be applied to locations where a hydraulic jump may occur at or near a levee.

An important consideration of levee analysis or design is the water surface profile along a stream. The profile can be obtained by evaluating the energy relationship. Without derivation (Henderson, 1966) the equation relating energy between two locations can be written as:

$$\frac{dE}{dx} = S_0 - S_f \qquad (7.12)$$

Where E is the total energy, x is the stream wise distance between points, S_0 is the bed slope and S_f is the friction slope. Combining this with the change in energy with respect to flow depth yields:

$$\frac{dE}{dy} = 1 - \mathrm{Fr}^2 \qquad (7.13)$$

Where Fr is the Froude number, as described previously. Combining equation 7.12 and 7.13 yields:

$$\frac{dy}{dx}(1 - \mathrm{Fr}^2) = S_0 - S_f \qquad (7.14)$$

The resistance equation, in the form of either Equation 7.12 or 7.14, is a differential equation that cannot generally be solved explicitly. Manual methods including numerical models are available to aid in obtaining a solution. These are covered in Section 7.3.8.

An important consideration for levees is the shape of the longitudinal water surface profile along its length. Before starting with detailed numerical integration of Equation 7.14, it is desirable to develop a general idea of the shape that may occur. This will aid in identifying potential problem areas where the levee alignment or height may need to be adjusted. For this exercise it is useful to rearrange Equation 7.14 as:

$$\frac{dy}{dx} = \frac{(S_0 - S_f)}{(1 - \mathrm{Fr}^2)} \qquad (7.15)$$

For a given flow, Q, friction slope and Froude number are functions of flow depth, y. For a semi-quantitative picture of how y varies with x it is only necessary to consider the relative sign (positive or negative) of the numerator and denominator of Equation 7.15 to obtain an understanding of the behaviour of the water surface profile. Figures 7.25a and b show longitudinal water surface profiles for two cases: a mild slope designated as M, and a steep slope designated as S. Similar profiles exist for adverse, horizontal and critical channel slopes (Henderson, 1966).

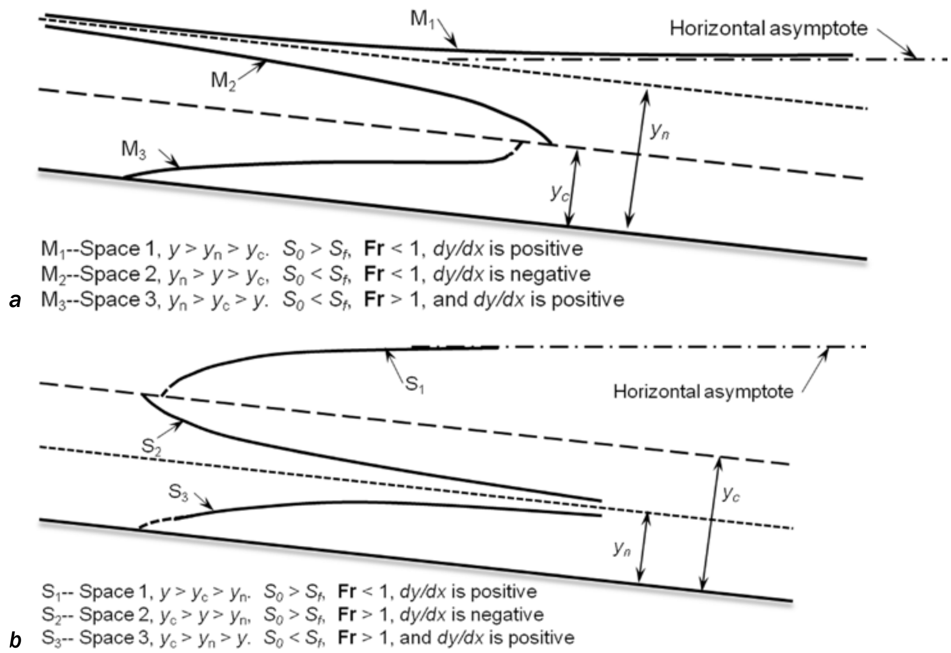


Figure 7.25 Longitudinal profiles on a mild slope (a) and on a steep slope (b)

Longitudinal profile analysis allows the prediction of the general shape of the flow profile that may occur for a given channel configuration and levee alignment. It also provides an insight into locations where hydraulic controls exist or where rapid changes in water level might be expected to occur. Levee alignments may then be adjusted to remove/reduce any effects on the profile or the crest elevations can be adjusted to accommodate anticipated local conditions.

7.3.6 Influence of bed roughness and river geometry on flow

Channel geometry is the shape of the flow area perpendicular to the direction of flow and is required for any hydraulic investigation. It is defined by the geometry of the section and the flow depth as shown in Figure 7.26. Channel geometry changes with time in response to varying influences such as hydrologic events, riparian vegetation, and anthropogenic changes. This is particularly true where river channels have mobile beds.

The longitudinal profile is the channel elevation versus channel length and describes the channel slope. Other identified parameters in Figure 7.26 are the water depth (h), top width (B), water area (A_c) and wetted perimeter (P). Hydraulic radius (R) is defined as the area of the channel divided by the wetted perimeter. In certain situations regime theory has established a relationship between hydraulic radius and discharge (for example Box 7.11)

Catchment topography establishes the slope of the land surface and heavily influences the channel slope. Elevations within the catchment and channel determine the amount of potential energy for a catchment. This potential energy converts to other forms of energy, principally kinetic energy, as runoff begins and flows down slope toward the basin outlet.

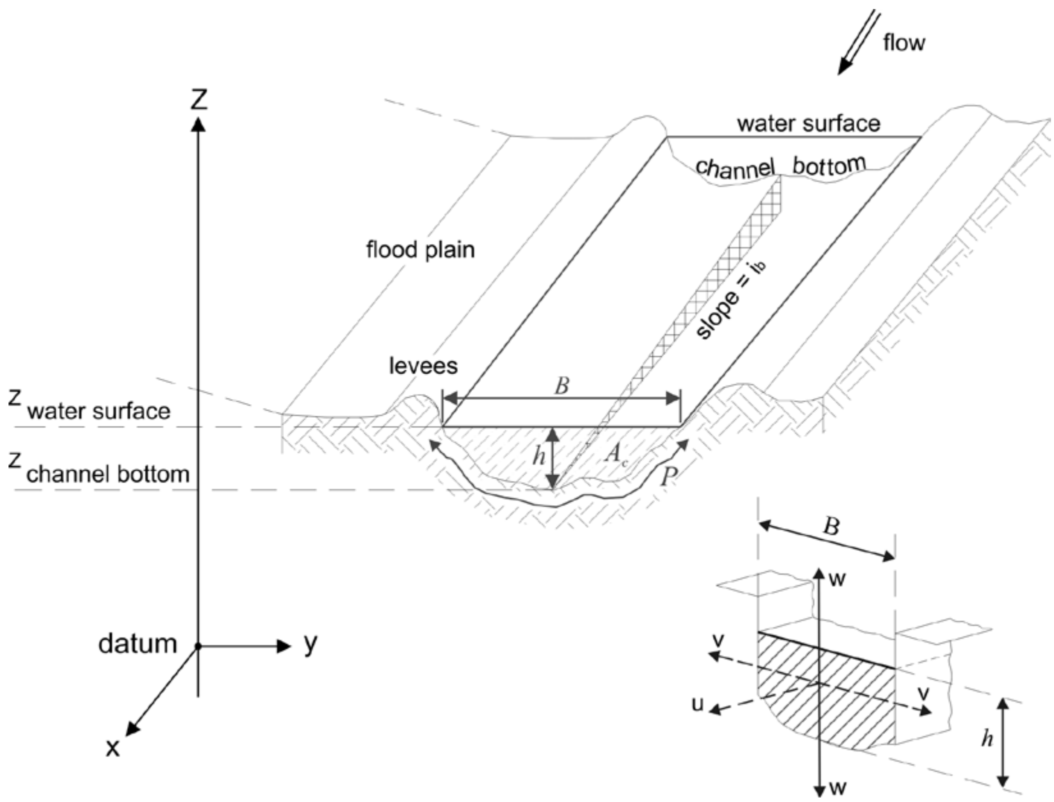


Figure 7.26 River geometry (courtesy CIRIA; CUR; CETMEF, 2007)

Box 7.11 *Lacey's simplified regime relationship between discharge and water level*

Lacey's regime equation relating discharge to typical water depth in terms of hydraulic radius (CIRIA; CUR; CETMEF, 2007) can be applied to many alluvial river channels and manmade canals with low sediment transport, ie where sediment concentrations range between 100 to 2000 mg/l and sediment grain sizes range between 0.1 mm to 0.5 mm. The regime equation includes Lacey's silt factor, *f*, which is estimated as being 0.3 to 1.0 for silty substrate:

$$R=0.47Q^{1/3} / f^{1/3} \tag{7.16}$$

Where:

R = hydraulic radius (m)
Q = discharge (m^3/s)
f = Lacey's silt factor

7.3.6.1 Effect of bed roughness on flow

The effect of bed roughness on flow can be assessed using either the Manning-Strickler or Chézy formulae.

Manning-Strickler formula

The cross-sectional averaged velocity, v (m/s), can be calculated using the Manning-Strickler formula given in Equation 7.17.

$$v = \frac{R^{2/3} i^{1/2}}{n} \tag{7.17}$$

where:

R = hydraulic radius (m), the ratio of the water area to the wetted perimeter
i = slope of the energy line or water surface slope
n = Manning's roughness coefficient

Manning's n, takes into account the roughness of the banks and bed. The roughness results in frictional head losses, which become more significant as roughness increases. Roughness depends mainly on the nature of the materials on the river bed and the vegetation along the banks.

Cowan (1956) presents a procedure for calculating Manning's roughness coefficient, n, based on a composite of various factors using Equation 7.18:

$$n = (n_0 + n_1 + n_2 + n_3 + n_5) \cdot m_5 \tag{7.18}$$

where:

n_0 = factor that depends on material of the channel, and determined by Strickler's formula (n_0=0.048$D_{50}^{1/6}$, or with n_0=0.038$D_{90}^{1/6}$ (Simons and Senturk, 1992), where D_{50} and D_{90} are grain size not exceeded by 50 or 90 per cent of the mass of the bed sediment. The relationship between n_0 and D_{90} is approximately constant for a range of relative depths given by 7 $<h/D_{90}<$150
n_1 = factor that depends on the degree of surface irregularity
n_2 = factor that depends on the variations in the cross-section form
n_3 = factor that depends on the effects of obstructions (bridges etc)
n_4 = factor that depends on the vegetation that modifies the flow conditions
m_5 = coefficient that indicates the degree of sinuosity of the channel.

Table 7.25 gives typical values of coefficients used in Equation 7.18.

Table 7.25 Values of Manning's coefficient proposed by US Soil Conservation Service (Chow, 1959)

Channel conditions		Components of *n*	
Material involved	Earth	n_0	0.020
	Rock cut		0.025
	Fine gravel		0.024
	Coarse gravel		0.028
Degree of irregularity	Smooth	n_1	0.000
	Minor		0.005
	Moderate		0.010
	Severe		0.020
Variations of channel cross-section	Gradual	n_2	0.000
	Alternating occasionally		0.005
	Alternating frequently		0.010 to 0.015
Relative effect of obstructions	Negligible	n_3	0.000
	Minor		0.010 to 0.015
	Appreciable		0.020 to 0.030
	Severe		0.040 to 0.060
Vegetation	Low	n_4	0.005 to 0.010
	Medium		0.010 to 0.025
	High		0.025 to 0.050
	Very high		0.050 to 0.100
Degree of meandering	Minor	m_5	1.000
	Appreciable		1.150
	Severe		1.300

James (1994) proposed a linear expression for m_5 (Equation 7.19) that depends upon the sinuosity I_s (see Section 7.3.1).

$$m_5 = \begin{cases} 1.0 & \text{for } I_s = 1.0 \\ 0.57 + 0.43 I_s & \text{for } 1.0 < I_s \leq 1.7 \\ 1.3 & \text{for } I_s > 1.7 \end{cases} \tag{7.19}$$

The Manning-Strickler formula (Equation 7.17) can be applied for the average value of n when discharges corresponding to the observed water surface profiles are known. If data show that n varies with water level, n should be determined from a curve of n versus water level or from the observed profile that most closely approaches the water level of the desired profile. If no records are available then values of n computed for similar stream conditions or values obtained from experimental data can be used as guidance to select appropriate values of n. Tables and photographs as provided by Chow (1959) may be used as a guide for selecting appropriate values of n. When discharge measurements are made to determine values of n, it is desirable to also obtain water surface slopes. Such data can be used to derive more reliable values of n than can be determined from high watermarks alone.

The Strickler coefficient, K, can be considered the inverse of the Manning coefficient ($K = 1/n$). Expressing Equation 7.17 as a function of K, other tables can be determined to characterise the riverbed roughness and to calculate hydrodynamic flow characteristics (Degoutte, 2001).

Recent work carried out in the UK on the estimation of flow conveyance has led to the development of a new approach for roughness characterisation (Knight *et al*, 2009). It is based on the concept of 'unit roughness', which describes identifiable segments of boundary friction. The unit roughness comprises three components:

- surface material (eg soil, rock)
- vegetation (in channel and on the floodplain)
- irregularities (larger elements such as tree roots and urban trash).

It is calculated as follows, to reflect the higher contribution of the largest component roughness:

$$n_l = (n^2_{veg} + n^2_{sur} + n^2_{irr})^{0.5} \tag{7.20}$$

This approach is encapsulated in a roughness adviser (Fisher and Dawson, 2003), which provides values for use and also advice on roughness in the absence of any survey data, but is specific for UK sites.

Chézy formula

The cross-sectional averaged flow velocity, v (m/s), can also be calculated from the Chézy formula as Equation 7.21:

$$v = C\sqrt{R \cdot i} \tag{7.21}$$

where:
R and i are defined as for the Manning-Strickler formula
C = bed friction Chézy coefficient ($m^{1/2}/s$)

The coefficient, C, is a measure of the riverbed and riverbank roughness and has been defined by Bazin (1897) as expressed by Equation 7.22:

$$C = \frac{87}{1+\frac{\gamma}{\sqrt{R}}} \tag{7.22}$$

where:
R is as per the Manning-Strickler formula
γ = parameter representative of the bed roughness ($m^{1/2}$)

Values of γ vary from 0.06 for smooth bed to 1.75 for grassed bed and cobbles. Further development of the method can be found in Christensen (1972) and Prandtl (1925).

There is a relationship between the Manning-Strickler and Chézy formulae through a description of C in terms of R and n.

Box 7.12 presents a method of determining Chézy coefficient based on grain and bed form roughness.

7.3.6.2 Effect of obstructions on flow

Structures in the flow such as bridge piers, abutments, caissons, cofferdams, weirs, bed (bedrock or natural monolithic material that restricts the ability of the channel to adjust its boundary), gate structures or training works can generate marked changes in the:

- shape of the vertical velocity profile
- local magnitude of the flow velocity
- water level
- level of turbulence of the flow.

These local changes in water level will be propagated upstream and downstream according to the water surface profile as discussed in Sections 7.3.5 and 7.3.9 and shown in Figure 7.25.

Box 7.12 ***Grain and bed form roughness – Chézy coefficient (from CIRIA; CUR; CETMEF, 2007)***

This box deals with methods based on bedform characteristics, particularly those developed by Van Rijn (1989). The hydraulic roughness consists of two parts:

- grain roughness, k_{sg} (m)
- bedform roughness, $k_{s\Delta}$ (m).

The grain roughness, k_{sg}, can be approximated by Equation 7.23 (Van Rijn, 1982).

$$k_{sg}=3D_{90} \tag{7.23}$$

For engineering purposes, the scatter of k_{sg} in the case of graded sediment can be described by k_{sg}/D_{90} = 1 to 3. Somewhat arbitrarily assuming that D_{90}/D_{50} = 2, which implies k_{sg}/D_{50} = 4 (actual estimates for D_{90}/D_{50} are given in CIRIA; CUR; CETMEF, 2007).

For **uniform sediment** the range of grain roughness is given by k_{sg}/D_{50} = 1 to 2. Despite scatter, on average the best results seem to be obtained using $k_s = D_{90} \cong 2\ D_{50}$ for fine sediments and $k_s = 2\ D_{90} \cong 4\ D_{50}$ for coarse material, assuming no bedform roughness.

The **bedform roughness**, $k_{s\Delta}$, should be calculated using the roughness predictors given in Van Rijn, 1989. The empirical relation (see Equation 7.24) is based on the dimensions of the dune bedforms that are present in the river bed.

$$k_{s\Delta}=1.1D_b\left(1-\exp(-25D_b/L_b)\right) \tag{7.24}$$

where D_b = average bedform height (m) and L_b = average bedform length (m).

Values for D_b and L_b depend on the flow regime and should be determined from echo-soundings of the river bed. The overall hydraulic roughness is given by Equation 7.25.

$$k_s=k_{sg}+k_{s\Delta} \tag{7.25}$$

In general, the contribution of k_{sg} to the hydraulic roughness is small compared with the contribution of $k_{s\Delta}$ Substituting k_s according to the above formulae in the equation for the Chézy coefficient should generally result in values in the range of: C = 25 to 60 $m^{1/2}/s$.

Note that for a silty bed (eg in estuaries), C may be up to 80 to 90 $m^{1/2}/s$.

Other methods of determining hydraulic roughness exist (for example EDF *et al*, 1992).

The presence of levees within the adjacent floodplain may also generate significant change in channel currents. Box 7.13 presents an example where the levee can have an influence on the river channel.

Vegetation in the river bed and banks can significantly affect the available cross-sectional shape and area, and potentially introduce major uncertainty in estimations of conveyance and water levels. In temperate climates vegetation cover, particularly above water level can vary in nature and density as a result of seasonal changes and maintenance operations. Characterising vegetation cover at an existing site, or the identification of local species in the case of a new levee design, is an important component of the site data acquisition (see Section 7.9.2 for surface cover survey). In hydraulics terms the impact of vegetation on river geometry is usually taken into account in the estimation of the hydraulic roughness of the river cross-section (see Section 7.3.6.1 on Manning's roughness coefficient).

Box 7.13 *Effect of a levee on a river channel*

Placement of a levee within a river floodplain results in a horizontal constriction, eg a reduction in width. This influences water depth and water surface gradient. Using the Chézy method (see also Section 7.3.6.1) where $B >> h$, the changes that may be expected can be assessed (note that subscript 0 denotes the initial, unaltered situation and subscript 1 denotes the altered state).

Equations for water discharge:

Continuity: $Q_0 = Q_1$ (7.26)

Motion: $\boxed{Q = BC\sqrt{h^3 S}}$ (Chézy equation) (7.27)

Equations for sediment transport:

Continuity: $S_0 = S_1$ (7.28)

Motion: $S = Bav^b$ (7.29)

where:

Q = discharge (m³/s)
v = average flow velocity (m/s)
B = top width (m)
C = Chézy coefficient ($m^{1/2}/s$)
h = flow depth (m)
S = slope (m/m)
a,b = coefficients which depend on the method used.

Given:

- $Q_0 = Q_1$ because there is no allowance for storage to reduce flow, so assumes no additional inflow for levee condition
- $B_0 > B_1$ due to levee encroachment on floodplain
- $S_0 < S_1$ because channel is constricted and $Q_0=Q_1$
- $h_0 < h_1$ expected due to levee encroachment on floodplain
- $A_{C0} > A_{C1}$ because channel cross-section area decreases due to levee

Assessment:

- velocity increases because S and *h* increase while *B* decreases and *Q* is constant
- sediment continuity is disrupted due to increase in velocity
- sediment motion increases due to higher velocity *v*, which will increase water depth *h*, ie lower the bed elevation
- lowering the bed increases the bank height and may cause bank instability.

7.3.7 Flow velocity distributions

Flow in a river is generally not uniform but varies in both the vertical and horizontal direction. Consequently, to evaluate shear stresses on the river bed or riverbank, it may be necessary to know more than just the depth-averaged velocity, v. The vertical and/or horizontal velocity distributions may have to be determined. This section discusses non-uniform velocities and other design considerations, which require a more detailed assessment of the current profile. These include:

- vertical velocity profile
- transverse or horizontal velocity distributions
- bed roughness
- composite cross-sections (banks and channels).

The bed shear stress is introduced in Section 7.3.9 and is an important parameter for the first two items listed above, affecting the flow pattern as well as the bed response.

In rivers, the dominating factor for levee analysis and design is the river discharge associated with the water level and the current velocity for a flood peak. A current velocity also represents a direct loading parameter through its interaction with the levee surface. Because velocity varies with discharge and the highest expected flood level may not produce the highest velocity, calculations should be made for a range of discharges to determine the highest velocity that acts on, or against, the levee.

The interactions of this section with other sections within the handbook are summarised in Figure 7.27.

Modern techniques for analysing flow velocities for levees involve use of numeric models. These hydraulic models may also include provisions for adding sediment transport calculations.

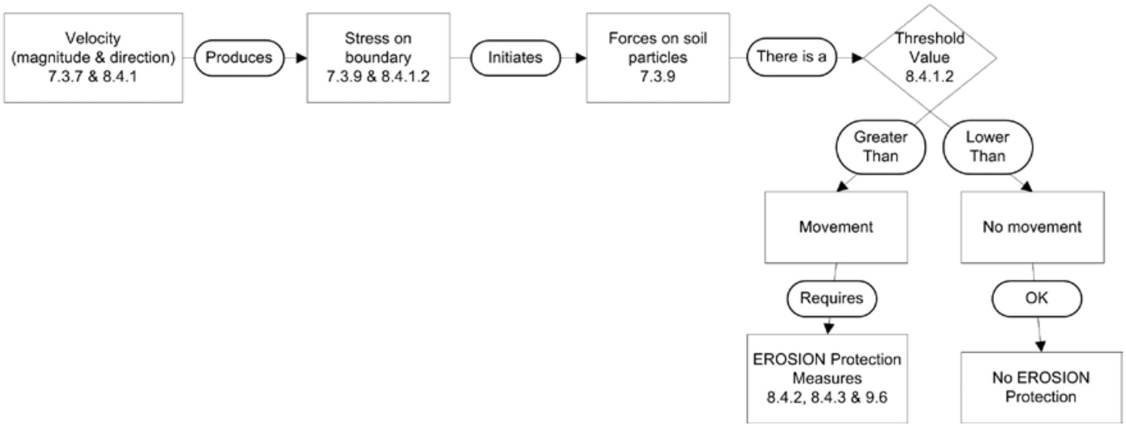


Figure 7.27 ***Relationship between sections in the handbook involving current and velocity***

7.3.7.1 Basic velocity distributions

The presence of a free surface and friction along the channel wall results in velocities that are not uniformly distributed in the channel section (Figures 7.28, 7.29, and 7.30). The maximum velocity is approximately 10 to 30 per cent higher than the cross-section averaged velocity ($v=Q/A_c$).

When the average depth, h (m) is small compared with the top width, B (m), for example where B/h is 20 or more, the cross-sectional averaged velocity can be approximated by Equation 7.30.

$$v = \frac{Q}{(B \cdot h)} \tag{7.30}$$

Where neither instantaneous discharge nor a rating curve is available, the depth-averaged velocity v in a river cross-section can be obtained for steady uniform flow by Manning-Strickler or Chézy formulae (Section 7.3.6.1).

Velocity distributions, both vertically and horizontally, can be computed if hydraulic conditions are known. For a hydraulically rough boundary $\frac{u_* \times D}{\nu} > 70$ the vertical flow distribution $u(z)$ is commonly used and determined with Equation 7.31.

$$u(z) = \frac{u_*}{\kappa} \ln\left(\frac{z}{z_0}\right) \tag{7.31}$$

Where:

D = grain diameter of the bed material or material size representative of k_s, the hydraulic roughness (m) (see Box 7.12)
u_* = shear stress velocity (m/s) (see Equations 7.46 and 7.47)
z_0 = reference level near the bed (m) – defined below
κ = von Karman's constant (κ = 0.4)

The reference level near the bed, z_0, is defined by $v(z = z_0) = 0$. For $u_* \cdot k_s/v > 70$, where k_s is the hydraulic roughness (m) and v the coefficient of kinematic viscosity (m/s), z_0 is defined by:

$$z_0 = 0.033k_s \tag{7.32}$$

Equation 7.31 implies that velocity is a maximum at the water surface so $v_{max} = u(h)$. The velocity $u(z)$ just equals v at $z = 0.37h$. In Figure 7.28, the velocity profile is shown in a non-dimensional logarithmic form.

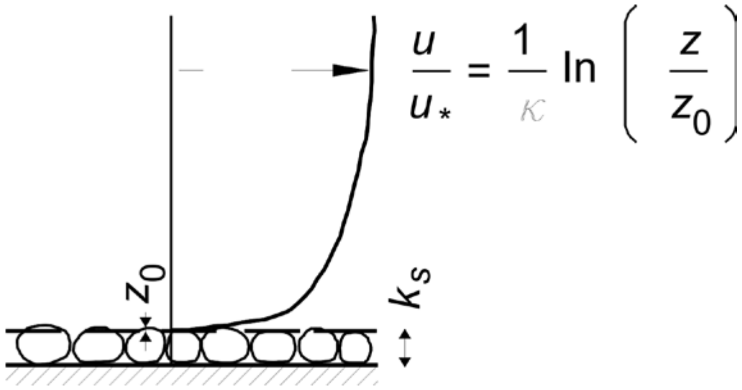


Figure 7.28 ***Vertical velocity profile – logarithmic (from CIRIA; CUR; CETMEF, 2007)***

For many engineering applications the velocity distribution can be approximated by a power function in z/h as in Equation 7.33.

$$u = u_{max} (z/h)^p \tag{7.33}$$

where:

u_{max} = maximum velocity at the surface

The exponent 'p' depends on the bed roughness and Reynolds number (Ackers, 1958). Typical values for 'p' are between 0.16 and 0.10.

Historically, more complicated cross-sections have been analysed using the Manning-Strickler method. Methods for analysing composite cross-sections divide the section into parts and each uses different coefficients to calculate mean velocity for the separate sub areas. The separate velocities may be combined to create a composite cross-section average or may be used individually to analyse boundary shear stress within that section. For example, near the bank in assessing armour requirements. Chow (1959), Henderson (1966), and James and Wark (1992) are sources for additional explanation of methods for analysing composite cross-sections.

7.3.7.2 Horizontal profile distribution

The horizontal velocity distribution results from the presence of banks on both sides of a channel. Figure 7.29 shows the velocity distributions in the transverse direction. The flow velocity near a bank in a straight channel, which is influenced by the shear stress acting along the banks, may be up to 25 per cent less than the velocity in the main axis of the channel. Conversely, where channels have a high degree of meandering, the velocity may be highest near the convex bank due to Coriolis effects and centrifugal action of the flow. The convex bank is the bank on the outer side of a meander bend while a concave bank is the bank along the inner side of a bend.

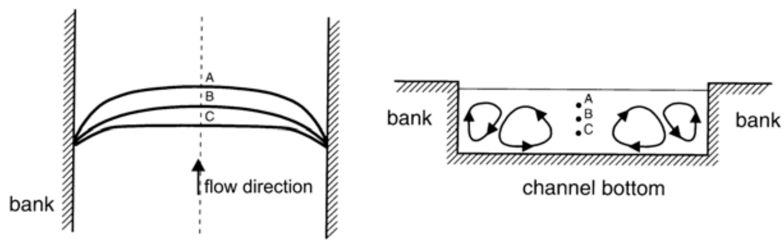


Figure 7.29 ***Horizontal velocity profile (from CIRIA; CUR; CETMEF, 2007)***

7.3.7.3 Cross-sectional distribution

The measured maximum velocity in ordinary channels usually occurs between 5 and 25 per cent of the flow depth below the surface. In natural channels the depth of the greatest velocity occurs deeper in the water closer to the banks. Figure 7.30 illustrates the general pattern of velocity distribution over various vertical and horizontal sections of a rectangular channel section and the curves of equal velocity in the cross-section.

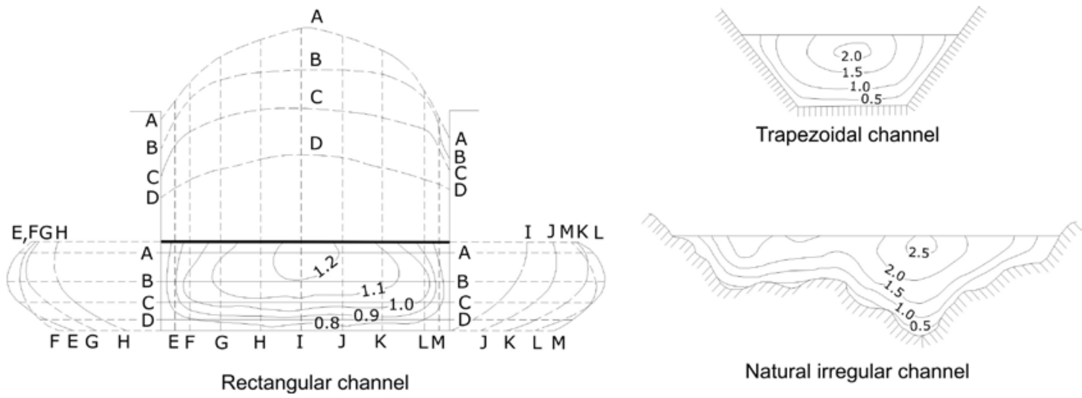


Figure 7.30 Velocity distributions in a rectangular channel, a trapezoidal and a natural irregular channel (from Chow, 1959 and CIRIA; CUR; CETMEF, 2007)

7.3.7.4 Local and secondary currents

Where changes in the geometry of the river, modifications in bed roughness, or structures are present the current distribution is 3D and appropriate models should be used. The velocity component in the transverse channel section is usually small and insignificant compared with the longitudinal velocity components. However, in natural rivers with irregular cross-section in channel bends, the flow velocity distribution differs from that in a straight channel. In curved channels, local currents can result in a spiral motion, which is an important phenomenon to consider in design. In bends, the curvature of the flow results in a transverse water surface slope, i_r, a secondary circulation develops and combines with the main flow into a spiral flow. In extreme cases this component may result in a super-elevation of water levels on the convex side of the channel, ie a higher water level exists in the outer bend when compared with the water level at the inner bend.

Interactions with structures contribute significantly to the local currents. Common examples are eddies between spur-dikes or where road crossings constrict flow in the floodplain.

Since the types of flow discussed here are highly dependent on local conditions and are of a complex nature, no general practical guidance can be given. Physical modelling is recommended along with 2D or 3D numerical modelling, which may be able to provide some insight into the expected flow pattern.

7.3.7.5 Turbulence

Turbulence may have a considerable local impact on the stability and movement of sediment and naturally occurring or placed armourstone. When added to the local time-average velocity, u (m/s), the random turbulent velocity component, u′ (m/s), causes an increase in the effective instantaneous velocity: u + u′. To assess the stability of sediment and stone, it is important to note that most stability formulae assume a normal turbulence. Where turbulence exceeds the normal level, a velocity correction should be applied. A common adjustment used to account for higher levels of turbulence uses the turbulence intensity, r.

Normal turbulence intensities can be characterised by $r \approx 0.1$ (10 per cent), which is found in uniform flow in laboratory flumes and rivers with a low flow regime and flat or rippled bed, excluding beds with relatively high sand dunes. Above a rough bed, values of $r \approx 0.15$ (15 per cent) apply. Turbulence levels in excess of these normal levels of 10 to 15 per cent are typically the result of hydraulic interaction between the flow and structures, including:

- flow separation (sudden widening of flow cross-section)
- vortex shedding (bridge piers, large stones)
- changes in the bed and/or slope roughness.

The application of the adjustment for turbulent effects results in an adjusted velocity of u(1 + r).

Highly turbulent flows can be found in a number of situations, as illustrated in Figure 7.31. Levee projects may encompass one or more of these situations and due consideration should be given to turbulent effects on currents near a levee.

Caution

Excessively high levels of turbulence can persist at considerable distances from a structure or hydraulic jump and should be considered in stability design.

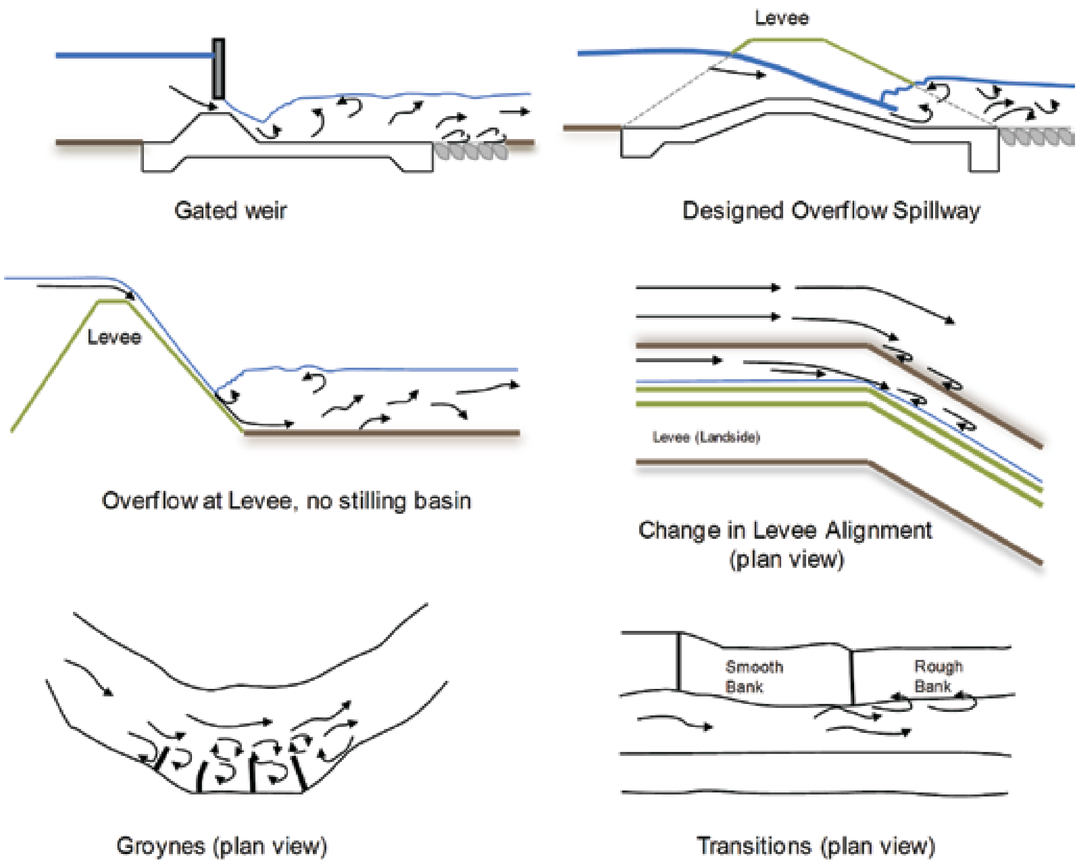


Figure 7.31 Examples of situations with high turbulence (from Escarameia, 1998)

7.3.8 Modelling of hydraulic processes

Analysis of available data provides only part of the picture needed to characterise levee projects. So, it is necessary to use various forms of models to assess project behaviour and the river response to that project. The type and complexity of the model depends on the size of the project, historic behaviour of the river, and any local conditions of special interest.

The primary purpose of any hydraulic model is to determine river water level information for sizing the levee. This includes developing water level-discharge relationships through the project reach. Similar to a water level-discharge rating curve derived from observed data for a gauging site (Section 7.3.2), the water level-discharge relationship derived from modelling may be used to describe conditions at any cross-section along the model reach and may incorporate hydrologic or other changes that result from project implementation.

Basic categories of hydraulic models include:

- analytical techniques including simplified empirical relationships or even rules-of-thumb developed from experience
- numerical (or mathematical) models, often these are computer based
- physical models – scale models are designed and operated by consideration of the laws of similitude. They are often only used to study complex behaviour and their coverage in the handbook is limited. The topic of physical models is covered extensively in Franco (1978), and Yalin (1971).

During an initial assessment, very simplified models may be used, including regional regression equations, empirical relationships or design charts. Use of these approaches is generally limited to assessing current site conditions. Modelling efforts become more involved as studies progress through options appraisal and on to the final project design.

Numerical models are essential tools to solve a set of mathematical equations for the variable(s) of interest. These equations represent the underlying physical processes including the energy and momentum equations for open channel flow. Numerical models may be used in evaluating various options by adjusting input parameters that reflect planned changes, thereby producing modified water level-discharge relationships. The modified water level-discharge relationships can then be compared to the baseline and future condition without project conditions. In this section, only a short outline is given.

Numerical (or mathematical) models can be made for simplified phenomena only because:

1. The understanding of the processes involved is still limited and needs to be expressed by mathematical equations.
2. The computational costs from both monetary and time perspectives should be acceptable.

The use of physical models is also limited for the following reasons:

- the costs of model investigations limits the scale of investigation
- a sufficiently small model should be used with a timescale that allows for testing within the available study time
- the reduction in scale introduces scale effects.

A mathematic model is not subject to scale effects, but it can only reproduce the phenomena included within the mathematical equations. Numerical models can be used to solve the steady or unsteady flow equations.

Caution

Confidence in any model approach is a function of how well it reproduces observed real world behaviour. A fundamental part of achieving a useful model includes adjustment of model parameters to reproduce observed data. This is known as model calibration. Calibration can seek to reproduce observed flood peaks, continuous event hydrographs (unsteady flow models), or general trends.

Box 7.14 presents an example of a software model used to estimate several parameters related to rivers.

Box 7.14 The UK Conveyance Estimation System (CES)

The CES-AES software (HR Wallingford, 2009) estimates conveyance (water levels, rating curves etc), spatial velocities and boundary shear stresses at river sections and also undertakes simple reach-based backwater calculations. It also provides a comprehensive database of river roughness, integrating diverse information from over 700 references, including photographs (linked to the River Habitat Survey) and advice on vegetation cutting and re-growth. In addition it can calculate the rise in water level (above normal) on the upstream side of bridges and other structures or obstructions, where the effective width of the waterway is reduced.

7.3.8.1 Analytical methods

The best fit transform may be established analytically with a statistical fitting procedure using standard regression analysis. Given those statistical procedures a model form is selected and the parameters of that model are estimated. The model output is then compared with the sample of discharge and water level to optimise the fit of the model to the data through the adjustment of the parameters. The model may be in the form of a simple power relationship using two or three parameters, as in Equation 7.34, or it may involve multiple parameters (Equation 7.35) as suggested by Freeman *et al* (1996). This technique is not suited to evaluating changed conditions that would result from planned levee projects.

$$Q = \mathrm{a}Z^{\mathrm{b}} \tag{7.34}$$

$$Z = \mathrm{a} + \mathrm{b}Q^{1/2} + \mathrm{c}Q^{1/3} + \mathrm{d}Q^{1/4} + \mathrm{e}Q^{1/5} + \mathrm{f}Q^{1/6} \tag{7.35}$$

where:

Q = discharge (m^3/s)

Z = water level (m)

a, b, c, d, e, f = parameters for the model that are estimated from the dataset

7.3.8.2 Simplified manual methods

Before the advent of numerical models, methods for the manual solution of governing hydraulic equations were used to calculate water surface elevations at select locations. These computed water levels were used in establishing the design water level for levee design as well as for analysing project impacts. Many 1D numerical models in current use adopt the same methods for solving the energy equations as presented below.

While not difficult, manual calculations to balance energy are iterative and can be time consuming especially for irregular channel geometry. The principle advantages of using numerical models to solve the equations are that many more cross-sections can be used to describe complex channel geometry and a wide array of options can be evaluated in a relatively modest period of time. In lieu of numerical modelling, hand calculations can be conducted using an expeditious selection of representative cross-section locations that capture significant changes in channel geometry.

The first thing required in calculating water surface profiles is to identify the cross-section locations needed to capture channel geometry and changes in roughness. The goal is to place cross-sections at intervals that will divide a total reach into a series of sub-reaches, each of which is as uniform in geometry and roughness as practical. Specific recommendations for locating cross-sections can be found in USGS (1992). General guidance is that streams with a fairly uniform geometry and roughness require fewer sections that are spaced further apart, while streams with highly irregular geometry and roughness require a larger number of sections with spacing to create segments that capture the hydraulic effects resulting from the irregularities.

Water surface profiles are computed from one cross-section to the next by solving the energy equation (Equation 7.36) between adjacent cross-section locations (Figure 7.32). For subcritical flow, calculations start at the downstream most cross-section and then move upstream in a step-wise manner. Calculations are performed in the reverse order for the supercritical flow regime. Conditions at the starting cross-section should be known or estimated from available data. Estimated starting water surface elevations may be calculated assuming normal depth (using the Manning's equation) for the section if no other data are available. The energy equation between adjacent sections is written as:

$$z_2 + y_2 + \frac{\alpha_2 V_2^2}{2g} = z_1 + y_1 + \frac{\alpha_1 V_1^2}{2g} + h_f \tag{7.36}$$

where:

z = elevation of the main channel inverts (m)

y = depth of water at cross-section (m)

V = average velocity (m/s)

α = velocity weighting coefficient

g = gravitational acceleration (m/s²)

h_f = energy head loss between locations 1 and 2 (m)

Subscript 1 indicates the location of a known condition and subscript 2 is the location to be calculated, respectively. Location 1, the known location, may be downstream or upstream of location 2 depending on whether flow is subcritical or supercritical, respectively.

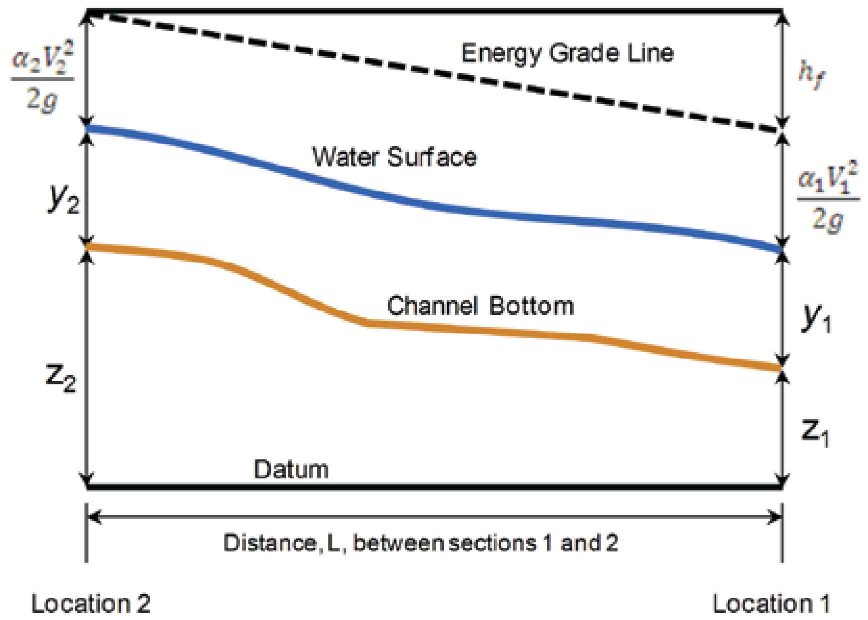


Figure 7.32 Representation of terms in Equation 7.36 for subcritical flow

The energy loss (h_f) between two cross-sections is comprised of friction losses and contraction or expansion losses. The equation for the energy loss can be formulated as:

$$h_f = LS_f + C\left|\frac{\alpha_2 V_2^2}{2g} - \frac{\alpha_1 V_1^2}{2g}\right| \tag{7.37}$$

where:

L = discharge weighted reach length (m)

S_f = representative friction slope between two sections

C = expansion or contraction coefficient

The discharge weighted reach length, L, is calculated

$$L = \frac{L_{lob}\, Q_{lob} + L_{ch} Q_{ch} + L_{rob} Q_{rob}}{Q_{lob} + Q_{ch} + Q_{rob}} \tag{7.38}$$

where:

L_{lob}, L_{ch}, L_{rob} = cross-section reach lengths specified for flow in the left overbank, main channel and right overbank, respectively

Q_{lob}, Q_{ch}, Q_{rob} = arithmetic averages of flows between sections for left overbank, main channel and right overbank, respectively

Figure 7.33 provides a visual definition of left overbank (sub area A), main channel (sub area B) and right overbank (sub area C).

The determination of total conveyance and velocity coefficient for a cross-section requires that flow be subdivided into units for which the velocity is uniformly distributed. One approach is to subdivide flow in the overbank areas using points where roughness (Manning's n) changes as break points (Figure 7.34). Conveyance within each subdivision is calculated using the Manning equation as:

$$Q = VA = KS_f^{1/2} \tag{7.39}$$

$$K = \frac{1}{n} AR^{2/3} \tag{7.40}$$

Where:

K = conveyance for subdivision (m^3/s)

n = Manning's roughness coefficient for subdivision ($s/m^{1/3}$)

A = flow area for subdivision (m^2)

R = hydraulic radius for subdivision (area/wetted perimeter of subdivision) (m)

Each of the incremental conveyances within the overbank subsections can be summed up to obtain the total overbank conveyances. Total conveyance for the section can be obtained by summing left overbank, right overbank, and main channel conveyance, Figure 7.34.

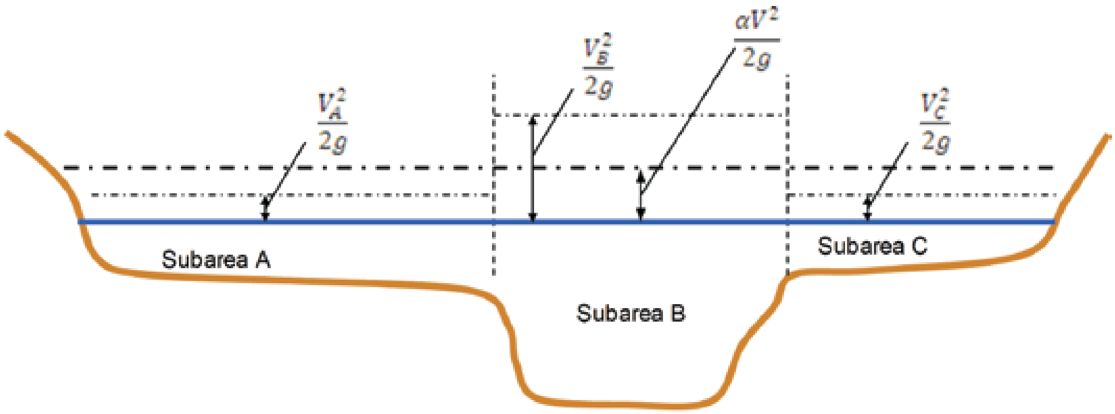


Figure 7.33 Example of how mean energy is obtained for a compound cross-section

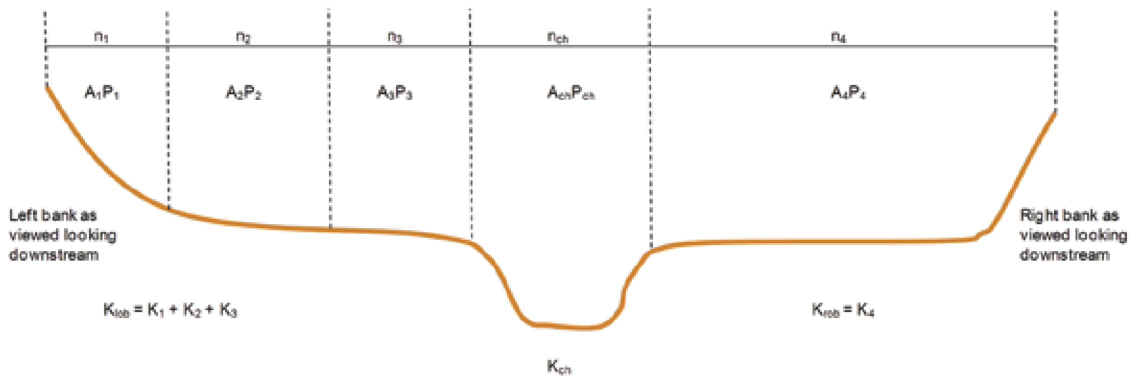


Figure 7.34 Method of calculating subdivided conveyance

In general terms, the velocity coefficient, a, is computed based on conveyance in all flow subdivisions: left overbank, main channel and right overbank.

$$\alpha = \frac{[Q_A V_A^2 + Q_B V_B^2 + Q_C V_C^2]}{QV^2} \tag{7.41}$$

Where:

$V_{A,B,C}$ = mean velocity for subareas A, B and C

$Q_{A,B,C}$ = flow for subareas A, B and C

Friction loss is evaluated as the product of the average S_f and L (Equation 7.37) between adjacent cross-sections. The friction slope, the slope of the energy grade line, at each cross-section is computed from Manning's equation as:

$$S_f = \left(\frac{Q}{K}\right)^2 \tag{7.42}$$

The average friction slope can be calculated by

$$\bar{S}_f = \frac{S_{f1} + S_{f2}}{2} \tag{7.43}$$

Contraction and expansion losses can be included by use of the coefficient C in Equation 7.37. Typical values for C are shown in Table 7.26.

Table 7.26 Typical values for expansion and contraction coefficient for standard step solution method and subcritical flow regime (after USACE, 2010)

Form of expansion/contraction	Contraction coefficient	Expansion coefficient
No transition loss	0.0	0.0
Gradual transitions	0.1	0.3
Typical bridge sections	0.3	0.5
Abrupt transitions	0.6	0.8

Computation procedure: the unknown water surface elevation at a cross-section is determined by an iterative solution of Equations 7.36 and 7.37 for each discharge of interest. Computations proceed as follows:

1. Select discharge for which the water surface profile is to be determined.
2. Identify necessary channel geometry and roughness information. Subdivisions and sub-reach lengths are computed.
3. Determine water surface elevation at downstream end (subcritical flow), location 1.
4. Assume a water surface elevation at the next upstream cross-section (downstream cross-section if in supercritical regime), location 2.
5. Based on assumed water surface elevation, determine the corresponding total conveyance and velocity head.
6. With values from step 5, compute $\bar{S}_f$ and solve Equation 7.37 for h_f.
7. With values from steps 5 and 6, solve Equation 7.36 for water surface elevation (WSEL) at location 2, WSEL2.
8. Compare the computed value of WSEL2 with the assumed value from step 4. Repeat steps 4 through 8 until the values agree to within a specified tolerance, for example 0.003 m.
9. The solution then moves one step, or sub-reach, further upstream. The calculated value for location 2 becomes the new value for location 1 (equivalent to step 3 above) and repeat steps 4 to 8.
10. Steps 4 to 9 are repeated sub-reach by sub-reach until the water surface profile has been computed for the entire study reach.

Criteria used to assume water surface elevations for a given discharge in successive iterations may vary from trial to trial. The initial trial may be obtained by projecting the previous cross-section flow depth to the new unknown cross-section. The second trial water surface elevation may be set to the assumed elevation plus 70 per cent of the error. Third and subsequent trials are generally made with a secant method of projecting the rate of change of the difference between computed and assumed values for the previous two trials.

Once a solution for water surface elevation is within the desired tolerance, a quick check should be made using Manning's equation (subcritical profiles should have depths greater than critical depth and supercritical profiles should have depths less than critical depth) to make sure that the elevation is in the appropriate regime.

This procedure is repeated in a step-wise fashion through all cross-sections by using the most recently solved water surface elevation as the 'new' location 1 in Figure 7.35, where information is known. Steps 1 to 5 are then repeated to calculate the water surface elevation for the next unknown cross-section (location 2 in Figure 7.35).

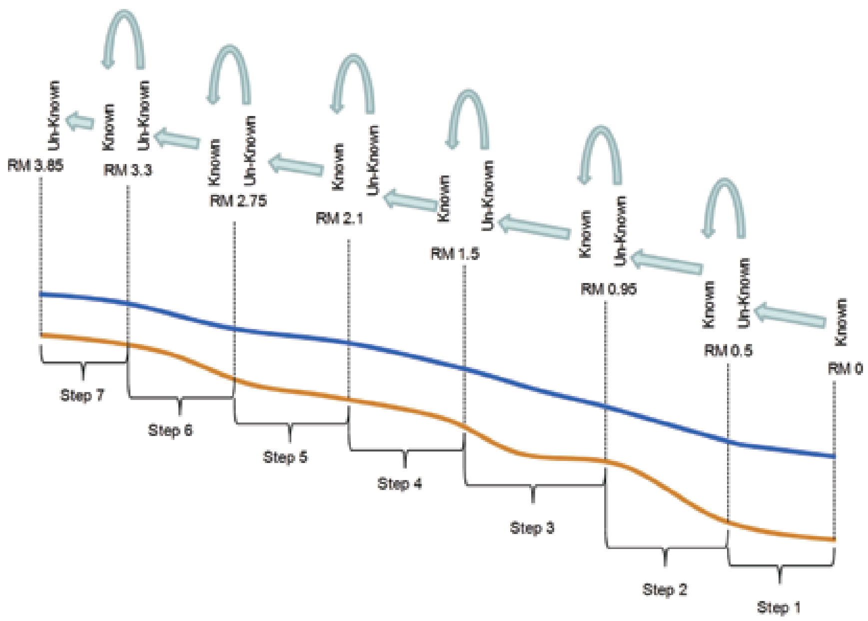


Figure 7.35 ***Step-wise solution of water surface elevations for subcritical regime***

An example using the step method to develop a water surface profile is given in Box 7.15.

Box 7.15 ***Example use of standard step method to develop water surface profile***

Problem description: a trapezoidal channel conveys 100 m^3/s of water. The channel varies in roughness, elevation, width and bank slope through the reach as indicated in Table 7.27 and Figure 7.36. Expansion and contraction losses are to be neglected. Compute the water surface profile (only calculations for the four sections are shown for illustration). Assume an average bed slope of 0.0006 m/m. Because there is no overbank flow, the velocity coefficient (α) is 1.00.

Table 7.27 ***Cross-section data for Figure 7.36***

Cross-section number	Manning's n value	Downstream reach length, Δx (m)	Bed elevation, z (m)
1	0.035	0	10
2	0.04	250	10.15
3	0.04	150	10.24
4	0.04	150	10.33
5	0.04	200	10.45
6	0.04	300	10.63

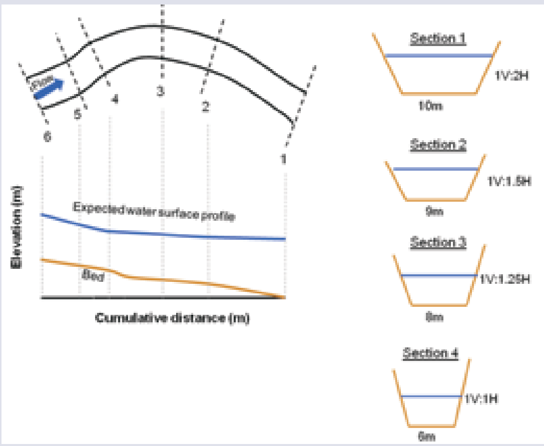


Figure 7.36 ***Plan and profile view of example stream reach***

Step 1: Q = 100 m^3/s given.

Step 2: see Figure 7.36 and Tables 7.28 and 7.29.

Step 3: the starting water surface elevation at section 1 will be taken as normal depth, solve Manning's equation iteratively for y_n.

Step 4: assume WSEL at 2 by adding depth at 1 to bed elevation at 2.

Step 5: calculate section parameters as shown in Table 7.29, hydraulic radius = Col 5/Col6, total head = Col 3 + Col 9.

Step 6: calculate K (Equation 7.40), S_f (Equation 7.42), $\bar{S}_f$ (Equation 7.43) and h_f (Equation 7.37).

Step 7: calculate water surface elevation at 2 (downstream cross-section Col 3 + downstream cross-section Col 9 – Col 9 + Col 15).

Step 8: compare calculated water surface elevation at 2 with assumed value, if not within 0.003 m, adjust assumed WSEL and repeat steps 4 to 8 until within specified tolerance (Col 16 – Col 3).

Step 9: once calculated WSEL at 2 is within specified tolerance, move upstream to calculate WSEL at 3. The calculated WSEL at 2 becomes the downstream known WSEL. Repeat steps 4 to 8.

Step 10: the preceding steps are repeated until all cross-sections have been evaluated.

Box 7.15 ***Example use of standard step method to develop water surface profile (contd)***

Table 7.28 ***Calculations for normal depth***

Iteration #	Y_n (m)	Wetted perimeter, P (m)	Cross-section area, A (m^2)	Hydraulic radius, R (m)	Q (m^3/s)
1	4.000	27.89	40.00	1.43	36
2	4.250	29.01	78.63	2.71	107
3	4.109	28.37	74.84	2.64	100

Table 7.29 ***Calculations for standard step method***

1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17
Cross-section/location	Iteration #	WS elevation (m)	Flow depth y (m)	Cross-section area A (m^2)	Wetted perimeter P (m)	Hydraulic radius R (m)	Mean velocity v (m/s)	Velocity head $\alpha V^2/2g$ (m)	Total head H (m)	Conveyance K	Friction slope S_f (x 10^{-4})	Avg friction slope S_f (x 10^{-4})	Reach length Δx (m)	Friction loss h_f (m)	Calculated WS Elev (m)	Error in guess (m)
1	1	14.109	4.109	74.84	28.37	2.64	1.34	0.091	14.200	4082	6	–	–	–	–	–
2	1	14.259	4.109	62.30	23.81	2.62	1.61	0.131	14.390	2957	11	9	250	0.218	14.286	0.028
	2	14.300	4.150	63.18	23.96	2.64	1.58	0.128	14.428	3015	11	9	250	0.213	14.284	-0.016
	3	14.283	4.133	62.82	23.90	2.63	1.59	0.129	14.412	2991	11	9	250	0.215	14.285	0.002
3	1	14.373	4.133	54.42	21.23	2.56	1.84	0.172	14.545	2548	15	13	150	0.199	14.439	0.066
	2	14.500	4.260	56.76	21.64	2.62	1.76	0.158	14.658	2699	14	12	150	0.187	14.441	-0.059
	3	14.443	4.203	55.70	21.45	2.60	1.80	0.164	14.607	2630	14	13	150	0.192	14.440	-0.002
4	1	14.533	4.203	42.88	17.89	2.40	2.33	0.277	14.810	1920	27	21	150	0.312	14.641	0.109
	2	14.800	4.470	46.80	18.64	2.51	2.14	0.233	15.033	2161	21	18	150	0.269	14.643	-0.157
	3	14.644	4.314	44.49	18.20	2.44	2.25	0.258	14.902	2019	25	19	150	0.292	14.642	-0.002

7.3.8.3 Numerical/computational methods

Numerical models conceptualise the governing equations of water flow using various simplifying assumptions that are part of the mathematical formulation contained in the model. Some models use a form of solution as presented in Box 7.15. There are many computer models in common use and a description of how each model represents the governing equations is left to the individual user manuals. Each model has parameters that should be adjusted through a calibration process. Selection of an appropriate model should consider both the purpose of the model and any underlying assumptions in how the model solves the governing equations.

The channel water level-discharge function for the baseline condition is developed using the selected model based upon:

- given channel properties
- initial conditions
- model parameters
- specified flow(s).

Levee projects may alter channel properties, principally those of the floodplain. To derive the water level-discharge associated with a proposed option plan, the appropriate model inputs are adjusted to reflect project conditions and the model is re-run to produce modified water levels for the plan.

Numerical models can be subdivided into three main types, 1D, 2D, or 3D, based on the underlying simplifications to the energy and momentum equations. Figure 7.37 compares the way each type of model solves the equations and provides an example of how results may be displayed. 1D model results generally produce plots of water surface elevation versus stream distance (water surface profiles) and cross-section plots indicating the water surface elevations. 2D and 3D models typically produce continuous mapping of water elevations, velocities, or water depths (when combined with terrain data). It is possible to produce inundation maps using results from a 1D model with geographic information systems (GIS) techniques (Section 8.11).

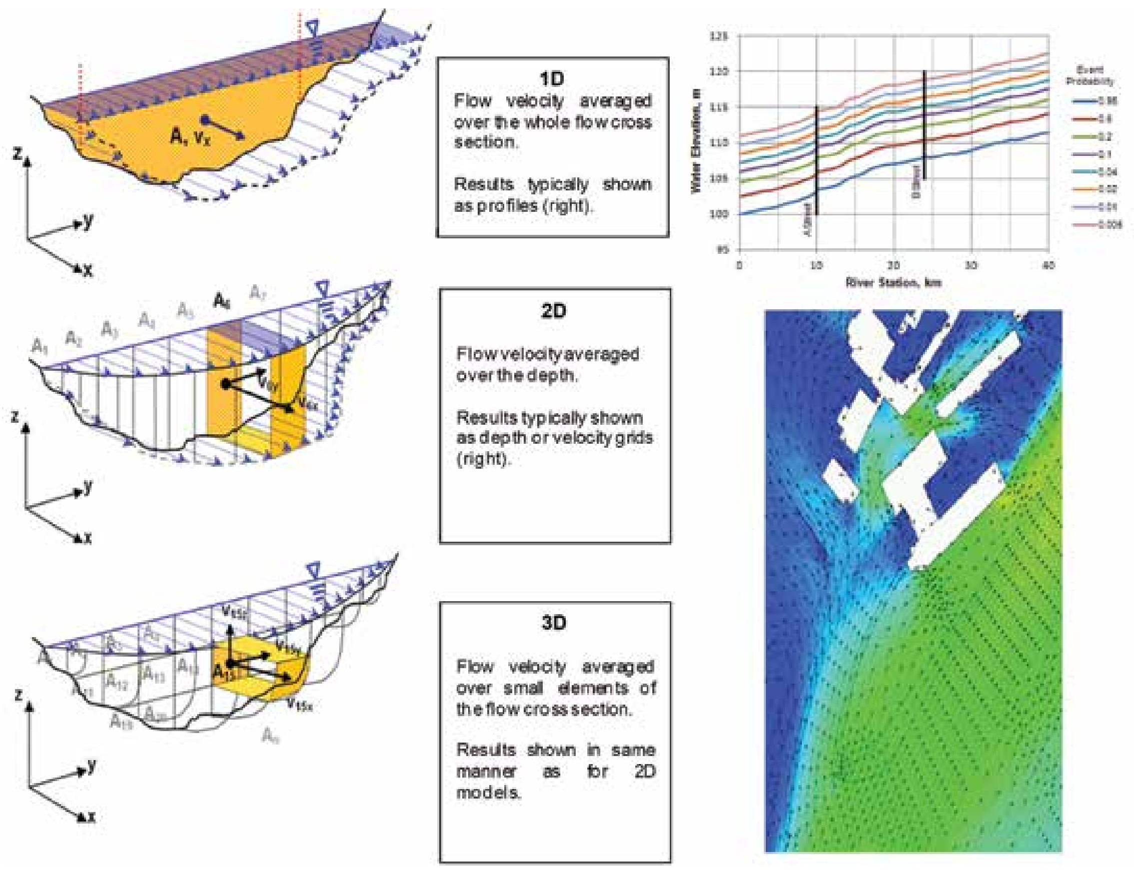


Figure 7.37 ***Discretisation in 1D, 2D and 3D flow models (courtesy Reinhard Pohl)***

7.3.9 Characterising sediment movement

Where an existing or proposed levee is in close proximity to a stream channel, analysis of bed and bank erosion warrants close evaluation to investigate the potential for impacts on the levee. Changes in stream system capacity over time can also have severe consequences on levee performance by altering water surface profile elevations. For this reason it is critical to evaluate sediment erosion or deposition within both the channel and the floodplain to the water side of the levee, and any impacts they may have on the water surface profile elevations.

Past morphological behaviour identified in the morphological assessment (see Section 7.3.1) is an important consideration in determining the level of investigation required. In systems that have a history of stable channels with minor changes over time, less intensive analysis may be justified. More exhaustive analysis will be necessary in the case where stream channels are known or expected to undergo rapid changes. Sediment transport modelling can be used to assess stream bed erosion or deposition and may be expanded to include deposition within the floodplain – floodplain deposition typically requires use of 2D or 3D sediment transport models. Results from sediment transport models provide a basis for the design of stabilisation measures required to protect the levee. Additional computations are also needed to evaluate the effect of currents that are near the levee embankment, ie floodwater that impinges on or flows along the slope of the levee.

Bed shear stress is a parameter often used to determine sediment transport and to assess stability of soils and armourstone. It is a typical parameter relating the hydraulic interactions and structural response along stream boundaries. Boundary shear stress and boundary material characteristics determine whether particles move with the current. Where boundary shear stress exceeds a threshold level, the movement of bed deposit occurs. The shear force is the shear developed on the wetted area of the channel and it acts in the direction of flow. This force per unit of wetted area is called the shear stress. This can be expressed as:

$$\tau_0 = \gamma R S_0 \tag{7.44}$$

where:

τ_0 = shear stress (N/m^2)

γ = specific weight of water (depends on fluid temperature and salinity) (N/m^3)

R = hydraulic radius (m), the ratio of the water area to the wetted perimeter

S_0 = bed slope (m/m)

7.3.9.1 Bed shear stress

As water moves over the surface of individual soil particles, which may range from very fine silt particles to very large cobbles or boulders, it exerts a shear force that tends to destabilise the particle from its original position and location. This force is due to friction at the soil-water interface. Counteracting the drag force is the self-weight of the particle due to gravity and any inter-particle forces such as cohesion, friction or capillary suction. The drag force is a function of particle shape, velocity of the moving water, and frontal area upon which the water acts. Stabilising forces are a function of particle size, particle density, proximity to adjacent particles, bed or embankment slope (if any), and the strength of cohesive and capillary influences. Figure 7.38 illustrates the relationship between destabilising and stabilising forces that act on a soil particle.

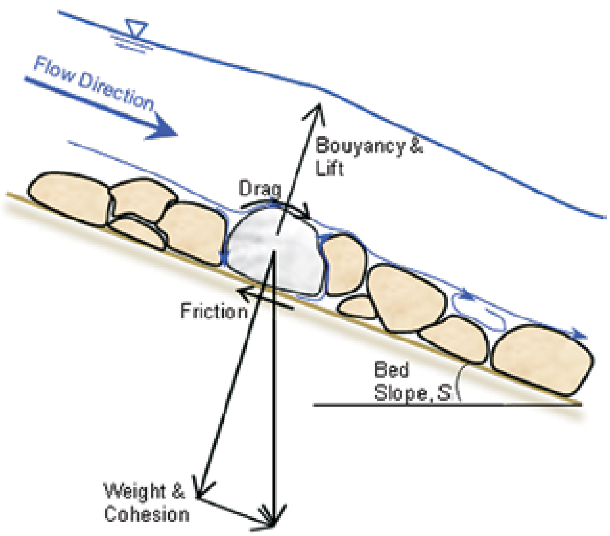


Figure 7.38 Diagram of forces acting on a particle in the stream bed

The simplest way to approximate the overall shear stress is to assume that the interaction between the current and the river walls (bed or banks) is only controlled by the bed shear stress, τ (N/m^2), and the mean gradient of the water surface, S. In this approximation, τ can be expressed as a function of the major hydraulic characteristics as:

$$\tau = \rho_w g R S \tag{7.45}$$

where:

ρ_w = water density (kg/m^3)

g = gravitational acceleration (m/s^2)

S = mean water surface gradient (m/m)
R = hydraulic radius (m)

A practical parameter, derived directly from the shear stress, τ, is the shear velocity, u_* (m/s), commonly defined as:

$$u_* = \sqrt{\tau/\rho_w} \tag{7.46}$$

or, expressed in terms of the hydraulic radius and water surface slope:

$$u_* = \sqrt{gRS} \tag{7.47}$$

Combining Equations 7.21 (where i is interchangeable with S) and 7.45, the relationship between the shear stress, τ, and the current velocity, V, is established by Equation 7.48:

$$\tau = \rho_w g(V/C)^2 \tag{7.48}$$

where:
C = Chézy coefficient

Because the Chézy coefficient, C, is basically a function of h/k_s or h/D, Equation 7.48 describes the dependency of the shear stress, τ, on the water depth, h, and the average current velocity, V. The importance of a proper estimate of the hydraulic roughness, k_s, is discussed in Sections 7.3.6 and 7.3.7.

A transverse velocity distribution is the result of an interaction with the riverbanks and/or sub-channels. Such a non-uniform velocity distribution in the transverse direction may even occur in the case of a long straight channel. Figure 7.39 illustrates the transverse shear stress over the boundary of a straight prismatic channel.

Measurements have indicated that the flow velocity near a bank can be about 40 per cent of the cross-sectional averaged velocity. This reduced velocity may be observed at the bank toe, which may be a critical factor in determining bank stability near levees. Usually, the cross-sectional averaged flow velocity is applied. Because of the lower flow velocity over the banks, the shear stress on the banks is also less than the bed shear stress. Measurements have shown that the bank shear stress may be reduced to approximately 75 per cent of the bed shear stress.

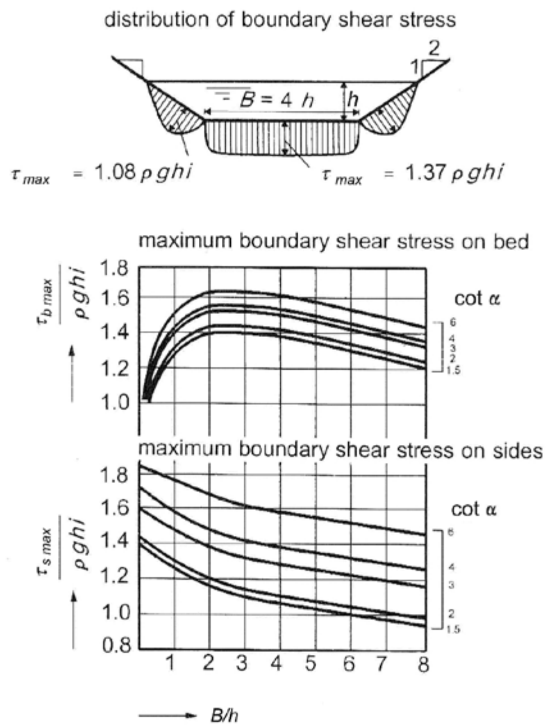


Figure 7.39 Shear stress in the transverse distribution (after CIRIA; CUR; CETMEF, 2007)

7.3.9.2 Sediment movement

Sediment transport occurs when shear stresses that act on the channel boundary material exceed the ability of the particle to remain stationary. This threshold depends on the particle shape, size and density.

The type of transport load depends on sources of sediment within the catchment, available material along the stream boundary, and stream energy. Sediment transport calculations are primarily made using empirically derived methods. Common transport functions used in modelling sediment transport and conditions for which they are applicable can be found in Simons and Senturk (1992). Sediment transport models yield net positive (higher/depositional) or negative (lower/erosional) changes in stream bed elevation with time as sediment is routed through the modelled reach.

The type of sediment model used to evaluate sediment transport for levee analysis depends on the phase of study and the complexity of the site. In the simplest cases (initial or reconnaissance phase of study or streams with limited evidence of sedimentation problems) the model may consist of a sediment budget that accounts for available sediment sources and the capacity of the stream to move available material sizes throughout the reach where impacts from levees may occur. More complex sites may require continuous sediment routing for a period of years using a 1D or even a 2D sediment transportation model. When deposition or erosion is expected to occur across the entire floodplain, a 2D model may be required. The most common modelling approach uses a quasi-unsteady (continuous hydrograph discretised into a series of relatively short constant flow steps) or fully unsteady flow hydrograph as the input boundary condition. In some limited cases a series of continuous constant flows may be used to assess sediment movement within the system.

Levee systems should be evaluated using sediment transport modelling to ensure that sediment deposition (or erosion) does not compromise their ability to pass the design discharge. They should be analysed for two conditions:

1. Long-term aggradation and degradation.
2. Aggradation or degradation during the design hydrograph.

7.3.10 Effects of wind on water levels and in generating waves

If the river reach adjacent to the point of interest is subject to wind over open water that wind may directly increase water levels at the levee and will also generate waves on top of the still water. Both phenomena need to be taken into account as for coastal levees. As explained in Section 7.4, wind-induced water level increases may be predicted empirically, through statistical analysis of observed elevations, or conceptually, with a model of the wind and wave setup processes. The former method is difficult, as observations of water level are not easily segregated into discharge-related and wind-induced components.

Wave action in rivers is dependent on the size of the open water body as well as the potential for wind across the body. In smaller rivers the fetch length may be small enough that the generated wave heights can be neglected. For larger rivers, fetch may be sufficient to develop waves of significant height (H_s) 0.3 to 1 m or more. The approach for estimating wave height and direction in coastal areas described in Box 7.19 is also applicable for estimating wave conditions in a fluvial environment. Waves in rivers can also be caused (CIRIA; CUR; CETMEF, 2007) by vessels navigating the waterway.

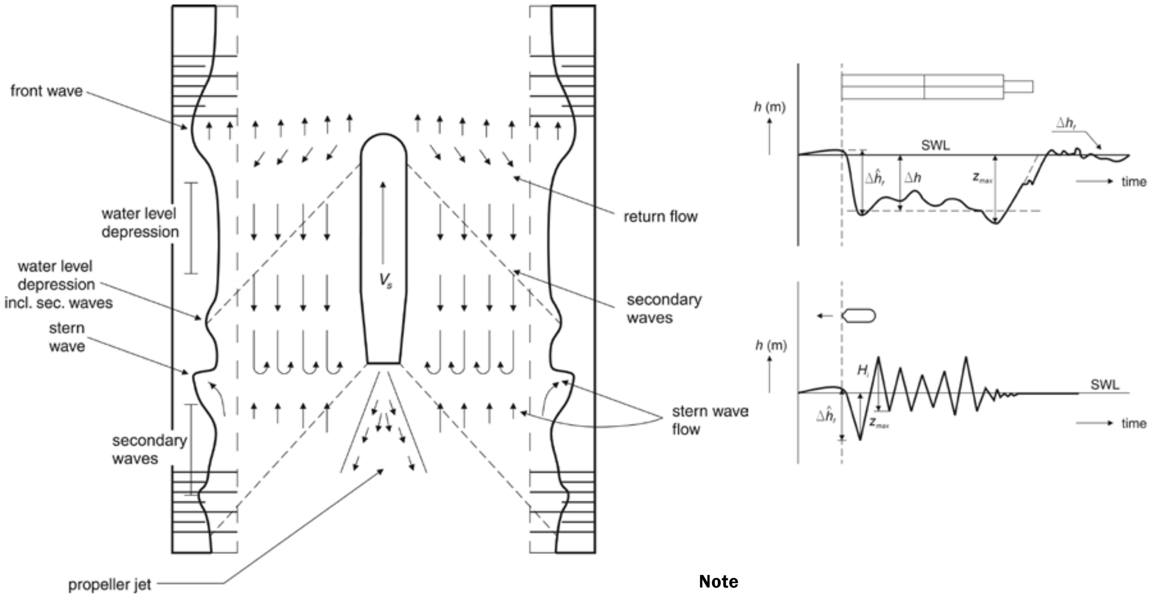
7.3.11 Ship induced currents

Navigation on inland waterways may have an impact on levee design by introducing waves and currents caused by vessel movement. Some governments suspend navigation during floods to eliminate this additional source of waves and associated currents on flood control measures including levees.

The relevant parameters for calculating the ship-induced water movements (Figure 7.40) are as follows:

- primary ship wave, consisting of (a) transversal front wave, (b) water level depression alongside the ship, and (c) transversal stern wave

- return current within the primary wave
- secondary ship waves
- propeller jet.



Note

Dotted lines represent the interference peaks of secondary waves of which the propagation direction is 350 with respect to the vessel's propagation direction.

Figure 7.40 ***Characteristics of ship-induced water movements related to bank stability (from CIRIA; CUR; CETMEF, 2007).***

Figure 7.41 presents eight steps for calculation of ship-induced water movements. The details of these basic relationships for evaluating ship-induced water movements in navigation channels can be found in Section 4.3 of CIRIA; CUR; CETMEF (2007). A more comprehensive discussion on ship-induced water movements in navigation canals can be found in PIANC (1987) and Przedwojski *et al* (1995).

1 Determine vessel's submerged cross-section, A_m

2 Calculate limit speed of vessel, V_L

3 Calculate actual speed, V_s, of vessel (or use admissible speed limit)

4 Calculate **mean** water level depression, Δh, and mean return flow, U_r

8 Calculate secondary ship waves, H_i, L_i and T_i

5 Calculate **maximum** water level depression, $\Delta\hat{h}$, and return flow, $\hat{U}_r$

6 Calculate front wave, Δh_f, and i_f

7 Calculate stern wave, z_{max}, i_{max} and u_{max}

Figure 7.41 ***Basic calculation scheme for ship-induced water movements (from CIRIA; CUR; CETMEF, 2007)***

7.3.12 Ice effects

Geometry of the river, climate, and character of the ice all play a part in the development of a surface ice layer and/or an ice jam. As such, the impact of ice dams on design water levels is site-specific. If sufficient data exists, calibration of a numeric model provides the best indication of ice effects. Generally, ice forces acting directly on a levee or ice induced erosion are not design issues unless there is a history of ice attack.

7.3.13 Uncertainty in data and analysis

Attention needs to be given to the uncertainty that exist in both data and analysis methodologies. This section will deal with uncertainty in hydrologic data, hydraulic data, water level-discharge functions derived from models, observed water level-discharge functions, and quantifying uncertainty in water level-discharge functions derived from hydraulic models.

7.3.13.1 Hydrologic uncertainty

With any of the methods identified in the preceding sections, the discharge predicted for a specified probability is not certain. Sources of this uncertainty depend both upon the data and the particular method of analysis used, but the most common sources for uncertainty stem from the following issues:

1 **Data uncertainties:**

a Inaccuracies in the stream flow data, primarily during measurement/estimation of the peak flow magnitude.

b Inadequate record length that leads to poor probability function parameters.

c Effects of flow regulation are partly or completely unaccounted for, eg small farm ponds may capture and hold flood runoff in agricultural catchments, or bridges and culverts may impede peak flows.

d Lack of knowledge regarding proper initial conditions for a rainfall-runoff model.

e Lack of stationarity in the historic data, or a climatic change from the historic conditions.

2 **Modelling uncertainties:**

a Errors in extrapolating stream data to another catchment, which could include a non-perfect relationship between the catchments, poorly fit regression models, or simply a poor translation of the data to the new catchment/site.

b Non-perfect fit or poorly selected model of a probability distribution function to gauge data.

c Numerical models that do not accurately reflect the runoff features of the catchment, inaccuracies in the precipitation information, poor/inappropriate numeric model that does not predict flows accurately.

d Errors in or uncertainty about calibration of the hydrologic model.

e Lack of experience of the modeller.

7.3.13.2 Hydraulic data uncertainty

Sources of uncertainty about a transform derived from observations of discharge and water level include:

- errors in measurements of water level. In many cases, these water surface elevations are measured under adverse conditions, perhaps from a boat or cableway during an event, or are made after the event including measurements made of high water marks
- errors in measurements of velocity from which discharge is estimated
- uncertain knowledge of channel geometry from which cross-section area is computed to estimate discharge indirectly as the product of velocity and area
- the topographic and bathymetric data from which geometry is determined may be surveyed with traditional methods or inferred from remotely sensed data. Even the best field surveys are subject to measurement errors

- lack of stationarity of the series of discharge and water level observations, as noted above
- uncertainty about the proper form of the mathematical function with which to represent the water level-discharge relationship. Sensitivity analysis on the function and function parameters should be considered to understand the nature of the uncertainties
- uncertainty about the parameters estimated for the function, particularly if the data set is small or if it is sparse in regions of highest flow.

7.3.13.3 Water level-discharge functions derived from models

Sources of uncertainty and error in a water level-discharge transform derived through application of a river hydraulics model include:

- choice of model type – models may be steady or unsteady, and 1, 2 or 3D. All are approximations of the true flow condition, the correct choice is never certain
- topographic and bathymetric data with which the model represents the channel and floodplain geometry
- measurements of dimensions and locations of structures in the channel and floodplain that impede flow. These structures include culverts, bridges, piers, levees etc
- existence of, and impact of, ice cover and ice jams in the stream
- downstream boundary conditions, which is especially complex if water level at the index point is influenced by backwater at a downstream confluence, reservoir or lake, or sea
- model parameters. For 1D models, parameters include Manning's n and expansion and contraction coefficients. For 2D models, additional parameters include eddy viscosity or the equivalent
- impacts of erosion and deposition before or even during a flood event
- impacts of wind and wave run-up on elevation, given the flow.

7.3.13.4 Quantifying uncertainty in observed water level-discharge functions

Developing a mathematical model of uncertainty about a graphically developed transform function is challenging. It is not possible to use many of the common statistical methods of analyses because of the graphical, rather than mathematical, form of the function.

1. Water level values determined with a graphical model can be compared with observations, and error statistics computed. These statistics include the mean error (difference in predicted and observed elevation) and standard deviation of the error. This approach describes only the uncertainty due to fitting the model, but it fails to account explicitly for other sources of uncertainty.
2. To account for additional uncertainties, sensitivity of the graphical function to various errors may be considered through sensitivity analysis. For example, the impact of systematic errors in measurement of elevation can be addressed by adding an increment or decrement of elevation to each observation, considering reasonable values of those errors derived from professional judgement. This approach allows an upper limit function and a lower limit function to be drawn, for a range of observed water levels for a given discharge. If the spread is assumed to be normally distributed (a common assumption for such errors), uncertainty model parameters can be inferred. With normally distributed errors in water level, 95 per cent of the error is within the range spanned by two standard deviations above and below the best estimate (mean value).

7.3.13.5 Quantifying uncertainty in water level-discharge functions derived from hydraulic models

In most cases, the analyst will be required to describe uncertainty for conceptually derived functions, because water levels computed with a channel hydraulics model for a specified discharge form the basis for developing the discharge-water level relationship for most risk analyses. A description of uncertainty about a discharge-water level transform can be derived through statistical comparisons with historical water level data, sensitivity analyses with professional judgement, or via detailed simulations that account for likelihood of contributing facts. These approaches are described as follows.

Chapter 5 describes how risk-based analysis can integrate the uncertainties about each of the functions included in the analysis. So, the uncertainties identified above should be quantified and represented mathematically.

Historical event analysis: comparisons can be made with historical data. As discussed above, uncertainty about the discharge-water level transform can be described, in part, through statistical analysis in which water levels computed with specified discharges are compared with observed water levels (high watermarks) for the same discharges. It should be noted that:

1. The standard deviation of error can be determined with standard statistical techniques.
2. This approach implicitly accounts for the impact of all uncertainties, but it is limited by the availability of data for comparison and by the diversity of the dataset. For example, if the data used for comparison do not represent cases of significant backwater, the uncertainty model parameter derived with this comparison cannot explicitly account for significant backwater effects. This method is highly dependent on having several medium to large events in the historical record. As this is often not the case, this method is not typically applied.

Sensitivity analysis and professional judgement: these can be used to compute the uncertainty in hydraulic model derived water level-discharge relationships. Uncertainty associated with water level-discharge functions computed with hydraulic models can be described by developing upper and lower limits on water levels, computed by inferring the uncertainty in model parameters.

It should be noted that these limits may be developed with standard hydraulics models, with model parameters set at reasonable limits. A realistic range of model parameters should be estimated to represent the 95 per cent confidence band of possible values. For example, Figure 7.42 shows water level profiles computed for a stream reach, with channel Manning's n values and other parameters taken in combinations and fixed at reasonable extremes. An upper reasonable limit on water level is computed and plotted, as is a lower limit. Assuming that the spread is normally distributed and that the upper and lower limits define the 95 per cent confidence range (±2 standard deviations) using standard statistical techniques.

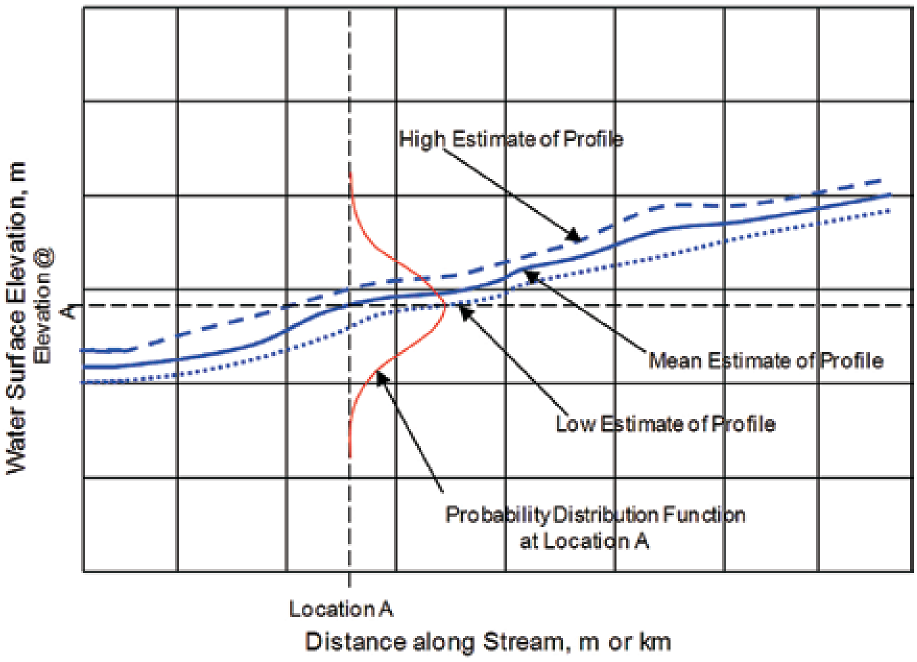


Figure 7.42 Example of computed water surface elevation sensitivity

7.4 MORPHOLOGY AND HYDRAULIC ACTIONS FOR COASTAL AND SHORELINE LEVEES

Proper understanding of water level, wave, current and morphological conditions is essential for the assessment or design of shoreline levees. As coastal and shoreline systems are dynamic and highly complex, the configuration of the coastline and its morphological evolution influence the nature and development of the hydraulic loads. The hydraulic loads that act on coastal levees vary geographically and over time and comprise of various combinations of water levels, currents and waves. These have short-term (daily), monthly, seasonal, decadal and long-term (extreme) characteristics. The mechanisms generating them include meteorological forcings (wind and pressure), astronomical forcings (tides), and seismic effects (tsunamis). Combinations of two or more of these forcings often determine the most extreme loadings on a coastal levee, for example, storm surges that raise the still water level to an unusual degree may well be coupled with simultaneous wave action.

This section provides methods for assessing the various types of coastal morphology and morphological change and the hydraulic events (water levels, waves and currents) and related loads, which might be generated at the location of the toe of levees. It discusses the determination of hydraulic conditions using deterministic, probabilistic and modelling techniques and explains how to deal with changes and uncertainties. Figure 7.43 presents a flow chart describing the interaction of the various morphological and other influences on the hydraulic loadings which affect coastal levees.

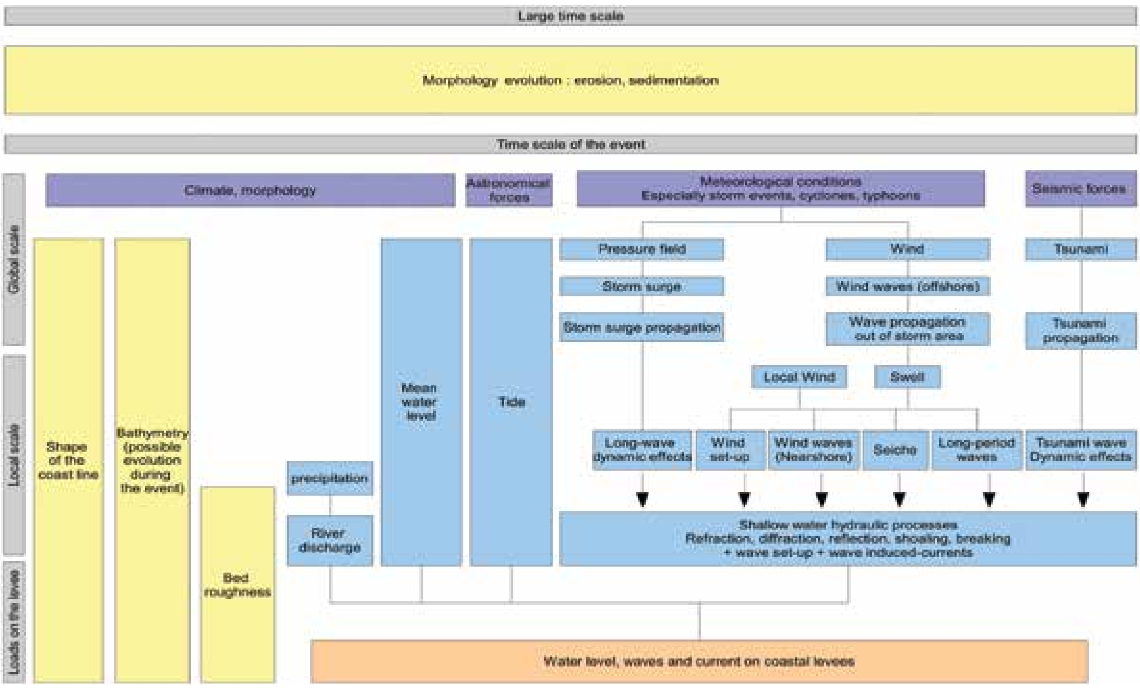


Figure 7.43 ***Influences affecting hydraulic loadings on coastal levees***

7.4.1 Coastal morphology

Knowledge of seabed bathymetry and its morphology is fundamental to the evaluation and design of coastal levees. For example:

- wave heights at the shoreline may be reduced from their offshore value by frictional and breaking processes over near-shore shoals
- steep near-shore gradients may cause rapid shoaling and breaking at the toe of the structure
- bathymetry influences on tide propagation and tidal range, amplified by the shape of a bay or estuary
- sub-tidal bathymetric features such as channels and banks can also steer flow in concentrated streams, which vary with time as the morphology changes.

In a coastal or marine environment, morphologic studies should be undertaken to establish an understanding of how the coastal zone responds to normal variations in flow regime (waves and tides) and to extreme events (hurricane, tsunami, or other surges). The stability of the shoreline and its adjoining hinterland depends on the physical character of the shore, which in turn is determined by the geology, geomorphology and the actions of winds, waves, and tides. The potential for changes in the shore location should be investigated to assess suitable sites for a new levee or to evaluate requirements for stabilisation works near an existing levee. Figure 7.44 illustrates changes along a shoreline in the Great Lakes, USA.

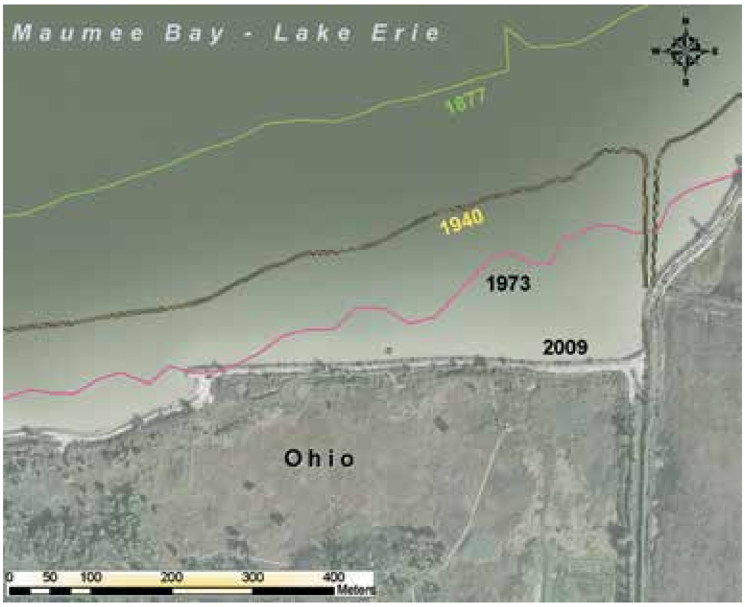


Figure 7.44 Shoreline change along Maumee Bay on Lake Erie, Ohio, USA (courtesy Andrew Morang)

The coastal morphology and bathymetry are controlled by the balance between the aggressiveness of physical processes, the resistance of the land, sediment supply, and, in some cases, biological productivity (eg mangrove coasts). The morphology of a coast changes if the applied forces or sediment supply change. The forces are induced by waves, tides, winds, currents, surges, and ice. Their impact on the landform depends on the type, magnitude and direction of the forces in combination with the materials strength. If these forces produce a change in landform shape, then a new relationship develops and continues to evolve until a new dynamic equilibrium is achieved. Sediment transport causes morphological changes and maintains this balance between landform and the hydrodynamic forces.

The sea bed and coastal zone exhibit a variety of bed patterns and landforms shaped by sediment transport mechanisms. Each planform has specific characteristics that should be considered when levees are located on the coast. Some of the major coastal features and their characteristics relative to levees are detailed on Table 7.30.

Table 7.30 Major coastal features with pertinent characteristics relative to levees

Feature	Characteristic
Sand waves	• most changeable of coastal bed patterns – height and position can vary considerably • bedform orientated almost perpendicular to dominant current direction • height can vary from 1 m to 10 m • can affect foreshore depth, current patterns and wave heights.
Sandbanks/shoals	• persistent but dynamic bed patterns, which change height and position • associated ebb-flood tide channels • fully submerged or drying at low tide levels • can influence wave and current patterns.
Mudwaves – Gyana	• similar features to sandbanks but formed from large accretions of fine sediment in the near-shore zone

Table 7.30 Major coastal features with pertinent characteristics relative to levees (contd)

Muddy foreshores, mudflats and saltmarshes	• exist on either net erosion or net accretion coastlines • exposed to cycles of erosion and deposition • transition zone to elevation where vegetation can colonise and bind/trap sediment • highly vulnerable to hurricanes and sea level rise • subject to subsidence caused by consolidation of underlying deltaic sediments (for example, Louisiana, Bangladesh).
Beaches	• cross shore profile – average slope between seaward and landward limits • in theory the average profile shape varies seasonally due to wave energy variations • longshore profile – large-scale shape of beach is controlled by long-shore movement of sediment • often nourished artificially to protect coastal levees (eg in the Netherlands).
Coastal dunes	• formed by sufficient supply of fine dry sand and sufficient wind to move it • may resist wave action but are vulnerable in unusually large storm surge events, tropical storms, or tsunamis • vegetation growth (natural or artificially planted) can provide added resistance to erosion • often built artificially as part of shore protection projects, sometimes containing a rock core.

Processes that drive cyclic development and destruction of these patterns are complex and beyond the scope of the handbook.

7.4.1.1 Approach to undertaking a morphological study in coastal settings

Unique challenges exist in performing a morphological study in a coastal environment. Factors such as landform geometry, land and seascape cover, and bathymetry create hydraulic effects that are not readily determined or modelled. Table 7.31 presents some information to be considered to perform a morphological study in a coastal environment. As examples of this variability, consider the following three different coastal conditions:

- a densely vegetal landscape on a low-lying continental shelf – the low gradient tends to disperse hydraulic energy over a large area and the vegetal cover provides greater roughness and reduces velocities
- an abruptly rising beach ridge – the hydraulic energy is intensely focused on a relatively narrow shore-water interface
- a funnel shaped bay – the hydraulic energy is directed inland towards the apex of the funnel.

Table 7.31 Some information to be considered to perform a morphological study in a coast environment

Information or data to be considered	
Coastal currents/ waves	• current and wave information for both normal and extreme events • event duration.
Sediment transportation	• the overall sedimentation system should be understood. This includes magnitude of sediment movement and the velocity, as well as seasonal and climatic effects • determine the regional sediment availability to be moved.
Existing beach or coastal stabilisation works	• the influence of the existing stabilisation works (groynes etc) on the natural morphological process should be evaluated. This needs to be undertaken not only at the site of interest but also up and down shore from the site in order to assess the functions of the entire system and evaluate the effects of changes to that system.
Geology	• both the onshore and offshore geological deposits should be evaluated as they provide natural replenishment sources to the foreshore deposits • geology also may provide constraints on long-shore development and vertical changes in near-shore and beach profile.
Bathymetric and topography data	• understanding the geometry of the shallow near-shore is a reasonable first approach to developing a map and prediction of future processes • LiDAR and bathymetric surveys are commonly used to map large areas • comparison of repeat coverage surveys allows a rapid determination of changes that have occurred in the area in the intervening time interval.

The data collected should be capable of differentiating changes resulting from:

- **'normal' coastal processes**
  - long-term changes, typically arising from long-shore movements, permanent onshore or offshore movements, or subsidence
  - seasonal changes, typically involving changes in beach profile without necessarily changing net beach volume
  - short-term changes, typically scour or deposition arising during individual storm events
- **'extreme' coastal processes:**
  - inundation by surge and waves
  - breach of natural barrier systems
  - channel avulsion.

Finally, long-term variation in climate change creates questions regarding accuracy of the historic data sets and injects an element of uncertainty in understanding future events. It is critical to include this dynamic in the procedure for predicting future processes.

7.4.2 Water levels

This section provides information on several aspects of importance to the determination of water levels:

- tidal and land data
- still water level
- wave set-up
- local variations in water level (including long period waves and seiche)
- numerical modelling of water level
- long-term changes in still water level.

7.4.2.1 Tidal and land based datum

Water levels have to be measured relative to some specified elevation or datum to have a physical significance. A datum is a base elevation from which vertical heights or depths are referenced. It is necessary that a datum represents some reference point, which is universally understood and meaningful, both onshore and offshore.

Two types of datum exist, **tidal based datum** and **land based datum** also called 'national fixed datum'. To avoid confusion, the type of datum used should be specified.

A sampling of typical tidal based datum is shown in Table 7.32. They vary along the coast as described in Sections 7.4.6 and 7.9.1. Examples of national fixed datum are presented in Section 7.4.6.

Table 7.32 Tidal based datum levels

Symbol	Name	Description
MSL	Mean sea level	Average over 19-year tidal epoch
HAT	Highest astronomical tide	Highest level caused by astronomical influences over 19-year epoch
LAT	Lowest astronomical tide	Lowest level caused by astronomical influences over 19-year epoch
MHHW	Mean higher high water	Average tidal height at higher high water during 19-year epoch
MLLW	Mean lower low water	Average tidal height at lower low water during 19-year epoch

1 2 3 4 5 6 7 8 9 10

The representative water level at a shoreline may be defined trpically as the temporal or time-averaged water surface elevation measured over a period of 10 minutes. To determine proper conditions it is necessary to distinguish:

- still water level that includes the effects of tides and surges
- wave set-up
- local variations that include long period waves and seiche.

7.4.2.2 Still water level

The still water level (SWL) is the water surface elevation, excluding local variation, but including the effects of tides and surges. It represents the water level averaged over a several minute period (removing the variation due to short-period waves). The SWL is typically referenced to a vertical geodetic datum. These may be decomposed into elements as described in the following.

1. Mean water level (MWL) for coastal waters open to the sea can, in most cases, be taken as a site-specific constant related to the mean sea level (MSL) of the oceans. The MSL is widely adapted as the primary datum since it can be accurately computed from tidal elevation records at any location. MSL is based on the average water surface elevation over a long time. A period of 19 years is used as it includes the long-term variability in tides and removes most meteorological effects.
2. Tides are water movements generated by the global response of oceans to astronomic effects. On continental shelves and in coastal waters, particularly bays and estuaries, this effect is amplified by shallow water and coastal platforms. The tidal range that coastal levees experience is the vertical difference in high and low water level due only to astronomical effects and varies geographically.
3. Storm surges represent the water surface response to the combined effects of local atmospheric pressure, and wind-induced surface shear stresses. The shear stress that is exerted by the wind on the surface of the water pushes water in the direction of the wind. Atmospheric pressure gradients raise water levels under the atmospheric depression, as water moves from regions of high atmospheric pressure into regions of low pressure. A storm surge will move across the sea with the depression and will be modified by bed and coastal forms. The presence of coastline irregularities and embayments can facilitate generation of locally higher storm surge. The phasing of the storm surge with the tide influences extreme water levels. Such surges may increase as they move into shallow water.

The contribution of wind to storm surge is often called wind set-up. Wind is most effective in creating wind set-up when it blows over shallow water, because the effect of wind on water level is inversely proportional to water depth. The shallower the water depth, the greater the influence of wind in generating storm surge. Wide, shallow coastal shelf regions are more prone to wind set-up than narrow, deep shelf regions. Wind set-up is also a function of fetch, or the distance over which a wind blows, together with the duration or persistence of wind. In general, the Coriolis-induced wind set-up/set-down from alongshore winds will be less than that from cross-shore winds. An example calculation of wind set-up ($\bar{\eta}_w$) is presented in Box 7.16 (Tilburg and Garvine, 2004).

Storm-induced surges can produce short-term increases in water level that rise to an elevation considerably above MWLs. Dynamic effects can cause a significant amplification of the rise in water level, such as when the depression moves quickly, the water level rise follows the depression. The height of the long wave generated by the depression may increase considerably as a result of shoaling in near-shore zones. Along the coasts of the southern North Sea, storm surges with a residual height of 6 m were recorded in 1953, and in 2005 Hurricane Katrina produced surges up to 9 m in the Gulf of Mexico.

Box 7.16 ***Detailed example of wind set-up (Tilburg and Garvine, 2004)***

$$\overline{\eta}_w = \frac{U_x \ell}{16300}\sqrt{\frac{U}{|U_x|}} - \frac{U_y U}{10^8 m}\ln\left(1 + \frac{10^3 m\ell}{D_c}\right) \tag{7.49}$$

where:

The x-axis is alongshore and the y axis is directed offshore

m = mean slope of the continental shelf typically 10^{-3} (1:1000)

D_c = water depth in front of the coastal wall, typically 1 to 5 m

U_x = alongshore wind speed component (km/h)

U_y = cross-shore wind speed component (km/h)

U = wind speed (km/h)

ℓ = distance of the shelf break from the coast (km), typically 100 km

Note that the constant 10^3 in the equation is a conversion factor from kilometres to metres

Application to Atlantic City, New Jersey, USA with a coast oriented N35°-N215°, wind blowing from N55°, U = 28 m/s (100 km/h), m = 0.001, D_c = 1 m, and ℓ = 100 km

Solution:

U_x = 94 km/h, U_y = -34 km/h and $\overline{\eta}_w$ = 0.59 + 0.16 = 0.75 m

7.4.2.3 Wave set-up

Wave set-up is the localised effect on the MWL at the shoreline as a result of the transfer of momentum from waves to the water column during the shoaling and breaking process. As waves break on a beach, wave heights decrease and the flux of wave momentum in the onshore direction is reduced. This creates a compensating force that is exerted on the water column and an increase in water level. The wave set-up depends on wave nonlinearity, wave breaking characteristics, beach slope, changes in slope, and wave propagation through vegetation. Wave set-up of the MWL is illustrated by the blue line in Figure 7.45, which also indicates diagrammatically the following parameters:

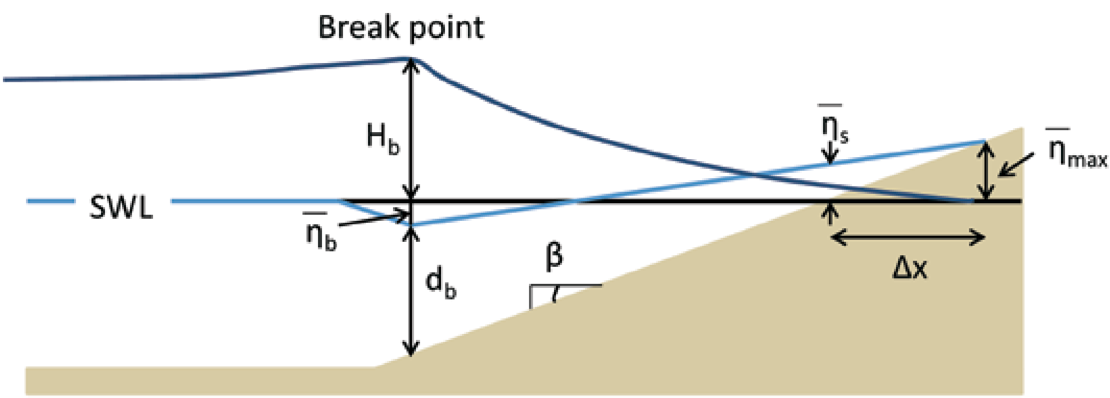


Figure 7.45 ***Schematic of wave set-up on the shoreline***

- break point water depth, d_b
- wave height, H_b
- wave set-down, $\overline{\eta}_b$
- shore slope, β
- mean water surface elevation or wave set-up, $\overline{\eta}_s$
- shoreward displacement of the shoreline, Δx
- maximum mean wave set-up, $\overline{\eta}_{max}$

In general, the wave set-up is lowest (negative or set-down) at the break point and increases linearly towards shore, reaching a maximum value inland from the still water line. So, the Δx and $\overline{\eta}_{max}$ values are important in levee design. An approximation for assessing wave set-up is presented in Box 7.17. More detailed methods are given in Smith (2003) and CIRIA; CUR; CEMTMEF (2007).

Box 7.17 Rule of thumb for wave set-up

Two methods are demonstrated: Method 1 is from Smith (2003), and Method 2 is based on work of Dean and Walton (2009).

Method 1 (use if breaking point wave data available):

Mean wave set-up at the still water line $\overline{\eta}_s$ can be approximated as:

$$\overline{\eta}_s \approx (0.10 \text{ to } 0.20) H_b \approx (0.08 \text{ to } 0.17) d_b \tag{7.50}$$

where a breaker depth index of $\gamma_b = H_b/d_b = 0.84$ is assumed.

Method 2 (use if offshore wave data available):

$$\overline{\eta}_s = (0.191 \pm 0.100) H'_{os} \tag{7.51}$$

where H'_{os} = equivalent deep water significant wave height (m).

Caution: it is important to note that methods to predict wave run-up levels or overtopping discharges on seawalls, revetments and related coastal/shoreline structures will (almost always) have been derived from (or validated against) measurements of run-up or overtopping on physical models in wave flumes of basins. Those measurements will have already included the on-structure set-up, and so no further allowance for set-up is required.

7.4.2.4 Local variations in water level (including long period waves and seiche)

On the foreshore localised variations in water level may increase the potential for levee overtopping, discussed in Section 8.2.1. The main phenomena are:

1. **Long-period waves** propagate with wave groups and become more important at the shoreline. Typically, long-period waves have a modest height of about 0.1 m to 0.3 m for most common situations. The long-period oscillations can aggravate the damage to coastal structures and increase overtopping.
2. **Seiches** are standing wave oscillations of the free surface of a body of water in a closed or semi-enclosed basin, such as a harbour or lake. These oscillations are of relatively long period, extending from minutes in harbours and bays to over 10 hours in the USA/Canadian Great Lakes. Any external perturbation to the body of water, such as earthquakes or landslides, can force an oscillation. In harbours, the forcing can be the result of short waves and wave groups at the harbour entrance. In enclosed or semi-enclosed bodies, seiches are primarily as a result of changes in atmospheric pressure and the resultant wind conditions during the passage of a squall and occur over the entire basin. Wind effectively tilts the water surface in the direction of the wind. When the storm abates and wind forcing is removed, the water that had accumulated against the downwind shoreline flows back across the body of water and a wave motion is excited as the water sloshes back and forth. Frictional losses cause the seiche amplitude to diminish as time passes. The frequency of oscillation is a function of the forcing, together with geometry and bathymetry of the system. Seiches may have the greatest effect near the ends of long, deep, and narrow water bodies.

7.4.2.5 Numerical water level modelling

Storm surge, wave set-up, long period waves, seiches, and circulation are routinely modelled numerically using the equations of continuity and momentum or the generalised wave continuity equation. Modelling inputs include bathymetry/topography, bottom roughness coefficients (eg Manning n coefficients derived from land-use maps covering the potential inundation area), tidal potential (or time history of tide levels at the model boundary for a small domain), river inflow boundary conditions, atmospheric pressure and wind fields over the domain, and wave radiation stress fields (to calculate wave set-up). Model grids should be developed with sufficient spatial discretisation to resolve bathymetry features and gradients in the forcing. Generally, storm surge models are depth integrated (2D in the horizontal), but may also be 3D. The models require calibration based on field measurements of water levels and/or currents and then validation with independent data. Models can range from simple finite-difference formulations applied over limited domains to multiple million node/cell grids in 2D or 3D covering entire ocean basins. Storm surge models solve for the time and spatial evolution of water level and current.

7.4.2.6 Long-term changes in still water level

Long-term changes in relative MSL arise from the combined effect of local changes in land elevation and global sea level changes arising from climate change. These changes result from five main processes. Several of these can occur simultaneously, but not all processes necessarily apply to all geographic locations:

- eustatic rise or global change in the oceanic water level. Examples of eustatic rise include melting of land-based ice sheets and glaciers and the expansion of near surface ocean water due to global ocean warming. Long-term measurements around the world indicate global rates of rise of 1 mm to 2 mm/year over the last century. The present consensus is that the rate of rise will probably increase to about 5 mm/year, with some regional variations, although as yet there is no evidence that this acceleration has started
- climatic fluctuations may also create changes in sea level. For example, surface changes produced by El Niño due to changes in the size and location of high pressure cells
- crustal subsidence or uplift from tectonic uplifting or down-warping of the earth's crust. These changes can result from uplifting or cooling of coastal belts, sediment loading and consolidation, or subsidence due to volcanic eruption loading
- seismic subsidence caused by sudden and irregular incidence of earthquakes
- auto-subsidence due to compaction or consolidation of soft underlying sediments such as mud or peat.

Increases in relative MSL are likely to cause related increases in all other water levels, including extreme water levels. In many cases, sea level rise may become an issue especially if the wave heights that are depth limited experience increases. Sea level rise would accordingly increase wave attack on coastal levee structures (CIRIA; CUR; CETMEF, 2007).

7.4.3 Waves

Waves impact levees by increasing the still water level (wave set-up) and by intermittent wave run-up and overtopping (Sections 8.2.1). Waves may modify the morphology due to shear stresses imposed by wave orbital velocities, run-up and overtopping velocities, undertow or reflection.

Ocean and coastal waves are composed of many different wave periods, heights, and directions and are referred to as random waves. Statistical parameters are used to characterise the wave field (see Box 7.18). Wave parameters of interest are significant height, peak and mean periods, direction, length, and maximum bottom orbital velocity. Waves may also be characterised by directional wave spectra, which describe the distribution of wave energy density as a function of frequency (f) and direction (θ). The directional wave spectrum $S(f, \theta)$ may be expressed as a product of the frequency wave spectrum $S(f)$ and a directional spreading function $D(f, \theta)$. A simplification of real-world random waves, which is often applied in design, is to represent wave fields by monochromatic or regular waves that consist of a single wave period, height, and direction. These monochromatic waves are often a quick and conservative way to estimate the effect of waves on levees and other coastal structures. Both regular and random waves are used in this chapter to represent wave effects on levees, depending on the ease of use and relative effort required to provide accurate design information.

Waves are generated by wind blowing across a fetch, the over water distance measured in the wind direction, and may present a significant design consideration for levees exposed to open water. Waves heights in deep water follow a Rayleigh or asymmetric distribution. The description of the sea surface with regard to wave action is called 'sea state'. The sea state can include two components:

1. **Wind sea:** waves that are actively growing because the wind speed is greater than the speed of the wave. These waves are variable in wave height, period and direction and characterised by short periods (typically 3 s to 15 s). The sea surface is irregular.
2. **Swell:** waves that have propagated away from the zone where they were generated. The local wind has little or no influence on them. Swell is characterised by a more regular pattern (more uniform wave height, period, and direction) than wind seas, with longer periods (typically 10 s to 30 s).

Box 7.18 *Wave parameters*

Significant wave height (H_s): ($H_s = H_{1/3}$): average height measured crest to trough of the highest one-third of the waves over about a 30 min record. Most measurements calculate significant wave height from the variance of the sea surface or integration of the wave spectrum, referred to as the zero-moment wave height $H_{mo} = 4(m_o)^{1/2}$, where mo is the zero moment of the frequency wave spectrum. $H_{1/3}$ and H_{mo} are equivalent in deep water but in shallow water at pre-breaking depths, $H_{1/3}$ may be up to 30 to 50 per cent larger (Thompson and Vincent, 1985).

Root-mean-square wave height: root-mean wave height defined by the zero-down crossing (or up-crossing) method in the time domain. $H_E = (8\ m_o)^{1/2}$ defined in the spectral domain, and may also be referred to as the mean energy wave height. These estimates may differ by several per cent or more, particularly in the near-shore zone.

Peak wave period (T_p): wave period corresponding to the frequency of the maximum value (peak) of the frequency spectrum, f_p, where they are related by $T_p = 1/f_p$.

Mean wave period (T_m): average wave period defined by the zero-down crossing method in the time domain or $T_{m02} = (m_o/m_2)^{1/2}$ defined from the zero and second moments of wave spectrum.

Peak wave direction (θ_p): direction of the waves corresponding to the peak of the directional wave spectrum. Wave direction is generally defined based on the direction from which the wave propagates. For sea-states of coexisting wind waves and swells, multiple peaks may appear in the spectrum.

Mean wave direction (θ_m): vector mean wave direction calculated from the directional wave spectrum.

Wavelength (L): distance between wave crests. Wavelength is a function of period, T, and water depth, d. In deep water, where d is larger and satisfies certain depth criteria, the deep water wavelength (L_o) is:

$$\boxed{L_o = \frac{gT^2}{2\pi}} \tag{7.52}$$

Waves interact with the bottom if the water depth is less than $L/2$. The general equation for wave length (L) in any water depth is:

$$\boxed{L = L_o \tanh \frac{2\pi d}{L}} \tag{7.53}$$

The wave length can be approximated as (Fenton and McKee, 1989):

$$\boxed{L = L_o \left(\tanh \left[\left(\frac{2\pi}{T} \left(\frac{d}{g} \right)^{0.5} \right)^{1.5} \right] \right)^{0.667}} \tag{7.54}$$

Maximum wave bottom orbital velocity (u_{bmax}): important during flooding when a levee is exposed to waves at an angle that can produce localised scour and erosion.

$$\boxed{u_{bmax} = \frac{H_{m0}\pi}{T} \frac{1}{\sin h \dfrac{2\pi d}{L}}} \tag{7.55}$$

Both wind sea and one or two swells are commonly present in coastal sites. Large wind waves are generated by storms, and the storm climate for a given site may include populations of both tropical cyclones (typhoons or hurricanes) and extra tropical storms. Extreme storms can produce significant wave heights of 12 m to 18 m, and peak periods of 15 s to 20 s in exposed coastal locations.

Wave direction relative to the shore and local bathymetry influences how waves transform. Directional spreading of waves is generally narrower in shallow water than in deep water due to wave refraction. This change should be taken into consideration when evaluating processes resulting from waves in shallow water. When wind sea and swell coexist, wave spectra exhibit multiple peaks, often termed bi-modal (two peaks) or multi-modal (multiple peaks) spectra.

Wave calculations can generally be broken into two regions:

1 **Generation:** wave generation dominates, little interaction with bathymetry.

2 **Transformation:** wave transformation processes (refraction, reflection, shoaling, breaking, dissipation) dominate as a result of strong interaction with bathymetry and coastal structures.

7.4.3.1 Wave generation

The characteristics of waves offshore can be obtained by field measurements, or predicted by simple methods or numerical models:

- **field measurements** are an excellent source of wave information but measurement locations are generally widely spaced and many records are too short to represent the wave climate for a location. National data collection programmes provide measurement resources. Section 7.4.6 provides some sources of local information
- **simple prediction methods** may be used for enclosed water bodies. Storm wave heights and periods may be estimated from wind speed, wind direction and fetch length using simple fetch-limited wave growth equations. An example calculation is presented in Box 7.19. Wind speeds should be adjusted to the standard 10 m elevation, and wind averaging times should be adjusted to find maximum wave conditions. Wave growth may be limited by how long the wind blows as well as fetch
- **numerical wave generation models.** For levees adjacent to the open ocean or a large sea, simple fetch-limited wave predictions methods are not appropriate because fetch is limited by storm size and basin geometry, wind fields are inhomogeneous, and waves propagate from distant storms (swell). So, wave generation models are generally used to hindcast wave conditions for extreme storms or representative climate. The evolution of wave energy density spectra is modelled with the energy balance equation using inputs of wind fields, bathymetry, and wave spectra at open ocean boundaries. Wave generation models require significant computational resources to run, require detailed input of bathymetry and wind fields throughout the basin, and should be validated with data from previous events.

Box 7.19 Fetch-limited wave growth

1 **Wind speed and direction:** estimate wind speed and direction from wind measurements or other climatic information. Wind speeds should be adjusted to a 10 m elevation:

$$U_{10} = U_z \left(\frac{10}{z}\right)^{\frac{1}{7}} \tag{7.56}$$

Where U_{10} is the 10 m wind speed, and U_z is the wind speed measured at elevation z (in metres).

Wind observations may be averaged over very short time periods (s to min).

Winds should be averaged over a period long enough for fetch-limited waves to develop (typically tens of minutes to an hour). Wind durations can be converted using:

$$\frac{U_t}{U_{3600}} = 1.277 + 0.296 \tanh\left(0.9 \log_{10}\left[\frac{45}{t}\right]\right) \qquad 1\text{ s} < t < 3600\text{ s} \tag{7.57}$$

$$\frac{U_t}{U_{3600}} = 1.5334 - 0.15 \log_{10} t \qquad 3600\text{ s} < t < 36{,}000\text{ s} \tag{7.58}$$

where t is the averaging time

2 **Fetch:** fetch X can be measured from a map. Starting from the levee, draw radials extending to the opposite shoreline. Fetch should be averaged over approximately 15° arcs. Wave height is a function of fetch and wind speed, so if wind speed varies with direction, multiple combinations of wind speed and fetch should be tested.

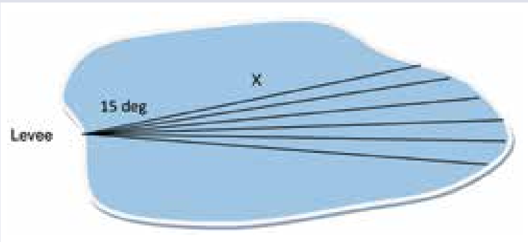


Figure 7.46 Measurement of fetch

3 **Apply fetch-limited wave growth equations** (Resio *et al*, 2008), apply consistent units for lengths (m), velocities (m/s), and times (s):

a Drag coefficient: $$C_D = 0.001(1.1 + 0.035 U_{10}) \tag{7.59}$$

b Wind friction velocity: $$u_*^2 = C_D U_{10}^2 \tag{7.60}$$

c Wave height: $$H_{mo} = 0.0413 \frac{u_*^2}{\sqrt{g}} X^{1/2} \tag{7.61}$$

d Wave period: $$T_p = 0.751 \frac{u_*^{1/3}}{g^{2/3}} X^{1/3} \tag{7.62}$$

4 If high wind durations are relatively short, duration t can be converted to an equivalent fetch and applied to the equations above:

$$X = 0.00523 g^{1/2} u_*^{1/2} t^{3/2} \tag{7.63}$$

7.4.3.2 Wave transformation

Wave transformation processes are very site specific and are driven by the interaction of waves with the local bathymetry and coastal structures within water depths less than half the wavelength.

The transformation processes include:

1. **Shoaling:** as waves propagate into shallower water the wave length decreases while the wave period stays the same, so the wave velocity (L/T) is reduced. The wave group velocity (speed at which the wave energy travels) also decreases with water depth, and as the flux of wave energy is conserved, the wave height increases to compensate for the decreased speed. An example calculation of linear shoaling is presented in Box 7.20.
2. **Refraction:** in shallow water, wave crests tend to align themselves with the bottom contours due to the decrease in wave velocity with depth (see Figure 7.47). If the wave is propagating normal to the bottom contours, no refraction will take place. If the bed contours are homogeneous along the shore, the change in wave angle can be calculated from Snell's Law (see Box 7.21). If the bathymetry contours are complex due to the presence of irregular bars, canyons etc the refraction patterns are also complex. Waves tend to focus energy in shallow water areas and defocus energy in deep water areas. Numerical models should be used in complex regions.
3. **Diffraction** is the process where wave energy spreads along the wave crest from a region of larger wave heights to lower wave heights, eg behind a breakwater (see Figure 7.48). Diffraction can be estimated from diffraction diagrams (Vincent *et al*, 2002) or with numerical models.
4. **Reflection** is the process where waves near steep/smooth structures may be reflected back toward the incident wave field, increasing wave heights and velocities by creating standing waves. Wave heights in front of a smooth, vertical wall can be twice those of incident waves.
5. **Breaking** occurs in very shallow water when the wave height is approximately equal to the water depth, resulting in dissipation of wave energy (see Figure 7.49). The ratio of wave height to water depth (d) at the incipient breaking point is a function of the wave steepness (H/L) and the bottom slope (β). Low steepness waves or steeper bottom slopes results in a larger breaker index (H/d). A typical wave height to water depth ratio (H/d) is 0.8 for incipient breaking of an individual wave and 0.6 for an energy-based wave height (H_{mo}). The wave height within the surf zone (region of wave breaking) can be estimated as being limited by the breaker index ($H_{mo} \leq 0.6d$). Broken waves travelling across a shallow region in front of a levee reform with a height of approximately $0.4d$.
6. **Dissipation** can result from interaction with a very rough seabed, such as coral reefs and submerged vegetation, or with fluidised mud on the bed. The inclusion of these effects generally requires measurements to calibrate dissipation coefficients.

Caution

The simple rules above for depth limited wave height can be used for short storm waves (not swell) and for low steepness bed slopes. If, however, the bed slope is steeper than 1:30, or the relative depth (d/gT^2) <0.01, then the depth-limited wave heights may be larger.

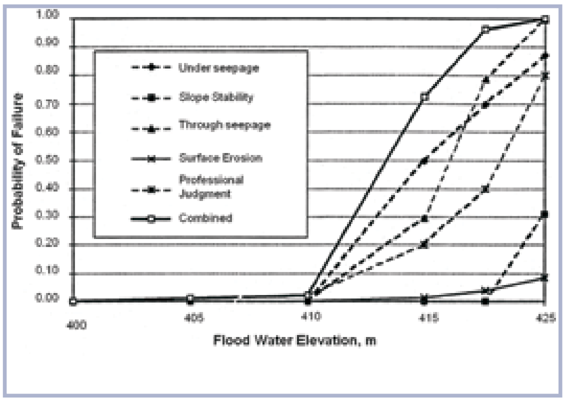


Figure 7.47

Wave refraction, shoaling, and breaking (courtesy USACE)

Figure 7.48

Wave diffraction at Sunderland breakwater entrance (courtesy William Allsop)

Box 7.20 *Calculation of shoaling*

Wave shoaling can be calculated by linear wave theory for monochromatic waves:

$$H_2 = H_1 \sqrt{\frac{C_{g1}}{C_{g2}}} \tag{7.64}$$

where the subscripts 1 and 2 refer to values at different water depths (eg offshore and near-shore) and the group velocity is given by:

$$C_g = \frac{1}{2}\frac{L}{T}\left(1 + \frac{\frac{4\pi d}{L}}{\sinh\frac{4\pi d}{L}}\right) \tag{7.65}$$

The definition of the wave length L was presented in Boxx 7.18.

Figure 7.49 *Waves breaking and reforming on northern California coast (courtesy Luc Hamm)*

Box 7.21 *Calculation of wave refraction using Snell's law*

Assuming bed contours are homogeneous along the shore, wave refraction of monochromatic waves can be calculated as follows:

$$\frac{\sin\theta_1}{C_1} = \frac{\sin\theta_2}{C_2} \tag{7.66}$$

Where the subscripts 1 and 2 refer to values at different water depths, θ is the angle between the bottom contour and the wave crest, and C (= L/T) the wave velocity.

The change in wave height (H) due to refraction is given by:

$$H_2 = H_1 \sqrt{\frac{\cos\theta_1}{\cos\theta_2}} \tag{7.67}$$

In the case of natural sea states, wave refraction is modulated by the directional spreading.

7.4.3.3 Numerical wave transformation models

In areas with complex near-shore bathymetry, wave transformation models provide an excellent tool for transforming random waves from deep waters offshore to the levee. Offshore boundary conditions may be applied from measurements or from a wave generation model hindcast/forecast. Models require near-shore bathymetry at a resolution that adequately characterises bathymetric features and shoreline variations. Additional inputs include local winds, water levels (tides and storm surge), and bottom roughness. Model outputs typically include wave heights, periods, directions, and wave spectra. Models can be broken into three categories:

1. **Phase-averaged models** (similar to wave generation models) do not represent individual waves, but the wave energy spectrum, including the processes of wind input, shoaling, refraction, breaking and dissipation. Diffraction is not included rigorously.

2 **Linear phase-resolving models** neglect local wave generation, but generally include shoaling, refraction, breaking, dissipation, diffraction and reflection.

3 **Nonlinear phase-resolving models** (Boussinesq-type models) also include the nonlinear deformation of waves and growth of low-frequency (infragravity) waves that contribute to run-up and overtopping, but require significantly more computation resources than linear models.

Generally, wave transformation models should be set up and run specifically for individual projects. Validation with field data for a specific site is recommended.

7.4.3.4 Modelling of wave breaking and its effects

When the modelling of energy dissipation due to depth-induced breaking in shallow water is important, it is recommended to simultaneously compute the variations in still water level (SWL) that result from wave breaking, set-down and set-up (see Section 7.4.2). For some phase-resolving models, these variations of SWL are computed within the wave model itself using the same equations. For phase-averaged models, the set-up is computed from the momentum and continuity equations, and so it is necessary to iterate between the wave and water level computations or couple the computations because variations of SWL affect the wave transformation.

7.4.4 Tsunamis

Tsunamis are gravity waves generated by earthquakes, landslides or volcanic activity. Tsunamis are characterised by wave periods that are in the order of minutes rather than seconds (typically 10 to 60 minutes). They generally originate from earthquakes below the ocean where water depths can be more than 1000 m and may travel long distances. In deep water the height of the tsunami may be very small (<1 m). However, as it approaches the coastline its height may increase significantly. Because of their large wavelength, these waves are subject to strong shoaling and refraction effects. The tsunami increases in height and steepness with complicated currents and multiple wave trains. Depending on beach slope and bathymetry, coastline irregularity, and incident wave direction, long-period waves may even propagate in the long-shore direction and become trapped on the continental shelf due to reflection and refraction. This phenomenon is called 'edge waves'. Reflections from adjacent shorelines may affect the number of tsunami waves and their amplitudes. One of the most damaging aspects of tsunamis is the run-up of water on a coastline. Vertical elevation of the run-up can be as high as 30 m in some cases due to focusing by the bathymetry and topography.

Approaching from quite large water depths, tsunami propagation can be calculated using shallow water theory. Wave reflection from the relatively deep slopes of continental shelves may also be an important consideration.

Some theoretical work is available (eg Wilson, 1963), as well as numerical models to describe tsunami generation, propagation and run-up over land areas (eg Shuto, 1991, Yeh *et al*, 1994, and Tadepalli and Synolakis, 1996) and also some large-scale experiments (eg Liu *et al*, 1995, and Rosetto *et al*, 2011).

7.4.5 Marine currents

Although waves are usually the dominant loading in a marine environment, currents should also be considered. Principal types of currents in marine and estuarine environments are:

- tidal currents
- river discharge
- wind induced currents
- density currents
- wave induced currents (ie long-shore currents)
- ocean circulation currents caused by the Coriolis effect induced by rotation of the earth.

On the coast, excluding estuaries, levees are mainly affected by wave-induced currents, which are presented in this section.

Currents depend on many factors and can have great geographical variability. They are strongly influenced by wind and local terrain under inundation situations, bathymetric variation, and by obstructions and constrictions to flow. Section 8.2 discusses overtopping and overflow situations and currents at transitions from higher to lower elevation features, and at transitions from hardened structures to earthen levees. The presence or absence of vegetation also influences currents.

7.4.5.1 Wave-induced currents

As waves break, the wave momentum parallel to the shore drives a long-shore current. A method of calculation with an example is presented in Box 7.22.

Box 7.22 Estimation of wave-driven current

An estimate of the velocity of the wave driven current in the mid-surf zone is given by (Komar and Inman, 1970):

$$V_{mid} = \sqrt{gH_s}\,\sin\theta_b \cos\theta_b \qquad (7.68)$$

where:

V_{mid} = mid-surf zone long-shore current
H_s = breaking wave height (m)
g = gravitational constant (m/s^2)
θ_b = breaking wave angle relative to shore normal (deg)

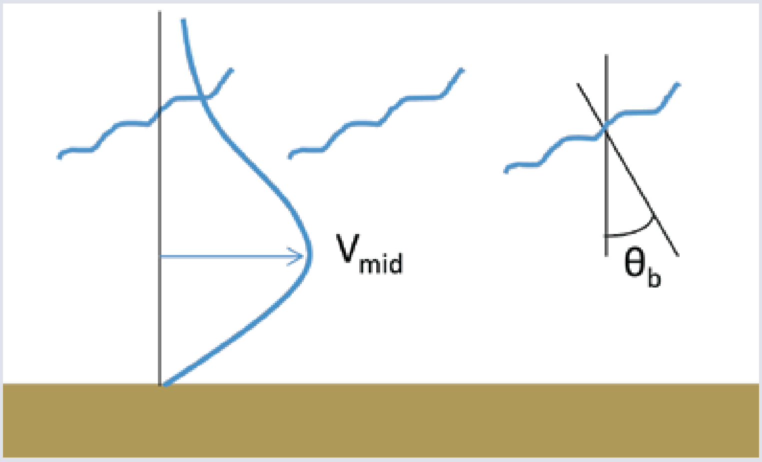


Figure 7.50 Relationship between mid-surf zone long-shore current and incident wave characteristics

Example:

For a breaking wave height of 2 m and a breaking wave angle of 10° relative to shore normal, the mid-surf zone long-shore current estimate would be 0.8 m/s.

Modern circulation models linked to wave transformation models can simulate wave-driven currents for complex bathymetries. When wave directions are near-shore-normal or there are long-shore irregularities in the bathymetry, near-shore circulation cells can develop with offshore directed rip currents.

7.4.6 Sources of global and local data

It is very important that reference datums are explicitly defined as they vary from country to country, and sometimes between different authorities within the same country. It should also be noted that chart datum (CD) levels vary between sea areas, and that the relation between the national land datum to CD will vary from chart to chart. There is further discussion of datum in Sections 7.4.2.1 and 7.9.1. A selection of national fixed land datum levels is presented in Table 7.33.

Table 7.33 National fixed land datum levels

Zone (continent, country or region)	Symbol	Name	Description
France national fixed datum (metropolitan territory)	NGF-IGN69	Nivellement Général IGN69	The 'zero' was established as the average sea level in Marseille for the period between 1884 and 1897.
Germany	DHHN, NHN	Deutsches Haupthöhennetz (German main elevation net)	On the basis of NN, revised, eg in 1992, 2004 (DHHN1992, DHHN2004).
	HN	Höhen Null	Old Surface elevation system referring to the Kronstadt (St. Petersburg, Rus) Sea Level Gauge (in the federal states of the Eastern part of Germany).
Ireland	OD	Belfast Ordnance Datum Malin Ordnance Datum	In Northern Ireland, OD for the Ordnance Survey of Northern Ireland is Belfast Ordnance Datum, the MSL at Clarendon Lock, Belfast between 1951 and 1956 In the Republic of Ireland, OD for the Ordnance Survey of Ireland is Malin Ordnance Datum: the MSL at Portmoor Pier, Malin Head, County Donegal, between 1960 and 1969.
The Netherlands	NAP	Normal Amsterdams Peil	Surface elevation system based on (normalised) Amsterdams Peil (dating from 1684).
	NN	Normal Null	Surface elevation system referring to the Amsterdam (NL) sea level gauge.
UK	OD(N)	Ordnance Datum (Newlyn)	ODN (Ordnance Datum Newlyn), defined as the MSL at Newlyn in Cornwall between 1915 and 1921. Before 1921, OD was taken from the level of the Victoria Dock, Liverpool (ODL). The difference between the different data varies across the country.
USA	NGVD63 or NGVD29	National Geodetic Vertical Datum	Fixed surface whose elevation does not change with time, revised in 1963.
	NAVD88	North American Vertical Datum	Fixed surface whose elevation does not change with time, revised in 1988.
USA/Canada Great Lakes	IGLD85	International Great Lakes Datum	Great Lakes and St. Lawrence River, revised in 1985. The 'zero' of the datum was established as the average of all hourly water surface water level readings at Pointe-au-Pere, Quebec, located on the Gulf of St. Lawrence for the period between 1941 and 1956.
	LWD	Low Water Datum	Established in 1933.

Tables 7.34 and 7.35 provide details of sources of global and national data on the wind and hydrodynamic measurement, and hindcasting and forecasting, respectively.

Table 7.34 Data sources for wind and hydrodynamic measurements

Zone (continent, country or region)	Source	Tides, currents, water levels	Wind and pressure	Waves and tsunamis
Global	Global Sea Level Observing System: **www.gloss-sealevel.org/data/**	X		
Global satellite coverage	European Space Agency: **www.globwave.org/**			X
Global/USA	US National Oceanic and Atmospheric Administration, National Data Buoy Center: **www.ndbc.noaa.gov/**		X	X
Australia	Australian Bureau of Meteorology: **www.bom.gov.au/marine/index.shtml**	X	X	X
Belgium	Flemish Banks Monitoring Network: **www.meetnetvlaamsebanken.be/Default.aspx?Page=&L=en**	X	X	X
France	Service Hydrographique et Océanographique de la Marine: **http://refmar.shom.fr/**	X		
	Centre d'études techniques maritimes et Fluviales: **http://candhis.cetmef.developpement-durable.gouv.fr/**			X
Germany	German Federal Maritime and Hydrographic Agency: **www.bsh.de/de/Meeresdaten/Beobachtungen/Seegang/index.jsp**	X		X
Italy	Istituto Superiore per la Protezione e la Ricerca Ambientale: **www.telemisura.it/** **www.mareografico.it**	X	X	X
Japan	Japan Oceanographic Data Center: **www.jodc.go.jp/index.html**	X	X	X
The Netherlands	Rijkswaterstaat: **www.rijkswaterstaat.nl/geotool/waterhoogte_tov_nap.aspx** **www.helpdeskwater.nl/onderwerpen/monitoring/landelijk-meetne**	X		X
	www.knmi.nl/waarschuwingen_en_verwachtingen/maritiem/golfhoogtes_en_waterstanden.html	X		X
	Royal Netherlands Meteorological Institute (KNMI) and National Institute for Coastal and Marine Management: **www.knmi.nl/samenw/hydra/cgi-bin/freqtab.cgi**		X	
Spain	Puertos del Estado: **www.puertos.es/en/oceanografia_y_meteorologia/redes_de_medida/index.html**	X	X	X
UK	British Oceanographic Data Centre: **https://www.bodc.ac.uk/data/online_delivery/ntslf/**	X		
	National Oceanography Centre, National Tidal and Sea Level Facility: **www.pol.ac.uk/ntslf/**	X		
	National Centre for Ocean Forecasting: **www.ncof.co.uk/**	X	X	X
UK (Coast)	Channel Coastal Observatory: **www.channelcoast.org/data_management/real_time_data/charts**	X	X	X
	WaveNet: **www.cefas.defra.gov.uk/our-science/observing-and-modelling/monitoring-programmes/wavenet.aspx**			X
UK (Off shore)	UK Met Office: **www.metoffice.gov.uk/weather/marine/observations/map.html**	X	X	X

Table 7.35 Data from hindcasting and forecasting

Zone (continent, country or region)	Source	Tides, currents, water levels	Wind and pressure	Waves and tsunamis
Global	Ocean Monitoring and Forecasting: **www.myocean.eu/web/24-catalogue.php**	X		
Europe	European Centre for Medium-Range Weather Forecasts: **www.ecmwf.int/products/**		X	X
France	Prévimer: **www.previmer.org/**	X	X	X
Germany	Bundesamt für seeschifffahrt und hydrographie: **www.bsh.de/en/index.jsp**	X	X	X
Japan	Japan Meteorological Agency: **www.jma.go.jp/jma/indexe.html** Japan Coast Guard: **www1.kaiho.mlit.go.jp/jhd-E.html**	X	X	X
The Netherlands	Rijkswaterstaat: **www.meetadviesdienst.nl/nl/water-en-weer/verwachtingen-water/noordzee/wind-en-golven.htm**		X	X
UK	National Centre for Ocean Forecasting: **www.ncof.co.uk/**	X	X	X
USA	National Ocean Survey, National Oceanic and Atmospheric Administration (Forecast models): **http://tidesandcurrents.noaa.gov/** **http://polar.ncep.noaa.gov/waves/download.shtml**	X		X
	USACE: **http://wis.usace.army.mil/**		X	X
	Ocean weather: **www.oceanweather.com/metocean/grow/index.html**		X	X

7.4.7 Analysis of extreme water levels and extreme waves

For risk analysis as well as for design, the knowledge of the probabilities of extreme water levels and of extreme waves is fundamental. Methods to obtain these are presented here. Section 7.4.8 then provides techniques for evaluation of the joint probability.

7.4.7.1 Coastal still water level analysis/prediction processes

Extreme water levels are expressed relative to the MWL, local chart datum (CD) or national land datum. To determine the extreme water levels all components of the water level should be determined as a function of the (average) probability of exceedance, alternatively expressed as average exceedance frequency or return period. Such exceedance curves are based upon a long-term distribution curve, obtained by fitting water level data to a standard statistical distribution. However, the lack of data for long return period events means that extrapolation is usually necessary. It is not recommended to extrapolate the return period more than twice the record length of the measurements. Extrapolation can be supported by numerical modelling of historical or synthetic storms. Synthetic storms are generally required for analysis of tropical storms due to their limited area of impact and scarcity in the data record.

In most cases the extreme water level may include tidal elevations, storm surges or long-period seiches, as appropriate, but excludes localised variations caused by waves (see Sections 7.4.2.3 and 7.4.2.4). The inclusion of wave set-up and long-wave oscillations in the water level depends on the application and on the formula or model. If a formula for the prediction of overtopping (see Section 8.2.1) requires the water depth at the toe of the levee, then all components of water level, including wave set-up and long waves, should be included.

Two methods can be used to determine extreme water levels resulting from the combination of tide and storm surges:

- separation of tides and storm surges
- statistic of the measured total water level.

The first method should be employed when the ratio of the tidal range/storm surge is greater than 1.5 (Haigh *et al*, 2010). Box 7.23 presents an example of extreme still water analysis based on the second method as it is more widely accessible.

Box 7.23 ***Example of extreme still water analysis***

For a site not affected by tropical cyclones, the method involves fitting peak water level data to an appropriate long-term distribution (CIRIA; CUR; CETMEF, 2007), for example the Gumbel distribution:

$P(X \leq x) = \exp\{-\exp(-ax+b)\}$, where a and b are parameters to be found

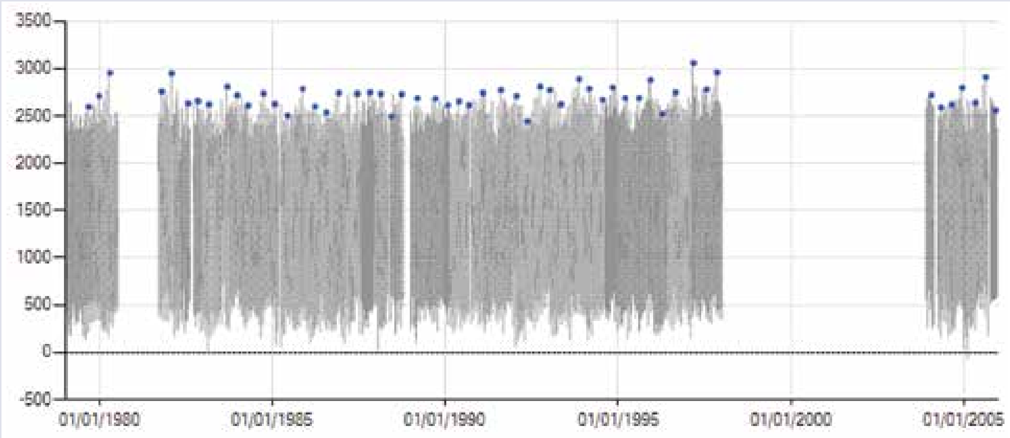


Figure 7.51 ***Source tide gauge data (mm Coastal Datum = mm CD) and dots indicating extracted peak water level data (minimum 100 day separation) (courtesy HR Wallingford)***

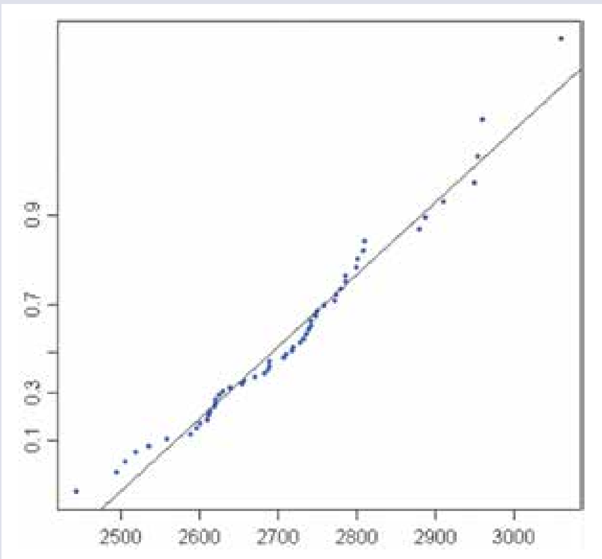


Figure 7.52 ***Gumbel distribution fitted to peak water levels (probability less than against mm CD) (courtesy HR Wallingford)***

Table 7.36 ***Return level estimates (mm CD for different return periods)***

Return period (years)	1	2	5	10	20	50	100
Sea level (mm CD)	2694	2782	2884	2956	3028	3121	3191

When few data on water levels are available for the site of interest, they may be available for a neighbouring location. In that case extremes can be estimated by analogy with extremes estimated at the neighbouring location. This approach should be applied with care and ideally only along open coastlines over which the tide range varies only gradually. Assume that the ratio below is the same at each of the two sites.

{(extreme level) – (mean high water spring level)} to (spring tide range, MHWS-MLWS)

7.4.7.2 Coastal wave climate analysis/prediction processes

For structures exposed to waves, the definition of hydraulic conditions requires knowledge, or estimation, of the probability of large and extreme events. The purpose of the determination of the long-term climate is to associate a wave height to a given return period and, if possible, with a confidence level. Typical return periods are 30 to 200 years with some (high value, or high hazard) structures requiring much longer return periods. In the Netherlands, return periods up to 10 000 years or even larger are used for some flood defences.

Extreme value analysis procedures are usually applied only to significant wave heights. The wave period associated with the return wave height can be determined by referring to empirical joint distributions of wave height and period of extreme wave data.

Extrapolation of the validity of a distribution beyond the range covered by the measurements should be done with care. However, this is generally the only way of predicting low-frequency (long return period) events. The procedure adopted is to fit to a theoretical extreme value distribution and then to extrapolate the fitted distribution to extreme values.

It is not recommended to extrapolate the return period more than twice the record length of the measurements. Often other distributions (not extreme value distributions) are fitted in a pragmatic way, because sometimes extreme value analyses yields non-robust results.

Selection, checking and preparation of data are probably the most important steps in the water level analysis procedure. This includes identifying data gaps, checking data quality, covering periods of record that include climate variability etc.

The cases of extra tropical storms and of tropical storms are addressed differently:

- **extra tropical storm:** for the extraction of storms wave height data, the peak over threshold (POT) method is recommended. The annual maximum method may be employed but the use of the total method sample is discouraged. In the POT method, the storm peak wave height above a chosen threshold (eg H_s = 3 m) for each storm is used in the extreme value analysis. It is recommended that the wave height threshold is selected to achieve an average number of selected data values per year (typically 5 to 10) that is equal to or less than the average number of events exceeding a certain threshold per year (typically 10 to 20). The dependence of the extreme values on the threshold should be investigated, and the outcome of this analysis should be used in the final choice of the threshold
- **tropical storms:** the occurrence of hurricanes that hit any coastal area is far less than 10 to 20 per year. As for the still water level, synthetic storms are generally required for the wave analysis of tropical storms due to their limited area of impact and scarcity in the data record.

Statistics of extreme sea state at a specific site should be established on the basis of measured wave data and/or hindcast wave data that cover as long a duration as possible. The method of wave hindcasting should have been successfully validated with field measurements, including several storms near the site of interest, especially if the near-shore region is highly dissipative due to the presence of soft mud or vegetation. Validation is important to quantify accuracy of and errors inherent in the prediction.

Caution should be exercised to ensure the water depth at which waves have been measured is not so shallow that it imposes an upper limit to the largest wave height due to wave breaking.

Section 4.2.4.9 of CIRIA; CUR; CETMEF (2007) and the Annex B of ISO 21650:2007, gives more information on the analysis of extreme waves and special considerations for shallow water conditions. Box 7.24 gives an example of extreme wave climate analysis based on CIRIA; CUR; CETMEF (2007) and Rogers *et al* (2010).

Box 7.24 ***Example of extreme wave climate analysis***

Extreme value analysis is usually applied only to significant wave heights (H_s) in order to associate a wave height to a given return period. An appropriate probability distribution is fitted to representative high values in the source data, and then extrapolated to extremes. For a site affected by tropical cyclones, usually cyclone and non-cyclone data is analysed separately. There is no theoretical argument in favour of the use of any particular probability density function but the three-parameter Weibull distribution is often used: $P(H \leq H_s) = 1\text{-exp}[-\{(H_s\text{-}a)/b\}^c]$, where a, b and c are parameters to be found. The procedure involves:

- selection of data for analysis, for example by identifying independent peak values from the source data
- fitting of distribution(s) to these data
- computation of return period values from the fitted distribution(s)
- checking and assessment of uncertainties.

The results would look similar to those shown in Box 7.23 but based on the total measured wave heights instead of the measured sea levels.

An alternative approach, which is often used for checking purposes, requires extremes to be estimated directly from the source data. Plot the source H_s data such that probability of exceedance is on a log-scale, extrapolating the best fitted line to rarer probabilities of occurrence and finding the return period estimates at the appropriate probabilities.

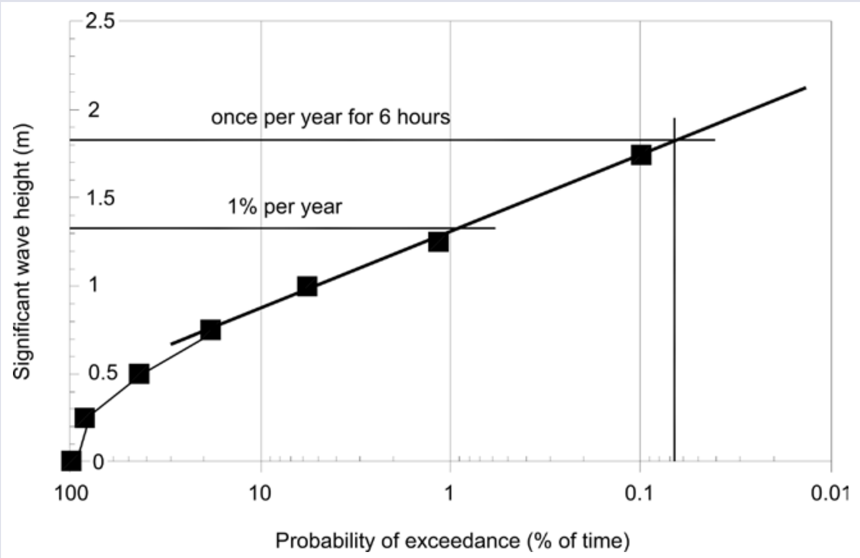


Figure 7.53 ***Example empirical fit to H_s data, showing the position of the estimated one-year return period value (CIRIA; CUR; CETMEF, 2007 and Rogers et al, 2010)***

It is usually necessary to estimate the mean wave period (T_m), often based on the joint distribution of H_s and T_m in the highest few per cent of the source data. Find the average wave steepness for those source data and use that steepness, $(2\pi H_s)/(gT_m{}^2)$, to estimate T_m values corresponding to the extreme H_s values.

7.4.8 Joint probability of waves and water level

This section describes calculation procedures for determining the probabilities of the joint occurrence of extreme water levels and extreme wave conditions. These joint probabilities are important for design and risk analysis.

7.4.8.1 Characterisation of the occurrence of wave and water level

In cases where ample data are available, marine water levels can be calculated reliably for existing conditions. The intermittent nature of wave recording, as well as the variability of wave conditions in coastal waters, makes prediction of extreme wave conditions more difficult and uncertain. An aspect of equal importance in design is characterising the probabilistic occurrence of large waves and high water levels occurring nearly simultaneously. Usually the peak wave and water levels do not occur exactly at the same time but at different times during the extreme event. Large waves and high water levels often occur together due to the following:

- certain weather conditions tend to produce both large waves and high storm surges. The correlation between water level and waves remains modest in areas where the astronomical component of the tide is much larger than the storm surge component. Conversely, it is more significant for areas with lower tidal influence
- the behaviour of waves is influenced by conditions of the near-shore zone. Shallow-water wave transformations depend upon water depth. If the wave prediction point is very close inshore or protected by sand banks, then wave conditions may be depth-limited, in which case there would be a strong correlation between large waves and high water levels.

Joint probability extremes can be calculated and described for both offshore and near-shore conditions. Offshore results are often applicable regionally but may at times be applicable only locally, depending on the variation of the bathymetry and the shape of the coastline. Wave predictions and joint probabilities are often calculated as a function of direction. This is important because general exposure to waves, correlation between large waves and high water levels, and near-shore transformation, may all depend upon storm direction.

The concept of a return period, when dealing with joint probabilities, is less straightforward than when dealing with a single variable. A joint probability extreme event can be defined in terms of the probability that a specific wave height is exceeded simultaneously with a specific water level. For any particular return period, there will be a range of combinations of wave heights and water levels, each of which is expected to be equalled or exceeded once, on average, in each return period. For example, consider a very severe wave condition with a modest water level, or a very severe water level with a modest wave condition – both will occur and both may have the same combined return period. This is of particular importance in considering wave overtopping as it is not necessarily obvious which combination will give the greater overtopping. It can also be noted that the combination of water level and wave condition that gives the highest overtopping may not be the combination that requires the largest armour.

Section 4.2.5 of CIRIA; CUR; CETMEF (2007) gives further information on the joint exceedance probability in independent and dependent cases. The methods of analysis are generally based on scatter diagrams.

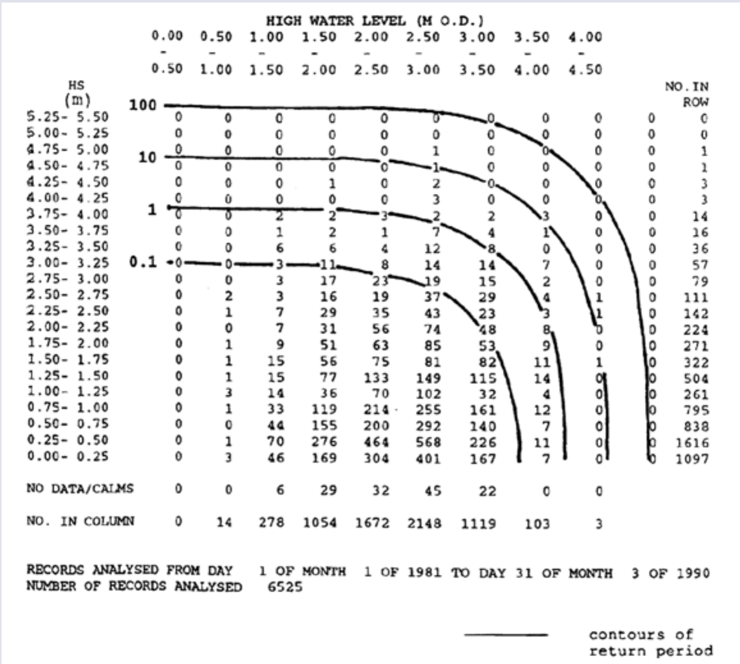
7.4.8.2 Extrapolated joint density approach (bi-variate model)

HR Wallingford and Lancaster University (1998) describe a method of transforming real wave and water level data into idealised bi-variate normal distributions whose dependence characteristics are already well-known. A Monte Carlo simulation method is applied to the transformed distributions, the results being transformed back to equivalent results for the original distributions. Extreme values can then be extracted from the long-term simulation without the need for further extreme analysis. In principle this method is not limited to two variables but is usually implemented with two variables – wave height and water level. Wave period is routinely included as an additional variable dependent on wave height. The coastal responses of interest (eg structural failure, overtopping) are then determined by integration over the joint probability contours. This approach requires specialised software and expertise.

There are many models in the literature for bi-variate distributions. The bi-variate normal model often underestimates the strength of the correlation in the extreme region. Box 7.25 presents an example of a joint probability analysis based on CIRIA; CUR; CETMEF (2007) and Rogers *et al* (2010).

Box 7.25 ***Example of joint probability analysis***

The dependence between large waves and high water levels, and the likelihood that they will occur together, is relevant in estimating joint extremes. The concept of return period is less straightforward than for a single variable. A joint exceedance extreme is defined in terms of the probability that a specific wave height is exceeded simultaneously with a specific water level. For any particular joint exceedance return period, there will be a range of combinations of wave heights and water levels, each of which is expected to be equalled or exceeded once, on average, in each return period. Dependence and joint probability extremes are best assessed with reference to several years of simultaneous wave and water level data. The return period of a particular combination of wave height and water level can be estimated directly from scatter diagram data. The diagram shows about 10 years of high-tide wave height and water level data. The 0.1, 1 and possibly 10-year return period contours can be sketched in manually, so as to capture approximately the correct number (eg, 10 1-year joint exceedance in 10 years of data, and one hundred 0.1-year joint exceedance) of source data in each square that could be drawn with its lower left corner on a return period contour. Probability contours of rarer events can then be drawn, retaining the shape of the know contours. The spacing between the extrapolated contours should be approximately equal for each factor of 10 increases in rarity of event represented. The positions of the contours as they meet the x- and y-axes are fixed by the values of the marginal extremes.


HIGH WATER LEVEL (M O.D.)

HS (m)		0.00 - 0.50	0.50 - 1.00	1.00 - 1.50	1.50 - 2.00	2.00 - 2.50	2.50 - 3.00	3.00 - 3.50	3.50 - 4.00	4.00 - 4.50		NO. IN ROW
5.25- 5.50	100	0	0	0	0	0	0	0	0	0	0	0
5.00- 5.25		0	0	0	0	0	0	0	0	0	0	0
4.75- 5.00	10	0	0	0	0	0	1	0	0	0	0	1
4.50- 4.75		0	0	0	0	0	1	0	0	0	0	1
4.25- 4.50		0	0	0	1	0	2	0	0	0	0	3
4.00- 4.25		0	0	0	0	0	3	0	0	0	0	3
3.75- 4.00	1	0	0	2	2	3	2	2	3	0	0	14
3.50- 3.75		0	0	1	2	1	7	4	1	0	0	16
3.25- 3.50		0	0	6	6	4	12	8	0	0	0	36
3.00- 3.25	0.1	0	0	3	11	8	14	14	7	0	0	57
2.75- 3.00		0	0	3	17	23	19	15	2	0	0	79
2.50- 2.75		0	2	3	16	19	37	29	4	1	0	111
2.25- 2.50		0	1	7	29	35	43	23	3	1	0	142
2.00- 2.25		0	0	7	31	56	74	48	8	0	0	224
1.75- 2.00		0	1	9	51	63	85	53	9	0	0	271
1.50- 1.75		0	1	15	56	75	81	82	11	1	0	322
1.25- 1.50		0	1	15	77	133	149	115	14	0	0	504
1.00- 1.25		0	3	14	36	70	102	32	4	0	0	261
0.75- 1.00		0	1	33	119	214	255	161	12	0	0	795
0.50- 0.75		0	0	44	155	200	292	140	7	0	0	838
0.25- 0.50		0	1	70	276	464	568	226	11	0	0	1616
0.00- 0.25		0	3	46	169	304	401	167	7	0	0	1097
NO DATA/CALMS		0	0	6	29	32	45	22	0	0		
NO. IN COLUMN		0	14	278	1054	1672	2148	1119	103	3		

RECORDS ANALYSED FROM DAY 1 OF MONTH 1 OF 1981 TO DAY 31 OF MONTH 3 OF 1990
NUMBER OF RECORDS ANALYSED 6525

Figure 7.54 ***Example of joint exceedance contours for return periods manually drawn over 10 years of high-tide wave and water level data (CIRIA; CUR; CETMEF, 2007)***

Alternatively, structure variable(s) could be continuously synthesised for whatever period of time series wave and water level data is available. For example, if 20 years of simultaneous wave and water level data were available, they could be used to hindcast rates of overtopping directly at an inshore site of interest. From this a probability distribution could be fitted to the overtopping rates and extrapolated to extreme values.

Analytical methods could be used to fit not only the distributions of each variable, but also the dependences between them. For example, JOIN-SEA includes a method of transforming real wave and water level data into a bi-variate normal distribution whose dependence characteristics are known (HR Wallingford and Lancaster University, 1998). A Monte Carlo simulation method is applied to the transformed distributions and the results are transformed back to equivalent results for the original distributions. Joint exceedance extremes can be extracted directly from this simulation, and equivalent structure variable distributions can be developed, without the need for further distribution fitting.

7.4.9 Uncertainties

With any form of predictive analysis there will be uncertainties in both the input data and the processes. These could include:

1 **Limitation of data and methods of measurement**

Consideration should be given to the limitations of the measurement methods to record the physical data. Data are collected in a hostile environment and technical problems are possible (autonomy, transmission, mechanical breakdown, electrical failure). These problems occur preferentially during storms which can result in an underestimate of the extreme values.

2 **Limitation of statistical analysis**

Uncertainties in the computed extreme values depend mainly on:

- inaccuracy or unsuitability within the source data
- inherent statistical variability, ie sampling variability
- uncertainty due to possible incorrect choice of the extreme value distribution
- uncertainty in the computation of significant wave height due to limited record length
- uncertainty as a consequence of extrapolation, conflicting physical behaviour. For example, wave heights cannot be extrapolated too far when the water depth is limited. A good understanding of the physical processes that can potentially occur is always necessary in addition to statistical extrapolation, especially with storm surges that depend on the trajectory of the storm

3 **Limitation of modelling – physical and numerical**

Physical models have inherent laboratory and scale effects due to:

- wave and flow generation equipment
- model boundaries
- inability to meet all similitude criteria for exact scaling of model material and forces.

Numerical models are only as good as the physics of coastal processes and responses to hydrodynamic forcing contained in the model. These 'physics' are often approximated by empirical expressions. Numerical models are constrained by spatial and temporal discretisation of the model domain and physical processes. Models are also limited by the quality of input conditions (bathymetry, boundary conditions, wind, pressure, wave, and water level forcing). Wave model accuracy is typically 10 to 20 per cent for the parameters of wave height and period. It is critical that the model application is consistent with model assumptions.

Further reading

The following reading is related to the material contained in Section 7.4:

- CIRIA; CUR; CETMEF (2007) *The Rock Manual*
- Rogers *et al* (2010) *The beach management manual*
- USACE (2002) *Coastal engineering manual, EM 1110-2-1100*

This is a good general reference for coastal processes (normal and extreme hydro and sediment/morphology, stabilisation works):

- Dean and Dalrymple (2001) *Coastal process with engineering applications*
- Wozencraft and Millar (2005) *Airborne lidar and integrated technologies for coastal mapping and charting*

7.5 MORPHOLOGY AND HYDRAULIC ACTIONS FOR ESTUARINE LEVEES

The principle concern for estuaries and their effect on levees derives from how their characteristics may influence water levels, velocities and wave heights. Estuaries are located at the transition where rivers enter large bodies of water such as very large lakes, seas, or oceans. They experience both tidal, current and wave dynamics and their flow behaviour is typically characterised by upstream controls from fluvial discharges and downstream controls from tidal and storm surge water level fluctuations. These tidal flows tend to dominate the estuarine morphology. This section outlines some key issues for levees of estuarine morphology but does not go into any detail on the hydraulics, because for levees the influences of currents and waves are similar to those described respectively in Section 7.3 for riverine levees and in Section 7.4 for coastal levees.

7.5.1 Estuarine morphology

An estuary is a transition zone between river basins and the sea, consisting of a complex system of channels, shoals, and flats. Because of the transitional nature of estuaries, processes that influence morphology and hydraulic behaviour originate from both sea and river sources. Sediment entering an estuary may originate from either marine or fluvial material, or both. The presence of tides results in a complex pattern of sediment transportation. Knowledge of the estuary channel and intertidal bathymetry and its morphology is fundamental to the evaluation and design of estuarine levees. Estuaries exhibit a variety of bed patterns and landforms shaped by sediment transport mechanisms. Processes that drive cyclic development and destruction of these patterns are complex and beyond the scope of the handbook. However, three basic types of estuary may be distinguished:

- short estuary with respect to the length of tidal waves
- long estuary with respect to the length of tidal waves
- tidal river.

Various more detailed morphological classifications for estuaries have been proposed, such as the example presented in Box 7.26 developed for the UK context.

In an estuarine environment, morphologic studies should be undertaken to establish an understanding of how the coastal zone responds to normal variations in flow regime (waves and tides) and to extreme events (hurricane, tsunami, or other surges). The stability of the estuarine shoreline depends on the physical character of the shore, which in turn is determined by its geology, its geomorphology and the actions of winds, waves, tides, currents, surges and ice. The morphology of the estuary changes if the applied forces or sediment supply change, which could include human interventions such as dredging. Their impact on the landform depends on the type, magnitude and direction of the forces in combination with the sediment strength. Sediment transport causes morphologic changes and maintains this balance between landform and the hydrodynamic forces.

Box 7.26 Estuary classification (from Dyer, 2002)

Table 7.37 An estuarine classification system as applied to estuaries around England and Wales by Dyer during the Futurecoast study

Type	Origin	Behavioural type	Sub-type
1a	Glaciated valley	Fjord	With spits
1b			No spits
2a		Fjard	With spits
2b			No spits
3a	Drowned river valley	Ria	With spits
3b			No spits
4a		Spit-enclosed	Single spit
4b			Double spit
4c			Filled valley
5		Funnel-shaped	–
6		Embayment	–
7a	Drowned coastal plain	Tidal inlet	Symmetrical
7b			Asymmetrical

This classification is an aid to understanding the particular estuary in which the levee is being placed and was further developed in the Estuary Research Programme funded by UK Government (Defra, 1998–2008).

Glacial valleys:

- fjord – exposed rock platform set within steep-sided relief and with no significant mud or sand flats
- fjard – low lying relief, with significant area of sand or mud flats.

Drowned river valleys:

- rias – exposed rock platform and no linear banks within them
- spit – enclosed estuaries with one or more spits
- funnel shaped – linear banks within them but no ebb/flood delta
- embayments – river or marine in origin with multiple tidal rivers meeting at or near the mouth with a bay width/ length ratio of one or greater and no exposed rock platform.

Drowned coast plain:

- tidal inlet – with barrier beaches or spits.

The classification is not a guide to prediction, although it has been used as the platform for system-based modelling. Instead, it provides a contextual framework for considering the historical development trajectory of an estuary.

7.5.1.1 Approach to undertaking a morphological study in estuary settings

Unique challenges exist in performing a morphological study in an estuary. It is helpful to consider the questions in Table 7.38 which were developed within the Estuaries Research Programme in the UK (Defra 1998–2008).

Table 7.38 ***Questions relating to estuary morphology***

1	How did present morphology arise?	6	Influence of underlying geology?
2	Influence of tidal processes?	7	Influence of associated ecology?
3	Influence of wave action?	8	Impact of sea level rise or climate change?
4	Influence of fluvial processes?	9	Impact of plan or activity on morphology?
5	Influence of sediment supply and dynamics?	10	Influence of changes in water quality?

Analysing each of these questions with access to appropriate quality and quantity of site data will enable a robust conceptual model to be developed, which will generate confidence in the results of a study by reducing uncertainty. It is assumed that the river study and coastal study provide understanding of the bounding elements on the estuary. The studies should seek to provide consistency between the three environments so that there are no spatial gaps in understanding and the results can be combined. Table 7.39 provides information to be considered when performing a morphological study in an estuary environment.

Table 7.39 ***Information to be considered to perform a morphological study of an estuary environment***

Information or data to be considered	
Estuary currents/ waves	• current and wave information for both normal and extreme events • event duration.
Sediment transportation	• the overall sedimentation system should be understood. This includes magnitude of sediment movement and the velocity, as well as seasonal and climatic effects • determine the regional sediment availability to be moved.
Existing works	• the influence of the existing works (eg port facilities) on the natural morphological process should be evaluated. This needs to be undertaken not only at the site of interest but also up and down estuary from the site in order to assess the functions of the entire system and evaluate the effects of changes to that system.
Geology	• both the onshore and offshore geological deposits should be evaluated as they provide natural replenishment sources to the foreshore deposits • geology also may provide constraints on long-shore development and vertical changes in near-shore and beach profile.
Bathymetric and topography data	• understanding the geometry of the estuary channel and foreshore areas/shoreline is a reasonable first approach to developing a map and prediction of future processes • LiDAR and bathymetric surveys together with aerial photographs are commonly used to map large areas • comparison of repeat coverage surveys allows a rapid determination of changes that have occurred in the area in the intervening time interval • dredging records at port facilities or in navigation channels provide a useful indication of sedimentation rates.

The data collected should be capable of differentiating changes resulting from:

- **'normal'** estuary processes (tidal currents and water levels, sediment erosion, transport and deposition, wave-induced stirring of sediment, effect of river flow, net sea level change)
- **'extreme'** estuary processes (tide and surge currents and water levels, sediment erosion, transport and deposition, large internally generated sea waves, surge inundation and passage of sea swell into the estuary, breach of natural barrier systems, channel avulsion or channel switching of tidal channels).

For further guidance on data collection, refer to Table 7.18.

While long-term variation in climate change creates questions regarding accuracy of the historic datasets, a robust conceptual model built with good quality data, and an understanding of the sediment budget, helps to reduce the uncertainty in understanding future events. It is critical to include this dynamic in the procedures for predicting future processes.

7.5.2 Hydraulic actions on estuarine levees

Determining hydraulic actions on estuarine levees involves a combination of river and coastal analysis techniques.

Wave effects need to be examined to determine how significant they are. In some areas of the world the primary forces developing the hydraulic loads that act on levees around very large estuaries or estuaries with a significant coastal exposure can be analysed using coastal methods for calculating water levels and loads from tide, surge, and wave overtopping. In other areas, externally generated waves may not easily find their way inside the estuary, depending on orientation to the prevailing wind and the local coastline shape and form of estuary at the mouth. Waves may, however, be generated internally within the estuary and these will depend on the combined wind field and fetch distances. In such conditions, river and tidal flow conditions will dominate and methods associated with developing top of levee elevations for rivers should be used with appropriate adjustments in downstream boundary conditions to reflect the local tides and waves/storm surge in the estuary. Where the consequences for damage are high, the joint probability of critical conditions from both the coastal and tributary river should be evaluated. National approaches to combining river and coastal hydraulic analyses in estuaries may vary between a completely coupled (eg joint-probability) analysis (see Section 7.4.8) to using a rather conservative approach where events of the same frequency are combined.

Water levels, which may be experienced by levees, are affected by the bathymetry and shape of the estuary. In some cases, tidal propagation and tidal range may be amplified by the shape of a bay or estuary. Similarly, surge levels may also be amplified as the surge passes along the estuary. Sub-tidal bathymetric features such as channels and banks can also steer flow in concentrated streams, which vary with time as the morphology changes. In the upper and middle reaches of the estuary river flow can also be important in increasing ebb currents at low tidal levels. Further information on estuarine hydraulics is available in standard texts (eg McDowell and O'Connor, 1977). The combined effect of tide, surge and river flow on currents and water level is often best assessed using an appropriate computational (1D, 2D or 3D) model.

In complex situations, use of numerical models is advised. Here, a circulation model, including the generation of tidal, fluvial and wind-driven currents, and storm surge is typically coupled with a wave model, which includes the generation and transformation of wind waves. The circulation model provides water levels and currents to the wave model and the wave model provides wave stresses to the circulation model to calculate wave set-up and wave-driven currents. ADCIRC (Luettich and Westerink, 2004) is an example of a model used for tidal and storm conditions in near-shore regions. It is a 2D, depth-integrated, finite element, ocean circulation model. It predicts circulation to evaluate changes in water surface elevation and depth-averaged velocity patterns. A wave model can be coupled with ADCIRC to provide the interaction between the waves, water levels and near-shore currents.

7.6 HUMAN ACTIONS ON LEVEES

Levees will experience various human actions throughout their life cycle, and these should be taken into account in assessment and design processes

Third party human interaction

Third party human interactions with levees can be subdivided as indicated in Table 7.40. All the actions should be considered in combination with the imposed hydraulic loads.

Table 7.40 Loads arising from third party human interaction

Loading category	Load type	Suggested loading assumptions or restrictions
Recreational use	Local pressures from feet/vehicle tyres cause rutting leading to accelerated deterioration and lowering of crest level below the design level.	Provide surfacing for traffic (gravel or reinforced grass path).
Terrorist or other attacks	Deliberate damage (lowering of crest).	Manage by routine inspection. In critical areas harden defences to reduce vulnerability to damage.
Boat traffic	Waves and currents generated by boat traffic.	Guidance is available in Section 7.3.11 and CIRIA; CUR; CETMEF (2007)
Unplanned structures	Unplanned development by local population (boundary walls and buildings).	Put measures in place to ensure that they cannot occur.
Transport infrastructure loads	See Box 7.27.	
Construction, maintenance and operational activities	See Section 7.6.2.	

Box 7.27 UK allowances for transport infrastructure vehicle loads on levees

UK (road)

Loads from road vehicles can be approximated to uniformly distributed loads (Table 70.6, ICE, 2012):

- 10 kPa – normal roads
- 20 kPa – trunk roads and motorways
- 37.5 kPa – routes with exceptional traffic

UK (rail)

The equivalent load for normal rail traffic is 133 kN/m, increasing to 150kN/m for heavy rail traffic (Table 6.1, BS EN 1991-2:2003). These are equivalent to a 2.6 m wide (typical width of a UK railway sleeper) strip load of 51 kPa and 58kPa, respectively.

Table 3, BS 6031:2009 suggests a strip loads over the area occupied by the track of:

- 30 kPa – light rail and underground
- 50 kPa – standard UK railways.

7.6.1 Construction, maintenance and operational actions on levees

The main categories of construction, operation and maintenance loadings are given in Table 7.41.

Loadings on levees arising from construction, operation and maintenance activities should not be assumed to arise independently from hydraulic loads. Typical load combinations that might be considered include:

- plant loads and stockpiles acting in combination with the design hydraulic load in situations where unscheduled or emergency works may be required during a flood event
- all other loads acting in combination with the imposed hydraulic loads.

If construction, routine maintenance and emergency response activities are to be carried out safely, it is desirable that loading assumptions used during design (for example, Box 7.28) are shown on drawings and in the project operation and maintenance manual. These may include limits on the size of equipment that is allowed to operate on the levee and limitations on future modifications to the as-built geometry of the levee. Further discussions on typical plant used in works on levees are presented in Section 10.4.3.

Table 7.41 Loads arising from construction, maintenance and operations

Loading category	Load type	Suggested loading assumptions or restrictions
Plant loads (for further discussions on types of plant see Section 10.4.3)	Construction and operation plant loads	Assume an imposed uniformly distributed load on all or relevant horizontal parts of a levee, such as the crest and berm, in a combination that has the most adverse effect on stability. The intensity of the load varies with country practice (Box 7.28). At the design stage these loads should be checked against manufacturer's data sheets and local design standards and codes.
	Maintenance plant, including grass cutting equipment	For safety reasons, state limits on the size of equipment in the operations and maintenance manual.
Temporary stockpiles	Construction materials	Avoid stockpile loads on levees during both construction and maintenance/operation by providing 'lay down' areas for stockpiling materials and plant in areas outside the levee footprint, unless they can be shown not to impact on the stability and performance of the levee.
	The adjacent landowner may use the levee as a working space and for stockpiling materials	Avoid these loads by imposing restrictions on stockpiling by landowners.
Loads arising from changes in geometry of the levee (and adjoining channel or beach)	***During construction works***	
	Over filling (surcharge) to accelerate settlement/facilitate edge compaction. Loads from armouring/revetment	Design to include constructability issues (in the UK the Environment Agency (2011) require designers to provide a 'buildability statement' with the design and specification for capital works, stating what the designer assumes are the constraints on construction methods).
	Raising crest level to allow for anticipated future settlement	Drawings should show what allowance has been made for construction settlement in the design.
	Programmed future raising	
	Crest raising as a reactive approach to climate change	Drawings and design to state to what extent and how the design has allowed for this.
	Maintenance and operation	
	Unplanned removal of berms/excavations at levee toe (service trenches)	Purpose of berms to be defined on drawings (long-term stability/seepage or whether only short-term stability/landscaping/spoil disposal/stockpile for future topping up of the crest).
	Enlargement of landside drainage channel	Consider defining a no-excavation zone at the toe of the levee on the drawings.
	Channel dredging, protection works/removal of silt deposited unevenly on channel bank	Allow for a minimum lowering of river bed level of 0.5 m for future dredging, or as agreed with the navigation authority for the river.
	Scour	Determine based on study of morphology of the water body.
Secondary function loads	Service corridor	Show on drawings what and where allowances have been made for services (loads, trench backfill/detailing, cover material, marker tape/pads).
	Crest structures and buildings	These include security barriers (fences or walls), buildings and crest wall, the latter being constructed to increase the standard of protection provided by the levee. They have self-weight and can be acted on by wind, hydraulic loads and the impact of floating debris to generate a secondary load. All may trap debris, further increasing the secondary loads.
Crest raising	Planned raising – demountable defences	Defined by design level from hydraulic loading.
	Unplanned emergency raising – sandbags, gabion baskets or other temporary barriers	Normally considered as an unusual load, so check whether reduced safety factor is acceptable.

Box 7.28 *Examples of construction plant loads to be considered during levee design*

General plant loads:

- UK: 10 kPa (Table 3, BS 6031:2009)
- US: 15 kPa (local practice).

Compaction plant:

- 20 kPa acting over a 2 m wide strip at the shoulder of the levee (local practice in some UK design offices).

Crane loads

Where a crane is required to operate on a levee the stability of both the levee and crane need to be assessed. In the UK the loads imposed by cranes and an outline method of assessing their stability is presented in Section 2.5 of Lloyd (2003) and BRE (2004). Refer also to Section 8.6.

Key points include:

- allow enough space for outriggers to be extended – typically 6 m wide
- stay out of the danger zone near slopes, ie stay behind 1V:2H line from toe of slope/retaining wall and at least four times the outrigger foundation width from the top of the slope
- allow for asymmetric loading – assume whole load from crane and load is transferred onto one track/outrigger.

The use of timber spreader mats is no longer seen as good practice in the UK following a fatality where a tracked crane slid off wooden spreader mats, which were covered in frost, into a water-filled drainage channel.

7.7 GROUND INVESTIGATION FOR LEVEES

Characterising the ground on which the levee is founded and the materials from which it is constructed, or to be constructed, through the processes of investigation and monitoring is fundamental to achieving a levee that fulfils the desired serviceability requirements. As with any engineering structure, a designer needs to know and understand the physical properties of the materials that they have to work with and how they will respond under load, whether during construction or in the longer term under normal and extreme operating conditions.

Soils are natural materials and their physical properties are enormously more variable than manufactured engineering materials. Due to the depositional history they can contain localised inclusions where the soil properties are significantly different from the general range of properties that are applicable to the broader expanse of the soil mass. It is these locations where serviceability criterion may not be achieved as a result of localised early deterioration of the levee through processes such as settlement, erosion, seepage or local instability. Additionally there is the potential for failure of the levee to occur, either during construction or under long-term operation.

Section 7.7 provides an overview of the principles of ground investigation for levees. A flow chart mapping the outline structure and contents of Section 7.7 is presented in Figure 7.55.

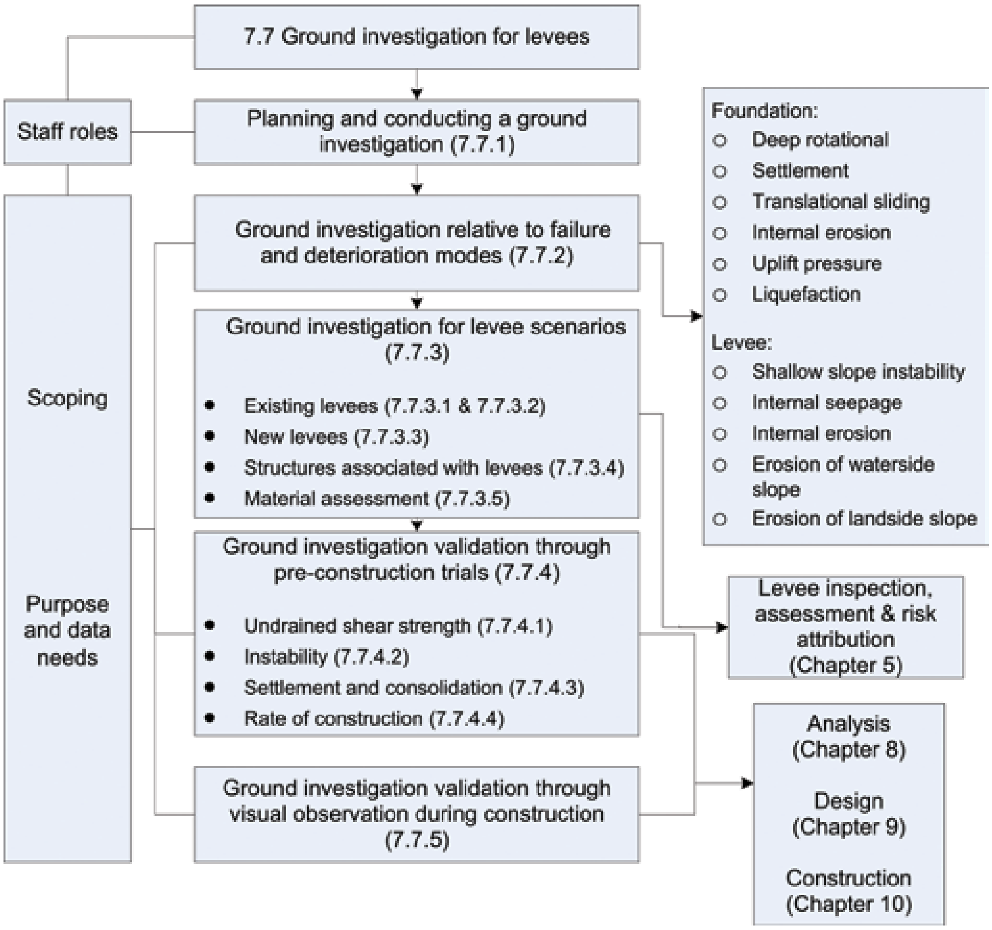


Figure 7.55 Structure and content of Section 7.7 and interaction with other subsections

Planning and conducting a ground investigation (Section 7.7.1): this section considers the interaction between the designer of investigations and any associated works (investigation designer) and the organisation undertaking the investigations (investigation contractor). Other arrangements could exist between these two parties as they could both be part of the same organisation, or one or both could be part of the owner/operator organisation. However, the broad principles discussed may still apply.

Ground investigations relative to failure and deterioration modes (Section 7.7.2): consideration is given to the general forms of investigation and monitoring that may be appropriate for the assessment of the overarching potential failure and deterioration modes discussed in Section 3.5.

Ground investigation for various levee scenarios (Section 7.7.3)

Existing levees – condition assessment and improvement works (Sections 7.7.3.1 and 7.7.3.2): the forms and approaches to the investigation of existing levees are discussed. When working on an existing levee it is important to understand the internal structure and the properties of the soils used to form the levee. The internal structure of existing levees is likely to be heterogeneous as a result of historic phases of raising using either locally won or imported materials or by design where a composite form of construction has been used. The heterogeneity can have an effect on the serviceability of the levee when it is required to resist increased loads such as hydraulic, vehicular, seismic etc.

New levees (Section 7.7.3.3): data from earlier investigations, existing levees or from other structures nearby whose past performance may provide clues as to the nature, characteristics and performance of the foundation soils may not be available that can be used to aid in the initial site characterisation during the reconnaissance phase. The characteristics of the site may have to be derived almost entirely through investigation.

Structures associated with levees (Section 7.7.3.4): earthen levees may include, or be required to include, penetrations or structural elements, such as pipes, services, piled seepage cut-off and crest wall, which can either have the potential to form a weak point in the levee or are included to enhance its operational performance during a flood event. Investigations are required to allow the interaction of an existing or new structure with the levee and foundation soils to be determined when under hydraulic load.

Fill material assessment (borrow material, Section 7.7.3.5): where a new levee is to be constructed or an existing one improved, a source of borrow material will be required. The extent and quantity of material available and its engineering properties will need to be defined so that they can be compared with established performance criterion (see Section 9.13.1) to determine its suitability or to establish what additional design measures are required to allow the borrow material to be used where its engineering properties are less than optimal. Where the likely source of borrow material is known at the start of the project, but is not from an established quarry, it can be investigated in parallel with investigation for the levee and foundations soils. Where the source is unknown in advance of the construction, the investigation principles outlined in this section still apply but may be undertaken as a separate phase of investigation.

Ground investigation validation through pre-construction field trials (Section 7.7.4): the construction of new large levees or the substantial raising of existing levees over a long length across soft foundation soils poses a significantly greater risk than more modestly sized levees. These design risks can be reduced through the formation of a designed pre-construction trial section of levee to validate or optimise the design by reducing the degree of uncertainty in the design parameters in terms of the stability of the levee during construction and the prediction of the amount and rate of post-construction settlement.

Ground investigation validation through visual observation during construction (Section 7.7.5): the investigation of ground conditions should be continued through to the construction stage where local variations in the ground conditions may only become apparent through observation in excavations or following a topsoil strip. The visual observation could suggest that ground conditions may locally be at variance with those assessed from the ground investigation. Other factors such as the forms of investigation and monitoring for construction control are covered in Sections 9.13 and 10.4.

7.7.1 Planning and conducting a ground investigation

To help ensure that the planned investigation will be successful, there should be early and frequent interaction between all parties involved. This includes the levee owner/operator, staff designing the investigation and staff conducting the investigation. The type of investigation to be conducted will be driven in part by the form of the intervention (condition assessment, improvement works or the construction of a new levee). The development of a scope of investigation and the supervision of the ground investigation in the field should be done by a person familiar with investigation techniques and equipment. This will facilitate execution, communication, and help ensure that activities of the ground investigation are providing the information required.

Scoping of the investigation

Conceptually the information required to develop a ground investigation and the basic principles to be considered in developing a scope of investigation is presented in Section 7.1. Selection of the tools available are presented in Section 7.9 and the assessment of the geotechnical parameters are discussed in Section 7.8. It is assumed that at the stage where the scope of investigation is prepared, the desk study has been completed and that it has been integrated within the CSM.

Levees are often located, or required to be constructed, over poor ground in remote locations where access may be restricted. Existing levees can have an undesirable geometry such as a narrow crest width and relatively steep side slopes. So, the early involvement of the investigation contractor in the scoping process to assess whether alternative or more effective methods and techniques can be used to achieve the objectives of the investigation is desirable. Possible interactions between the investigation designer and contractor during the early phase of scoping a ground investigation are presented in Table 7.42.

Table 7.42 Possible interactions of the investigation designer with the investigation contractor during the development of a scoping of ground investigation

Sequence	Actions by	Activity
Draft scope: • project goals • data needs.	Investigation designer	• identify owner requirements and constraints • identify site constraints • establish credible failure and deterioration modes • identify information required to evaluate the failure and deterioration modes • if appropriate, identify possible sources of borrow • develop, check, cost and programme an outline scope of investigation • discuss requirements with owner.
Ground truthing with investigation contractor	Investigation designer and ground investigation contractor	• visit site together • discuss and evaluate site constraints • discuss and evaluate safety issues • discuss scope of investigations and confirm and/or identify alternative approaches • identify any high cost/risk activities.
Finalise scope	Investigation designer	• update owner • revise scope in light of interaction with ground investigation contractor • prepare schedule of investigations and plans • undertake formal check of scope and documents.

There are various contractual mechanisms and relationships that may result in a revised sequence from those given in Table 7.42. Some organisations require investigation contractors to bid on a scope of work, without up-front involvement. In this case, the investigation designer may need to incorporate items within the documents that allow the flexibility to adapt the work later to accommodate changing conditions, without halting the progress of field work. Every possible field condition cannot be anticipated, and in some cases modifications during the field work are unavoidable. The investigation designer is responsible for ensuring the owner/operator is kept informed throughout field work on progress and when changes (technical, cost, and programme) are required.

The scoping of an investigation should be given the same due diligence as the preparation of design calculations or reports. So, it should be undertaken, or overviewed, by a competent professional and checked. It is often the case that the investigation designer who developed the scope of the investigation is not the same person supervising site activities. The investigation contractors' primary source of information is essentially the scoping document from which to evaluate the requirements of the investigation. To help others who have to check and implement the investigations in the field, a schedule can be prepared detailing what information is required and how it is anticipated that it will be obtained. In the case of intrusive investigations, comprising exploration holes, this could be a table detailing the exploration hole number, location, anticipated ground conditions, reasons for the exploration hole, depth, method of investigation, sampling and *in situ* testing requirements, and details of any instrumentation.

Execution of the ground investigation

It is important that an investigation is adequately supervised and managed with appropriately qualified and experienced staff to ensure that the investigation is progressing technically, adhering to the scope, and being performed safely. The structure of levees and the foundation soils can be locally variable, requiring changes to the scope of the investigation as the field work progresses. Additional constraints may be identified during the execution of the investigation. Large investigations may require a senior staff member supported by junior staff on site full-time, whereas a small investigation may only require a junior staff member on site (either full or part-time) with senior staff providing office based support. The key activities of the supervisor are as an administrator and technical director in terms of ensuring that the information required by the investigation designer is obtained. These roles are summarised in Table 7.43.

Table 7.43 ***Some activities to be undertaken by the investigation supervisor during execution of the ground investigation***

Role	Activities
Administrator	Overview the day to day running of the contract: • ensure compliance with the contract, including methods, and health and safety • observe and record site activities so that progress can be assessed and evaluated • agreeing to the measurement of work done for payment • update owner on costs and programme.
Technical director	Ensure that the investigation delivers the required information: • the investigation should be viewed as a programme of research that retains the flexibility to be modified in response to the new information it provides. The scope should be continually reviewed on the basis of the new data, both from finds in the field as a result of the investigations and scheme modification made by the engineer or directed by the client. A key activity to achieving a good overview of field data is the production of a geological section from the intrusive investigations as logs are received and including on the section a summary of the *in situ* tests results. Where geophysical data are available the findings of the intrusive investigation could be overlaid or compared with the geophysical profile. This approach allows the supervising engineer to: • visualise the information and compare new data with existing information, which could be used to form an initial geological cross-section before starting the current phase of investigation on site • identify anomalies in the information and target additional investigations to resolve then • ensure a robust distribution of more specialised *in situ* testing and laboratory testing within a given soil horizon is achieved when scheduling testing • if appropriate, shorten the depth of intrusive investigations where soil descriptions and *in situ* tests show the soil horizon to be competent and relatively uniform • identify any operator dependent variability in the investigation • confirm installation details of scheduled instrumentation • ensure that the results of *in situ* tests are consistent with the soil descriptions and compatible with the information obtained from other forms of investigation.

7.7.2 Ground investigations relative to failure and deterioration modes

The fundamental objectives of a ground investigation are to provide quantitative supporting information to understand, assess and mitigate potential levee failure and deterioration modes, or allow them to be managed. These are discussed in Section 3.5. Tables 7.44 and 7.45 consider, respectively, the principle modes of failure and deterioration, and outline the forms of intrusive investigation and monitoring that may be appropriate to evaluate them. While Tables 7.44 and 7.45 specifically address intrusive investigations it should be seen as part of a phased approach to the investigations of the ground, which includes the assessment of published and existing data, site observations, non-intrusive and geophysical investigations etc.

Table 7.44 ***Foundation soils failure and deterioration modes – indicative ground investigations and monitoring***

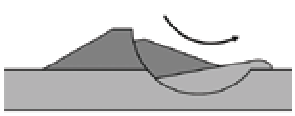
Failure/deterioration mode		Process	Investigation	Monitoring
Deep rotational failure		• shear failure through levee and foundation soils during construction or embankment raising works.	• shear strength of fill and foundation soils, in particular undrained shear strength of soft compressible soils (clays/peat) • longer term gain in strength of soft compressible soils due to consolidation.	• pore water pressures in soft compressible soils below levee • vertical settlement of levee crest and lateral displacement of levee toes.

Table 7.44 Foundation soils failure and deterioration modes – indicative ground investigations and monitoring (contd)

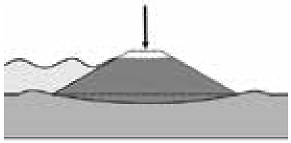
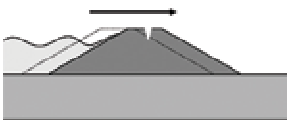
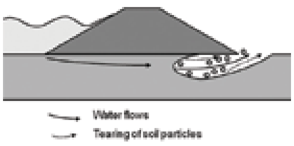
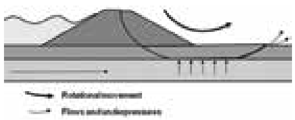
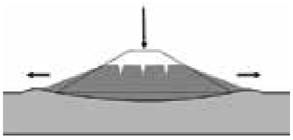
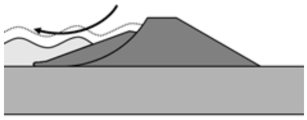
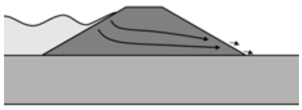
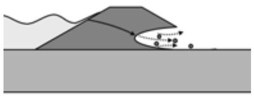
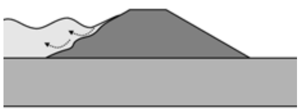
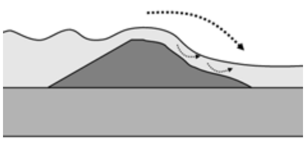
Settlement		• self-weight compression of fill material and consolidation of compressible foundation soils.	• compression characteristics of compacted fill and consolidation characteristics of foundation soils • secondary compression characteristics of compacted fill and foundation soils.	• pore water pressures in compressible foundation soils below levee • vertical settlement of levee crest.
Translational sliding		• lateral hydraulic force exceeds shear resistance of founding soils along base of embankment or desiccation of organic fill leading to a reduction in weight and shearing resistance.	• shear strength of soft clays and organic soils directly beneath the levee • extent of desiccation of peat and organic fills.	• imposed hydraulic loads • lateral displacement at levee toe • pore water pressure in foundation soils below levee.
Internal erosion		• under-flow of floodwater in high permeability soils leading to erosion and instability (note that the four forms that this can take are discussed in Section 8.5).	• permeability, particle size distribution, composition and inter-layering of foundation soils.	• imposed hydraulic load and pore water pressure response in high permeability foundation soils • visual inspection during high water event.
Uplift pressures		• build-up of pore water pressure in confined permeable foundation soils due to hydraulic continuity with flood or high water on water side • confining pressure can lead to the formation of boils and piping failure.	• presence of highly permeable foundation soils beneath and landward of the levee that are capped by light weight peats and low permeability alluvial clays.	• imposed hydraulic load and pore water pressure response in high permeability foundation soils • uplift and lateral displacement of landside levee toe.
Liquefaction		• large seismic accelerations cause settlement and loss of effective shear strength in loose fine grained non-cohesive foundation soils, leading to loss of bearing pressures and lateral spreading of levee.	• presence of loose, saturated fine grained non-cohesive foundation soils in seismically active areas.	• seismic accelerations • pore water pressures in saturated fine grained non-cohesive soils • settlement of levee crest and lateral displacement of toe.

Table 7.45 Levee structure failure and deterioration modes – indicative ground investigations and monitoring

Failure/deterioration mode	Process	Investigation	Monitoring
Shallow slope instability 	• shear failure during levee construction or raising • instability during rapid draw-down after a flood • loss of strength due to increase or equalisation of pore water pressures • erosion of toe on water side due to channel or beach morphology.	• compaction of fill material in relation to moisture content • swell potential of over-consolidated clay fill • erosion resistance of levee fill and foundation soils • effective shear strength of fill material in relation to slope angle • desiccation of fill material.	• pore water pressures in slope • movements on slope • channel or beach morphology.
*Internal seepage** 	• excessive seepage caused by desiccation, fine fissuring or highly permeable fill material, exacerbated by burrowing animals and vegetation.	• plasticity of clay leading to fissuring • local composition and internal structure of levee • high permeability soils.	• optical fibres to identify seepages • pore water pressures on seepage path • visual inspection during high water event.
Internal erosion 	• detachment and movement of soil particles by the seepage flows (note that the four forms that this can take are discussed in Section 8.5).	• erosion resistance and fissuring potential of levee fill • local composition and internal structure of levee.	• optical fibres to identify seepages • visual inspection during high water event • turbidity of seepage.
Erosion of waterside slope 	• wave erosion of earthen levee and rapid changes in pore water pressure over and below revetment.	• erosion resistance and fissuring potential of levee fill • integrity and suitability of vegetation cover and/ or revetment • characteristics of revetment • local composition and internal structure of levee.	• condition and nature of surface cover (revetment or vegetation) • rate of erosion.
Erosion of landside slope 	• over topping.	• erosion resistance and fissuring potential of levee fill • suitability of landside slope profile, topsoil, vegetation or geotextile • slope geometry.	• condition and nature of vegetation or geotextile cover • extent of fissuring • water level and crest elevation.

Note

* All levees are prone to seepage to some degree without there being any secondary detrimental effects, such as internal erosion of the levee or foundation. So, seepage alone is not considered to be a 'deterioration mode' in Section 3.5.

7.7.3 Ground investigations for different levee scenarios

The process of implementing an investigation through a phased approach is discussed in Section 7.1.1. Some objectives and circumstances under which ground investigations may be undertaken in connection with levees are summarised in Table 7.46.

Table 7.46 Ground investigation for levees: circumstances and objectives

	Circumstances	Objectives
Existing levees	Condition assessment (Chapter 5)	• quantify the performance of the levee under the action of new loads where there is no visual or recorded information that indicates that the levee has inherent problems • reassessment to ensure the levee satisfies current or updated criteria for safety where there is no visual or recorded information that indicates that the levee has inherent problems • assessment of known problematic sections that could affect the levee performance and that can be related to the development of a potential failure or deterioration mode identified in Sections 3.5 and 7.7.2.
Existing levees	Design of improvement works	• increase the standard of protection where there is no visual or recorded information that indicates that the levee has any inherent problems and investigations are required to assess the effects of the works on the existing levee • prevent the development of failure or to mitigate the effects of deterioration where problematic sections have been identified relative to the potential failure or deterioration modes identified in Sections 3.5 and 7.7.2, and works are required to reinstate the existing standard of protection • a failure has occurred, emergency works may have been implemented and a permanent repair is now required.
New levees	Design of new levees	• provide first time flood defences • form managed retreats to create habitat, regulate upstream water levels or to mitigate the use of water side structures, such as piling, in favour of softer engineering options.
New and existing levees	Penetrations and structures	• support the evaluation of the geotechnical performance and condition assessment of existing penetration or structure at locations where visual inspection has identified performance issues • evaluate the nature of the geotechnical works required to upgrade an existing penetration or structure • facilitate the design of a new penetration or structure.
New and existing levees	Evaluate borrow materials	• the economics of constructing or undertaking improvement works on an existing levee may be dependent upon being able to use site won borrow material • other off-site sources of fill may have been identified as potential borrow areas.

Where ground investigations are required on existing levees it should be remembered that in many cases they are old structures that have been raised over time to suit evolving needs. While a section of levee may not show evidence of failure or deterioration in its present state, there could have been maintenance or remedial works undertaken in the past that need to be considered when developing an investigation. Some of the factors to consider are summarised in Table 7.47.

Table 7.47 *Some factors to be considered when undertaking ground investigations on existing levees*

Timing of activity	Factors to be considered
Construction	• internal structure of levee may be homogeneous or heterogeneous in nature. It may have a core or be layered as a result of historic phases of raising and widening • internal structure may hide elements of the existing or former flood defence like steel sheet piling, wooden piles, road foundations or old stone revetments (Figure 3.96) • contaminated materials may have been used to form the levee • the phreatic groundwater level in a heterogeneous levee may be unpredictable because of variations in permeability. Groundwater level in the levee can be higher than the high water level because of precipitation.
Improvement	• side slopes may have been built up using various materials, some of which may be of poor quality such as 'dredgings' from an adjacent channel • crest level may have settled over time requiring maintenance raising, possibly using different materials, to maintain the standard of protection • crest level may have been raised, possibly using different materials, to improve the standard of protection.
Repair	• seepage may have been mitigated by forming an apron, installing sheet piling or deep soil mixing • failure of sections may have occurred and been repaired. The repair works could include structural elements.

Intrusive investigations within the structure of a levee have the potential to weaken it in terms of its ability to withstand hydraulic load. The needs for, potential consequences of, and method of reinstatement following intrusive investigations should be fully explored to avoid leaving the levee with a residual weak point. The method of investigation can also weaken the levee. Rotary drilling methods can cause 'hydraulic' fracture of the levee where the return of the flush medium (air/water/mud) is restricted by a blockage and the pressure of the flush medium increases rapidly at the drill tip. This could be mitigated by including a pressure release or limiting valve in the flush pipe line.

A summary of some of the characteristics to be evaluated by the ground investigation are summarised in Table 7.48, and are expanded on in Sections 7.7.3 to 7.7.5.

Table 7.48 *Summary of some ground characteristics to be evaluated*

Element	Some geotechnical characteristics to be evaluated
Foundation soils	• ground profile and geotechnical properties below and outside the levee footprint, including any localised geological features that may affect levee performance • groundwater regime and response to external hydraulic loads in higher permeability soils • seismicity of area.
Levee material	• internal structure, noting potential issues identified in Table 7.47, and any evidence of subsurface/internal deterioration of the existing levee • geotechnical properties of the levee fill material(s).
Borrow material	• geotechnical properties of the borrow material and their implications on the construction works and long-term performance of the levee • the extent (lateral and depth) and quantity of acceptable fill material • level of contamination.
Other factors	• information on associated structures that pass below, through or are located on the levee • information at tie-in points with existing flood defences (levees, walls, buildings) or higher ground.

Where risk assessments show it to be appropriate, studies for the assessment of UXO, contamination and archaeology can be incorporated into the investigations. These are specialist techniques and the requirements will vary with local legislation. Specialist input should be sought in scoping and integrating these within the ground investigation.

7.7.3.1 Ground investigation for the condition assessment of existing levees

The aim of the condition assessment is to define how an existing levee will perform under hydraulic load and to identify and assess the effect of any adverse feature that could affect the serviceability of the levee. The process of undertaking a condition assessment of a levee is presented in Chapter 5, to which reference should be made. Ground investigation only forms part of that process in support of visual observations and monitoring.

The extent of the investigations may depend on the nature of the levee to be assessed. It may be an old levee system with little information, and for which a full programme of investigation is required to establish the geotechnical composition of the levee and its foundation soils. Alternately, it could be a well-managed system about which there is existing data, and some limited investigation is required to verify that the current condition of the levee and its foundation soil are compatible with the required performance.

The general principles of undertaking ground investigation for the condition assessment of a levee follow those detailed in Sections 7.1 and 7.9, and a phased approach to any investigation is advised. Specific to the condition assessment process, some countries adopt a tiered approach to levee condition assessment (Section 5.5). This is summarised in Table 7.49 (although three tiers are shown this does not imply that there would be only three tiers), and is consistent with the phased approach to investigations as outlined in Section 7.1.2.

The tiered approach is similar to the three phases of investigation for a new levee (reconnaissance, feasibility, detailed), but with subtle differences. While both consist of collecting progressively more data in an iterative fashion, the investigation process for a new levee proceeds to final design. Tiered investigations for existing levees are both phased and scaled to be commensurate with the performance issues, consequences, and overall risk. More intensive investigations and studies are done for levees having more serious issues.

Table 7.49 Example of a tiered approach to the condition assessment of a levee

	Objectives	**Investigations**
Tier 1	• obtain information to inform managers about potential weak points and areas of deterioration and/or damage • identification of the nature and intensity of data needed to predict future performance of the levee and to inform the scope of the Tier 2 assessment.	• desk study of historic and published data, including maintenance records and information on observed performance under hydraulic load, visual inspections and the application of limited topographic survey.
Tier 2	• obtain information to assess the potential damage or deterioration identified in Tier 1 • the results of non-intrusive investigations, which may be combined with the potential risks associated with the levee, can be used to determine the need for remedial action or additional, more detailed non-intrusive and/or intrusive investigations under Tier 3.	• collection of new information through non-intrusive investigations such as geophysical data, updated aerial photography, morphologic mapping, or topographic survey data.
Tier 3	• obtain detailed information where a comprehensive assessment of the levee condition is required. This may be aimed at 'high risk' levees that protect large numbers of people and property or vulnerable critical infrastructure, vulnerable natural resources and services • investigations may also be required to confirm deterioration processes indicated by remote sensing methods.	• collection of additional information through non-intrusive and intrusive techniques, which may include the installation of instrumentation • it is sometimes necessary to conduct intrusive investigations when other techniques are unsuitable or incapable of gathering the required data, or for pinpointing the source problems within the levee.

7.7.3.2 Ground investigation for improvement works on existing levees

When undertaking investigations for improvement works on an existing levee there is a need to understand the current condition of the levee as this will influence the nature of the works. The principles of condition assessment are presented in Chapter 5 and Section 7.7.3.1. There is also a need to understand how the improved levee and foundation soils will respond to the additional load from any new fill material and increased hydraulic load, where the standard of protection is to be enhanced, along with the properties of any borrow material. The process of ground investigation could follow the general philosophy outlined in Sections 7.7.3.3 and 7.7.3.5.

In the case of repairs to a failed section of levee, location of the failure could be evaluated in the context of the geological environment as presented in the CSM. Ground LiDAR, aerial photographs, and historic investigation data may indicate that the failure has occurred at a location where a localised ground feature is present, which suggests that local ground conditions may be different from the more 'general' ground conditions on the levee alignment, where no failures have occurred. Any investigation may need to be undertaken using hand-held or lightweight equipment to limit the load imposed on the failed section of levee and it should aim to provide sufficient information to understand the cause of the failure and assess whether it has stabilised under the present condition. The information needed for this may include:

- failure geometry (including the profile of the slip surface, if applicable)
- soil profile and properties (pre and post failure)
- loads (hydraulic, imposed, pore water pressure) acting at the time of failure.

The information may be used to back analyse the failure and derive geotechnical parameters. This approach is considered in Section 7.7.4.1. Where the failure is localised the investigation should also extend to adjacent sections that appear to have performed adequately. This will allow a direct comparison to be made of the ground conditions at both locations and may provide additional clues as to the nature and cause of the problem, and identify whether sections of adjacent levee also require improvement works to enhance their stability or resilience.

7.7.3.3 Ground investigation for new levees

The majority of ground investigations undertaken are for existing levees. In the case of a new levee there may not be data from earlier investigations that can be used to aid in the initial site characterisation during the reconnaissance phase. Similarly there may not be data on the past performance of an existing levee or other structures nearby that may have provided clues as to the nature, characteristics and performance of the foundation soils and levee fill material under the action of periodic external and hydraulic loads, sustained self-weight and aging.

However, where such information is not available, the characteristics of the site may have to be derived almost entirely through investigation. In cases where imported borrow material is required the source may not be known until the construction contract is let. The information needed to characterise the foundation soils is summarised in Table 7.50. Indicative forms of investigation for evaluating the foundation soils are provided in Table 7.51. The process of ground investigation should follow the general philosophy of a phased approach outlined in Section 7.1.1 and the guidance offered in Sections 7.8, 7.9.6 and 7.9.9.

Table 7.50 Information required characterising the levee foundation soils

Foundation material	Characteristic to be assessed	Method
All	• ground profile • groundwater profile.	• survey • non-intrusive investigations • intrusive investigations with soil sampling, classification and full engineering description, including soil fabric • instrumentation: piezometers.
Clays	• moisture content • Atterberg limits • grain size distribution • permeability • undrained shear strength • effective shear strength • compressibility • chemical aggressivity to other building materials.	• empirical relationships • routine *in situ* and laboratory testing • pre-construction field trials to confirm undrained shear strength and compressibility.

Sand and gravels	• grain size distribution
	• permeability
	• response of groundwater to variations in external water level during a tidal or fluvial event
	• maximum and minimum density and relative density (seismic evaluation)
	• effective friction angle (seismic evaluation).
		• empirical correlations
		• routine *in situ* and laboratory testing
		• *in situ* pumping test to determine permeability
		• instrumentation: piezometers
		• effective friction angle:
		  ◦ full displacement of shear box test to determine (ϕ'_{cv})
		  ◦ cyclic triaxial tests (seismic evaluation).

Peat	• permeability • undrained shear strength • effective shear strength • compressibility (including secondary compression).	• routine *in situ* and laboratory testing • extended consolidation tests to evaluate secondary compression.

Table 7.51 Some indicative forms of investigation for levee foundation soils

Foundation material	Investigations
All	• during feasibility, using information from the CSM, undertake geophysical investigations (seek appropriate specialist advice) followed by targeted CPT soundings on the line of the proposed levee and outside the levee footprint to provided cross-sectional information • consider doing a limited number of targeted boreholes during feasibility to aid in ground truthing both geophysics and CPT soundings • use the information within the CSM and the new data obtained from the geophysics and CPT soundings to target the locations of other intrusive investigations, which may be adjacent to the CPT sounding. This will help to maximise the information gained from sampling and *in situ* testing to aid in site geology interpretation, obtained geotechnical parameters, target the installation of monitoring instrumentation and resolve specific questions and uncertainties.
Clays and peat: superficial deposits of alluvium	• target clusters of other forms of intrusive investigation (boreholes, vane shear tests) adjacent to selected CPTs to characterise clays, peat, and other organic soils. Use data from intrusive investigation to develop site specific corrections between *in situ* and laboratory measure parameters (undrained shear strength, compressibility) and CPT data. Use site specific CPT correlations to infer geotechnical properties at other CPT locations across the site.
Sand and gravel	• CPT soundings where the density and particle size distribution allows full penetration • perform *in situ* testing to assess permeability • install piezometers. Where external water level varies cyclically as a result of the tide or relatively frequent fluvial events, monitor the external water level and piezometer response to establish a correlation. Observed data can be used to calibrate soil permeabilities in a transient seepage model. However, extreme events may change boundary conditions by removing lower permeability riverbed silts that may have attenuated the hydraulic response of the groundwater in the sand and gravels.

7.7.3.4 Ground investigation for assessment of structures associated with levees

Levees can include a variety of structures and penetrations. There are possibly three main forms:

- crest wall: ground bearing or embedded wall
- cut-offs: bentonite cement slurry walls or sheet piles
- penetrations: pipeline, culverts or services.

Investigation may be required to establish their interaction with the levee and performance under hydraulic load. The structural assessment of these elements is outside the scope of the handbook and specialist advice should be sought. Some issues associated with structural elements and possible methods of investigation are given in Table 7.52.

Table 7.52 Some issues associated with structural elements and possible methods of investigation

Levee structures	Issues	Investigations
Crest wall	• differential settlement along levee resulting in compression or opening up of joints between ground bearing wall panels, or the formation of voids below the panel bases where they span across areas of localised settlement.	• geophysical techniques may be used, following appropriate specialist advice, to detect voids under the base slab, with trial pits to ground truth the results • trial pits to assess presence of voids at locations targeted through visual observation of the settlement profile • intrusive investigation may be required to evaluate the cause of the settlement and internal issues within the body of the levee or the foundation soils.
	• lateral deflection/ movement of embedded walls resulting in the formation of a gap between the wall and the soils, causing an increased lateral hydraulic loading on the wall.	• intrusive investigations to determine the ground profile and properties, including stiffness, of the soils within the zone of influence of the embedded wall.
	• determination of toe level for an existing embedded wall.	• intrusive investigation: vertical or inclined boreholes or probes to intercept the wall at a range of elevations • magnetometer on CPT, or down the borehole, adjacent to wall where steel sheet piles have been used.
Cut-offs	• determination of toe level.	• intrusive investigations: vertical or inclined boreholes to intercept cut-off where it comprises bentonite/cement grout • magnetometer on CPT or down the borehole, adjacent to the wall where steel sheet piles have been used.
	• ineffective cut-off.	• assess recorded evidence of ineffectiveness of cut-off • determine the toe level of cut-off, as above • focused and/or broader coverage geophysical investigations, following appropriate specialist advice, with intrusive investigations, scaled to suit the magnitude of the perceived issues, to understand the profile and properties of the internal structure of the levee and/or the foundation soils in 3D • where cut-off is within the foundation soil, the existing groundwater regime will need to be understood which may require the installation of piezometers.

Table 7.52 Some issues associated with structural elements and possible methods of investigation (contd)

Penetrations	• settlement of levee around and over penetration.	• visual internal inspection of penetration, where possible, to assess integrity and evidence of material being washed into penetration through joints or fractures • visual evidence of material being removed by seepage through backfill or bedding material around penetration or through overlying backfill • geophysical investigation, following appropriate specialist advice, and/or intrusive investigation to evaluate the condition of the backfill relative to adjacent *in situ* sections of levee and foundation, and the presence and nature of any bedding material.
	• seepage along line of penetration.	• geophysical investigation, following appropriate specialist advice, and/or intrusive investigation to evaluate the condition of the backfill relative to adjacent *in situ* sections of levee and foundation, and the presence and nature of any bedding material.

7.7.3.5 Ground investigation for material assessment (borrow material)

The costs associated with the transportation of borrow materials used to improve existing or construct new levees can be a major proportion of the total project cost. These costs may be reduced if the material is locally sourced, and minimised if it can be obtained on site, possibly through the excavation or enlargement of an existing drainage channel on the landward side of the levee. In coastal areas sea dredged materials could be brought ashore for use in composite levees. Levees may also be formed using sands and gravels quarried on shore.

The investigations need to be sufficient to adequately define the consistency, or variability, and extent of the potential borrow materials. Additionally, the investigations should define the geotechnical properties so that acceptability can be assessed (Section 9.13.1).

The information needed to characterise borrow material is summarised in Table 7.53. Indicative forms of investigation for evaluating a source of borrow material are outlined in Table 7.54. Where the source of borrow material is from an established source (on or off shore) with well documented information on the nature of the materials, then consideration could be given to reducing the amount of investigation and testing. The contents of the tables focus on the evaluation of engineering properties. Consideration will also need to be given to assessing whether the material contains unacceptable levels of contamination. Some countries require contamination testing of potential sources of borrow material. Investigations for this are not covered in the tables and specialist advice should be obtained.

Table 7.53 Information required to characterise borrow material

Borrow material	Characteristic to be assessed	Methods
All	• ground profile • groundwater profile.	• survey • non-intrusive investigations • intrusive investigations with soil sampling, classification and full engineering description, including soil fabric • instrumentation – piezometers.
Clays	• moisture content • Atterberg limits • grain size distribution • undrained shear strength • compaction characteristics • effective shear strength • permeability. • dispersion and erosion resistance • chemical aggressivity to other building materials.	• empirical relationships • routine *in situ* and laboratory testing • compaction – including one specimen compacted at 'as received' moisture content. Undrained shear strength on top and bottom of each compacted specimen • effective shear strength and permeability on re-compacted specimens at natural moisture content – assuming material to be used 'as dug'.

Sand and gravels	• grain size distribution
	• compaction characteristics.
	• maximum and minimum density (for seismic evaluation)
	• permeability
	• effective shear strength.
		• empirical relationships
		• routine *in situ* and laboratory testing
		• effective friction angle:
		  ○ full displacement of shear box test to determine (ϕ'_{cv})
		  ○ cyclic triaxial tests (seismic evaluation).

Table 7.54 Some indicative forms of investigation for borrow material depending upon its nature and source

	Nature and source of potential borrow	Investigations
Clay	Superficial deposits of alluvium (desiccated crust obtained on site through excavation and enlargement of landside drainage channel)	• observations in existing landside drainage channel • CPT soundings and/or trial pits to assess thickness and extent of potential borrow material (assuming only desiccated crust is suitable) • trial pits for visual inspection of material, with *in situ* testing comprising hand shear vane tests in walls of trial pits or intact blocks of clay removed by excavator bucket to determine shear strength depth profile. Obtain bulk samples for classification and laboratory testing.
Clay	Solid geology (off site from a large and deep borrow pit/existing quarry)	• review published and available information on engineering properties of geological formation • trial pits to allow visual inspection of material *in situ* and obtain bulk samples at shallow depth for laboratory testing • where depth of borrow pit is likely to extend below the depth achievable by trial pits, boreholes may be required to allow the material at depth to be visually inspected and sampled • where the clay is underlain by higher permeability water bearing strata the investigation should also consider the stability of the clay layer forming the base of the excavation against uplift.

Table 7.54 Some indicative forms of investigation for borrow material depending upon its nature and source (contd)

Sand and Gravel	Land source	• geophysics, following appropriate specialist advice, and/or CPT sounding to assess thickness and extent of deposits • trial pits above the water table to allow visual inspection of the material and obtain bulk samples for testing • boreholes to obtain disturbed samples where excavation of borrow material will be required below the water table.
	Off shore source (dredged)	• geophysics to profile the subsurface deposits and define the extent of the deposits • vibrocores and grab samples for ground truthing near surface geophysics and to provide samples for visual inspection, classification and laboratory testing • over water boreholes may be required where the dredge depth is likely to extend below the depth achievable by vibrocores, or the material is too coarse/dense for this technique, to allow the material at depth to be sampled for visual inspection and laboratory testing.

7.7.4 Ground investigation validation through pre-construction trials

Where large and long sections of new levees are to be constructed or existing levees substantially raised over soft foundation soils the design, cost and construction risks are significantly greater than they are for more modestly sized levees. This is particularly true where the ground conditions or the height of the levee is outside the envelope of routine construction and there is no local experience of similar forms of construction. It may also be desirable on financial and environmental grounds to use site sourced borrow material, which may be derived from the desiccated crust of the alluvial deposits and so may not be the most suitable material for the construction of a levee.

These risks can be reduced through the formation of a pre-construction trial section of levee to validate or optimise the design. This will tend to reduce the degree of uncertainty in the stability of the levee during construction and the prediction of the amount and rate of post-construction settlement, which influences the magnitude of the in-built settlement allowance or future maintenance commitments to preserve the standard of protection. It will also demonstrate the acceptability and workability of potentially marginal site won borrow material.

Pre-construction site trials can be expensive due to the costs associated with the complexity and intensity of instrumentation and testing that is normally required, and the staff costs associated with the frequency of monitoring and real time interpretation of the data, which is required for the onset of failure to be detected during construction. As such they may only be justified where there are residual design risks that will have a significant influence on the overall cost of the project or construction programme. However, some overall saving may be made by subsuming the trial section of the levee into the main construction, assuming the trial is not aimed at achieving failure of the foundation soils and that it achieves the desired factors of safety and quality required by the final design and specification.

The formation of a pre-construction trial section of levee to assess the strength and consolidation characteristics of the foundation soil is considered in this section. Issues relating to fill material acceptability, site compaction trials and plant operations are addressed in Sections 9.13 and 10.4.

The derived mass soil properties of the foundation soils will feedback into the CSM and the design process. This will reduce the margins of uncertainty inherent in the judgement of the designer when extrapolating the results of tests on small, possibly unrepresentative, soil specimens to interpret the geotechnical properties of the soil mass.

The selection of a suitable location for the construction of a trial section of levee is important if the results are to be meaningful and representative of site conditions. As such the location can only effectively be identified once sufficient investigation has been undertaken. A review of the results from the available investigation in combination with other information in the CSM (LiDAR, aerial photographs, geophysics) can be used to guide the identification of a suitable location where ground conditions are representative of, or slightly worse than, typical conditions across the site. If data on

1 2 3 4 5 6 7 8 9 10

ground conditions at the trial location are limited then additional *in situ* testing and sampling for laboratory testing should be undertaken when sinking exploratory holes to install the subsurface instrumentation. A good understanding of the ground conditions and geotechnical properties will be required to allow a robust comparison to be made between the calculated and actual field performance. The selection of a location may also take into account the convenience of site access but this should not be the driver behind the selection of a location.

A typical length of trial levee might be at least three times the base width to limit the influence of 3D effects. The size should also be sufficient to allow the effective movement and operation of construction plant.

An initial assessment of the acceptability, workability and compactive effort required to achieve the desired level of compaction can be determined through field compaction trials. These should be considered as an integral part of a pre-construction trial and should also be seen as routine practice at the start of earthworks operations associated with levees.

7.7.4.1 Assessment of undrained shear strength

The undrained shear strength of the foundation soil can be assessed by designing the trial section of levee to fail at a predetermined height or, more safely, through the excavation of a toe trench. The failure geometry is then back analysed through iterations of the shear strength of the foundation soils and levee fill material using slope stability techniques (Section 8.6), until a factor of safety of unity is achieved. The strength and density of the fill material needs to be known to undertake the analysis. Forming the trial section from non-cohesive fill, rather than cohesive fill, will provide a more consistent and predictable fill strength. The trial section could be instrumented to identify the onset of failure. This could include extensometers and settlement gauges to monitor settlement of the levee, and inclinometers and surface displacement stations to measure lateral displacements of the levee toe and/or piezometers to measure associated changes in pore water pressure under the levee. These will be destroyed as the levee fails. However, the observed responses leading up to failure will help with the interpretation of any instrumentation installed during the construction phase.

7.7.4.2 Assessment of the onset of instability

The onset of instability of a levee during construction on soft clay can be assessed by monitoring displacements based on a correlation between vertical settlement below the levee and the lateral displacement of the toe (Wakita and Matsuo, 1994), and by monitoring the increase in pore water pressures in the soft clay as the levee is raised. The response of the pore water pressure to the applied load can also be used to evaluate the yield stress (σ'_y) (Section 7.8.3.4). The use of pore water pressure monitoring to control stability is considered to be a more reliable indicator of the onset of instability than the measurement of displacements. Details of both methods are discussed in this section.

Stability control by monitoring displacements

Wakita and Matsuo (1994) proposed an observational design method for embankments on soft clay following several observed embankment failures in Japan. They observed that there was a correlation between the settlement S on the centre line at the bases of the embankment (original ground level) and the ratio δ/S, where δ is the horizontal displacement at the toe of the embankment.

Measurements of horizontal and vertical displacement made during construction can be used to evaluate S and δ/S. Figure 7.56 shows the typical displaced path for an embankment loaded to failure. If construction is paused the displacement path follows the direction of the dotted arrow if the stability factor is sufficient. When S and δ/S are plotted on the base data of Wakita and Matsuo (1994) (Figure 7.57) they define a point which can be used to interpolate the degree of safety against failure.

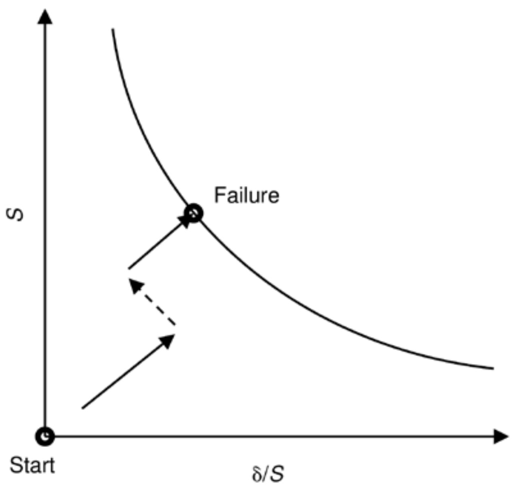


Figure 7.56 *Application of Wakita and Matsuo method to construction control and assessing onset of failure*

They also observed that a family of curves existed on a plot of S against δ/S that corresponded to points of equal values of the ratio q/q_f, where q is the embankment load and q_f is the load at failure. They called the final curve where q/q_f =1, failure criterion line. The family of curves can be defined by Equation 7.69, where the coefficients a, b, and c are as presented in Table 7.55 and are dependent upon the ratio q/q_f.

$$\boldsymbol{S} = \boldsymbol{a}.\exp[\boldsymbol{b}(\frac{\boldsymbol{\delta}}{\boldsymbol{S}})^2 + \boldsymbol{c}(\frac{\boldsymbol{\delta}}{\boldsymbol{S}})] \tag{7.69}$$

where:

S = settlement

δ = horizontal displacement at embankment toe

a, b and c = regression factors

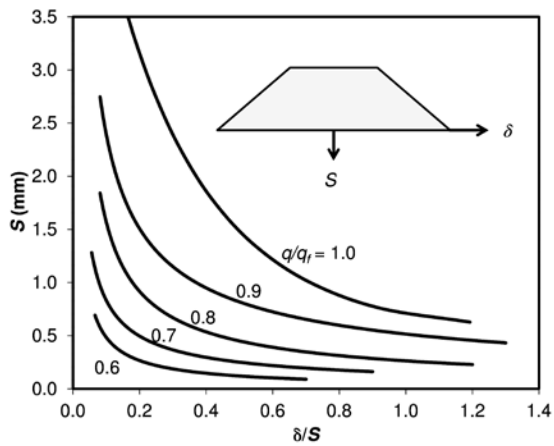


Figure 7.57 *Variation in S with δ/S for values of q/q_f of 0.6 to 1.0 (failure)*

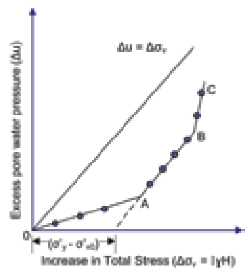
Table 7.55 *Value of regression coefficients for Equation 7.69*

Load intensity (q/q_f)	Regression coefficients			Range of application
	a	b	c	
1.0	5.93	1.28	-3.41	0 <δ/S <1.4
0.9	2.80	0.40	-2.49	0 <δ/S <1.2
0.8	2.94	4.52	-6.37	0 <δ/S <0.8
0.7	2.66	9.63	-9.97	0 <δ/S <0.6
0.6	0.98	5.93	-7.37	0 <δ/S <0.6

Stability control by monitoring pore water pressures

The development of pore water within a soft clay foundation below the levee depends on the degree of over consolidation and how close the soil is to failure. The increase in pore water pressure (Δu) due to the applied load, or increase in total vertical stress ($\Delta\sigma_v$), shows three well defined and sequential responses. It is convenient to take $\Delta\sigma_v$ as γH (unit weight density of the fill times the height of the embankment). However, at depth below the embankment this assumption is conservative and an influence factor (I) should be used to assess $\Delta\sigma_v$, such that $\Delta\sigma_v = I\gamma H$ (Table 7.56). For this method to be effective sufficient measurements of fill thickness and pore water pressure should be made during construction to define the relationship between Δu and $\Delta\sigma_v$. This may require readings to be taken immediately before and after each layer of fill has been placed.

Table 7.56 Relationship between increase in total vertical stress and pore water pressure

Increase in total stress ($\Delta\sigma_v = I\gamma H$)	**Soil state**	**Increase in pore water pressure (Δu)**	Change in pore water pressure with increase in total vertical stress
Below σ'_y (O–A)	Over consolidated	Less than $\Delta\sigma_v$	
σ'_y to σ'_f (A–B)	Normally consolidated	Equal to $\Delta\sigma_v$	
Above σ'_f (B–C)	Onset of local failure	Greater than $\Delta\sigma_v$	

The method of control is relatively straightforward. A plot of Δu versus $\Delta\sigma_v$ is generated. As soon as the slope of the line is greater than unity then there is a risk of failure occurring. At this point fill placement should stop. It may even be necessary to remove fill to avert a failure.

Where the levee is built in stages with a pause for consolidation between each stage, then each construction stage can be assessed independently by applying the same principles.

On the plot of Δu versus $\Delta\sigma_v$ (shown in Table 7.56), project the straight line portion (A–B), representing the normally consolidated state, on to the $\Delta\sigma_v$ axis. The value of the intercept is equal to $\sigma'_y - \sigma'_{vo}$ at the depth at which the pore water pressures were measured.

7.7.4.3 Assessment of settlement and coefficient of consolidation

Where failure of the foundation soil is not an objective of the trial, it may be more appropriate to install more complex forms of instrumentation (extensometers, piezometer arrays, inclinometers) to monitor the response of the soil to the applied load from the trial section of levee. This form of trial may not confirm the ultimate limit state undrained shear strength of the foundation soil. However, it should confirm that the design factor of safety is adequately robust, by demonstrating that deformations and the response of excess pore water pressures to the applied load remain within acceptable limits. This is discussed in Section 7.7.4.2, and provides a validation of the design.

The early prediction of the consolidation characteristics of the foundation soils based on limited field data will be unreliable and an extended period (up to one year or more) of post-construction monitoring may be required. Even so, practically it is unlikely that monitoring will continue for sufficient time to record secondary compression in the foundation soils. Given the extended time frame required for a pre-construction trial, the need for one should be recognised at an early stage of the project so that it can be allowed for in the programme.

The amount of consolidation settlement may be calculated with a reasonable degree of accuracy within acceptable limits given good quality laboratory test data. However, the assessment of the rate of consolidation is more difficult to determine as it depends on the rates of drainage allowed by the soil fabric and the drainage boundary conditions. A method of predicting the final settlement and assessing the coefficient of consolidation (c_v) was developed by Asaoka (1978), and is presented in Box 7.29.

Box 7.29 ***Assessment of settlement and the coefficient of consolidation (c_v) based on settlement/time plots (from Asaoka, 1978)***

Requirements:

- settlement readings are taken at constant time intervals (Δt) or are interpolated from the time settlement plot
- the structure and properties of the foundation soils should not be too variable
- the field condition is analogous to the settlement of a layer of thickness (H) according to Terzaghi's theory of consolidation
- the predicted settlement will essentially be that due to primary consolidation.

Procedure:

- draw the time settlement curve ($s(t)$) for the foundation soil
- select a time interval (Δt) and assess the settlement (s_i) at times $(t_0 + i\Delta t)_{i=0,1,2,3}$ (Figure 7.58)
- re-plot the settlement data as $(s_i \text{ versus } s_{(i-1)})_{i=0,1,2,3}$ (Figure 7.59)
- draw a best fit straight line through data points. Early data points may not be aligned with the later data and these are usually ignored. This may in part be due to the length of the construction period, non-uniform initial pore water pressures and/or heterogeneous ground
- the average c_v can be calculated from the slope of the line (β) through the data points:

$$c_v = \frac{-5H^2}{12\Delta t}\ln\beta \tag{7.70}$$

- sketch on the plot a line at 45°, ie $s_i = s_{(i-1)}$. The line through the data (s_i versus $s_{(i-1)}$) intersects the 45° line at s_∞, the final settlement.

Variations on the Asaoka plot:

- stage construction results in a series of approximately parallel straight lines on the plot. Each can be treated independently
- under constant load a change in slope can indicate the transition from primary settlement to secondary compression. The continuation of the secondary compression line to intersect with the $s_i = s_{(i-1)}$ gives a s_∞, which includes secondary compression
- curvature of the line passing through the data points can indicate a reduction in c_v with increased effective stress.

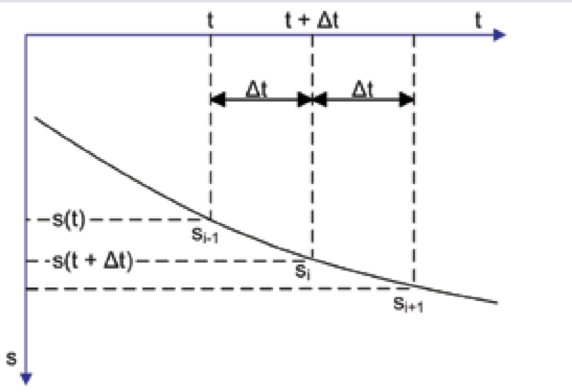


Figure 7.58 ***Settlement/time plot***

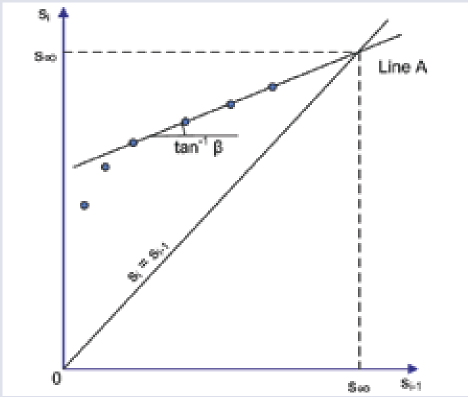


Figure 7.59 ***Relationship between S_i and S_{i-1}***

7.7.4.4 Rate of construction

During construction of a trial section of levee the rate at which it is raised will be dictated by either the rate of production that can be achieved given the monitoring constraints or a restriction placed on the rate of filling for geotechnical reasons to maintain stability. To maintain stability the amount of fill placed over any time interval would normally be expected to reduce as the height of the levee is increased. In reality, under normal circumstances with no ground improvement, unless there is a significant programmed pause in construction of the order of six to 12+ months, the gain in strength of the foundations soils and the improvement in stability achieved during construction will be limited. Where the rate of filling is not constrained by stability issues, an artificial restriction could be imposed, such as restricting the number of layers of fill that can be placed in a week. This serves the following purposes:

- provides a rate of filling that may be more representative of those achievable during the construction works
- provides time for the field data to be gathered, analysed and interpreted while filling is still ongoing
- if data indicate that the trial is approaching failure then there may be time for action to be taken to halt filling or even remove placed fill.

7.7.5 Ground investigation validation through visual observation during construction

The processes of ground characterisation are generally based on data from non-intrusive and intrusive investigation. By their nature, intrusive investigations only recover very small volumes of soil for inspection and testing at relatively widely spaced, discrete locations. However, variability in the ground can occur over small distances. So, there is the probability that despite the best planned and executed investigations some ground features may not be identified, or the true extent of features identified may not be realised.

As part of the construction works on levees, there are a number of activities that will result in large exposures of the ground surface and subsurface:

- removal of topsoil from below the new levee footprint or over the surface of the borrow area
- removal of topsoil from the surface of the levee and excavations within the body of the existing levee
- excavation or enlargement of the drainage channel, which parallels the levee and that may be used to provide a source of fill material
- excavations in the borrow area
- other excavations (service diversions, drainage runs, associated civil works etc).

All of these activities provide fresh and extensive exposures of the surface and subsurface deposits, which can provide clues as to the nature of the ground and can be used to identify locations where ground conditions are consistent or at variance with those assessed from the ground investigation data. In this way localised adverse ground conditions may be identified and, if considered appropriate, their potential effects mitigated through location specific design revisions.

Full opportunity should be taken to examine all fresh exposures to ensure that the ground conditions are consistent with those assessed through the ground investigation process. For this, the observer needs to have a good understanding of the ground conditions and design assumptions so that any features that conflict with the interpretation can be identified. When a conflicting feature is discovered it should be cross referenced with information in the updated CSM to assess whether there is existing data on the feature, whether it may have been misinterpreted and whether it will affect the design. If the causes of these features and their likely effects on the design cannot be assessed based on the existing information then additional ground investigations should be undertaken. In the first instance consideration could be given to undertaking additional trial pits using existing site-based plant, if ground conditions are suitable, under the supervision of an appropriately qualified person. If this does not provide sufficient information then other forms of investigation should be considered required. Examples of some features that may be evident during construction actives are included in Table 7.57. This is not an exhaustive list but serves as an illustration.

Table 7.57 Examples of some field observations during construction and their potential implications on levee performance

Localised field observation	Possible cause	Potential implications on levee performance
• low ground at variance with the general topography • darker soil colour • peat deposits • rutting by earth moving plant • encountering weaker or more organic soils • localised failure of excavations (side slopes and base heave).	• possible paleochannels in-filled with softer or organic/peat soils	• greater thickness of fill placed to achieve defence level • larger settlements due to greater compressibility of foundation soils and increased fill thickness • instability due to weaker foundation soils • fill unacceptable or increased shrinkage/ fissuring where used

Table 7.57 ***Examples of some field observations during construction and their potential implications on levee performance (contd)***

• higher ground at variance with the general topography • lighter soil colour • non rutting by earth moving plant • seepage • non-cohesive deposits, more permeable horizons or land drains.	• possible paleochannels in-filled with coarser grained soils • possible point bars • possible deposits of coarse grained material laid down during a flash flood.	• less settlement due to lower compressibility foundation soil • seepage and piping through the foundation • instability through uplift on landside where higher permeability soil is capped by lower permeability soil • fill unacceptable or seepage, piping and erosion where used.

7.8 GEOTECHNICAL PARAMETERS

The geotechnical parameters of soils relate to their classification, mechanical and hydraulic properties, and their spatial distribution within the soil profile. They are required to assess how the levee will behave during construction and interact with the environment, external actions and foundation soils in the longer term. As the soils forming the levee and the foundation soils are natural materials they will be laterally and vertically variable, even within a defined horizon, which itself will vary in thickness. As such, data from the tests for any given property will vary within limits, reflecting natural soil variability. It will also be affected by sample and specimen disturbance and different testing methods. Data from an investigation has to be interpreted in order to establish characteristic values or range of values, to use in the design or assessment. This section considers the determination of geotechnical properties that are relevant to levees and an approach to the interpretation of the dataset to arrive at the characteristic values.

A flow chart mapping the outline structure and contents of Section 7.8 is presented in Figure 7.60, with details given in Table 7.58.

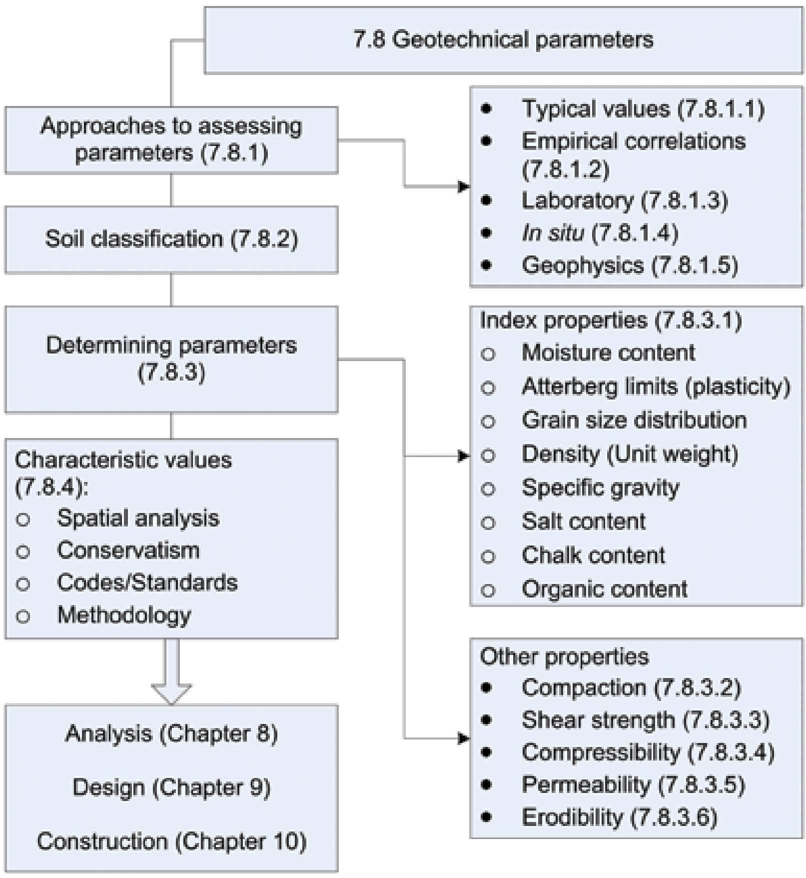


Figure 7.60 ***Structure and content of Section 7.8, and interaction with other sections***

Table 7.58 Geotechnical parameters and their relevance to levees

Description	Methods	Seepage and internal erosion	Stability	Settlement	Erodibility	Seismicity	Construction QA/QC
Soil classification	Section 7.8.2	X	X	X	X	X	X
Moisture content	Section 7.8.3.1	X	X	X	X	X	X
Atterberg limits			X		X		X
Grain size distribution		X	X	X	X	X	X
Density (Unit weight)		X	X	X	X	X	X
Specific gravity			X				X
Salt content		X				X	X
Chalk content				X	X		X
Organic content			X	X	X		X
Moisture-density relationship • optimum moisture content • maximum dry density.	Section 7.8.3.2	X	X	X	X		X
Undrained shear strength	Section 7.8.3.3		X			X	X
Drained shear strength			X			X	
Compressibility • elastic settlement • primary consolidation • secondary compression • undrained elastic modulus • compression index and compression ratio • coefficient of volume compressibility • yield stress • coefficient of consolidation • coefficient and modified coefficient of secondary compression.	Section 7.8.3.4	X	X	X		X	X
Permeability	Section 7.8.3.5	X	X				X
Erodibility	Section 7.8.3.6	X			X		

7.8.1 Approaches to assessing geotechnical parameters

Geotechnical parameters can be derived in a number of ways. It is preferable to assess them using more than one approach to validate the parameters.

- typical values
- empirical correlations
- measurement in the laboratory
- measurement *in situ*
- measurement using geophysical techniques.

7.8.1.1 Typical values

Many geotechnical publications, codes, standards and guides include tables giving a typical range of values for parameters based on generalised soil descriptions. Typical values serve as a broad guide when assessing the validity of the magnitude of a parameter evaluated by other means, such as empirical correlations or direct measurement. Their value lies in allowing the development of an understanding of how the levee may perform qualitatively when only existing geological data gathered during the desk study is available. Unless the typical values are site or regionally specific, any quantitative calculation should be treated with extreme caution. Some typical values for parameters are included in Section 7.8.3.

7.8.1.2 Empirical correlations

Published correlations can be used to relate classification test data to mechanical and hydraulic parameters, and other properties. They provide a valuable method of validating test results or establishing parameters where no direct test data are available, eg during the early stage of development of a project as part of the process of collating the initial CSM. Some typical empirical correlations for a range of parameters are presented in Section 7.8.3.

7.8.1.3 Measurement in the laboratory

The purpose of laboratory testing is to describe and classify the samples, investigate the fundamental behaviours of the material and establish geotechnical parameters relevant to the technical objectives of the investigation. The way in which the levee and foundation soils will respond to the imposed loads, through an understanding of the failure and deterioration modes (Section 3.5, and Tables 7.44 and 7.45) should be understood to ensure that appropriate laboratory tests are scheduled. Information on locating intrusive investigations, and the sample types and sampling frequency are presented in Sections 7.9.6 to 7.9.8.

Scheduling laboratory testing

While many samples may be taken as part of an investigation, not all will be tested. The selection of samples for testing through the preparation of a testing schedule is not a random process. Scheduling should be undertaken with a hierarchical structure (Table 7.59). Index tests are usually undertaken on as many samples as possible, which should include, but not be restricted to, those samples taken at the depth of *in situ* tests and those samples to be tested for the determination of mechanical (strength, consolidation, compaction) and hydraulic (permeability, erosion resistance) properties. This will allow the directly measured mechanical or hydraulic properties of samples to be inferred to samples with similar index properties and compared with those derived through empirical correlations.

To ensure a suitable distribution of test types and numbers within each stratum, it is advisable to establish an understanding of the ground profile. For example, by sketching a geological section as the investigation progresses and identifying the locations of scheduled tests on the section (Table 7.43). Scheduling tests on samples taken at relatively close centres within a horizon from the same exploration hole can be more informative of the variation in soil properties than a series of tests at different depths across a range of exploration holes.

When scheduling tests, due consideration needs be given to the quality of the sample, which should be compatible with the test to be performed (Section 7.9.8). Consideration also needs to be given to the *in situ* loads and the change in load that will be applied to the *in situ* material as a result of the presence of the levee, and how these could cause the material to behave (deform, consolidate, shear [compression, direct, extension, anisotropy], flow of water etc). The loads applied in the laboratory need to encompass the range of loads anticipated in the field.

Table 7.59 ***Generic illustration of a hierarchical system of scheduling laboratory tests***

Field work – sampling and *in situ* testing					**Laboratory testing**	
Notional depth zone		*Sampling*	*Routine in situ testing (eg SPT, vane shear, may be within each depth zone)*	*More costly routine and non routine in situ testing (eg in situ permeability, pressuremeter)*	*Index properties*	*Mechanical/ hydraulic properties*
Increments of increasing depth	A	■				
	B	■	■		■	
	C	■				
	D	■	■	■	■	■
	E	■				
	F	■	■		■	
	G	■				
	H	■	■		■	■
	I	■				
	J	■	■			
	K	■				
	L	■	■		■	
	M	■				
	N	■	■	■	■	■

Notes

Investigations in depth zone:

- D and N allow comparison of *in situ* and laboratory measured parameters with those derived from correlations based on index properties and, if appropriate, routine *in situ* tests.
- H allow comparison of laboratory measured parameters with those derived from correlations based on index properties and, if appropriate, routine *in situ* tests.
- B, F and L allow *in situ* and measure parameters to be inferred based on soils with similar index properties and, if appropriate, routine *in situ* tests.

7.8.1.4 Measurement *in situ*

In situ tests routinely form part of a phased programme of investigation and the quality control programme during the construction of a levee. They are usually employed where:

- a prompt result is required
- where the mass characteristics of the soil are expected to be different from those derived through laboratory tests on samples
- when it is difficult to obtain representative samples (for example, non-cohesive soils)
- to complement the results of laboratory tests.

The simple forms of *in situ* testing (*in situ* density, SPT, vane shear, permeability, CPT dissipation) are common place within routine investigations and provide supporting data for direct comparison with laboratory tests or empirical correlations with index properties. However, the volume of soil affected by these routine *in situ* tests is limited and the results may not be entirely representative of the mass characteristics of the ground. Where a more robust assessment of a soil parameter is required, larger scale *in situ* tests may be justified that are designed around known ground conditions and are located to maximise the information they provide. These could include a pumping test for the assessment of permeability (Section 7.8.3.5) and a trial section of levee for the assessment of the undrained shear strength or compressibility characteristics of the foundation soils (Section 7.7.4). These forms of test would generally only be undertaken on larger projects where a robust assessment of these characteristics is required as they have a significant impact on the project risk and cost.

In situ testing frequency

The factors to be considered in assessing the frequency of *in situ* testing are consistent with those discussed for sampling (Section 7.9.8.3). The location and distribution of *in situ* tests can be identified on a geological section, as discussed for the laboratory testing. The frequency of *in situ* testing is generally a reflection of the cost of the test, which is indicative of the staff and equipment employed to perform the test and the 'down time' of other equipment while the test is being undertaken. Basic forms of *in situ* tests such as SPT and vane shear tests may be undertaken at a frequency consistent with sampling. Other forms of routine *in situ* testing are a little more costly, such as rising and falling head permeability tests, and CPT dissipation tests. These may be undertaken at a frequency that is sufficient to provide a dataset against which parameters derived through laboratory tests or empirical correlations can be compared for a given soil type.

Some forms of *in situ* testing, such as the penetration shear vane, do not require a borehole as the equipment is jacked or pushed into the ground from the ground surface. So, there is no conflict of interest between the frequency of *in situ* testing and the requirements for sampling.

7.8.1.5 Geophysical measurements

Geophysical techniques, which are described in Section 7.9.6, along with their applications, provide a means to determining some *in situ* properties as well as the stratigraphic and variations in engineering properties of the levee foundation soils and the internal structure of the levee. Resistivity techniques may be used to indicate assess variations in stiffness, moisture content or porosity, and clay/sand content, while seismic techniques may be used to indicate variations in shear strength. Determination of the broad characteristics of the levee structure and foundation soils using geophysical methods can significantly reduce subsurface uncertainty.

However, geophysical data are generally dependent on other factors. Measured geophysical properties (eg resistivity, seismic velocity) are generally dependent on intrinsic physical soil properties. For example, variations in resistivity can be a function of changes in porosity, water saturation, pore shape, presence of clays and water chemistry. Geophysical data can be used as a screening process to highlight potentially anomalous variations in geotechnical parameters, which can then be tested using intrusive investigation techniques.

Geophysical measurement of geotechnical parameters

Some geophysical techniques can be used to assess geotechnical parameters. The geotechnical properties are mainly limited to *in situ* elastic properties such as small-strain shear, bulk and Young's moduli and Poisson's Ratio but other physical properties may be evaluated, such as porosity, density, permeability etc. Table 7.60 presents a summary of the primary parameters that can be measured directly using geophysics and the other parameters that can be inferred from these primary parameters.

Table 7.60 Soil properties measured using geophysics and related geotechnical parameters

Measured property using geophysics	Primary geotechnical parameter(s)	Other related properties
Electrical Resistivity	Porosity Permeability	Clay content Saturation Salinity
Electromagnetic Conductivity	Porosity Permeability	Clay content Saturation Salinity
Seismic velocity (P-wave)	Dynamic bulk modulus	Porosity Density Saturation
Seismic velocity (S-wave)	Dynamic shear modulus	Density Porosity
Seismic velocity (Rayleigh wave)	Dynamic shear modulus	Dynamic bulk modulus
Gravitational field strength	Density	Porosity
Magnetic field strength	–	Presence of ferrous material

Caution

Measured geophysical quantities may be dependent upon one or a number of geotechnical and other physical or chemical parameters. Only seismic velocity can be measured directly. For example resistivity can vary as a function of porosity, but also as a function of water saturation and composition, pore geometry and clay content. Prediction of geotechnical characteristics in particular from geophysical data requires calibration against reference information usually acquired from boreholes or probing.

7.8.2 Soil classification

The classification of soils is important as it allows the nature of a soil to be recorded and communicated using terms that have defined meanings and from which, as a first step, engineering behaviour can be inferred. Soil is classified based on the relative percentages of different particle sizes (clay, silt, sand, gravel) in combination with other aspects of the material content (eg organics or shell) and material properties and behaviour (eg strength, density, plasticity). Classification serves as a common language for communicating a description of soil so that others, who were not in the field and did not observe the soil first-hand, will have the same understanding of the soil type and composition. Classification is complicated by the fact that:

- multiple systems are in use, which are generally standardised by nation
- fundamental soil components, such as 'sand', are defined differently in each system
- soils are mostly classified in the field using visual manual methods, while accurate classification requires laboratory testing and only a small amount of soil inspected in the field samples is sent to the laboratory for testing.

The key point is that when using historic data and geological studies, it is important to find out which classification system was used to ensure an accurate interpretation of the soil descriptions.

Natural soils comprise individual particles. At the extreme fine limits (clay) these particles occur as very thin plates while in coarser soils (silts and sands) the particles are more spherically shaped and typically composed of mineral grains. The very large specific surface areas of the clay plates cause them to have an electrochemical reaction with water, giving them different characteristics to the coarser grained soils. The size of the particle is used to classify the soil. A number of classification systems have been developed that have some differences. Figure 7.61 shows a graphic comparison of particle size definitions used by some different systems with notes of discrepancies between systems shown in Box 7.30.

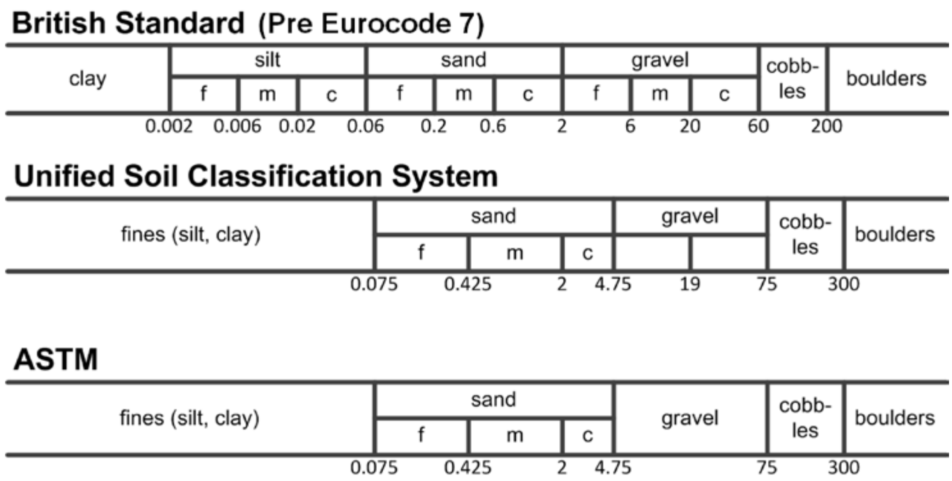


Figure 7.61 Particle size classifications as defined by various systems (pre-Eurocode British Standard illustrated)

Most of the definitions across the various systems are similar, but not identical. The greatest difference is in the maximum particle diameter size for defining coarse sand as given by:

- Unified Soil Classification System (USCS) (after Casagrande, 1948): 4.75 mm

- ASTM D2487-11 (2011): 4.75 mm
- ASTM D422-63 (2007): 4.75 mm
- BS 5930:1999+A2:2010: 2.0 mm
- ISO 14688-1:2002: 2.0 mm
- AASHTO M145-91-UL (2008): 2.0 mm
- Wentworth (1922): 2.0 mm.

As a result a given particle could be defined as either a coarse sand or fine gravel depending on the system.

Box 7.30 Notes on BS (pre and post Eurocode) and Wentworth classification systems

British Standard (BS 5930:1999+A2:2010) includes an update following the implementation of Eurocode, which adopted the ISO (EN ISO 14688-1:2002). Part of the undate included the terms for defining the particle sizes. The update means that particle size diameters previously defined by a 6 are replaced with 63. So, under the new British Standard sand ranges between 0.063 mm and 0.2 mm, compared with 0.06 mm and 0.2 mm, previously.

Many geologists and geomorphologists, who do not have an engineering geology background, may be accustomed to using the Wentworth scale, which defines gradation boundaries using the phi scale:

Diameter (mm) = $2^{-\Phi}$

In the Wentworth system, particle size steps occur at powers of two. The particle diameter equals two raised to the negative phi exponent. For example, medium sand is between 2^{-2} (0.25 mm) and 2^{-1} (0.5 mm).

Fine grained soils comprising silt and clay are classified based on their consistency and particle size. The remoulded consistency of the soil depends on the moisture content. Four consistency states are acknowledged: liquid, plastic, semi-solid and solid. The moisture contents at which fine grained soils transitions between the liquid and plastic states and the plastic and semi-solid states are defined as the liquid limit (w_l) and the plastic limit (w_p), respectively. Collectively they are also known as the Atterberg limits. Their value is affected by clay content, clay mineralogy, exchangeable cation and organic content.

Fine grained soils are classified based on the plasticity chart (Figure 7.62) by plotting liquid limit against plasticity index (plasticity index = liquid limit – plastic limit). The chart is zoned to classify the soil, as having a high (H), or low plasticity (L), some classification systems include a zone for intermediate plasticity, for example the BS system. The chart aims to differentiate clay soils from silts by the A-line – soils that plot above the A-line are clays, while those that plot below are silts. A second line, the U-line, defining the upper limit for soils is also often included on the chart. Figure 7.62 shows the plastic chart based in the USCS.

Under the BS system the plasticity of the fines is further subdivided over a range of liquid limits, as detailed in Table 7.64.

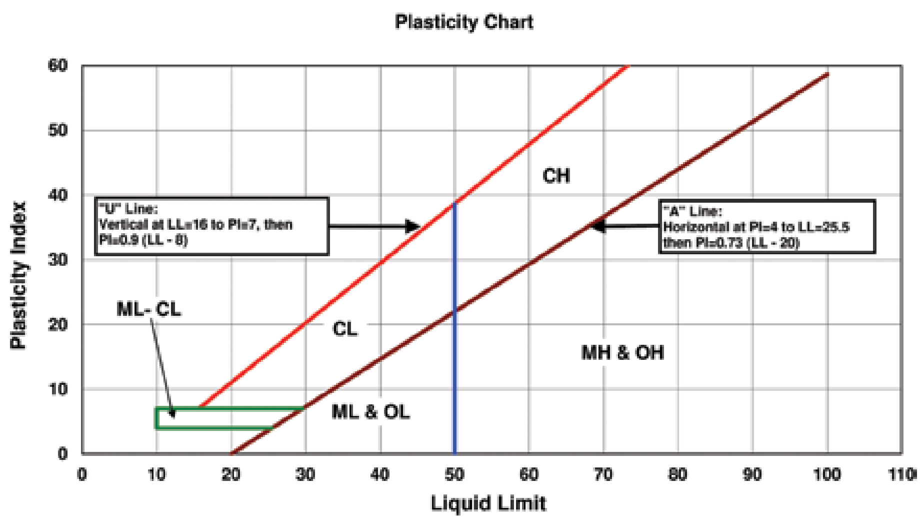


Figure 7.62 Example of a plasticity chart based on the USCS of soil classification

Comparison of classification systems

The two systems in widest use are the BS and the USCS, which is almost identical to the ASTM method. Both use laboratory testing methods of classification and field descriptions based on a visual-manual procedure. They both divide soils broadly into coarse or fine grained and then add progressive descriptive subdivisions based on other properties. Coarse grained soils are sub-classified by the degree of grading and the percentage of fine grained soils included within them (fines content). Fine grained soils are sub-classified by plasticity.

Tables 7.61 to 7.64 summarise and compare the USCS and BS systems for coarse and fine grained soils. The tables are not intended to be comprehensive and do not show a full representation of the classification systems but serve to illustrate some key differences.

The respective plasticity charts should be used to classify fine grained soils in both systems.

Symbols are often used as a shorthand to describe a soil group as shown in Figure 7.62. These may comprise:

- G = gravel, S = sand, M = silt, C = clay, O = organic, F = fine grained
- W = well graded, P = poorly graded
- L = low plasticity, H = high plasticity, I = intermediate, V = very high, E = extremely high

Table 7.61 USCS: coarse grained soils

Unified Soil Classification System (USCS)								
Less than 50% fines (<0.075 mm)								
Grading	**GRAVELS 50% or more of coarse material >4.75 mm**				**SANDS 50% or more of coarse material <4.75 mm**			
	Clean gravels		*Gravels with fines*		*Clean sands*		*Sands with fines**	
	Well-graded GRAVEL	Poorly-graded GRAVEL	Silty GRAVEL	Clayey GRAVEL	Well-Graded SAND	Poorly-Graded SAND	Silty SAND	Clayey SAND
% fines	0–5	0–5	>12	>12	<5	<5	>12	>12

Table 7.62 BS: coarse grained soils

British Standard (BS 5930:1999+A2:2010)		
No fines (<0.063 mm)		
Term	*Principal soil type*	*Approximate % secondary constituent*
Slightly sandy or gravelly	SAND or GRAVEL	<5
Sandy or gravelly		5–20
Very sandy or gravelly		>20
	SAND and GRAVEL	Equal proportions

Less than 35% fines (<0.063 mm)						
Grading	**GRAVELS 50% or more of coarse material >2 mm**			**SANDS 50% or more of coarse material <2 mm**		
	Slightly silty or clayey GRAVEL	Silty GRAVEL Clayey GRAVEL	Very silty GRAVEL Very clayey GRAVEL	Slightly silty or clayey SAND	Silty SAND Clayey SAND	Very silty SAND Very clayey SAND
% Fines	<5	5–20	20–35	<5	5–20	20–35

Table 7.63 *USCS: fine grained soils*

Unified Soil Classification System (USCS)							
More than 50% fines (<0.075 mm)							
Grading	**SILTS and CLAYS Liquid limit <50%**			**SILTS and CLAYS Liquid limit >50%**			
	SILT Inorganic silts Rock flour	LEAN CLAY Inorganic clays (low plasticity)	ORGANIC SILT or CLAY (low plasticity)	ELASTIC SILT Inorganic silts (high plasticity)	FAT CLAY Inorganic clays (high plasticity)	ORGANIC CLAY ORGANIC SILT (high plasticity)	PEAT (highly organic soils)

Table 7.64 *BS: fine grained soils*

British Standard (BS 5930:1999+A2:2010)					
More than 35% fines (<0.063 mm)					
Grading	**Gravelly or sandy SILTS and CLAYS 35% to 65% fines**		**SILTS and CLAYS >65% fines**		
	Gravelly SILT Gravelly CLAY	Sandy SILT Sandy CLAY	Slighty gravelly SILT Slighty gravelly CLAY	Slighty sandy SILT Slighty sandy CLAY	SILT CLAY
Liquid limit %			Plasticity		
<35			Low		
35–50			Intermediate		
50–70			High		
70–90			Very high		
>90			Extremely high		

Field descriptions of soil

Field descriptions of soil are made based on visual-manual methods to estimate relative percentages and plasticity without the benefit of laboratory test results where relative percentages are determined on the basis of dry weight, which can be difficult to assess visually. Sand gauges are commercially available to aid in the description of granular soils. These cards show sand grains of various sizes, with varying degrees of angularity and roundness. Sand gauges exist for each of the major systems. Silt and clay are difficult to differentiate based on grain size, as the particles are smaller than what can be discerned visually, and so are identified based on their behaviour. Silts will display dilatancy, have a low dry strength, and tend to have lower plasticity. Clays will not be dilative and will have medium to high dry strength and higher plasticity. Estimates of plasticity based on air dry strength are shown in Table 7.70.

The soil description includes more information than a summary of the soil classification and captures details such as colour, moisture, consistency (strength or density), soil structure (layering and fabric), and other features that are difficult to preserve. Country codes and standards detail the format for describing soils. While nomenclature varies, a description generally consists of the strength or density, colour, primary component preceded by the secondary component as a modifier, soil classification symbol, followed by ancillary components and their relative per cent and particle size (fine, medium, coarse), any other observations (odour, roots, organics etc) and geologic origin. An example classification of a soil type is given in Box 7.31.

Box 7.31 *Example classification (USCS): silty SAND (SM)*

Description

Loose, light grey, silty (10%–20%) fine SAND (SM), wet, trace organics, odour, grains are sub-rounded, some stratification (alluvium).

It is essential that modifying terms (much, some, trace), which may be defined in the various country codes, are understood by all staff to ensure consistency in descriptions. Other descriptive terms may include an estimate percentage of each size fraction in parenthesis or the use of further modifying terms (slight, very).

Good advice for the engineer or geologist going into the field is to develop simple, laminated 'cheat sheets' summarising the classification system that is being used on a project. Field staff should make the opportunity to calibrate their visual classification skills by comparing their field descriptions with laboratory classifications. It may be preferable for field logs to show both the original field descriptions and the laboratory classifications.

Classification of peat and organic soil

Peat represents an extremely important soil type for levees given the frequency with which it is encountered in the alluvial environment, and the issues it represent in terms of long-term settlement, low density, typical low strength and shrinkage potential. Specialised classification systems have been developed to provide a greater level of detail and description to the properties of the peat.

The description of peat and other organic soils can be described according to the degree of decomposition, determined by squeezing, and fibre content. The Von Post (1922) Scale of humification is a botanical classification system based on the depth of sample, peat type, degree of humification, moisture content, fibre content, and presence of woody remnants. In ISO 14688-1:2002 standard, a classification is outlined based on the moisture content and fibre content. ASTM D4427-13 classifies peat in terms of absorbency (retained moisture content), acidity, ash content (after burning), botanical composition and fibre content (determined by sieving). The three classification systems for peat are compared in Table 7.65.

Table 7.65 Comparison of peat classification systems

ISO 14688-1:2002		Von Post (1922) Scale of humification		ASTM D4427-13	
Classification	**Description**	**Classification**	**Description**	**Classification**	**Description**
Fibrous peat	Fibrous structure, easily recognisable plant structure, retains some strength	H1 to H2	Completely, to almost entirely, un-decomposed peat, when squeezed releases clear to yellowish water. Plant remains easily identifiable. No amorphous material present, no peat passes between the fingers.	Fibric	Peat with greater than 67% fibres.
Pseudo-fibrous peat	Recognisable plant structures – no strength of apparent plant material	H3	(Very) slightly to moderately high decomposed peat, releases muddy brown water and contains no amorphous material to very 'muddy' water with some amorphous granular peat escaping between fingers when squeezed.		
		H4	Slightly decomposed peat, plant remains are slightly pasty.	Hemic	Peat with between 33 and 67% fibres.
		H5	Moderately decomposed peat, plant structure still recognisable, residue is very pasty.		
		H6	Moderately high decomposed peat, with a very indistinct plant structure.		

Table 7.65 Comparison of peat classification systems (contd)

Amorphous peat	No visible plant structure, mushy consistency	H7 to H10	Highly decomposed to fully decomposed peat, with a lot of amorphous material and very faintly to hardly any recognisable plant structure. When squeezed, half to almost all material escapes as a fairly uniform paste.	Sapric	Peat with less than 33% fibres.
Gyttja	Decomposed plant and animal remains. May contain inorganic constituents	Not defined		Not defined	
Humus	Plant remains, living organisms and the excretions together with inorganic constituents, form the topsoil	Not defined		Not defined	

The organic content of soil is associated with lower specific gravity, and higher compressibility, secondary compression and shrinkage potential, as well as unsatisfactory strength characteristics. These adverse properties are the reason why classification by organic content is important when planning the construction and assessing the long-term serviceability of levees.

A number of engineering classification systems (ASTM D2487–11 and ISO 14688-2:2004) provide some form of soil classification based on organic content.

The ASTM D2487–11 (2011) classification system considers organic soils as a subgroup of fine-grained soils. A soil is termed organic if its liquid limit, as determined from oven dried and re-wetted samples, is less than or equal to 75 per cent of the original wet prepared liquid limit. However, in the USA the usual practice is to measure the organic content directly through a burn-off test. The system classifies whether a soil is a highly organic soil or peat based on the prevalence of organic matter, their dark colour and organic odour. For methods for determining organic content see Table 7.82.

ISO 14688-2:2004 includes organic content as a key characteristic used in classifying soils. Soils with between two and six per cent organics are termed 'low organic', while those with six to 20 per cent organics are termed 'medium organic'. Soil with an organic content of greater than 20 per cent are classed as highly organic soil but there is no indication of when the organic content become larger enough for the soils to be classified as a peat. Peats are classified based on the degree of decomposition, determined in the wet state by squeezing, and on its fibre content (Table 7.65). The intensity of odour and colour also indicates the proportion of organic matter, and should be described.

7.8.3 Determination of geotechnical parameters and methods

This section provides an overview of typical methods of estimating the geotechnical parameters of soils used in the assessment of levees, along with their applications and limitations. The methods include laboratory and *in situ* tests, together with an indication of how the properties can be estimated through correlations with index properties. Typical values of the parameters are also presented, where appropriate. The parameters are considered under the broad groupings of:

- index properties
- compaction

- shear strength
- deformability
- permeability
- erodibility.

When selecting the method used to assess the parameter consideration needs to be given to:

- appropriateness to the phase of investigation, taking account of the form of the levee and level of risk
- relevance of the method to how the result will be used in assessing the performance of the levee
- how representative the sample is of the ground profile
- effects of sample quality on the property to be measured, including other related factors such as transportation, sample handling and specimen preparation
- quantity of representative material available for testing
- time available for testing based on the project programme
- budget available for testing
- required precision of parameter.

The geotechnical parameters obtained through testing should not be used without being validated to ensure that they are reasonable in the context of other data obtained during the investigation. Methods of validation might include comparisons with:

- local experience on similar soils
- published typical values based on a wider range of data
- values derived through published correlations with index properties, taking account of geographic influences
- alternative test methods
- results of *in situ* testing on similar materials
- levee past performance and maintenance history
- data from earlier investigations.

Preferred methods of testing and procedures may vary from country to country, and reference should be made to the relevant country codes and standards.

The determination of the geotechnical parameters for deposits of made ground (artificial, not naturally occurring) and peats can be particularly problematic. General points to consider when dealing with these materials are summarised in Table 7.66.

Table 7.66 Some considerations when determining the geotechnical parameters of made ground and peat

Soil type	Considerations

Made ground	• it can be highly variable in composition, containing natural and/or manmade materials
	• some of the materials could comprise large particles, which can obstruct intrusive investigations and hinder construction
	• it may contain contaminants, which could be leached/mobilised through changes in groundwater level and flow caused by the construction of the levee
	• it may not be adequately compacted and experience some collapse settlement when inundated with floodwater
	• where the made ground is found to be relatively uniform it can be sampled and tested as appropriate for a comparable natural soil. Tests may cost more if the samples are contaminated
	• where the made ground is variable it may be appropriate to:
	  ○ undertake *in situ* tests so that a larger and more representative soil mass is tested
	  ○ adopt cautious design parameters
	  ○ adopt engineering solutions, which mitigate reliance on critical parameters (eg permeability – install a cut-off, obstructions permitting).

Peat	• it ranges in composition from a relatively fresh mass of organic matter to a completely decomposed mass where no plant remains can be identified • it generally has a high moisture content and exhibits very high compressibility and significant long-term secondary compression • it loses mass and volume on drying. At the surface it is prone to oxidation and dispersion by the wind • the measurement of failure strength is problematic as the peat fibres can act to 'reinforce' the specimen. Laboratory testing protocol in the Netherlands suggests that it is not evident that peat behaviour can be described with the Mohr-Coulomb shear model • the behaviour of small samples of heterogeneous fibrous peat tested in the laboratory may not be representative of the *in situ* behaviour • the very low *in situ* effective stresses in peat can not normally be replicated in laboratory shear strength testing equipment (ie triaxial cells) • the tolerable shear strain is an important design consideration. Peat mobilises low shear strength at low shear strains. Deformations may be large if high shear strength and large shear strains are permitted • measured shear strength (total and effective stress measured *in situ* or the laboratory) can be high particularly where the peat is fibrous. It may be appropriate to undertake routine strength tests but to adopt a cautious pragmatic approach to assessing a characteristic value.

7.8.3.1 Index properties

Index properties are fundamental soil characteristics that define how the soils forming the levee and the foundation soils will perform. They are used to determine the uniformity or variability of a soil type across a site or a number of sites, allowing measured geotechnical properties to be extrapolated to soils with similar index properties. Many empirical correlations have been developed over the years, which relate index properties to other geotechnical properties and behaviours. Common index properties appropriate to levees include:

- moisture content
- Atterberg limits
- grain size distribution
- density (unit weight)
- specific gravity
- salt content
- chalk content
- organic content.

They can readily be determined at low cost and only require disturbed samples, unless the soil has a high level of soil fabric or specific sub-horizons. They are used to classify soils but the system of classification can vary for different countries (Section 7.8.2).

Moisture (water) content

The moisture (or water) content (w) of a soil is a key parameter that determines its behaviour. It tends to be referenced against other parameters, such as Atterberg limits, and not in isolation. In levees it is important as in cohesive soils it will control the undrained shear strength (Section 7.8.3.3), compressibility (Section 7.8.3.4) and shrinkage potential (Section 9.12.2). It also governs the degree of compaction that can be achieved in a soil for a given compactive effort (Section 7.8.3.2), and is a factor influencing the occurrence of liquefaction (Section 8.8.4) and the behaviour of granular soils during transient seepage conditions (Section 8.3.1).

It is defined as the mass of moisture removed from a soil by oven drying (M_w) as a ratio of the mass of the dry soil (M_d). It is usually represented as a percentage.

$$w\,(\%) = 100 \text{ x } M_w / M_d \tag{7.71}$$

Where a soil contains coarse grained particles and the moisture content is to be related to the Atterberg limits (determined on the fines fraction) for use in empirical correlations, the moisture content needs to be corrected to give the equivalent moisture content of the fines fraction.

Methods for determining moisture content are provided in Table 7.67.

Table 7.67 Methods of determining moisture (water) content

Method	Applications	Limitations
Moisture content probe	• rapid field measurement during earthwork operations.	• may not be suitable for granular soils with little or no fines in contact with probe • initial checks advisable against oven drying.
Nuclear density probe	• rapid field measurement during earthwork operations.	• not suitable where there is surface water • initial checks advisable against oven drying • a bias may be needed to account for soil type or material composition • moisture determinations below 75 mm to 100 mm depth are typically not accurate.
Oven drying at 105 to 110°C (some country codes allow over drying at 105 to 115°C)	• definitive method for all soils.	• 16 to 24 hrs before results are available • soils with minerals containing water of crystallisation (ie gypsum) should be dried at 80°C • peats and organic soil should be dried at 60°C to prevent oxidation.
Speedy: calcium carbide	• rapid field measurement during earthwork operations • best suited to granular soils.	• approximate method. Check against oven drying • can be unreliable in clay soils • moisture content related to mass of wet soils rather than dry soil.
Sand bath	• rapid field measurement during earthwork operations.	• approximate method • check against oven drying.
Microwave or stove	• rapid field measurement during earthwork operations • microwave offers favourable comparisons with oven drying tests on fine-grained soils.	• sample may become overheated or unevenly heated • non-metal containers (microwave) • field power source needed (microwave) • not appropriate for organic soils.
Relevance to levees		
• compaction control during construction of levee • control of shrinkage in clay soils • high moisture content could indicate soft and/or organic soil with low shear strength/high compressibility leading to potential stability/settlement and shrinkage problems • estimate of the engineering parameters of soils, in combination with other index properties, through empirical correlations.		

Atterberg limits

The Atterberg limits are applicable to fine grained soils (clays/silts) or for the fines fraction of a more broadly graded soil. They determine the characteristics and behaviour of a soil. An acceptable range of values can be defined for a soil over which these behaviours are considered to be suitable for use in the construction of a levee.

The Atterberg limits define the range of moisture content, usually expressed as a percentage, over which the soil behaves in a plastic state. At the higher moisture contents, those above the liquid limit (w_l), a soil begins to behave as a viscous liquid. At the lower moisture contents, those below the plastic limit (w_p), the soil begins to become brittle. These limits typically vary for soils of different geological origin, reflecting its clay content, clay mineralogy, exchangeable cation and organic content.

Sample preparation is an important consideration as the results can be affected by the method of drying. Air drying is preferred over oven drying. However, wet preparation is the most preferable method as it minimises the potential for changes, or reduction, to occur in the liquid limit when the sample is drying below the as-received moisture content. Tropical residual soils are particularly susceptible to changes due to the method of sample preparation.

The plasticity index (I_p) is the range of moisture content over which the soil behaves plastically.

$$I_p = w_l = w_p \tag{7.72}$$

The I_p of a soil provides an indication of its classification when considered in combination with the liquid limit. It can be used to infer engineering behaviour such as shrinkage and swelling potential, fissuring potential (Section 9.12.2), effective friction angle (Section 7.8.3.3) and erosion resistance (Section 7.8.3.6).

The liquidity index (I_l) can be used to obtain an indication of the consistency or undrained shear strength (Section 7.8.3.3) of a soil. It is reported as a number: zero when the moisture content is equal to the plastic limit, and 1 when the moisture content is equal to the liquid limit. As the I_l increases, soil compressibility increases and undrained shear strength decreases.

$$I_l = (w - w_p)/I_p \tag{7.73}$$

The activity is an indication of the dependence of the plastic limit on the clay fraction of a soil (the percentage of particles finer than 2μm) and is a function of the clay mineralogy. It is reported as a number, and increasing values indicate a stronger influence on soil properties by the clay minerals.

$$\text{Activity} = I_p/\text{clay fraction} \tag{7.74}$$

A description of the relative activity of a soil as defined by its activity, along with some typical clay minerals, is presented in Table 7.68. Methods for determining plastic and liquid limits are given in Table 7.69. Estimation of plasticity based on air dry strength is shown in Table 7.70.

Table 7.68 Activity of clays

Description	Activity (typical clay mineral)
Inactive clay	<0.75 (kaolinite)
Normal clay	0.75–1.25 (Illite)
Active clay	1.25–2.00 (montmorillonite)
Highly active clay	>2.00 (sodium montmorillonite)

Table 7.69 Methods of determining plastic and liquid limits

Method	Applications	Limitations
Plastic limit: roll to 3 mm thread	• definitive method for plastic fines fraction.	• a small amount of soil is used, so precision is key for reliable results • accuracy/repeatability a function of the skill of the operator.
Liquid limit: four point cone penetrometer	• definitive method for plastic fines fraction.	• a small amount of soil is used, so precision is key for reliable results.
Liquid limit: Casagrande apparatus	• accepted alternative method.	• values may vary slightly from those obtained using the four point cone penetrometer • poor maintenance and adjustment of equipment can result in errors.
Liquid limit: single point cone penetrometer	• where rapid assessment is required.	• less accurate than the four point method • the design of the equipment varies according to the requirements of national standards and should be taken into account.
Relevance to levees		
• classification of soils and assessment of variability • I_l can be used as a screening tool to evaluate soil compressibility and undrained shear strength • acceptability criteria for use as engineered fill • estimate of performance characteristics and engineering properties, in combination with other index properties, through empirical correlations • high I_p in near surface slopes will generally require more maintenance due to soil fissuring and slope stability • erosion resistance.		

Table 7.70 Estimation of plasticity based on air dry strength (after Sower, 1979)

Plasticity	I_p (%)	Dry strength	Field test on air dry sample
Non plastic	0 to 3	Very low	Falls apart easily
Slightly plastic	3 to 15	Slight	Easily crushed with fingers
Medium plastic	15 to 30	Medium	Difficult to crush with fingers
High plastic	>30	High	Impossible to crush with fingers

Grain size distribution

The grain size distribution of a soil determines its classification and acceptability for use as a fill material in the construction of levees. It provides an indication of permeability (Section 7.8.3.5), filtering characteristics, erosion resistance (Sections 7.8.3.6), and compaction characteristics (Section 7.8.3.2). Typical methods for determining grain size distribution are given in Table 7.71.

Soils are a particulate material. They can comprise particles that are essentially single sized or cover a range of sizes. The grain size distribution is a measure of the percentage of material present in the soil that is smaller or larger than a specific size determined by passing the soil through a sieve of known mesh size in the case of coarse grained soils. For fine soils the equivalent percentage passing is determined by the application of Stokes' Law to a soil/water suspension as the soil particles settle under gravity over time. The mass of the soil particles remaining in suspension can be determined either by direct measurement on a subsample of the suspension or assessed from the specific gravity of the suspension.

Tests are not normally undertaken to determine the distribution of the fine soil fraction unless it exceeds 10 per cent of the soil mass.

An indication of the soil classification can be obtained from the grain size distribution curve by determining the coefficient of uniformity (C_U), derived from the ratio of the grain sizes for which 10 per cent and 60 per cent of the grains are finer, D_{10} and D_{60}.

$$C_U = D_{60}/D_{10} \tag{7.75}$$

C_U <4 to 6 soils tend to be poorly or uniformly graded
C_U >4 to 6 soils tend to be well graded

Table 7.71 ***Methods of determining grain size distribution***

Method	Applications	Limitations
Dry sieving – coarse grained soils	• quantitative assessment for coarse grained soils down to fine sand size • suitable for routine testing of dry soils with little fines on site.	• fines retained on soil particles will affect the results and it should not be used unless shown to give the same results as wet sieving.
Wet sieving – coarse grained soils	• quantitative assessment for coarse grained soils down to fine sand size.	• difficult for field use as water source required • more time required for soaking and drying material during test.
Pipette – fine grained soils	• quantitative assessment of fines for percentages of silt and clay • considered to be the primary method compared with the hydrometer method.	• extensive period of time required for test • expensive equipment required • not suitable for routine testing on site.
Hydrometer – fine grained soils	• quantitative assessment of fines for percentages of silt and clay • suitable for routine testing on site.	• results can be less accurate than pipette method but sufficiently accurate for engineering purposes.
Visual accumulation tube – sands	• sediment survey for morphological studies.	• accuracy affected by particle size distribution (lower where there is a concentration of coarse sands) and manual reading.
Bottom withdrawal tube	• sediment survey for morphological studies to determine sedimentation rate • can be used in the field.	• flocculation can inhibit proper readings • best results may be obtained when the sample is tested using water from the project site, saline or otherwise.
Relevance to levees		
• primary index property when assessing hydraulic conductivity parameters for seepage evaluation • acceptability criteria for use as engineered fill • can be used as a screening tool to help determine the suitability of other testing to be performed • liquefaction for foundation soils • morphology of bed sediments • susceptibility to the development of piping • implications on soil strength.		

Density (unit weight)

The density (or unit weight) of a soil can be used to assess the level of compaction achieved during construction of the levee and is an indicator, with other factors, of the susceptibility of the foundation soils to liquefaction. It also provides the soil self-weight load components when assessing the stability and settlement of a levee (Section 8.7). Typical methods for determining soil density are given in Table 7.72.

The bulk density(ρ) of a soil is the total soil mass (M) divided by the total volume (V) of the mass.

$$\rho = M/V \text{ (Mg/m}^3\text{)} \tag{7.76}$$

The assessment of the dry density of a soil also requires the measurement of the moisture content (w) of the soil mass. The dry density (ρ_d) can be calculated as follows:

$$\rho_d = \rho / (1 + 0.01w) \text{ (Mg/m}^3\text{)} \tag{7.77}$$

Table 7.72 Methods of determining density

Method	Applications	Limitations
Linear measurement of regular shaped specimens	• cohesive open tube samples or rock cores.	• sample disturbance can affect results.
Weighing in water	• intact irregular lumps that cannot readily be measured.	• cohesive materials/rock • sample coated with wax to prevent ingress of water, which needs to be taken into account in calculations.
Nuclear density probe	• rapid field measurement during earthwork operations • fine, medium and coarse grained soils.	• not suitable where there is surface water • only applicable at exposed ground surface • material specific calibration needed to account for soil type or material composition • prove suitability by undertaking comparison with sand cone/sand replacement.
Sand cone/sand replacement	• field measurement during earthwork operations • fine, medium and, with larger pouring cylinder, coarse grained soils.	• time consuming and labour intensive • dry conditions required • collapse of excavation on non-cohesive soils • proper training required • surface calibrations should be performed in the field.
Core cutter	• field measurement during earthwork operations in cohesive soils free from coarse soils.	• correct equipment should be used to drive in the core cutter • driving in the core cutter could impart additional compaction to the sample • prove suitability by undertaking comparison with sand cone/sand replacement.
Water replacement	• field measurement during earthwork operations in coarse and very coarse soils.	• the diameter of the ring used should be at least five times the size of the largest particle • side and base of the cut void need to be smooth to prevent the membrane from bridging over irregularities.
SPT (See Table 7.75)	• *in situ* assessment of relative density during intrusive investigations for non-cohesive soils.	• based on empirical correlations • affected by drilling method and boring type, especially below groundwater • appropriate correction of SPT *N* values required when using empirical correlations.
CPT (See Table 7.75)	• *in situ* assessment of relative density during intrusive investigations for non-cohesive soils.	• based on empirical correlations • dilation in fine non-cohesive soils influencing tip resistance • difficult to penetrate soils with significant gravel content or large particles.
Relevance to levees		
• compaction control during construction of levee • low dry density could indicate soft soil with low shear strength leading to potential stability problems • low density fine non-cohesive soils may be prone to liquefaction • estimation of the engineering properties of soils, in combination with other index properties, through empirical correlations.		

Some typical natural unit weights of soils are tabulated in BS 8002:1994. A selections of these typical values of unit weights that are relevant to levees are presented in Tables 7.73 and 7.74.

Table 7.73 Typical unit weight of cohesive soils

Cohesive material and consistency	Saturated unit weight (kN/m³)
Peat (very variable)	12.0
Organic clay	15.0
Soft clay	17.0
Firm clay	18.0
Stiff clay	19.0
Very stiff clay	20.0
Stiff or very stiff glacial clays	21.0

Table 7.74 Typical unit weight of non-cohesive soils

Non-cohesive material	Moist unity weight (kN/m³)		Saturated unit weight (kN/m³)	
	Loose	Dense	Loose	Dense
Gravel	16.0	18.0	20.0	21.0
Well graded sand and gravel	19.0	21.0	21.5	23.0
Coarse or medium sand	16.5	18.5	20.0	21.5
Well graded sand	18.0	21.0	20.5	22.5
Fine or silty sand	17.0	19.0	20.0	21.5
Rockfill	15.0	17.5	19.5	19.0

The SPT and CPT can be used to provide an indication of relative density for coarse grained soils. The relative density ranges between 0 and 100 per cent, where 0 per cent represents the loosest state of the soil and 100 per cent the densest state. A correlation between relative density, density index and SPT *N* values and CPT cone resistance, as presented in BS EN 1997-2:2007, is summarised in Table 7.75 but boundaries may vary with other country codes and standards.

Table 7.75 Assessment of relative density from SPT N values and CPT cone resistance

Relative density	Density index (%)	SPT $(N_1)_{60}$ (blows per 300mm)	CPT cone resistance (MPa)
Very loose	0–15	0–3	0.0–2.5
Loose	15–35	3–8	2.5–5.0
Medium	35–65	8–25	5.0–10.0
Dense	65–85	25–42	10.0–20.0
Very dense	85–100	42–58	>20

Specific gravity

Specific gravity (G_s) of a soil is not normally used independently for characterisation purposes. It is used with other index properties to calculate phase relationships such as voids ratio, degree of saturation and air voids ratio. In terms of levees the phase relationships can provide an indication of settlement potential and the level of compaction achieved. Values of specific gravity are dependent on the mineralogical composition of soils. For soils containing soluble salts the test may be undertaken using kerosene or white spirit in preference to water.

Specific gravity is the ratio of the density of soil particles (ρ_s) to the density of water (ρ_w) and is dimensionless, Equation 7.78. Typical methods for determining specific gravity are given in Table 7.76.

$$G_s = \rho_s / \rho_w \qquad (7.78)$$

Table 7.76 Methods of determining specific gravity

Method	Applications	Limitations
Density bottle (small pycnometer)	• fine grained soils up to sand size particles.	• fine grained soils • small specimen size requires expensive, high precision measuring equipment.
Gas Jar	• all soils up to gravel size particles • suitable for use in the field.	• soils with particle size up to 37.5 mm • need to ensure that shaking removes all air from the soil.
Pycnometer	• site use with medium to coarse grained soils.	• not applicable to soils with high organic content • less accurate than the other two methods.
Relevance to levees		
• typically assumed or measured values used in assessment of grain size distribution (hydrometer) and phase relationships (void ratio, air voids) in consolidation, triaxial and compaction tests • determination of air voids content, which can be used as a compaction control measure.		

A natural soil will contain a number of minerals with different particle densities and particle sizes. So, the test method gives an average value for the soil. A typical value assumed for most common soils is in the range of 2.64 to 2.72. Typical values for some common soil minerals are presented in Table 7.77.

Table 7.77 Typical values of specific gravity for some common soil minerals

Mineral	Specific gravity
Montmorillonite	2.50–2.80
Kaolinite	2.60
Illite	2.66–2.72
Quartz	2.66
Silica	2.60
Peat	1.0 or less

Salt content

Clays forming the foundation to a levee or the levee structure may have been deposited with a pore fluid whose composition may change over time by natural leaching processes or by the construction of the levee.

The clays may have originally been deposited in a high saline environment. Uplift of the ground and the impounding influence of the levee can result in a gradual leaching of the saline water by the infiltration of fresh water. A lowering of the electrolytic content of the pore water can result in a reduction in the attraction between the clay particles, a lowering of strength and the potential for greater dispersion and reduction in erosion resistance. Such clays are normally termed ‘quick clays’.

Salt content can be defined as the mass of salt per litre of soil moisture (NaCl g/l) or the percentage by dry mass of the soil. Typical methods for determining salt content are given in Table 7.78.

Table 7.78 Methods of determining salt content

Method	Applications	Limitations
Qualitative assessment of salt content – soil/water solution tested with reagents	• preliminary test to assess whether the salt level is negligible and further testing is not required.	• non-quantitative assessment.
By 2:1 (or 1:1) water/soil extract for water soluble salt	• quantitative assessment of water soluble salt • suitable for cohesive and non-cohesive soils.	• only suitable for soils which have had recent contact with, or immersion in, saline water.
Extraction of pore water from soil sample by squeezing	• suitable for fine grained soils having a moisture content of 14% or greater.	• not generally applicable for determining the soluble salt content of the pore water extracted from coarse grained soils.
Relevance to levees		
• instability through loss of strength and erosion through dispersion.		

The clay can be classified depending on its salt content. The ranges reported in Table 7.79 have been used in an agricultural context but serve as a point of reference.

Table 7.79 Classification of salinity and a suggested acceptable limit

Qualitative description	Salt content (mg/l)	Acceptable level (mg/l)
Non saline	0 to 300	<4000[1]
Slightly saline	300 to 5000	
Highly saline	>5000	

Note

[1] TAW (1996).

Chalk content

The chalk (calcium carbonate) content of clay can act as a cementing agent and have a detrimental influence on its erosion resistance when used in levee construction.

Typically the methods of determining the chalk content rely on the effervescent reaction of the calcium carbonate with acid (hydrochloric, HCl). The intensity of this reaction can be determined qualitatively (audibly and visually) or quantitatively by assessing the amount of gas given off by the reaction. An alternate method relies on raising the temperature of the soil to a high level so that CO_2 is driven off. Typical methods for determining chalk content are given in Table 7.80.

Table 7.80 Methods of determining chalk content

Method	Applications	Limitations
Diluted hydrochloric acid (10% HCl)	• rapid relative indicator for use in the field or laboratory (Table 7.81) • to establish whether additional quantitative testing is required.	• qualitative assessment only.
Calcium carbonate content chamber – pressure increase generated by 1 g of soil in response to HCl	• rapid quantitative indicator for use in the field or laboratory.	• quantitative assessment based on correlation with excess pressure generated by effervescence.
CHI mass loss – volume of CO_2 given off by reaction with CHI	• laboratory determination.	• calculation requires pressure and temperature adjustment if test not performed under 'standard' conditions (20 °C and 760 mmHg).
Mass loss on ignition: sample heated to 800°C in crucible	• laboratory determination.	• high energy input and long duration test as sample is heated and cooled over several cycles (probably four days).
Relevance to levees		
• chalk content can result in reduced erosion resistance and some loss of strength of a clay used to form a levee.		

Table 7.81 Qualitative assessment of chalk content

Qualitative description	Audible/visual reactions	Acceptable level (%)
Chalk free	None audible or visible	<25[1]
Chalk poor	Audible and not visible or clearly audible with bursts of effervescence	
Chalk rich	Strong and prolonged reaction	

Note

1 TAW (1996)

Organic content

The environment in which clay soils were deposited can influence the amount of organics they contain. The presence of organics can increase the liquid limit, increase compressibility and shrinkage/fissuring potential, while acting to reduce strength and resistance of erosion when compared with inorganic soils. All these factors can have an adverse influence on the serviceability of a levee.

Organic soils can usually be identified by their odour, or dark grey/black colour. If there remains a question as to whether a soil is organic the liquid limit can be determined on two specimens of the soil – one which has been oven dried and rewetted, and one which has been wetted up from the natural state. If there is a difference in the liquid limit of more than 25 to 30 per cent then the soil can be considered to be organic. However, it should be noted that the presence of clay minerals such as halloysite can also result in a reduction in the liquid limit on oven drying. Despite this observation, it may be more usual practice to determine organic content directly using the loss on ignition (burn off) test. The temperature at which the loss on ignition test is conducted can vary significantly and reference should be made to the relevant country codes and standards. Typical methods for determining organic content are given in Table 7.82.

Table 7.82 Methods of determining organic content

Method	Applications	Limitations
Loss on ignition – sample heated to 440°C or 750 to 800°C in crucible to burn off organics	• more suited to sandy soils with little or no clay or chalk • only pre-treatment required is oven drying • 440°C appropriate for all percentages of organics and peat • 750 to 800°C may be more appropriate where peat is required for a fuel source.	• chalk content can influence results • high energy input.
Peroxide oxidation method	• laboratory determination.	• hydrogen peroxide has only limited influence on undecomposed plant remains such as roots and fibres • protracted process requiring cycles of heating, boiling and cooling of peroxide/ soil mixture, which may need up to two days for very organic soils.
Dichromate oxidation method	• laboratory determination.	• soils containing sulphates and chlorides can give high results but can be removed by appropriate pre treatment • protracted and complex test method • assessed organic content is not absolute but is adequate for engineering purposes.
Relevance to levees		
• organic content has an adverse influence on the soil properties and engineering behaviour.		

The classification of a soil based on its organic content is presented in Section 7.8.2.

7.8.3.2 Compaction

Compaction testing provides information on the moisture content at which the most effective compaction can be achieved for a given soil and compactive effort. It provides a measure for specifying and controlling earthworks operations during the construction of levees (Sections 9.13.2 and 10.4).

Compaction is the reduction of air voids of a soil. It is considered in terms of the dry density (Section 7.8.3.1) of the soil. Under a standard compactive effort the dry density that can be achieved will vary with the moisture content. The dry density that can be achieved will reach a maximum for given moisture content. The density and moisture content at which this occurs are termed the maximum dry density (MDD) and optimum moisture content (OMC) (Figure 7.63). The ratio of field dry density to the maximum dry density of the soil determined in the laboratory is usually referred to as a percentage and is termed the 'relative density'.

Where the test soil is cohesive, additional useful information can be gained from the test by:

- preparing one specimen at the natural moisture content. This ensures that the range of moisture contents covered by the test encompasses the moisture content of the as dug material.
- measure the undrained shear strength of each compacted specimen. This will provide an indication of the variation in undrained shear strength of the compacted fill with moisture content and may allow the undrained shear strength to be used as an acceptability criterion instead of using moisture content. It will also provide an indication of the trafficability for construction plant on the compacted fill material.

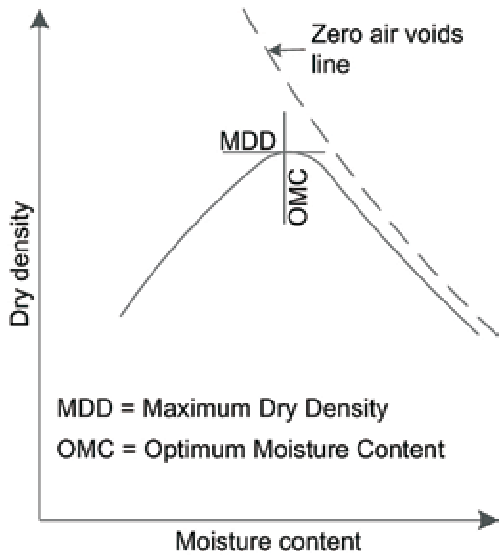


Figure 7.63 ***Typical compaction curve for soils***

Some soils may contain particles that are susceptible to crushing during compaction. Under these circumstances it is necessary to prepare test specimens using separate batches of soil at different moisture contents from the sample, so that each specimen is compacted only once. If the same sample was used each time with a modified moisture content, then the characteristics of the material would progressively change. Consequently, a much larger sample is required (Section 7.9.8.2).

The acceptability of a cohesive material for the construction of levees is in part controlled by the moisture content as this will define the undrained shear strength and level of relative compaction that can be achieved. The measurement of undrained shear strength has been used as an indirect method of moisture content control. However, moisture content is often used as a method of site construction control, which is compared to other soil properties, such as plastic limit and optimum moisture content. The properties of a soil in terms of moisture content and plastic limit can vary significantly both locally and across a site. Under these circumstances the use of moisture content as a method of site control may not be entirely satisfactory. This is especially the case where the soil contains a variable amount of coarse grained soils, which distort the moisture content of the soil mass. These issues are overcome by the compaction related moisture condition value (MCV) test (Parsons, 1976).

Different laboratory procedures have been developed to model the various compactive efforts generated by construction plant and the nature of soils to be compacted, as shown in Table 7.83. Typical values of maximum dry density and optimum moisture content for a range of soils are shown in Table 7.84.

Table 7.83 ***Method of determining soils compaction characteristics***

Method	Applications	Limitations
Proctor/CBR mould (standard compaction – 2.5 kg rammer)	• clayey soils • CBR mould used for coarser soils.	• if >30 per cent retained on 20 mm sieve then soil is too coarse to test • sands and gravels tend to be displaced by rammer rather than compacted.
Proctor/CBR mould (heavy compaction – 4.5 kg rammer)	• where heavy compaction plant is to be used • clayey soils • CBR mould used of coarser soils.	• if >30 per cent retained on 20 mm sieve then soil is too coarse to test • sands and gravels tend to be displaced by rammer rather than compacted.
Vibrating hammer and CBR mould	• sand and gravels.	• max particle size 37.5 mm and not >30 per cent retained on 20 mm sieve.

Table 7.83 Method of determining soils compaction characteristics (contd)

One point field compaction method: comparison of result against a family of site specific compaction curves (Box 9.51)	• provides a rapid method of determining the MDD and OMC from a single point test on a specimen compacted dry of OMC • allows determination of the per cent compaction achieved in the field appropriate to the location of the *in situ* density test • used where compaction characteristics of the soil are variable.	• requires trained field staff and a site based laboratory • bulk sample needed from around the location of each *in situ* density test to establish a location unique MDD and OMC from the single point compaction test • where the soil compaction characteristics are very variable the projection of a probable compaction curve profile from a single point can be less accurate than the two point method.
Two point field compaction method: comparison of result against a family of site specific compaction curves (Box 9.52)	• provides a rapid method of determining the MDD and OMC from a two point test on specimens compacted dry of OMC • allows determination of the per cent compaction achieved in the field appropriate to the location of the *in situ* density test • used where compaction characteristics of the soil are variable.	• requires trained field staff and a site based laboratory • bulk sample needed from around the location of each *in situ* density test to establish a location unique MDD and OMC from the two point compaction test.
Moisture condition test (MCV): hammer blows continued to full compaction. MCV related to number of blows between n and 4n for 5 mm penetration of the hammer	• provides earthworks control, which is independent of moisture content and plasticity of cohesive soils • can be related to trafficability of earthworks plant.	• determination in granular soils is difficult and liable to error.
Relevance to levees		
• earthworks quality control during construction of levee • low compaction could lead to potential stability problems, self-weight settlement, collapse settlement on inundation with water and during a seismic event, increased permeability, reduced erosion resistance and increased maintenance requirements.		

Table 7.84 Typical values of maximum dry density and optimum moisture content for a range of soils types for standard compaction (from Carter and Bentley, 1991)

Soil Type	**Range of maximum dry density (kg/m³)**	**Range of optimum moisture content (%)**
Gravels and sand mixes		
Well graded and clean	2000–2150	11–8
Poorly graded and clean	1850–2000	14–11
Well graded with small silt content	1900–2150	12–8
Well graded with small clay content	1850–2000	14–9
Sand and sandy soils		
Well graded and clean	1750–2100	16–9
Poorly graded with small silt content	1600–1900	21–12
Well graded with small silt content	1750–2000	16–11
Well graded with small clay content	1700–2000	19–11
Fine grained soils of low plasticity		
Silts	1500–1900	24–12
Clays	1500–1900	24–12
Organic silts	1300–1600	33–21

Table 7.84 Typical values of maximum dry density and optimum moisture content for a range of soils types for standard compaction (from Carter and Bentley, 1991) (contd)

Fine grained soils of high plasticity		
Silts	1100–1500	40–24
Clays	1300–1700	36–19
Organic clays	1050–1600	45–21

7.8.3.3 Shear strength

The shear strength of the foundation soils on which the levee is constructed and the materials from which it is formed are key parameters that need to be defined to ensure the stability of the levee during both construction and serviceability.

The shear strength of a soil depends on the effective stress between the soil particles forming the soil skeleton. Under fully drained or dry conditions the load is carried by the soil particles, resulting in an increase in shear strength proportional to the applied load, a reduction in volume and no increase in the pore water pressure. In a saturated soil where no drainage is allowed, the increase in load applied to the soil is carried by the pore water, resulting in an increase in the pore water pressure, no reduction in volume and no increase in the effective stress between the soil particles. As a result there is no increase in the shear strength of the soil. In the case of a partially saturated soil loaded under conditions that do not allow drainage, some of the applied load is carried by the pore water pressure and some by the soil particles. As a result there is some increase in shear strength along with a reduction in soil volume as the air voids compress and the air goes into solution within the pore water. With increased applied load the soil eventually becomes fully saturated and behaves as an undrained soil.

Whilst soils exhibit complex nonlinear viscoelastic behaviour, for most practical applications simple linear models of time independent elastoplastic theory are used to model the stress-strain relationship of soils, ie the soil is assumed to have a linear elastic behaviour to the yield point and then act perfectly plastically. The most commonly used model for representing soil strength is the Mohr-Coulomb failure criteria. The Mohr-Coulomb criteria can be written ass

$$\tau = \sigma_n \tan \phi + c \quad \text{(for a dry soil)} \qquad (7.79)$$

$$\tau = (\sigma_n - \mathrm{u}) \tan \phi + c \quad \text{(for a saturated or partially saturated soil)} \qquad (7.80)$$

where:

τ = shear strength at failure

σ_n = normal stress on failure plane

u = pore water pressure

ϕ and c = strength parameters defining the friction angle and cohesion

It should be noted that ϕ and c are not inherent properties of the soil but parameters used to represent the linear model defining the Mohr-Coulomb failure envelope. However, the actual failure envelope can be nonlinear (Figure 7.64).

The stress-strain behaviour of a soil in the plastic state is either one of strain hardening or strain softening (Figure 7.65), ie there is some increase or reduction in strength with strain in the plastic zone depending upon the nature and initial stress conditions of the soil. Dense sands and over-consolidated clays are dilatant soils that increase in volume as they are strained past their peak shear strength (ϕ_p) and strain soften to a limiting state known as the critical voids ratio or fully softened state, ϕ_{cv}. Loose sands and normally consolidated clays are contractive soils that reduce in volume and increase in shear strength as they strain harden. Their peak strength corresponds with ϕ_{cv}, which typically occurs at about 10 to 20 per cent strain. At very large strains, which are typically more than twice those required for ϕ_{cv}, ϕ reduces further to the residual state, ϕ_r. In cohesive soils ϕr is typically several degrees lower than ϕ_{cv}. However, for cohesionless soils ϕ_r is typically equal to ϕ_{cv}.

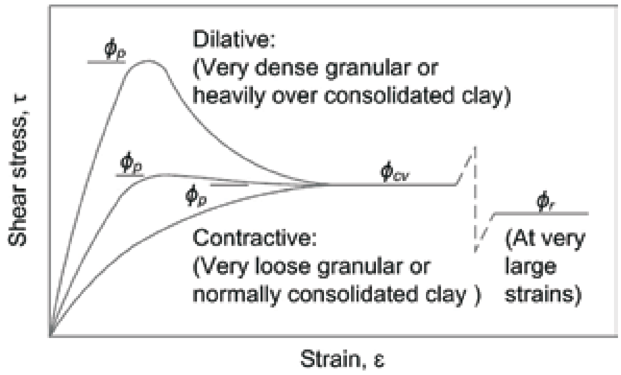


Figure 7.64 Stress strain behaviour of soils

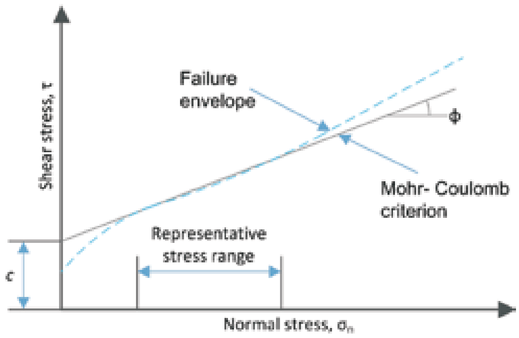


Figure 7.65 Mohr-Coulomb failure envelopes

During the construction of a levee, which can encompass improvement works, or during a flood event where there is a rapid change in loading conditions, cohesive soils can behave undrained. For non-cohesive soils drained conditions invariably apply under short term loading. In the longer term, under relatively steady state conditions the soils can behave in a drained state and effective stress strength parameters may be applicable.

Where significant movements have occurred within the body of the levee and/or the foundation soils as a result of the complete or partial failure of the levee, and preferential shear planes have developed within the soil mass (such as may occur as a result of a failure during construction or an earthquake), then the drained (ϕ_r) or undrained (c_{ur}) residual shear strength of the soil may need to be considered during the design of remediation works.

Undrained shear strength

The undrained shear strength of a cohesive soil is not a unique property. It is dependent upon a number of factors. Some of the key ones are listed below and considered in more detail in Table 7.85:

- test method
- orientation of the shear plane relative to the soil structure
- specimen disturbance
- rate of shearing
- sample size
- geological history or stress path.

Table 7.85 Factors affecting undrained shear strength

Influence factor	Discussions
Test method	• different test methods impose different boundary conditions on the soil during shearing and cause the shear plane to develop in a defined direction • the values of undrained shear strength mobilised in different directions can be measured in the laboratory by, for example, triaxial compression (TC), direct shear (DS) and triaxial extension (TE). The relevance of these to the modes of shear under a levee is shown in Box 7.33. Typically the relative magnitude of the strengths measured by these methods lies in the order TC>DS>TE for a given soil sample. The corrected field vane strength is thought to represent the average shear strength mobilised along a shear surface developed below a failing embankment.
Orientation of the shear plane relative to the soil structure	• a soil can have different shear strength depending on the direction of the shear plane relative to the soil structure, as well as the method of shearing. This is due to geometrical anisotropy (or soil fabric) resulting from the depositional history and stress anisotropy resulting from stress and depositional history.
Specimen disturbance	• if a specimen is disturbed by sampling, the processes of transportation or during test preparation, the measured undrained strength will be less than the *in situ* value. For this reason high quality samples are required (Section 7.9.8.1) along with the disturbance mitigation measures (Table 7.158). These factors are particularly important for soft (low strength) sensitive cohesive soils. For good quality samples the strain at failure will generally be significantly less than about six per cent.
Rate of shearing	• the greater the time to undrained failure the lower the maximum undrained shear strength will be. This needs to be considered when applying the results of field or laboratory tests, which may take of the order of 10 minutes to complete, compared with a rate of loading of weeks or months in the field during construction of a levee. For every log cycle decrease in strain rate there is about a 10 per cent reduction in undrained shear strength • the undrained strength of soils under rapid loading conditions should be evaluated using undrained tests appropriate to the *in situ* loading condition (eg monotonic or cyclic tests) • cyclic loading can result in a reduction in undrained shear strength. Where cyclic strains are less than half the strain at failure under non-cyclic conditions, the reduction in undrained shear strength is minimal.
Specimen size	• the size of the test specimen can have an influence on the undrained shear strength as disturbance reduces with increase in sample size. Larger samples will be more representative of the soil mass in terms of soil fabric and, in over consolidated stiff clays, the distribution of fissures • the outer edges and ends of a sample are prone to the greater disturbance. Larger samples allow more representative specimens to be prepared from the least disturbed sections of the sample by hand trimming.
Geological history or stress path	• the past stress history of a soil can influence its undrained shear strength, which can be considered in terms of the *in situ* effective stress and over consolidation ratio (defined in Section 7.8.3.4) • soils that have not experienced a state of effective stress greater than the present *in situ* effective stress are considered to be normally consolidated. Alternately, and more commonly, they are lightly over-consolidated as a result of a number of factors, which could include secondary compression, variations in groundwater level and surface desiccation. In practical terms, these clays are usually very soft, soft and firm recent deposits upon which many levees are constructed. Where a stronger desiccated crust is present, the over consolidated surface layer could also be used as a source of borrow material for the construction of levees • where the soil has experienced the removal of a significant thickness of overburden, the past maximum *in situ* effective stress will be a lot more than the present condition and the soil may be considered to be heavily over-consolidated. In practical terms, these are usually the stronger, stiff to very stiff deposits, which form the underlying geology and form a source of borrow material for the construction of levees.

Measurements of undrained shear strength can be obtained from *in situ* tests in the field or from tests in the laboratory. *In situ* tests can be integrated in the ground investigation programme and are usually less expensive when compared with laboratory testing, which includes the associated sampling requirements. *In situ* tests can also provide an assessment of the distribution of undrained shear strength across different geological units, which can be refined through a laboratory testing programme on layers of significant interest.

Laboratory tests for undrained shear strength typically need a high quality, Class 1 sample (Section 7.9.8.1). The conditions under which the specimens are tested should match the conditions anticipated in the field. Table 7.86 summarises some typical laboratory tests that may be used to assess undrained shear strength.

Table 7.86 Some in situ and laboratory methods of assessing undrained shear strength

Method	Applications	Limitations
Field shear vane test (penetration or down the borehole): • 55 mm diameter by 110 mm • 65 mm diameter by 130 mm	• assess undrained shear strength for very soft to firm clays • multiple tests can be performed to get strength profile with depth at one location • develop site specific correlations with other forms of *in situ* test, specifically CPT • penetration shear vane can be jacked or pushed into the ground without the need for a borehole.	• may overestimate shear strength of very soft high plasticity clays and requires a correction factor to be applied to reduce the shear strength (see Box 7.32) • not reliable if coarse grained layers or particles, or fibrous material is encountered • assumes uniform shear strength over all surfaces of the cylinder of soil sheared by the vane • ground obstructions restrict depth of penetration shear vane • regular calibration required, at least annually and before the start of a large project • may not be reliable in sandy clays/silts due to effects of partial drainage.
Hand shear vane: • 19 mm diameter by 29 mm • 33 mm diameter by 50 mm Tor vane: • various sizes	• field assessment in walls of trial pits, intact blocks of soil removed from trial pits, end of tube samples and during earthworks operations at the source of the borrow material and in compacted fill.	• small volume of soil tested • shear strength may be assessed by direct reading of the scale or calculation based on the surface area of the sheared cylinder of soil • may not be reliable in sandy clays/silts due to effects of partial drainage.
Laboratory shear vane: • 12.7 to 25.4 mm diameter and length to diameter ratio of 1:1 or 1:2	• may be used to determine strength of soils, which are too soft to extrude and/ or allow preparation of specimens for other laboratory strength tests • samples may be tested while still retained within sample tube.	• small volume of soil tested • laboratory mini vane is only suited for soft soils • may not be reliable in sandy clays/silts due to effects of partial drainage.
Pocket penetrometer: • 6.3 mm diameter probe pushed to penetrate to reference mark on probe	• quick preliminary field evaluation of strength in walls of trial pits, intact blocks of soil removed from trial pits, end of tube samples and during earthworks operations at the source of the borrow material and in compacted fill • aid in the field classification of cohesive soils.	• does not replace normal laboratory testing • surface layer of material tested. Surface should be fresh and free of obstructions • the scales on some penetrometers give a direct reading of shear strength (c_u), others give uniaxial compressive strength, equivalent to $2 \times c_u$ • may not be reliable in sandy clays/silts due to effects of partial drainage • equipment may be difficult to recalibrate.
Standard penetration test (SPT)	• provides initial estimate of undrained shear strength based on empirical correlations.	• data can be misleading where locally stronger bands or obstructions are encountered • estimates are sensitive to non-standard test equipment and procedures that may not be evident from the data review • correlations should be treated with caution for soft clays • appropriate correction of SPT *N* values required when using empirical correlations (see Caution box).

Table 7.86 Some in situ and laboratory methods of assessing undrained shear strength (contd)

Cone penetration test (CPT)	• provides continuous profile of subsurface conditions and an indication of soil fabric if pore water pressures are measured • published correlations available to estimate soil classification and undrained strength • site specific correlations can be developed with other forms of testing, specifically the field shear vane test.	• correlation factors can vary with local geology and need confirmatory boreholes and test data to develop site specific correlations • obstructions can limit penetration • satisfactory de-airing of piezocone filter element can be problematic leading to a potential 'flat' pore water response profile • desaturation of piezocone may occur in desiccated or gassy soils.
Menard and other pressuremeters	• provides deformation modulus, creep pressure and limit pressure • estimate of undrained shear strength for clay soils based on published correlations • primarily used for foundation design.	• drilling disturbance and pre drilled borehole wall instability may result in low results • in soft soils borehole diameter may reduce before insertion of pressuremeter • correlations with shear strength depend on soil consolidation state • loading direction is horizontal, which may not correspond with the load direction in the field.
Direct shear test	• simple and quick • quick undrained tests on clay take <20 min, so virtually no consolidation • failure envelope is virtually horizontal (ϕ_u= 0°) • suitable for samples with a defined shear plane.	• cannot measure pore pressure or control drainage • direction of shear may not be along the weakest plane • stress concentrations at sample boundaries • stress path and rotation of principal stress direction is uncontrolled • failure envelope may be curved for over-consolidated clays.
Simple or pure shear test	• minimises stress concentrations by developing a fairly homogeneous stress state in the specimen.	• test complexity requires trained laboratory technician • does not measure pore water pressure • no control of stress path.
Unconfined compression test	• simple and quick • portable equipment suitable for site use.	• as sample is open to the air, the preparation and handling of very soft specimens may be problematic.
Triaxial test: unconsolidated undrained (UU)	• quick test • shear plane develops along weakest inclined plane in specimen.	• assumes pore water conditions generated in the test are similar to those in the field • partially saturated samples undergo consolidation during test that is hard to control and match with field conditions. As a result: $\phi_u \neq 0°$.
Relevance to levees		
• assessment of the stability of the levee during construction activities, such as improvement works or the construction of a new levee • quality control of borrow material • evaluation of the performance of an existing levee subject to a rapid change in external load, resulting in undrained loading.		

Caution

***In situ* test – standard penetration test (SPT) – N, N_{60} or $(N_1)_{60}$**

There are a number of ways in which the SPT N values can be reported and used in conjunction with empirical correlations:

- *N* value:

The blow count as recorded directly in the field

- N_{60} value:

  The blow count recorded in the field, normalised to a hammer efficiency of 60 per cent.

  It is important to document the hammer system used for the SPT and its energy efficiency, when available, so that *N* values can be normalised or corrected to a standardised hammer efficiency to allow direct comparison of *N* values determined using different field equipment and systems of controlling the hammer drop energy. Some countries use a pulley and cathead system, or other systems that are very operator dependent. However, automated hammer systems are now far more common and are more efficient at delivering the hammer energy into the ground, which can result in lower blow counts (*N* values) compared to those from the older hammer systems. Older systems can have an energy delivery efficiency of about 60 per cent and formed the basis for many of the established empirical correlations used today.

- $(N_1)_{60}$ value:

  The blow count recorded in the field, normalised to a hammer efficiency of 60 per cent and an effective overburden pressure of 100kPa.

  For a given granular soil the penetration resistance is also proportional to the strength of the ground, which is a function of the mean effective stress, which itself is related to the vertical effective stress and over consolidation ratio. Some correlations have been developed, which require the use of the $(N\ _1)_{60}$ value, for example in the assessment of liquefaction.

It is clear that there can be significant confusion, which can introduce errors when using SPT *N* values to assess soil properties through empirical correlations if the reported SPT *N* values are not compatible with the SPT *N* values used to develop the empirical correlation.

Clayton (1995) provides a commentary on the SPT, which includes detail of how the above corrections can be made. It also includes a table (Table 9), which summarises the correction factors that are usually required when assessing a variety of soil parameter through empirical correlations. For soil properties that are appropriate to levees the corrected *N* values may be reported as:

effective friction angle (ϕ')	$(N_1)_{60}$
undrained shear strength (c_u)	N_{60}
undrained elastic modulus (E_u)	N_{60}
coefficient of volume compressibility (m_v)	N_{60}

However, the user should be satisfied that the reported SPT *N* values are appropriate when using them to assess soil properties through empirical correlations.

Some tables detailing correlations between soil parameters and SPT *N* values are included in Section 7.8.

Box 7.32 Bjerrum's correction factor for field vane shear strength

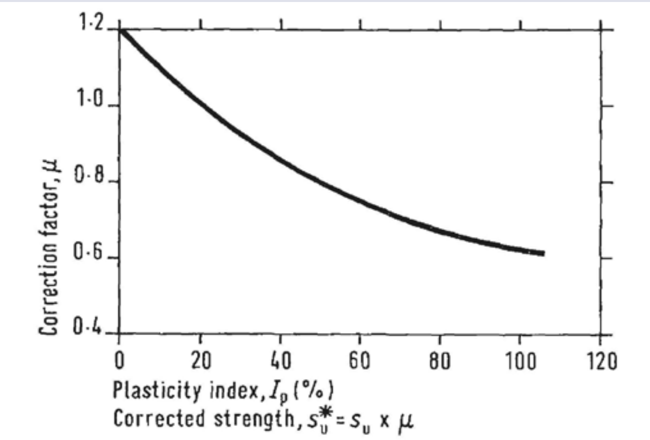


Figure 7.66 Correlation between plasticity index and Bjerrum's (1972) vane shear strength correction faction (μ) (from Meigh, 1987)

Bjerrum (1972) analysed a large number of slope failures using a circular slip surface and undrained shear strengths measured using the field shear vane. The results where plotted against the plasticity index of the soils and showed that in high plasticity clays the mobilised undrained shear strength was less than that measure by the field shear vane. As a result the follow correct factor was proposed (Figure 7.66).

Other correction factors have been proposed and reference should be made to local country codes and standards.

Some typical empirical correlations for evaluating consistency and undrained shear strength in the field are presented in Table 7.87.

Table 7.87 ***Designation of undrained shear strength and field assessment of consistency/undrained shear strength through tactile behaviour and correlations with in situ tests***

Undrained shear strength[(a)]	Shear strength, c_u (kPa)	Field descriptive term of consistency	Field consistency test	SPT N_{60} value[(b)]	CPT cone resistance, q_c (MPa)[(c)]	PMT Menard pressuremeter test limit pressure, p_L (MPa)
Extremely low	<10	Very soft	Extrudes between fingers	0 to 4	<0.30	<0.15
Very low	10 to 20					
Low	20 to 40	Soft	Moulded by light finger pressure	4 to 8	0.3 to 0.6	0.15 to 0.30
Medium	40 to 75	Firm	Cannot be moulded, rolls to a thread	8 to 15	0.6 to 0.13	0.30 to 0.50
High	75 to 150	Stiff	Crumbles, breaks, remoulds to a lump	15 to 30	1.13 to 2.25	0.50 to 0.80
Very high	150 to 300	Very stiff	Crumbles, does not remould, can be indented by thumbnail	30 to 60	>2.25	>0.80
Extremely high	>300	Hard		>60		

Notes

(a) Terms used to designate undrained shear strength according to the results of laboratory or *in situ* tests (after EN ISO 14688-2:2004).

(b) After Table 8, Clayton (1995).

(c) Based mid-range N_k of 15.

Values should only be used for initial field evaluation of consistency.

Table 7.88 contains some examples of empirical correlations that could be used to assess the undrained shear strength of a cohesive soil. Other methods are available but all empirical correlation should be treated with caution, and where possible the investigation should aim to confirm them or develop site specific correlations.

Table 7.88 ***Some empirical correlations for the assessment of undrained shear strength***

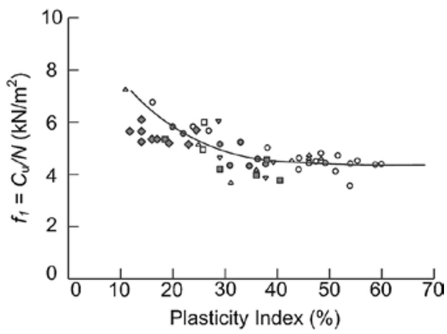
Empirical correction	Correlation charts
SPT: Stroud (1975) developed a correlation between c_u and SPT *N* value ($f_1 = c_u/N$) in insensitive over-consolidated clays, where c_u was measured on 100 mm diameter triaxial specimens. The result showed a correlation with I_p. Others (De Mello, 1971) reported a c_u/N ratio ranging between 0.4 and 20, but this included a wider range of soil types. c_u values derived using the Stroud correlation in soils other than insensitive over-consolidated clays should be treated with caution.	Correlation between c_u with SPT *N* value with I_p (after Stroud, 1975) 

Table 7.88 Some empirical correlations for the assessment of undrained shear strength (contd)

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CPT:

Cone tip resistance (q_c) has been correlated with c_u by a number of investigators (Lunne *et al*, 1997). Possibly the most applicable and simplest correlation for levee evaluation is that between total cone resistance (q_c) and vane shear strength (s_u)

$s_u = (q_c - \sigma_{vo})/N_k$ (7.81)

Where:

s_u = vane shear strength – uncorrected for PI

q_c = measured cone resistance

σ_{vo} = total overburden stress at depth of cone

N_k = empirical cone factor

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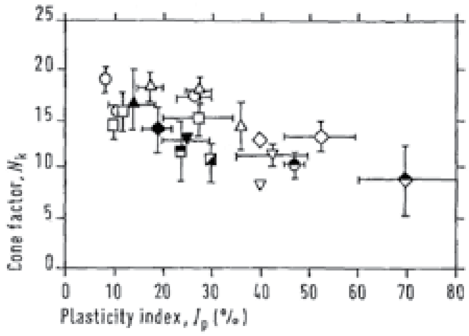
Correlation between s_u with q_c and I_p (from Meigh, 1987)



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Liquidity index:

Correlations have been developed between Liquidity Index (I_l) and the c_u for a remoulded soil and the sensitivity (ratio of natural to remoulded c_u) of a soil (Skempton and Northy, 1952). Based on these correlations, Carter and Bentley (1991) proposed a correlation between natural c_u and I_l.

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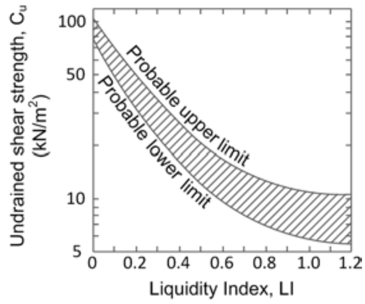
Correlation between natural c_u and I_l (after Carter and Bentley, 1991)



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Vertical effective overburden stress:

For normally consolidated clays c_u has been shown to be a function of the vertical effective overburden stress (σ'_{vo}) and I_p. Grace *et al* (1957) established the following correlation:

$c_u/\sigma'_v = 0.11 + 0.0037\ I_p$ (7.82)

Other correlations have been developed by Bjerrum (1972) and Kenney (1976)

Mesri (1975) proposed that if the vane shear correction factor, μ (Bjerrum, 1972), and over-consolidation due to aging are taken into account, then: $c_u/\sigma'_{vo} = 0.22$

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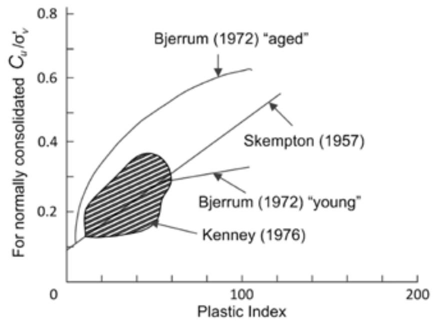
Correlations between c_u, σ'_v and I_p (based on Carter and Bentley, 1991)



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Drained shear strength

The drained shear strength of a soil is dependent upon a number of factors. Some of the key ones are listed below and considered in more detail in Table 7.89:

- test method
- orientation of the shear plane relative to the soil structure
- specimen disturbance
- rate of shearing
- sample size
- geological history or stress path.

Due to the difficulty of obtaining undisturbed samples in non-cohesive soil, some of these factors may not be replicated in laboratory tests without careful selection and reconstruction of the test specimen.

Table 7.89 Factors affecting drained shear strength

Influence factor	Discussions

Test method	• as detailed in Table 7.85, different test methods impose different boundary conditions on the soil during shearing and cause the shear plane to develop in a defined direction. The values of effective friction angle, ϕ', relative to the values measured in triaxial compression (TC) are:
	  • triaxial extension = 1.22/1.12 (TC)
	  • plane strain compression = 1.10/1.12 (TC)
	  • plane strain extension = 1.34/1.25 (TC)
	  • direct shear: for sands the ratio is a function of density. At low density there may be little difference (1.0 TC). At high densities the increase could be about four degrees (about 1.10 TC)
	Where '1.22/1.12' indicates the ratio for normally consolidated cohesive soils/non-cohesive soils.

Orientation of the shear plane relative to the soils structure	• cohesive and non-cohesive (*in situ*) soils (see Table 7.85) • non-cohesive samples: laboratory specimen formed from restructure disturbed sample that may be unrepresentative of the *in situ* state.
Specimen disturbance	• cohesive soils (see Table 7.85) • non-cohesive soils: due to the difficulty in obtaining undisturbed samples, laboratory tests are performed on re-compacted specimens that may be unrepresentative of the *in situ* state. To overcome this, consideration could be given to shearing the specimen to ϕ'_{cv}, which is independent of the initial density state. Design codes may allow a lower partial or overall factor of safety when ϕ'_{cv} is used.
Rate of shearing	• cohesive soils. The rate of shear will determine whether the soil behaviour is fully or partially drained, or undrained (see Table 7.85) • non-cohesive soils: the rate of shear will determine whether the soil behaviour is fully or partially drained, or undrained. Rapid rates of loading, such as seismic loading, can impose undrained conditions. In strongly dilatant soils the undrained shear strength measured by laboratory tests may be larger than the drained strength. In the field, under rapid loading, the undrained shear strength will be limited by cavitation effects that may not have occurred during the laboratory test • in lower density soils cyclic loading can result in the development of excess pore water pressures and liquefaction • the rate of shearing under drained conditions has only a minimal effect on ϕ'. Under drained condition, vibration and repeated loading can cause loose soils to densify and dense soils to loosen, resulting in an increase and decrease, respectively, in ϕ'.

Specimen size	• cohesive soils (see Table 7.85)
	• non-cohesive soils: the specimen size is typically governed by the size of the soil particles. For direct shear tests the maximum size of particles present in significant quantities should not exceed:
	  • 60 × 60 mm = 2.00 mm
	  • 100 × 100 mm = 3.35 mm
	  • 305 × 305 mm ≤ 37.50 mm

Geological history or stress path	• cohesive soils: the shear strength parameters (c' and ϕ') depend on the pre-consolidation pressure. When the imposed effective stress is a large fraction of the pre-consolidation pressure, corresponding to a low over-consolidation ratio, ϕ' with be slightly less than the normally consolidated values and c' with depend on the magnitude of the pre-consolidation pressure. The reverse situation applies when the effective stress is small compared with the pre-consolidation pressure
	• non-cohesive soils:
	  • where the soil particles are cemented the soil may behave like a soft rock at small strains below a critical stress state. At higher stress levels it may behave like an uncemented material as the cementation breaks down and pore water pressures may increase
	  • increasing the confining pressure can increase the strain to failure, decrease dilatancy and reduce the brittle characteristics of the stress-stain curve
	  • the grain size distribution and grain angularity influences ϕ'. A well graded soil with angular grains has a higher ϕ' at a given density than a uniformly graded soil with rounded grains
	  • The presence of mica in the soils can result in lower values of ϕ'.

The measurement of drained shear strength can be obtained from *in situ* tests via empirical correlations and laboratory tests. Due to the difficulty in obtaining undisturbed samples of non-cohesive soils the drained shear strength tends to be based on empirical correlations with *in situ* tests, unless the material is to be used as fill and will be engineered to a known state that can be replicated in the laboratory. By contrast undisturbed samples of cohesive soil can readily be obtained. Table 7.90 summarises some typical methods of assessing drained shear strength through empirical correlations for non-cohesive soils.

Table 7.90 Some in situ methods of assessing drained shear strength of non-cohesive soils

Method	Applications	Limitations
Standard penetration test (SPT)	• drained shear strength parameter of non-cohesive soils assessed through empirical correlations • where disturbance of non-cohesive soils at the base of the borehole is suspected the test can be continued for a further four increments of 75 mm to penetrate below the zone of potential disturbance (note that this test will not be compliant with BS EN ISO 22476-3:2005).	• data can be misleading in presence of obstructions such as larger particle sizes, eg cobbles • estimates are sensitive to non-standard test equipment and techniques (can be operator dependent) that may not be evident from a data review • results prone to disturbance at the base of the bore, particularly in fine non-cohesive soils, and a positive water head is required to prevent 'boiling' • appropriate correction of SPT *N* values required when using empirical correlations. Results may need to be corrected for hammer energy, effective overburden pressure and rod length if values are to be used to derive soil properties or correlate with CPT data.
Cone penetration tests (CPT)	• provides a continuous profile of subsurface conditions and an indication of soil fabric if pore water pressures are measured • published correlations available to estimate soil classification and drained shear strength.	• correlations can vary with local geology and need confirmatory boreholes • obstructions and gravels can limit penetration.

Laboratory tests for shear strength typically need a high quality, Class 1 sample (Section 7.9.8.1). The conditions under which the specimens are tested should match the conditions anticipated in the field in terms of pore water pressure, stress path and direction of shear. This is of particular importance for over-consolidated or layered soils. Table 7.91 summarises some typical laboratory shear strength tests that may be used to assess drained shear strength.

Table 7.91 Typical laboratory shear strength tests that may be used to assess drained shear strength

Method	Applications	Limitations
Direct shear test	• measurement of drained shear strength parameters in both cohesive and non-cohesive soils. Simple and quick in high permeability cohesionless soils • can be used to measure ϕ'_{cv} by continuing to shear beyond the peak value • can be used to assess ϕ' by pre-cutting specimen on 'shear plane' or by cycles of reverse shearing.	• cannot measure pore pressure or control drainage • direction of shear may not be along the weakest plane • stress concentrations can occur at sample boundaries • stress path and rotation of principal stress direction is uncontrolled • extended consolidation phase under applied normal stress and slow shearing rate required for cohesive soils to allow pore water pressure to dissipate.
Simple or pure shear test	• suitable for performing consolidated shear test on cohesive soils • minimises stress concentrations by developing fairly homogeneous stress state in sample.	• test complexity requires trained laboratory technician • does not measure pore pressure parameters • no control of stress path.
Triaxial test: consolidated drained (CD) test	• comprehensive test that provides detailed measurement of pore water pressure and information on stress path and stress state • orientation of failure plane direction is not restricted • suitable for non-cohesive soils that can quickly dissipate pore water pressures • improved measurement accuracy and test control can be achieved using automated test equipment.	• an expensive shear strength test specifically on cohesive soils. The equipment is occupied for a long period due to the pre-test consolidation phase and the slow rate of shearing required to limit the development of excess pore water pressures • test period increases significantly for larger specimen diameters due to the increased length of drainage path • test complexity requires trained laboratory technician • performing test over a stress range that is inappropriate to the *in situ* conditions.
Triaxial test: consolidated undrained (CU) test with pore water pressure measurement	• comprehensive test that provides detailed measurement of pore water pressure and information on stress path and stress state • provides total and effective strength parameters under the defined confining pressure • orientation of failure plane direction is not restricted • suitable for non-cohesive and cohesive soils • improved measurement accuracy and test control can be achieved using automated test equipment.	• test complexity requires trained laboratory technician • rate of strain during testing may be too rapid for soft soils that are sensitive to testing rate • test is more expensive than most field and laboratory tests.
Ring shear	• determination of ϕ'_r on a remoulded specimen of cohesive soil by shearing in one direction.	• shear strain cannot be calculated but shear displacement can be determined.
Relevance to levees		
• assessment of the stability of the levee during construction activities, such as improvement works or the construction of a new levee • quality control of borrow material • evaluation of the performance of an existing levee subject to a change in hydraulic loading • design of repair works where a levee has failed or a slip has occurred.		

Typical drained shear strength parameters for a range of soil types are presented in Table 7.92. They should only be used as guide values in the absence of other site specific data.

Table 7.92 Typical values of drained parameters for a selection of soils

Soil type		**Effective friction angle (ϕ')**	
Compacted cohesive soils	Clay of high plasticity	19	
	Clayey silts	25	
	Clays of low plasticity	28	
	Silt and clayey silt	32	
	Clayey sands	31	
Non-cohesive soils		***Loose***	***Dense***
	Uniform sand, round grains	27	34
	Well graded sand, angular grains	33	45
	Sandy gravel	35	50
	Silty sand	27–33	30–34
	Inorganic silt	27–30	30–35

Simple qualitative field tests, such as SPT and CPT, undertaken as part of a routine geotechnical investigation can be used to provide a field assessment and initial estimate of the effective friction angles of non-cohesive soils. Some examples of these correlations for field assessment are present in Table 7.93.

Table 7.93 Field assessment of shear strength of non-cohesive soils through penetration resistance (after BS EN 1997-2:2007)

Field descriptive term	**Field test**	**SPT $(N_1)_{60}$ value**	**CPT cone resistance, q_c (MPa)**	**Effective friction angle, ϕ' (°)**
Very loose	Can be dug by spade, 50 mm peg easily driven	0–3	0.0–2.5	29–32
Loose		3–8	2.5–5.0	32–35
Medium dense		8–25	5.0–10.0	35–37
Dense	Need pick for excavation, 50 mm peg hard to drive	25–42	10.0–20.0	37–40
Very dense		42–58	>20	40–42

Empirical correlations have been developed for cohesive and non-cohesive soils, which relate the drained shear strength (peak, constant volume and residual) to the index properties such as plasticity index and clay fraction or in the case of non-cohesive soils, the effective overburden pressure. Some of these empirical correlations are shown in Tables 7.94 and 7.95.

Table 7.94 Some empirical correlations for the assessment of drained shear strength (cohesive soils)

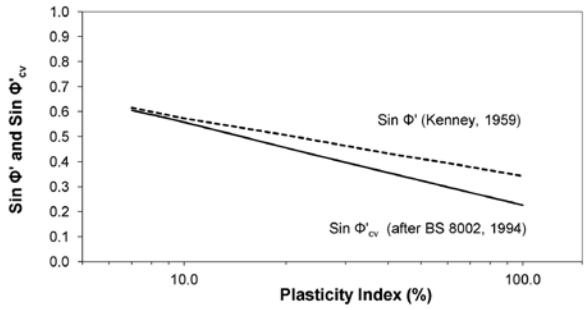
Empirical correction: fine grained soils	**Correlation charts**
Peak and constant volume shear strength Relationships between I_p and ϕ' and ϕ'_{cv} have been developed. Correlations can be developed as both plasticity index and shear strength reflect the clay mineralogy. Figure shows the variation in ϕ' for natural and remoulded normally consolidated clays (Kenney, 1959), and ϕ'_{cv} (after BS 8002:1994).	Variation in Sin ϕ' with I_p for normally consolidated natural and remoulded clays, and Sin ϕ'_{cv} 

Table 7.94 Some empirical correlations for the assessment of drained shear strength (cohesive soils) (contd)

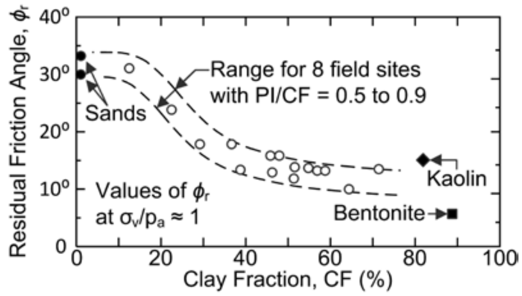
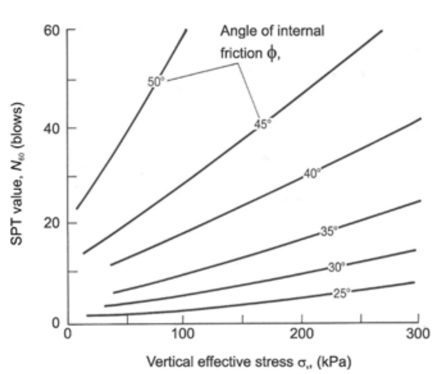
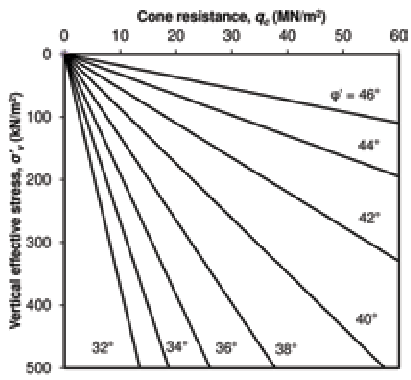
Residual shear strength: Relationships between clay fraction (% <2µm) and ϕ'_r have been developed. Figure illustrates the variation on ϕ'_r. with clay fraction (after Kulhawy and Mayne, 1990). It shows that for clay fractions less than 15 per cent, ϕ'_r is not dissimilar to φ′ and it behaves like a non-cohesive soil. For clay fractions greater than 50 per cent, ϕ'_r is significantly less than ϕ' and is controlled by the reorientation of the clay particles on the shear plane.	Variation in residual effective friction angle, ϕ'_r, and clay fraction 

Table 7.95 Some empirical correlations for the assessment of drained shear strength (coarse grained soils)

Empirical correction: coarse grained soils	**Correlation charts**
SPT: Many correlations have been developed which seek to relate SPT N_{60} values to ϕ' To allow the comparison of test results for different systems that are used internationally the *N* value may be adjusted to a hammer efficiency of 60 per cent, denoted as N_{60}. Refer to country codes and standards. The correlation example presented in the figure (right) takes account of the effective overburden pressure, σ'_{vo}.	Suggested relationship between N_{60}, σ'_{vo} and ϕ' (after Mitchell *et al*, 1978) Angle of internal friction ϕ, degrees 
CPT: Dougunoglu and Mitchell (1975) derived a correlation for ϕ' based on bearing capacity theory. This is considered to provide a reasonable lower bound value for the types of sand tested, essentially uncemented, and normally consolidated quartz sand. These may be modified if the sand has a higher compressibility by up to 2° and a further 2° if the sand is over-consolidated. For high confining stresses the chart does not take account of curvature of the Mohr-Coulomb envelope and ϕ' can be reduced by up to 8° in very dense sands.	Suggested correlation between q_c, σ'_{vo} and ϕ' for normally consolidated, uncemented quartz sand (after Dougunoglu and Mitchell (1975) 

7.8.3.4 Compressibility

Levees are often constructed on recent deposits of clay and peat of low strength and high compressibility. The assessment of the amount and rate at which settlement occurs due to the imposed load from the levee is a fundamental consideration in the design of a levee. This is because it drives the determination of the construction crest level and the long-term maintenance requirements to ensure that the required defence level is sustained throughout the life of the levee. It also influences the geometry of the levee in terms of achieving a stable profile because it is associated with the rate at which the foundation soil increase in strength during the consolidation process.

The compression of a soil comprises three independent and sequential components (Figure 7.67):

- elastic compression
- primary consolidation
- secondary compression.

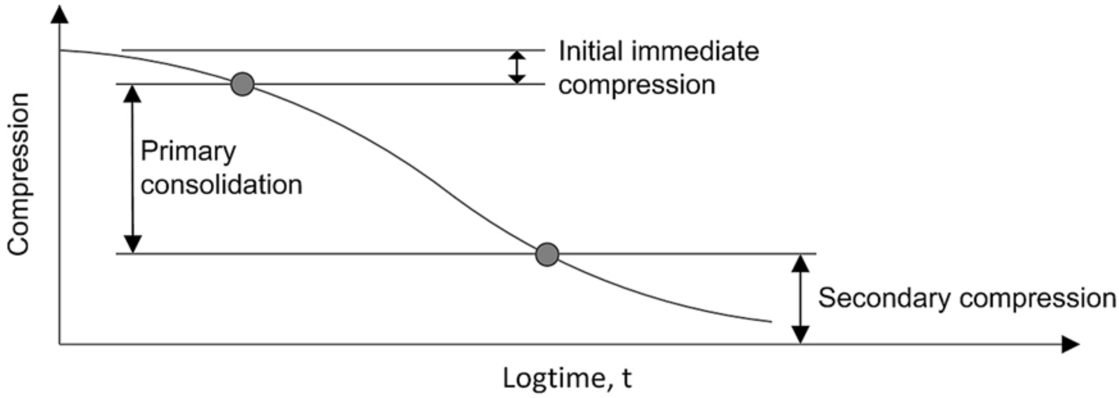


Figure 7.67 The three components of compression

The compressibility of non-cohesive deposits is not usually considered when assessing the settlement of levees because it is relatively small and essentially occurs during construction.

Elastic settlement

Elastic settlement in cohesive soils is instantaneous and recoverable, and occurs under undrained conditions. So, settlement resulting from elastic compression is complete during construction and does not influence the post-construction settlement. These deformations are mostly small compared with the consolidation settlement and may be considered to be within the limits of accuracy of a settlement calculation. However, for high levees forming part of a large project it may be appropriate to assess as it will result in some increase in the quantity of fill required and some misinterpretation of initial field settlement data if all recorded settlement is assumed to be derived solely from primary consolidation. Methods of calculating elastic settlement are presented in Section 8.7.3.1. The undrained elastic modulus (E_u) is dependent upon, among others, plasticity, over consolidation ratio, and stress level.

Primary consolidation

Primary consolidation is time dependent and only partly recoverable. It is the movement of water from the soil voids as a result of the excess pore water pressures generated by the applied load that causes a reduction in volume. The resulting settlement is not linear with the applied load and may be ongoing for an extended period. The assessment of primary consolidation is based on the compression index (C_c) obtained from the gradient of the virgin compression line on an e-log p' plot (see figure in Table 7.97), or the compression ratio (CR)

$$C_c = (e_1 - e_2)/\log (p'_2/p'_1) \quad (7.83)$$

$$CR = C_c/(1+e_o) \quad (7.84)$$

An alternative parameter, which is more appropriate to soils of higher strength and lower compressibility, is the coefficient of volume compressibility (m_v). It defines the settlement that occurs as result of a change in vertical effective stress. The value of m_v reduces with increase in effective stress and is evaluated for each load increment applied during a laboratory test. For calculation purposes the value used may be evaluated over the field stress range, p'_o to ($p'_o + \Delta p'$), where p'_o is the existing *in situ* vertical effective stress and $\Delta p'$ is the change in vertical effective stress due to the imposed load from the levee.

$$m_v = [(e_1 - e_2)/(1+e_1)].[1000/(p'_2 - p'_1)] \quad (\mathrm{m^2/MN}) \quad (7.85)$$

Some settlement calculation techniques require the drained constrained elastic modulus, M, which can be assessed from m_v.

$$M = 1/m_v \quad (\text{MN/m}^2) \tag{7.86}$$

Primary consolidation only occurs when the vertical effective stress in the soil exceeds a certain critical value known as the yield stress (p'_y), sometimes referred to as the pre-consolidation or maximum past pressure. It is determined from the e-log p' plot (see figure in Table 7.97) through direct assessment, where good quality test results are available, or graphical reconstruction such as that developed by Casagrande. The magnitude of the yield stress is influenced by the past loading history and age of the deposit. In normally consolidated soils, it is equal to p'_o. However, this is seldom the case and soils are usually a little over-consolidated as a result of the removal of overburden, changes in groundwater level, desiccation and self-weight secondary compression. So, p'_y is usually greater than p'_o.

The ratio of p'_y/p'_o is termed the over-consolidation ratio (OCR). The magnitude of this ratio defines the *in situ* state of the soil relative to the current effective overburden stress. Over-consolidation may also be defined by the over-consolidation margin (OCM) defined as ($p'_y - p'_o$). Table 7.96 summarises the range in these factors and the definition that is usually applied to them.

Table 7.96 Descriptive terms and associated over-consolidation ratio and margin

Descriptive terms	Over-consolidation ratio	Over consolidation margin (kPa)
Under consolidated	<1.0	<0
Normally consolidated	1.0 but <1.5*	0
Lightly over-consolidated	1.5–4.0* (<2.0**)	0 to 100
Over-consolidated	–	100 to 400
Heavily over-consolidated	>4.0* (>3.0**)	>400

Note

* Look (2007), ** Atkinson (2007)

Table 7.97 Assessment of primary consolidation parameters form e-log p′ plot

Compressibility coefficients:

Coefficient of consolidation:

$$C_c = (e_1 - e_2)/\log(p'_2/p'_1) \tag{7.87}$$

Compression ratio:

$$CR = C_c/(1+e_o) \tag{7.88}$$

Coefficient of volume compressibility:

$$m_v = [(e_1 - e_2)/(1+e_1)].[10^{-3}(p'_2 - p'_1)] \ (\text{m}^2/\text{MN}) \tag{7.89}$$

Constrained elastic modulus:

$$M = 1/m_v \ (\text{MN/m}^2) \tag{7.90}$$

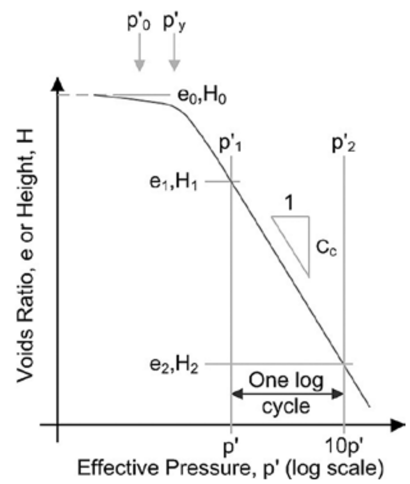
Over-consolidation ratio (OCR):

$$OCR = p'_y/p'_o \tag{7.91}$$

Over-consolidation margin (OCM):

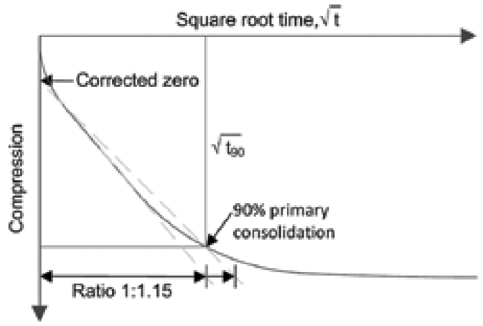
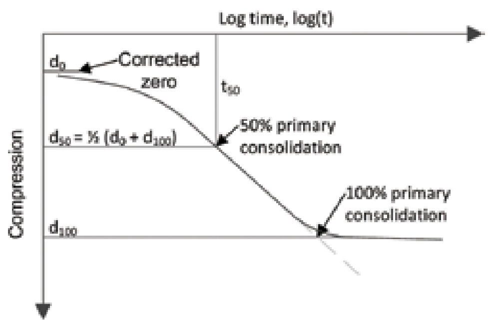
$$OCM = p'_y - p'_o \tag{7.92}$$

Assessment of C_c, CR, m_v, p'_y, p'_o, OCR and OCM from a consolidation test e-log p' plot



The rate at which primary consolidation occurs is controlled by the coefficient of consolidation (c_v). It is usually determined for each load increment during the test by graphical construction on a plot of the settlement of the specimen of thickness, H, under constant applied load against time (t), either $\sqrt{t}$ or log t. Equation 7.93 in Table 7.98 gives the determination of c_v based on the square root time method Equation 7.94 gives the determination by the logarithm time method.

Table 7.98 ***Assessment of coefficient of consolidation from a given loading increment***

Determination of c_v by square root time method	***Determination of c_v by logarithm time method***
	
Coefficient of consolidation (square root time method):	***Coefficient of consolidation (logarithm time method):***
$c_v = 0.111H^2/t_{90}$ (7.93)	$c_v = 0.026H^2/t_{50}$ (7.94)
Where t_{90} is the time to achieve 90 per cent consolidation	Where t_{50} is the time to achieve 50 per cent consolidation

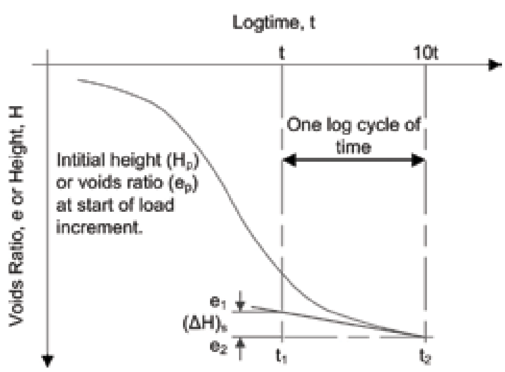
Using the log-time method to determine c_v is the preferable approach if the settlement curves are of a conventional shape. However, the corrected gauge reading, d_0, for the start of the test at time t_0, may not be well defined in some soils and the root-time may offer a better approach. Both methods should be used until one has been established as preferable for a given soil type.

Special consideration needs to be given to interpretation of these curves for silty soil, unsaturated clays and peat (Head, 1982).

Secondary compression

Secondary compression is also time dependent and occurs once the excess pore water pressures driving primary consolidation have essentially dissipated, ie it occurs at constant effective stress. It is defined by the coefficient of secondary compression (C_α) determined from the final straight line section of the compression/log(t) plot for a given applied load and is the compression per log cycle of time. For this reason, where the assessment of C_α is required, a load increment is often maintained for an extended period to better define the straight line section of the plot, or an extended load test is performed on a separate sample loaded to a stress equivalent to that anticipated in the field under the levee in the long-term. As an alternative to C_α the modified secondary compression index ($C_{\alpha\varepsilon}$) can be determined, this is the strain per log cycle of time, related to the voids ratio at the end of primary consolidation (e_p) (Table 7.99).

Table 7.99 ***Assessment of secondary compression coefficients from a given loading increment***

Secondary compression coefficients: ***Coefficient of secondary compression*** $C_\alpha = (e_1 - e_2)/\log(t_2/t_1)$ (7.95) Modified secondary compression index $C_{\alpha\varepsilon} = C_\alpha/(1+e_p)$ (7.96)	 ***Determination of C_α and $C_{\alpha\varepsilon}$***

There are a number of techniques that can be used to assess the compressibility characteristics of a soil, some of which are summarised in Table 7.100. These can range from standard laboratory tests to full scale instrumented field trials. In laboratory tests the specimen may be prone to disturbance that, together with size limitations, is likely to mean the test may not be representative of the mass behaviour of the *in situ* soils as a result of soil fabric and the natural drainage conditions.

Table 7.100 Methods of determining compressibility characteristics

Method	Applications	Limitations
Standard oedometer: load applied in increments that are twice the previous value	• routine testing • relatively quick and relatively low cost – one day per load increment • specimens can be prepared on vertical plane of sample to assess horizontal coefficient of consolidation (C_h).	• small specimen size less representative of *in situ* soil • progressively larger load increments applied so p'_y may be poorly defined on an e-log p' plot • coefficient of consolidation routinely assessed for vertical drainage • no account taken of any horizontal soil fabric • rate of field consolidation usually much greater than that assessed from the oedometer test.
Modified odometer: load applied in constant small increments to p'_y, then as standard oedometer method	• better determination of p'_y as smaller load increments applied up to p'_y • can use automated equipment minimising manual intervention.	• small specimen size less representative of *in situ* soils • each load increment at stress levels below p'_y is applied for a short period until the end of 'primary consolidation' and so several load increments can be applied in one day. Significant technician input required during initial stages of test • coefficient of consolidation routinely assessed for vertical drainage. No account taken of any horizontal soil fabric • rate of field consolidation usually much greater than that assessed from the oedometer test.
Consolidation (Rowe) cell: load applied hydraulically with measurement of compression and pore water pressure	• large specimens sized more representative of *in situ* soil • greater control over loading and drainage boundary conditions.	• needs a high quality large size sample • test may be of long duration depending upon nature of soils • natural variations within specimen may lead to misleading results.
Field trials: construction of an instrumented embankment	• assess field consolidation characteristics where residual uncertainty in performance presents a high project risk.	• duration and cost of test may be prohibitive • measurement of coefficient of secondary compression unlikely due to practical time constraints imposed on trial by project.
SPT: correlation between SPT *N* value and coefficient of compressibility (m_v)	• initial assessment in the absence of other data • heavily over-consolidated soils.	• empirical correlations using uncorrected SPT *N* values • correlation factor is a function of I_p • in soft soils the results should be treated with extreme caution.
CPT piezocone: correlation with cone resistance and coefficient of compressibility (m_v)	• initial assessment in the absence of other data • compare with other site specific test results to develop site specific correlation.	• assessment of a drained compressibility from the undrained parameter of cone resistance using a non site specific correlation can result in significant error.

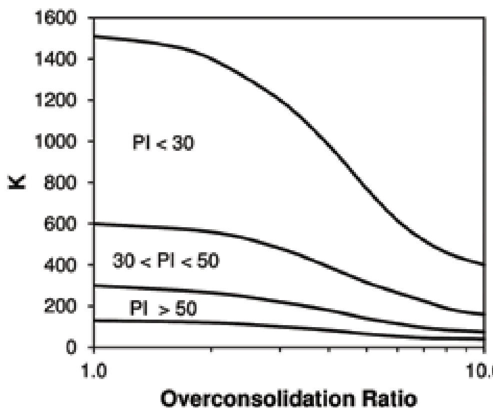
Table 7.100 Methods of determining compressibility characteristics (contd)

CPT piezocone: dissipation test to determine coefficient of horizontal consolidation (c_h)	• initial assessment in the absence of other data • compare with other site specific test results to develop site specific correlation.	• dissipation tests are only indicative of a small volume of soil around the piezocone filter • stress levels in the soil during the test are not representative of those in the field post-construction of the levee
Piezometer: permeability test for determination of coefficient of consolidation (c_h)	• *in situ* testing to complement laboratory test data.	• smear on side of borehole can reduce permeability and of the assessed values of c_h • stress levels in the soil during the test are not representative of those in the field post-construction of the levee.
Relevance to levees		
• the amount and rate of settlement is a key consideration in the design and construction process. Post-construction it is a driver of the future maintenance programme • high quality specimens are required if good results are to be obtained from laboratory tests • peats and organic clays experience a large amount of secondary compression. A single increment sustained load test may be required on selected specimen to obtain adequate secondary compression data. The applied load should be comparable with that anticipated in the field.		

Undrained elastic modulus

The undrained elastic modulus is normally correlated with undrained shear strength and is a function of plasticity index and over-consolidation ratio, as indicated in Table 7.101.

Table 7.101 Example of a method of determining undrained elastic modulus

Undrained elastic modulus: $E_u = K\, c_u$ (7.97) where: c_u = undrained shear strength of clay (kN/m^2) K = coefficient obtained from chart For practical purposes a lower caution value of E_u could be taken as: $E_u = 200\, c_u$ (7.98)	 *Chart for the estimating of undrained modulus (E_u) for clays (after Duncan and Buchignani, 1976)*

Compression index and compression ratio:

Table 7.102 presents some indicative values of compression index (C_c) for some typical soils together with empirical correlations with index properties.

Table 7.102 ***Compression index: degree of compressibility, typical values and empirical correlations***

Degree of compressibility	C_c	Empirical correlations	C_c
Low	<0.2	Normally consolidated clay	$0.007\ (w_l-10)$
Moderate	0.2 to 0.4	Low to medium sensitivity clay	$0.009\ (w_l-10)$*
High	>0.4	All clays	$1.15\ (e_o-0.35)$
Typical values		Inorganic cohesive soil	$0.30\ (e_o-0.27)$
Normal consolidated clays	0.2–0.5	Very low plasticity soil	$0.75\ (e_o-0.50)$
Organic clays	4.0 +	Organic soil	$0.0115\ w$
Organic silts and clayey silts	1.5–4.0		
Peats	10.0–15.0		

Note

w_l denotes liquid limit (%).

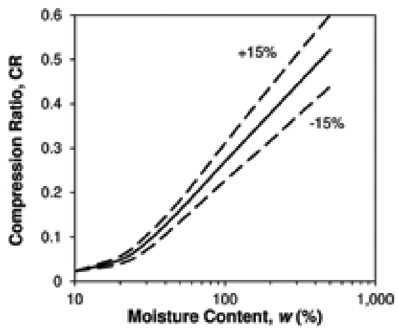
e_o denotes initial void ratio and w denotes natural moisture content.

* denotes reliability range ±30 per cent for inorganic clay with a sensitivity ≤4 and w_l ≤100 per cent.

The compression ratio (*CR*) is effectively a normalised version of the compression index (C_c), which reduces the data scatter. Some indicative values are presented in Table 7.103, which are based on a limited dataset and should be used with caution, together with a correlation for *CR* with natural moisture content developed by Lambe and Whitman (1979).

Table 7.103 ***Compression ratio: typical values and empirical correlations***

Degree of compressibility	*CR*
Very low	<0.05
Low	0.05 to 0.1
Moderate	0.1 to 0.2
High	0.2 to 0.35
Very high	>0.35
Typical values	
Inorganic clays and silts	0.05 to 0.20
Organic silts and clayey silts	0.15 to 0.30
Peats	0.30 to 0.60



Correlation of compression ratio with natural moisture content (from Lambe and Whitman, 1979)

Coefficient of volume compressibility

Table 7.104 presents some indicative values of m_v for some typical soils. There is an absence of empirical correlation with index properties, probably because the parameter is stress dependent.

Table 7.104 ***Coefficient of volume compressibility: degree of compressibility and typical values (after Carter, 1983)***

Degree of compressibility	Type of clay	Typical range of coefficient of compressibility, m_v (m^2/MN)
Very high	Peat and highly organic alluvial clay	>1.5
High	Normally consolidated alluvial clays. Estuarine and delta deposits, and sensitive clays	0.3–1.5
Medium	Firm clays, glacial outwash clays, lake deposits, weathered marls, firm boulder clays, normally consolidated clays at depth and firm tropical red clays	0.3–0.1
Low	Boulder clay, marls, very stiff tropical red clay	0.05–0.1
Very Low	Heavily over-consolidated boulder clay, weathered mudstone and hard clay	<0.05

A method of assessing values of m_v from SPT N values was proposed by Stroud and Butler (1975). Equation 7.99 is an approximation of the correlation for clay with a plasticity index greater than 25 per cent. This relationship was derived on over-consolidated clays and should only be used with caution for other clays that may have low SPT N values.

$$m_v \approx 1/(0.45N)\ (\mathrm{m^2/MN}) \tag{7.99}$$

Correlations of CPT cone resistance (q_c) with compressibility are based in the drained constrained modulus (M). For normally consolidated soils (M) measured in the odometer at stress equivalent to p'_y can be estimated from Equation 7.100

$$M = 1/m_v = \alpha_m . q_c\ \ (\mathrm{MN/\ m^2}) \tag{7.100}$$

Where values of α_m vary with soil type and measured cone resistance as detailed in Table 7.105.

Table 7.105 ***Variation in coefficient α_m with soil type and measured cone resistance (after Lunne et al, 1997)***

Soil type	**Range of measured cone resistance q_c (MN/m$_2$)**	**Range of coefficient α_m**
Clay of low plasticity (CL)	<0.7 0.7 to 2.0 >2.0	3 to 8 2 to 5 1 to 2.5
Silt of low plasticity (ML)	>2.0 <2.0	3 to 6 1 to 3
High plasticity silt and clay (MH,CH)	<2.0	2 to 6
Organic silt (OM)	<1.2	2 to 8
Peat and organic clay (P_t, OC)	<0.7, with water content (w) 50%<w<100% 100%<w<200% w>200%	 1.5 to 4.0 1.0 to 1.5 0.4 to 1.0

Note

Lunne *et al* (1997) presents other methods that correlate cone resistance to constrained modulus *M*.

Yield stress

A number of correlations have been developed that relate index properties and properties measured *in situ* to σ'_y (Kulhawy and Mayne, 1990). However, all the methods have a common link with the assessment of undrained shear strength (c_u), confirming that there is a fundamental correlation between c_u and σ'_y (or p'_y). So, the undrained shear strength of the soil may initially be determined using the methods outlined in Section 7.8.3.3.

Empirical correlations exist that relate c_u of a normally consolidated clay to the applied vertical effective stress (σ'_v). These expressions are usually in the form (Table 7.88):

$$c_u/\sigma'_v\ (\text{or } c_u/p'_y) = f(y) \tag{7.101}$$

If it is assumed that the undrained shear strength reflects the apparent maximum σ'_v that the soil has experienced in the past it can be equated to p'_y and the expression rearranged:

$$\sigma'_v = p'_y = c_u/f(y) \tag{7.102}$$

Investigators have observed that the c_u/p'_v (or c_u/σ'_v) ratio varies with the plasticity index (I_p) and the degree to which the soil is 'aged'. Table 7.88 presents a number of these correlations. Using the plot for a 'young' clay (Bjerrum, 1972) and making the substitution of p'_y for σ'_v, p'_y can be assessed for a known value of plasticity index (I_p) and c_u from Equation 7.102, where f(y) is the ratio of c_u/σ'_v reported on the y-axis.

A simple correlation for assessing the yield stress from the undrained shear strength in the absence of any other data is presented in Equation 7.103.

$$p'_y = c_u/0.25 \text{ or } 4c_u \qquad (7.103)$$

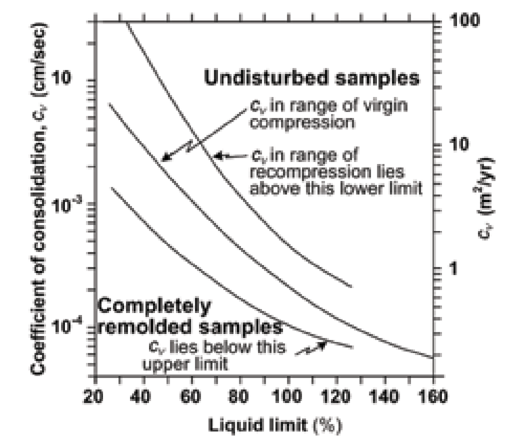
Coefficient of consolidation

Laboratory tests are conducted on relatively small samples. In the field factors such as soil fabric, composition and direction of the drainage path will influence the rate of primary consolidation which may occur between five and 15 times quicker than that assessed using laboratory derived values of c_v.

In the absence of measured data Table 7.106 presents some typical values of c_v for various soil types and a correlation of c_v with liquid limit, depending upon the condition of the soils.

Table 7.106 Coefficient of consolidation: typical values (after Lambe and Whiteman, 1979)

Soil type	Range of plasticity index (I_p)	Coefficient of consolidation, c_v (m^2/yr)	
		Undisturbed	*Remoulded*
Clay high plasticity	>25	0.1 to 1.0	About 15 to 50 per cent of the undisturbed value
Clay medium plasticity	25 to 15	1.0 to 10	
Clay low plasticity	<15	10 to 100	
Silt		>100	



Correlation between c_v and w_l (after UFC, 2005)

Coefficient and modified coefficient of secondary compression

The coefficient of secondary compression (C_α) has been correlated with the coefficient of consolidation (C_c) for a number of soils (Mesri and Goldlewsk, 1977) and the modified coefficient of consolidation has been correlated with the moisture contents (Mesri, 1973). These are presented in Table 7.107.

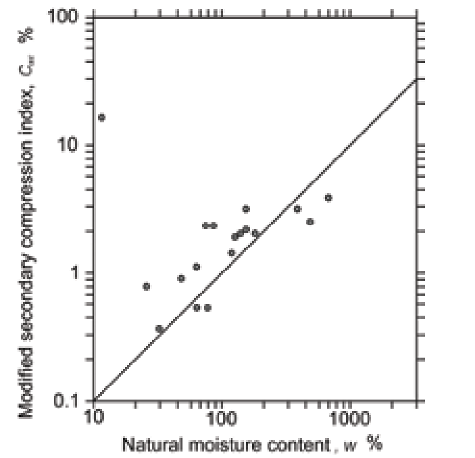
The ratio between C_α and C_c is approximately constant for most normally consolidated clays at loads typically within engineering practice.

$$C_\alpha / C_c = 0.05 \pm 0.01 \qquad (7.104)$$

Variations in this relationship with soil type are presented in Table 7.107.

Table 7.107 ***Coefficient and modified coefficient of secondary compression: typical values and correlation with natural moisture content***

Soil type	**Coefficient of secondary compression (C_α)**
Normally consolidated clay	0.005 to 0.02
Very plastic clay	≥0.03
Organic clay	≥0.03
Over consolidated clay (OCR>2)	<0.001
Soil type	**C_α/C_c**
Inorganic clay and silts	0.040±0.01
Organic clays and silts	0.050±0.01
Peat	0.075±0.01
Note that the ratio $C_\alpha/C_c = C_{\alpha\varepsilon}/CR$	
For the correlation presented in the figure, $C_{\alpha\varepsilon}$ is related to the moisture content (w%) by: $C_{\alpha\varepsilon} = 0.0001w$ (7.105)	



Variation of natural moisture of soil as a function of the index of secondary compression $C_{\alpha\varepsilon}$ (after Mesri, 1973)

7.8.3.5 Permeability

Permeability is the property that measures how easily water flows through soil. It is one of the most important soil characteristics in evaluating levee performance. Excessive seepage through the levee and foundation material can lead to failure due to structural instability or internal erosion. It is also difficult to estimate the condition of a levee that experiences seepage forces based on a visual inspection. This is because the seepage will only occur when the hydraulic conditions are appropriate and seepage is evident, and often only in extreme events, leading to the formation of boils, erosion and piping, and slope instability. This makes the measurement and assessment of permeability characteristics critical to assessing the performance of a levee.

When subject to a hydraulic head the laminar flow of water through the saturated portion of the levee and foundation soils can be defined by Darcy's Law. The typical simple conceptual hydraulic model of the flow of water in a saturated soil is presented in Table 7.108, together with Darcy's Law.

Table 7.108 ***Assessment of permeability in saturated soil using Darcy's Law***

Darcy's Law:

$q = A\,k\,i$ (7.106)

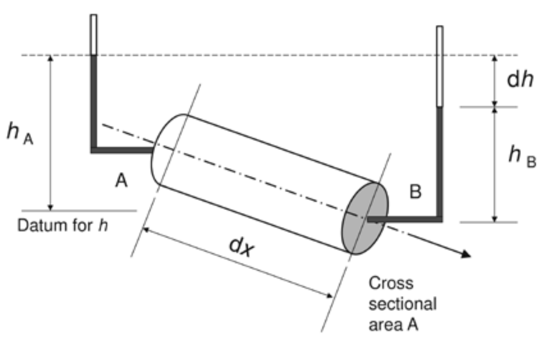
where:

q = volumetric flow rate (m^3/s)

A = cross-sectional area of flow (m^2)

k = coefficient of permeability (m/s)

i = hydraulic gradient in the direction of flow, dh/dx



Schematic representation of hydraulic gradient in a saturated soil

For unsaturated soils the permeability is less than for saturated soil because of the interaction of the water surface tension with the soil particles slowing down the flow. This influence decreases as the degree of saturation increases. That is, permeability increases with increased saturation. When the degree of saturation is less than 80 per cent, much of the air within the soil would be continuous throughout the soil voids and Darcy's Law is not applicable. If the degree of saturation is greater than 80 per cent, most of air present in the soil will be discontinuous in the form of small occluded bubbles, and Darcy's Law will provide an approximation of behaviour.

In the event that a transient seepage analysis is performed then unsaturated permeabilities will need to be used, which can be obtained from references such as, Wösten *et al*, 2001.

There are a number of factors that need to be considered when assessing the permeability characteristics of a levee and the foundation soils as these can make the measurement and subsequent analyses of levee performance complex. These are detailed in Table 7.109.

Table 7.109 Some soil factors to be considered when assessing permeability

Soil factor	Influence
Macro affects	• water pressure finds the path of least resistance and variable ground conditions make it difficult to get representative laboratory samples for testing • larger scale field tests, such as pumping tests, are often required to get a more representative assessment of soil mass permeability • an initial assessment of permeability can be made using presumptive values such as those detailed in Table 7.111, which have been derived on the basis of field observation.
Anisotropy	• often the horizontal permeability of a levee and the foundation soils is higher than the vertical permeability. This is due to the horizontally bedded nature of both materials resulting from the construction process and fissuring, and depositional history. Laboratory and field tests need to consider this anisotropy during sampling, testing and evaluation • higher permeability soils tend to be non-cohesive and so are not amenable to undisturbed sampling methods, which would preserve the soil structure.
Partially saturated soil	• levees may only be required to retain floodwater for a limited period of time. For the majority of the time they do not impound water. Measured permeabilities are usually based on fully saturated steady conditions, which overestimates the permeability of the partially saturated soil.
Sampling method	• bulk samples are usually obtained in non-cohesive soils. The sampling technique may result in some loss of fines, specifically where the borehole is advanced using cable percussion boring techniques (Table 7.149), which can affect the assessed or measured permeability.

There is a variety of *in situ* and laboratory test methods, and correlations that can be used to assess the permeability of soils. Laboratory tests need undisturbed samples, typically sample Class 1 or 2, but these can only be obtained in cohesive soils. For non-cohesive soils disturbed samples do not preserve *in situ* structure and there may be a loss of fines. Some of the commonly used methods of assessing permeability are discussed in Table 7.110.

Table 7.110 Some in situ and laboratory methods of assessing permeability

Method	Applications	Limitations
CPT dissipation tests	• horizontal permeability of soils around the CPT piezocone can be estimated from either the soil classification or calculations based on the time for partial dissipation of excess pore water pressure • provides an initial estimate of permeability.	• influence limited to small volume of soil around the CPT and soil fabric • results should be verified by other methods.
Surface permeameter or infiltration test: open and close systems	• provides a measured amount of surface infiltration or percolation • open flow systems are used to measure permeabilities in the range 10^{-5} to 10^{-8} m/s • closed systems are used where the permeability is less than 10^{-8} m/s.	• depending on soil permeability, the test could run for either several minutes or several days • care is needed in setting up the apparatus to prevent erroneous reading from inadvertent loss of water at the outer edges • deep fissures in the soils could provide a preferential leakage path • infiltration may vary over time as a result of closure of the fissures • limitations on the water head that can be applied, which is usually less than 1 m.
Pump tests – pumping water from a well and measuring draw-down curve at equilibrium	• provides estimates of permeability on a macro scale • can provide a good source of data for relief well design.	• more expensive and time consuming than other tests • may require disposal of significant amounts of water from a drawdown test in a well • does not provide data on soil layers above water table • fluctuation in the water table over duration of test can affect results.
Rising and falling head tests in boreholes	• provides estimates of permeability on a macro scale • suitable for soils with permeability in the range 10^{-6} to 10^{-9} m/s • falling head test may be performed in unsaturated zone above water table but the results should be treated with caution.	• in unstable ground where drill casing is to be partly extracted the test section needs to be prepared using a perforated pipe or a gravel pack to provide support • fluctuation in the water table over duration of test can affect results • results only relate to the soils influencing the flow of water in the test zone • any sediment suspended in the water column can settle out during falling head test, reducing the permeability of the system.
Constant head test in the borehole	• provides estimates of permeability on a macro scale • suitable for soils with permeability in the range 10^{-4} to 10^{-7} m/s • may be performed in unsaturated zone above water table.	• in unstable ground where drill casing is to be partly extracted the test section needs to be prepared using a perforated pipe or a gravel pack to provide support • fluctuation in the water table over duration of test can affect results. • results only relate to the soils influencing the flow of water in the test zone • any sediment suspended in the water column can settle out during an inflow, reducing the permeability of the system.
Rising and falling, and constant head tests in a piezometer	• measure the horizontal permeability of individual soil layers below a water surface.	• permeability of piezometer filter or sand cell must be greater than the permeability of the soil being measured.

Table 7.110 Some in situ and laboratory methods of assessing permeability (contd)

Slug tests – adding or removing a measured quantity of water in a well rapidly and measuring recovery with time	• relatively low cost test and limited time needed • can be conducted in existing boreholes.	• provides estimate of permeability for material close to the well screen • permeability of sand pack in well may influence results.
Constant head test: permeameter	• laboratory assessment for coarse grained soils • permeability in the range 1 to 10^{-2}m/s (large constant head cell) and 10^{-2} to 10^{-5} m/s (standard constant head cell).	• applicable for soils with less than about 10 per cent fines or with a permeability $>10^{-6}$ m/sec • rate of flow should be low enough to maintain laminar flow through soil.
Falling head test	• laboratory assessment for fine grained soils • sample can be prepared to assess either horizontal or vertical permeability • permeability in the range 10^{-5} to 10^{-9} m/s	• sample has to be uniform and representative of field conditions • time for completion of test increases rapidly with increase in fines content • achieving full saturation of specimen prior to testing is essential • applied hydraulic gradients need to be compatible with soil permeability.
Oedometer tests	• laboratory assessment for fine grained very low permeability soils • permeability of soil can be calculated from consolidation test data • view as an initial assessment or confirmatory test data • permeability $>10^{-9}$ m/s.	• results need to be checked with data from other sources due to smaller sample size • load increments should be maintained for the full duration and not curtailed • measured specific gravity should be used in calculations.
Triaxial cell	• laboratory assessment for fine grained low permeability soils.	• achieving full saturation of specimen before testing is essential • application of proper hydraulic gradients will affect results • filter strips should not be used on the sides of the specimens for achieving rapid saturation.
Relevance to levees		
• determination of seepage, seepage pressures, hydraulic gradients and evaluating the potential for the development of piping and erosion both within the levee and the foundation soils.		

These values are typically based on back calculation from local historic performance records and tests on similar geological units. Examples of typical permeabilities for a range of soil types are presented in Table 7.111 and Figure 7.68.

Table 7.111 Ranges of permeability for typical soil types (from ICE, 2012)

Soil type	Degree of permeability	Typical values of permeability (m/s)
Clean gravels	High	$>10^{-3}$
Sand and gravel mixtures	Medium	10^{-3} to 10^{-5}
Very fine sands, silty sands	Low	10^{-4} to 10^{-7}
Silt and interlaminated silt/sand/clays	Very low	10^{-6} to 10^{-9}
Intact clays	Practically impermeable	$<10^{-9}$

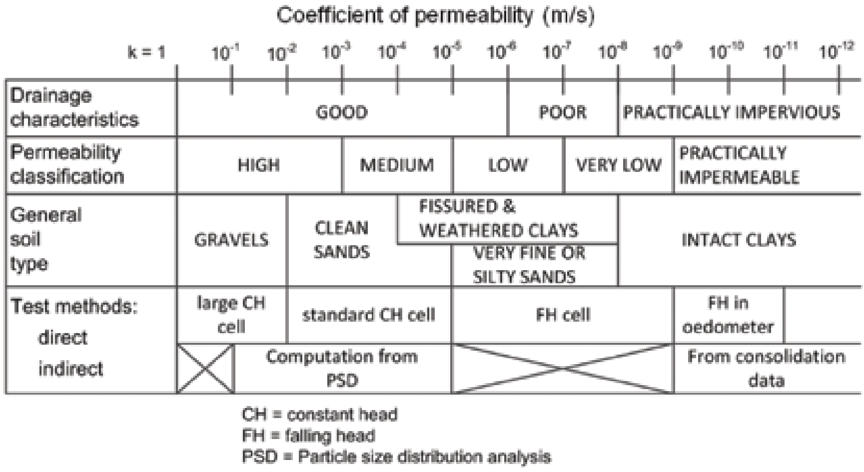


Figure 7.68 Typical permeability for a range of soil types (Head, 1982)

Figure 7.69 presents a correlation between grain size (D_{10}), voids ratio (e) and permeability for clean, coarse grained soils.

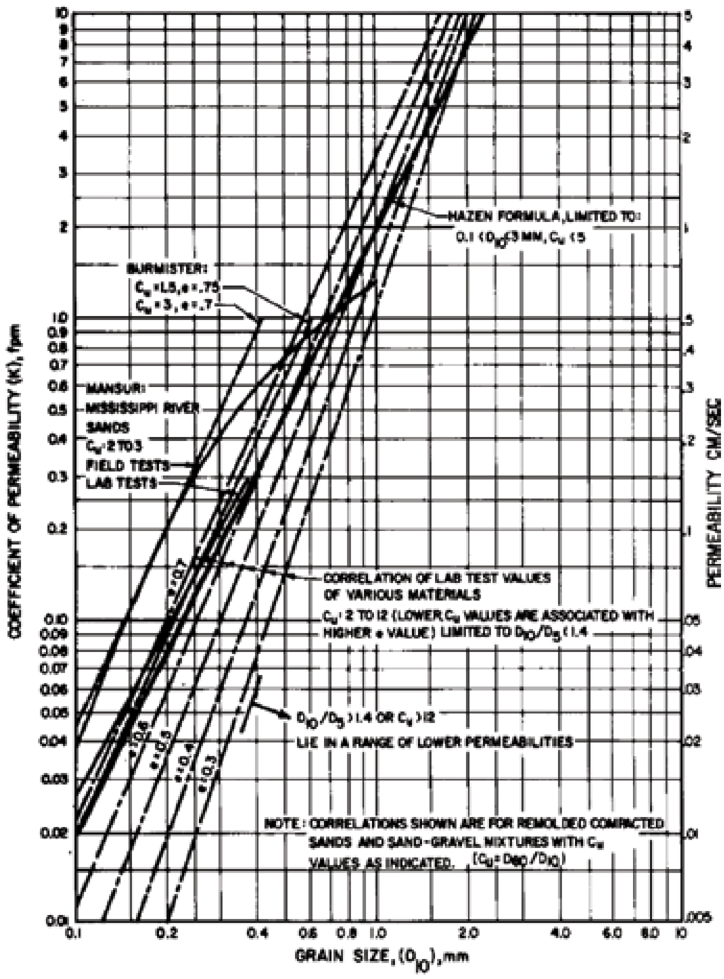


Figure 7.69 Permeability of sands and sand-gravels mixtures (from UFC, 2005)

Permeability characteristics can also be estimated using empirical formulae that correlate with grain size distribution. The results from these methods often vary by orders of magnitude and the results should be treated with caution. Table 7.112 presents some empirical approaches that have been used to assess permeability from grain size distribution. By their nature these methods are only applicable to coarse grained soils with permeabilities in the range 10^{-1} to 10^{-5} m/s.

Table 7.112 Some empirical correlations for the assessment of permeability in coarse grained soils

<table>
<tr><th colspan="3">Method</th><th>Application</th><th>Limitation</th></tr>
<tr>
<td>Hazen (1892)
$k = 0.01D_{10}^2$ (m/s) (7.107)
More generally:
$k = CD_{10}^2$ (m/s) (7.108)
where:
k = permeability (m/s)
C = coefficient varying with C_U
D_{10} and D_{60} are the grain size diameters in mm corresponding to 10 and 60 per cent passing</td>
<td>$C_U = D_{60}/D_{10}$

1.0 to 1.9
2.0 to 2.9
3.0 to 4.9
5.0 to 9.9
9.9 to 19.9
>20.0</td>
<td>C

0.011
0.010
0.009
0.008
0.007
0.006</td>
<td>• simple equation to use for preliminary assessment of permeability</td>
<td>• applicable to clean sands from which the equation was developed through experimental work
• ignores influence of voids ratio on permeability, which is significant.</td>
</tr>
<tr>
<td colspan="3">Kozeny – Carman Equation
Carman (1939)
$$k = \frac{\rho_w g}{C \eta_w S^2} \times \frac{e^3}{1+e} \qquad (7.109)$$
where:
k = coefficient of permeability (m/s)
g = acceleration due to gravity (9.81m/s²)
ρ_w = mass density of water (1.00 mg/m²)
η_ω = dynamic viscosity of water (1 mPas at 20°C)
e = voids ratio of soil
S = specific surface area of grains (mm⁻¹)
C = grain shape factor (five for spherical particles)
This can be simplified to:
$$k_{20} = 1.962 \frac{1}{fS^2} \times \frac{e^3}{1+e} \qquad (7.110)$$
where:
k_{20}= coefficient of permeability at 20°C (m/s)</td>
<td rowspan="2">• takes account of the full grain size distribution. For granular soils containing a range of grain sizes the angularity is assigned to the fraction retained on each sieve (Section 7.8.3.1). The specific surface area is also assessed for each section. This gives a value of fS^2 for each sieve size. A combined factor for the whole sample is obtained by combining the separate factors in proportion to the percentage retained on each sieve.</td>
<td rowspan="2">• more complex calculation
• assessment of voids ratio also required
• considered not suitable for fine grained soils. However, work by Chaouis and Aubertine (2003) suggests that it can be used to estimate permeabilities in the range 10^{-1} to 10^{-11} m/s to within 0.33 to three times the measured value.</td>
</tr>
<tr>
<td>f = grain angularity factor.

Typical values are:
f = 1.10 (Rounded)
f = 1.25 (Sub rounded)
f = 1.40 (Angular)</td>
<td colspan="2">S = specific surface area of grains (mm⁻¹)
$$S = \frac{6}{\sqrt{(d_1 d_2)}} \qquad (7.111)$$
where:
d_1 and d_2 are the ranges of grains size diameters being considered (mm)</td>
</tr>
</table>

7.8.3.6 Erodibility

The erosion of soils forming the levee results from the removal of soil particles or aggregates of soil particles by, in this context, the action of water. Erosion of the soils underlying a hard revetment can contribute to damage of the surface protection. On levees that do not have a revetment, erosion can occur at the zone of wave attack or where water flows are locally concentrated by structures or natural obstructions.

Erosion may occur because of the following:

1 **Dispersion:** the separation of clay particles from the surface of the soil mass in the presence of water. This can occur under no flow conditions and results from a loss of the electro-chemical bond between the clay particles causing a loss of cohesion. For some soils it can be an important factor in a changing fresh-salt water environment.

2 **Surface flow of water:** the movement of water over the surface of a soil can loosen and remove individual soil particles or aggregates of particles. This process is generally more pronounced in sandy soils than higher plasticity clays. However, if the clay has a well developed soil structure then it too can have a low resistance to erosion.

3 **Wave action:** the forces generated by breaking waves on the levee slope cause the disturbance and removal of soil particles and aggregates. The wave action can also cause the build-up of pressure around blocks of fissured clay causing them to become dislodged. A similar mechanism can occur on the landward slope during the overtopping of a levee.

There are a number of *in situ* and laboratory tests that can be undertaken to assess the erodibility of a soil. These are summarised in Table 7.113.

Table 7.113 Some laboratory and in situ methods of assessing erodibility

Method	Applications	Limitations
Crumb test: visual assessment of the degree of dispersion of soil crumbs in distilled water or sodium hydroxide solution	• qualitative assessment of dispersion potential through a visual assessment.	• does not prove a direct quantitative assessment of erodibility due to the action of water.
Cylinder test: extension of crumb test but performed on a reconstituted soil sample	• quantifies the disaggregating geometry of an immersed unsaturated soil specimen as a function of time • visual assessment of hydration and dispersion.	• does not prove a direct quantitative assessment of erodibility due to the action of water.
Dispersion test: particle size distribution (hydrometer method) determined on pre treated and untreated samples	• provides an indication of the natural dispersive characteristics of clay soils.	• method may not identify all dispersive clay soils. It has about an 85 per cent probability of predicting dispersive behaviour.
Chemical test: determination of sodium absorption ratio	• provides an indication of whether a clay is dispersive • determines the amount to sodium in the pore water relative to other cations.	• complex test procedure required equipment usually found in a chemical laboratory.
Pin hole test: distilled water passed through a 1 mm diameter hole on a re-compacted sample	• provides an indication measurement of dispersion and colloidal erodibility of clay • three alternative methods of classifying the dispersion of clay based on the results of the test.	• not a quantitative test method • provides a relative assessment of the erosional performance of a clay.
Mobile Jets erosion test (MoJET): rotating six nozzle jets provide erosive action to surface of soil	• quantitative and qualitative determination of erosion in laboratory or field • surface material can be included.	• requires experienced staff to perform.
Hole erosion test (HET): distilled water passed through a 6 mm diameter hole on a re-compacted sample	• provides a quantitative characterisation of piping erosion on a remoulded sample • provides a more comprehensive measurement of erosion parameters.	• laboratory based test • requires experienced staff to perform.

Table 7.113 Some laboratory and in situ methods of assessing erodibility (contd)

Jet index erosion test (laboratory or field): single submerged jet nozzle with a constant pressure head	• quantitative estimation of erodibility of soils • laboratory method performed on undisturbed or remoulded samples • field method performed on *in situ* soils.	• not suitable for soils that have a dominant grain size of >70 mm. Larger grain sizes can increase erodibility/detachment of the finer grains • the degree of saturation before testing can affect the results • requires well equipped and experienced staff to perform.
Relevance to levees		
• provides qualitative and quantitative indicator of propensity of soils to disperse or erode under the action of water.		

The erosion resistance of a soil may also be assessed from index properties such as grain size distribution and Atterberg limits.

It has been found that soils with more than 40 per cent sand can be eroded quickly with relatively low flow rates. Soils with a plasticity index of less than 18 per cent also erode quickly. However, clays with liquid limits greater than 45 per cent generally have a high resistance to erosion, even at sustained high flow rates. Some clay with a liquid limit greater than 45 per cent can show a strong degree of erosion but these had a low plasticity index (TAW, 1996). The findings of the TAW (1996) report are summarised in Table 7.114.

Table 7.114 Assessment of erosion resistance category based on particle size distribution and Atterberg limits (from TAW, 1996)

Erosion resistance category	Index properties
Erosion resistant clay	Liquid limit: >45% Plasticity index: above A-line on plasticity chart Sand content: <40%
Moderately erosion resistant clay	Liquid limit: <45% Plasticity index: >18% Sand content: <40%
Clay with little erosion resistance	Liquid limit: below A-line on plasticity chart Plasticity index: <18% Sand content: <40%

The suggested requirements for the Atterberg limits are summarised in Figure 7.70.

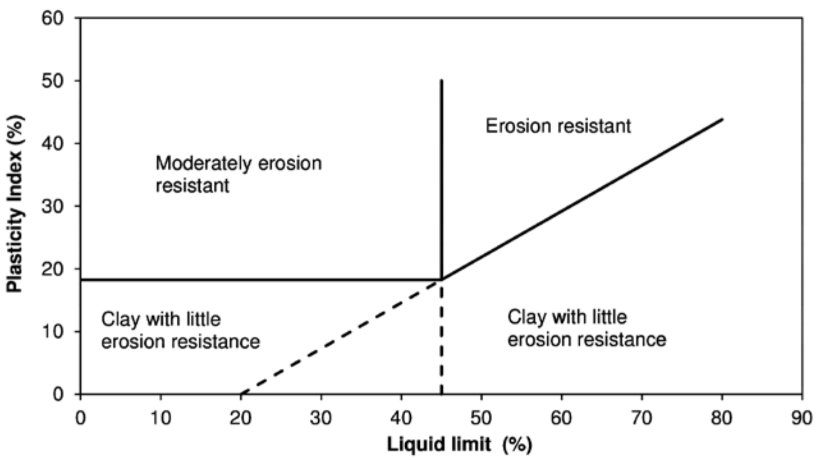


Figure 7.70 Erosion resistance in relation to the plasticity chart (after TAW, 1996)

7.8.4 Determination of characteristic values

Site characterisation can generate a large and diverse array of data. However, design calculations require a single value for each parameter that is appropriate to the limit state for each soil unit over a given region. The array of data should also be interpreted to provide simplified characteristic subsurface models that are appropriate to the method of analysis.

The derivation of a characteristic value requires professional judgement. Some factors to consider include:

- design codes and standards being used
- degree of conservatism appropriate for the limit state being analysed
- applicability of the data to the limit state being evaluated
- in the case of soil strength, whether the soil will strain harden or strain soften as it shears
- method by which empirical coefficients or correlations were originally derived
- quantity and quality of available data
- comparable experience.

> "The selection of characteristic values of relevant ground parameters is probably the single most important task that a geotechnical engineer undertakes in design. Although partial factors [or other methods of incorporating safety in a design] provide a degree of reliability, they cannot compensate for gross errors of judgement in interpretation of the operational conditions in the ground."
>
> Source: Bond and Harris, 2008

Considerations when assessing a characteristic value

Data that appear to be outliers or anomalous require special attention. Testing methods should be checked for errors, both in methodology and transcription, previously assessed soil layer boundaries may need to be re-evaluated and the context of the data considered spatially. The results may be compared with other data in order to provide supporting or contrary evidence as to the validity of the data. If the results are deemed valid, then the designer has to consider the possibility that the values are representative of local conditions at that point and develop an approach that gives sufficient allowance to that data. Designing an entire levee system based on the worst case soil conditions could make project costs prohibitive. Ignoring an apparent anomaly/outlier in design could result in an unrecognised vulnerability remaining in the levee system. So, data of a common type should not be grouped and evaluated indiscriminately across the site, and account needs to be taken of the spatial distribution in the context of the geological setting.

The spatial evaluation of data on plan and in elevation (or depth/depth below top of a soil unit) may be achieved by reviewing the data in 2D on a plan and through vertical sections, and in 3D, where possible. If 3D visualisations/plots are not available, then key data can be plotted on a plan view as an aid to identifying the lateral variability in properties. There may be trends and changes that can be related back to the geological setting or previous site history, or that are simply an indication that something is different at a specific location and needs further investigation. Cross referencing the locations of apparently anomalous data with information contained in the CSM (Section 7.1.3) may provide clues to its origins. Some degree of real time data interpretation can be done during the investigation field work to allow validation and/or updating of the CSM as necessary, and to identify areas needing further investigation while equipment is still on site. Giving consideration to the spatial distribution of data can contribute to the identification of reaches of levee with similar characteristics of both soil types and geotechnical properties.

In order to ensure that the laboratory and field *in situ* test results are consistent and credible, they can be compared against each other, and against other sources of data, which may include:

- information contained within the CSM
- geological setting
- geophysical and other remote sensing methods

- field logs and observations
- specimen descriptions
- variations in results between drilling crews and phases of the investigation
- correlations with index properties
- data from other similar sites
- published values
- case studies, especially those dealing with back analysis of limit state conditions.

There should be an awareness of the effects of scale and the method of testing on the results. This can be reflected in the size of the sample tested in the laboratory, and between laboratory tests and *in situ* test results. Generally the larger the volume of soil tested, specifically where it contains fabric and structure, the more representative the result will be of the *in situ* soil mass. So, appropriate interpretation of the data is required.

Different forms of a test that outwardly measure the same parameter can give different results and should be used appropriately in calculations. For example, in clays the undrained shear strength could be measured by triaxial compression/extension, simple or pure shear and the shear vane. Each will give a different value, some of which are applicable to different discrete sections of the ultimate limit state rotational failure surface along the edge of a levee. This is illustrated in Box 7.33. Data from the test that best replicates the limit state condition should be used.

Box 7.33 ***Relevance of test method to the development of ultimate limits state***

Triaxial compression is applicable below the levee on the declining section of the slip surface.

Direct shear is appropriate over the horizontal section of the slip surface.

Triaxial extension is applicable over the inclined section of slip surface beyond the levee toe.

The field shear vane test, corrected with Bjerrum's μ factor (Box 7.32), provides an assessment of the average shear strength on the slip surface.

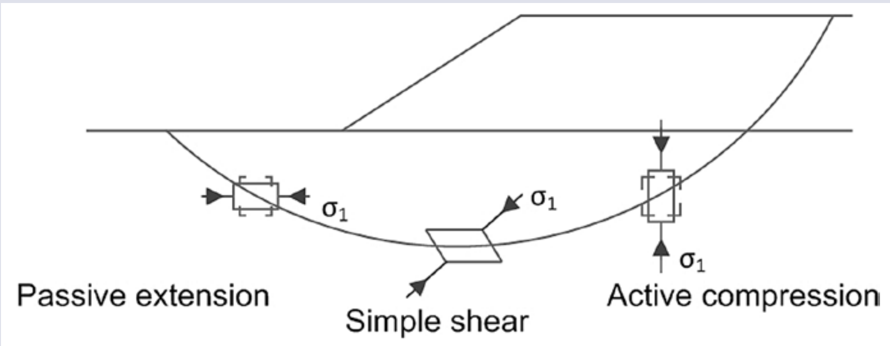


Figure 7.71 ***Relevance of laboratory shear tests to modes of shear in the foundation of a levee***

Characteristic values need to be sufficiently conservative to account for residual uncertainty at the end of the site characterisation process. Soil types, properties and boundaries can vary significantly over short distances, both vertically and laterally, because of the complex and dynamic geological settings along rivers, coasts and estuaries. Even with good practice, given the inherent variability of the subsurface, it is possible, even likely, that the investigations may miss both the very worst and best soil conditions present on site. As levees are long linear structures there only needs to be a local failure at one weak point for the entire system to fail.

There is significant variation in how the overall conservatism, or factor of safety, is handled in an analysis to avoid the occurrence of an ultimate limit state. This conservative bias can be built solely into the characteristic value, or applied separately in the calculations through partial factors on actions (loads) or to their effects, or ensuring that restoring actions exceed disturbing actions by a defined proportion (lumped factor of safety), or it may be a combination of these. Refer to the relevant country codes and standards for further information, and Section 9.10.1.

Various terminologies have been used to describe qualitatively the different degrees of conservatism that may be applied in deriving characteristic values for geotechnical parameters. For each case additional factors may be built into actions within the analysis to ensure the ultimate limit state condition is not exceeded:

- **cautious estimate:** "cautious estimate of the value affecting the occurrence of the limit state." (paragraph 2.4.5.2(2)P, BS EN 1997-1:2004)
- **representative:** "conservative estimate of the properties of the soil as it exists *in situ*... properly applicable to the part of the design for which it is intended." (BS 8002:1994)
- **moderately conservative:** "conservative best estimate" (Padfield and Mair, 1984) or "a low cautious average" (Bond and Harris, 2008)
- **worst credible:** "the worst which the design could realistically believe might occur. Not the worst physically possible but – a value which is unlikely to be exceeded." (Padfield and Mair, 1984).

Where the worst credible value is adopted, lower factors of safety or partial factors may apply.

Characteristic values need to ensure the bias is set reasonably. In a stability analysis, lower strength values will give a more conservative result but in a seepage assessment high permeability values are more conservative. Alternately, lower values of permeability may be more conservative for a lower permeability soil that is underlain by a higher permeability soil and the failure mode is due to uplift of the overlying lower permeability soil. So, there are situations where both high and low characteristic values may be appropriate for the same parameter in the same soil, but the value depends on the limit state condition being assessed. Higher characteristic values might be more appropriate under extreme conditions (eg accidental or seismic situations) but not for persistent or variable conditions.

Where the long-term serviceability limit state is to be assessed, it is good practice to undertake a sensitivity analysis to explore the consequences of the credible range in the data. For example, when assessing the long-term settlement of a levee it may be appropriate to consider not only the settlement due to the characteristic value of compressibility but also to assess the most likely magnitude of settlement using the mean value of the parameters and possibly the upper limiting value of the parameters to get a feel for the likely range of settlement that could occur.

Probabilistic forms of analysis explore the consequence of variability in one or more parameters. As such the mean and standard deviation of the dataset for the parameter is usually required. However, due consideration of the points made here needs to be taken as simple statistical techniques may not be appropriate and a pragmatic 'by inspection' approach to defining these values may be more applicable.

Quantifying characteristic values

Various degrees of conservatism have been described that qualitatively define the characteristic value. The use of quantitative statistical methods of assessing a characteristic value should be treated with caution as the application of the statistics could detract from the use of good engineering judgement based on all the available information, not solely the test results.

Bond and Harris (2008) present an extensive discussion on the evaluation of the characteristic value in the context of EC7, but the general principles may apply in other cases. However, specific reference should be made to the requirements of other country codes and standards.

When selecting the characteristic values for a ground parameter, consideration needs to be given as to how much ground is involved in the occurrence of the limit state. When failure of the 'system', for example the whole foundation, requires a failure surface that extends over a large area/volume of soil, it is the average properties of the soils affected by the limit state that govern its occurrence. The characteristic value is a cautious estimate of the spatially averaged value of the parameter relevant to the area/volume of soil associated with the occurrence of the limit state. If the system failure occurs as a result of a local failure then the spatially average value is not the relevant parameter. In the context of EC7, it requires the characteristic value to represent a 95 per cent confidence assessment of the mean value. In the context of the lower characteristic value, this is a value (or spatial value where the parameter varies with, for example depth) that has a 95 per cent probability of being exceeded by the spatial average. Statistically, the characteristic value will be further from the spatial mean where there are fewer data available and/or only a small volume of soil is affected by the ultimate limit state.

Caution

BS EN 1997-1:2004 defines a characteristic material property in terms of the five and 95 per cent fractiles, ie where low values are unfavourable the characteristic value should be defined by the five per cent fractile, and where high values are unfavourable they should be defined by the 95 per cent fractile.

This definition only really applies to structural members such as manufactured materials whose properties can be expected to lie within narrow bands, and not whole systems. Soils are very variable and a statistical approach may not be applicable as datasets are often small, not normally distributed about the mean, and contain significant scatter. However, there may be circumstances where comparable experience in some countries shows this approach to be applicable to specific techniques.

There are a number of other statistical approaches that may serve as a starting point to aid in the selection of a characteristic value. This may include statistical tools such as:

- mean
- standard deviation
- 95 per cent confidence value – assuming a normal distribution
- log-normal distribution and geometric mean.

The simple mean (average) describes the central tendency of the dataset, and the standard deviation reflects the variability of the dataset. A cautious estimate might use the mean plus or minus some multiple (or fraction) of the standard deviation.

Confidence intervals describe the upper and lower range of the data, assuming the data is normally distributed about the mean. The greater the confidence level, the further the value is from the mean. To calculate a characteristic value with a 95 per cent confidence level (ie 95 per cent of the data fall above or below the value.), the following can be used, but this assumes a normal distribution and sample size of 30 or more.

$$F_{95\%(\mathrm{Lower})} = x_{\mathrm{mean}} - 1.7 \text{ standard deviation} \qquad (\text{For } n \geq 30) \qquad (7.112)$$

$$F_{95\%(\mathrm{Upper})} = x_{\mathrm{mean}} + 1.7 \text{ standard deviation} \qquad (\text{For } n \geq 30) \qquad (7.113)$$

The 1.7 multiplier varies depending on the size of the dataset. The smaller the dataset, the larger the multiplier becomes. Student's t-values should be used for small datasets (n <30 data points). Comparing the mean to the 95 per cent confidence level also helps illustrate the amount of variability in the data. The greater the difference between the mean and the confidence value, the more variable the data.

The simple mean and standard deviation calculation implies a normal distribution of data, the classic bell shaped distribution, but many types of data in nature do not have a normal distribution, and are asymmetrical (skewed). When plotted logarithmically some datasets will reflect a bell-shaped curve and are said to have a log-normal distribution. For these datasets, log-normal statistics could be adopted. There are statistical tests to determine whether a dataset has a normal or log-normal distribution. For a log-normal distribution, the geometric mean is used in place of the mean, and the standard deviation is calculated and applied differently.

The geometric mean can sometimes provide a better sense of the central tendency when dealing with skewed data sets, and can also reduce the influence of outliers. The geometric mean is calculated as:

$$x_{\mathrm{geomean}} = (x_1 \times x_2 \times x_3 \times \ldots x_i)^{1/i} \qquad (7.114)$$

While statistics may serve as a useful tool to help the designer analyse data and inform the determination of characteristic values, they should not become the master. Statistical results should never be indiscriminately applied, and should never replace professional judgement based on a good understanding of all relevant data, not just numerical data.

7.9 SITE INVESTIGATION METHODS

The act of acquiring data on a site is a continuous process, whether it is subjective, through formal or informal visual observation, or quantitative, using physical techniques. One of the challenges faced by the designer is to identify methods of acquiring data at a level of confidence and detail that is appropriate to the stage of development of the project.

The forms of data required to characterise a site include

- topographic information and aerial photographs to contextualise the levee in its environment
- information defining the physical processes that affect and drive the hydraulic loads
- investigations to defined the stratigraphy and properties of the soils that form the levee and on which it is founded.

It is the aim of this section to present a brief outline of the available methods of acquiring data to characterise a site together with their applications and limitations. The level of detail presented for each method is sufficient to make the designer aware of the processes involved in the method and to assess its appropriateness to conditions at a given site. While there may be many methods by which data can be acquired, only those that are appropriate to the characterisation of levee sites are included. Due to the constraints of space the information is presented concisely as short text or in tabular form and maybe supplemented by appropriate references for further reading.

A flow chart mapping the outline structure and contents of Section 7.9 is presented in Figure 7.72.

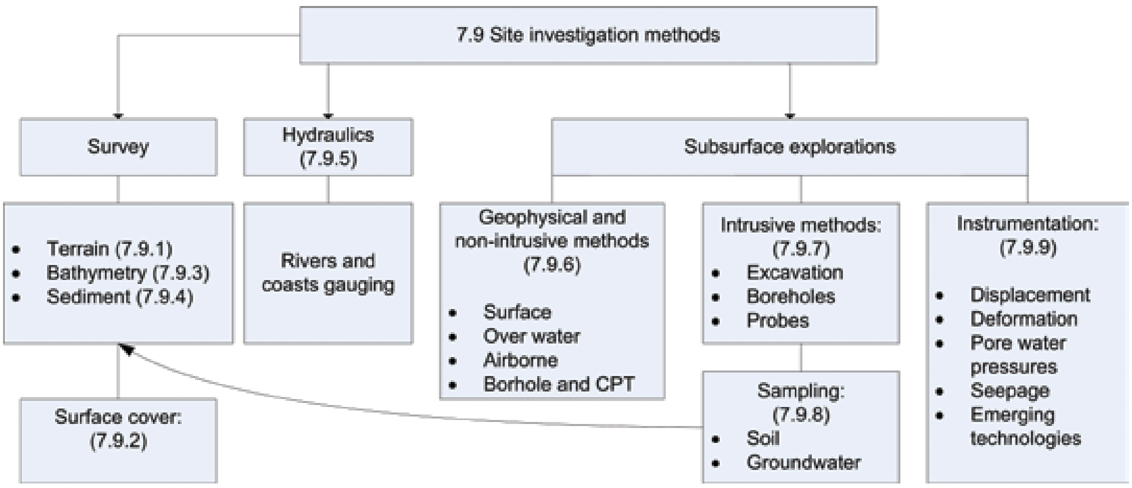


Figure 7.72 Structure and content of Section 7.9, and interaction with other sections

7.9.1 Terrain survey methods

This section covers dimensional survey techniques to support the engineering assessment of levees and to map the environs in which they are sited, or in the case of new levees, will be located.

7.9.1.1 Defining the scope of works

Before commissioning a survey its objectives and the deliverables required should be defined. Typically, these may be:

- levee crest levels to ensure they achieve the required elevation and to identify any low spots
- size and shape of levee to support a geotechnical assessment and to provide a basis for developing improvement works

1 2 3 4 5 6 7 8 9 10

- detect vertical movement to monitor rates of uplift or settlement
- define the limits, gross movement and key features associated with problem areas to contribute to the understanding of the cause and allow remedial works to be designed
- define the topography of the environs adjacent to the levee to contribute to an understanding of ground conditions, hydraulic modelling and, in the case of new levees, understanding the height and form that the levee will take.

Any long-term requirements of the survey should be defined. This could include future flood modelling, design, construction, monitoring, operation and maintenance.

Survey work should be undertaken by a qualified surveyor who will be able to advise on the most appropriate techniques to obtain the required information.

7.9.1.2 Survey control and datum

All survey techniques require survey control, These are fixed reference points with a defined location on the ground and an elevation defined against either a national, local or project specific datum. The density of the survey control station network will depend upon the requirements of the survey and the techniques used. They should be permanently stable, easily visible monuments located in or around the survey area. At least three survey control stations should be used so that the movement of any one station will be evident. Without permanent control stations it may be difficult to validate survey data or determine the reliability of data obtained at different times. Where displacement and deformation are to be monitored over time stable reference survey monuments may need to be established, located clear of the structure being monitored (Sections 7.9.9.3). This may require a deep datum to be installed, which is anchored into the stable soils at depth. The reference survey monuments may also serve as survey control stations.

There is the possibility that different survey control reference systems and datum may have been used when considering data from a number of historic sources. To avoid this it is good practice to:

- ensure that for new projects the same survey control system is used throughout the life cycle of the levee
- show survey control details on all drawings and include in reports together with the co-ordinates and elevations.

The location at which the land elevation datum is determined could have changed historically or subsequent surveys may have redefined the datum, resulting in a shift in the datum elevation. Where the level of the land varies by a few metres above and below the datum some regional projects may adopt a datum 100 m below the national datum to avoid reporting small negative and positive elevations, and the errors that this could introduce. Greater variability in the datum can occur between land and marine surveys. Elevations on maritime surveys are generally reported to chart datum. The difference between chart datum and the land datum can be several metres and varies from point to point around the coastline of a given landmass, which itself may have one datum.

7.9.1.3 Survey product deliverables

The two common approaches to survey are direct-observation and remote measurement.

With direct-observation survey techniques, which include levelling and global navigation satellite system (GNSS), discrete points are surveyed and used to produce drawings, such as topographical plans, long sections and cross-sections. For deformation monitoring, the engineer may require the surveyor to present changes in elevation on a plan, which may include presenting contours of the changes, or through a 3D representation.

Remote measurement includes LiDAR and aerial photogrammetry. LiDAR data provides a point cloud that can be used to create a digital ground model from which sections on any alignment can be produced in addition to 3D views and a 'fly through' of the digital ground model. By comparing ground models at different times, isopach plans can be produced to show areas of uplift and settlement, and can be overlain on an orthophotographic background.

It is considered good practice for surveys to be accompanied by a survey report, which includes at least details of the control measure used, observation data, calculations and achieved accuracies.

7.9.1.4 Terrain surveying methods

All surveying techniques have their advantages and disadvantages that depend upon many factors, including site conditions. Tables 7.115 to Table 7.119 present a summary of the common terrain survey techniques, including:

- levelling
- global navigation satellite system (GNSS) and 'total station'
- static laser scanning
- mobile laser scanning from land or air vehicle
- photogrammetry.

Small unmanned aerial vehicles (UAV) increasingly offer a cost effective platform for undertaking high resolution remote surveys. These can provide aerial photographs, as well as 2D and 3D mapping for small localised sites or scheme wide areas.

Table 7.115 Levelling

<table>
<tr><td colspan="4">Equipment required: optical or digital level</td></tr>
<tr><td colspan="4">Methodology: vertical height differences are measured using an optical levelling instrument and level staff or rod. Elevations are calculated with respect to the given or assumed datum</td></tr>
<tr><td colspan="4">Applications to levees:
• survey control and monitoring work
• measuring the elevation at a defined location, eg crest of a levee, water levels, pipe inverts and spillway levels.</td></tr>
<tr><th>Advantages</th><th>Limitations</th><th>Cost factor</th><th>Achievable accuracy</th></tr>
<tr><td>• simple technology
• good for discrete survey tasks
• relative accuracy can be very high.</td><td>• positional information not established
• requires surveyor(s) on ground.</td><td>• low cost</td><td>• using low precision equipment: +/-2 mm root mean squared error (RMSE)
• using geodetic equipment: +/-0.1 mm (RMSE) over 500 m.</td></tr>
</table>

*Table 7.116 Global navigation satellite system (GNSS) and Total Station (TS)**

<table>
<tr><td colspan="4">Equipment required: global navigation satellite system – GNSS receiver(s) and/or Total Station (TS)</td></tr>
<tr><td colspan="4">Methodology: for GNSS position established through accurate measurement of travel time of encoded signal from an array of satellites. For TS, position is established using electronic theodolite (transit) integrated with an electronic distance meter (EDM) to read slope distances from the instrument to a particular point.</td></tr>
<tr><td colspan="4">Applications to levees:
• survey control and monitoring.
• levee long section and cross-sections, topographical features and discrete small area surveys.</td></tr>
<tr><th>Advantages</th><th>Limitations</th><th>Cost factor</th><th>Achievable accuracy</th></tr>
<tr><td>• quick to collect and process data
• provides position in plan and elevation.</td><td>• GNSS: unreliable unless sky view is unobstructed and there are no reflective structures nearby (manmade or natural, such as buildings or cliffs)
• TS: direct line of sight required
• requires surveyor on ground.</td><td>• low: standard accuracy
• medium: high accuracy.</td><td>• plan and level position to between 3 mm and 35 mm (RMSE). Depends upon techniques and equipment being used.</td></tr>
</table>

Note

* a theodolite with integrated distance measurement system and on-board processing

Table 7.117 Static laser scanning

<table>
<tr><td colspan="4">Equipment required: tripod mounted laser scanner</td></tr>
<tr><td colspan="4">Methodology: the laser scanner rapidly scans millions of points, which are visible from the scanner position to produce a ‘point cloud’ of 3D points. A survey will typically involve a number of scan scenes that are connected together using common survey control and features to produce a single point cloud</td></tr>
<tr><td colspan="4">Applications to levees:
● remote measurement of simple or complex structures, specifically where the working environment may be hazardous, such as near water
● good for surveying discrete areas but not normally suitable for mapping large areas.</td></tr>
<tr><th>Advantages</th><th>Limitations</th><th>Cost factor</th><th>Achievable accuracy</th></tr>
<tr><td>● health and safety: remote non-contact measurement
● mapping of complex structures.</td><td>● detects vegetation and animals that are at the scene. (data can be filtered to remove extraneous features).</td><td>● low and cost effective for small complex areas
● medium to high for larger areas and other techniques should be considered.</td><td>● position (x,y) and level (z) between 3 mm and 35 mm (RMSE). Depends upon techniques and equipment being used.</td></tr>
</table>

Table 7.118 Mobile laser scanning from land or air vehicle

<table>
<tr><td colspan="4">Equipment required: vehicle mounted laser scanner (land or air based) with GPS or GNSS, and inertial measurement unit (IMU)</td></tr>
<tr><td colspan="4">Methodology: survey uses laser scanner (LiDAR) to produce a data point cloud. The survey control comes from the smoothed best estimate trajectory (SBET). It is commonly mounted on the following platforms:
● helicopter
● fixed wing but accuracy and density of the point cloud data is less than can be achieved using a helicopter. Fixed wing platforms can cover many kilometres of levee very quickly
● ground vehicles but this is only practical if it is possible to drive along the levee. Scan will only survey what can be seen.
If detail survey observations are required, permanent survey control stations may be needed to position the GNSS base stations to the survey platform. The mobile platform may include a precise GNSS receiver and an IMU.</td></tr>
<tr><td colspan="4">Applications:
● good for mapping large areas and long linear features
● can be useful in areas of foliage and deep canyons.</td></tr>
<tr><th>Advantages</th><th>Limitations</th><th>Cost factor</th><th>Achievable accuracy</th></tr>
<tr><td>● rapid data capture and short processing time
● good for mapping large areas
● useful in areas of foliage and deep canyons as penetration on return travel possible
● less seasonal and weather dependant than photogrammetric methods.</td><td>● all objects in field of view will be scanned (data can be filtered to remove extraneous features)
● interpretation can be difficult without field verification
● suitable for large areas only.</td><td>● low – for appropriately sized sites
● ratio of survey area to mobilisation cost is important to minimise survey cost per square metre.</td><td>● position (x,y) and level (z) between 30 mm and 150 mm (RMSE). Depends upon techniques and equipment being used
● high accuracy is possible. Around 30 mm (RMSE) error in 3D but depends on site conditions (eg obstructions to GNSS signals, multipath effects and the degree of integration between the GNSS and the IMU). The inclusion of fixed ground controls will allow the survey to achieve high accuracy.</td></tr>
</table>

Table 7.119 Photogrammetry

Equipment required: vehicle (land or air), calibrated camera, GPS and IMU.			
Methodology: high resolution overlapping vertical aerial photography allows features to be viewed and measured to form a 3D image, either manually or automatically by the production of a point cloud. Other forms include near infrared photogrammetry and multi/hyperspectral, which captures image data at a range of wavelengths.			
Application to levees: • quick mapping of large areas • near infrared photogrammetry can highlight areas of water • near infrared photogrammetry and multi/hyperspectral can highlight differences in vegetation type, which could be correlated with seepage issues.			
Advantages	**Limitations**	**Cost Factor**	**Achievable accuracy**
• readily interpreted and provides a record of site conditions • wider coverage per pass than LiDAR • features not detectable by LiDAR may be surveyed.	• shadows make analysis difficult • weather can restrict clear view of the target • cannot penetrate vegetation • more post-processing than LiDAR.	• generally low but it can be relatively expensive to fly long sinuous features.	• plan and level position to between 35 mm and 150 mm. (RMSE) dependent upon techniques and equipment being used • high quality colour images can provide a resolution of typically 50mm/pixel.

Further reading

There are a number of guidance notes and other useful reading materials, which can be found at:

The Survey Association website: **www.tsa-uk.org.uk**

Royal Institution of Chartered Surveyors: **www.rics.org**

Chartered Institution of Civil Engineering Surveyors: **www.cices.org**

UK Ordnance Survey: **www.ordnancesurvey.co.uk**

Environment Agency: **www.environment-agency.gov.uk**

National Standard Contract and Specifications for Surveying Services

Flood Risk Management Consortium: **www.floodrisk.org.uk**

Long *et al* (2011) *Performance based inspection of flood defence infrastructure: integrating visual inspection and quantitative survey measurements*

7.9.2 Surface cover survey methods

Surface cover on a levee and the waterside ground surface, such as the floodplain or foreshore, will affect the hydraulics of the system (Manning's n, Section 7.3.6). It will also affect the way in which the levee responds to the hydraulic load in terms of external erosion from waves, currents on the waterside slope and the crest and landward slope from overtopping. There may also be cases where vegetation grows on islands within the channel when the highest elevation is above the normal water level for a sufficient period during the growing season. The nature of the surface covering should be surveyed so that its effects can be taken into account when assessing system hydraulics and the performance of the levee. Surface cover can be beneficial or detrimental to the hydraulics and levee performance as detailed in Table 7.120. In the case of new levees there may be no cover material until natural or seeded vegetation has established, or temporary erosion resistance may be provided by biodegradable matting until the grass cover has developed. Examples of forms of cover are given in Table 7.121.

Table 7.120 Forms of surface cover and effects on levee

Form of surface cover	Effects on levee
Matting	• provides scour protection against currents, waves overflow and overtopping (under mild conditions a 'soft' surface cover, such as grass, or in the short-term biodegradable geofabrics, may provide adequate erosion resistance. Under aggressive conditions 'hard ' surface cover may be required, such as placed stone, rip-rap, asphalt etc, which may also reduce wave run-up) • encourages accumulation of sediments (living vegetation, such as saltmarsh, may or may not grow with the rate of accumulation).
Linear	• is often used to provide stability at the toe of the levee but can be detrimental higher up the embankment • captures flow debris (restricting overall quantity of flow) • restricts natural movement of *in situ* deposits (encourages accumulation and restricts natural replenishment of *in situ* deposits down flow).
Discrete	• encourages localised scour (accelerated velocities around obstacles) • locally captures flow debris (resulting in a larger flow obstruction).

Table 7.121 Example forms of cover

Cover	Matting	Linear	Discrete
Natural and bioengineering	• grass • saltmarsh • reed beds • biodegradable geofabrics.	• hedging • willow spilling.	• trees and shrubs.
Engineering	• non-biodegradable geofabrics • open-cell mats • rock and concrete armouring • gabion mattresses.	• boundary fences and walls • toe protection (rock/piling) • groynes • gabions • gravity walls • sheet piling.	• structures • power and communication towers/poles • isolated rocks.

Surface cover should be recorded and described based on visual observation with supporting photographs, and investigations and testing, where appropriate. The information to be recorded should include:

- description
- density of cover and condition (matting)
- dimensions, location, alignment and any localised detrimental effects (strip and discrete).

Specific information to be recorded for some common forms of surface cover on levees is presented in Table 7.122.

Table 7.122 Information to be recorded for some common forms of surface cover on levees

Form of surface cover	Information recorded
Grass	• species of plants • size of bare patch • percentage cover • plant/seedling density • root density and depth.

Table 7.122 Information to be recorded for some common forms of surface cover on levees (contd)

Placed stones	• material type (limestone, granite) and strength • dimension (LxBxH) • shape and degree of interlocking • nature of filler material between stones • nature of filter material under the stones • evidence of deterioration of stone and infill.
Asphalt	• layer thickness – ground penetrating radar • stiffness modulus – falling weight deflection meter • fatigue properties – relation between failure stress and number of load cycles) • evidence of deterioration of asphalt.

7.9.3 Bathymetric survey techniques

Bathymetry is the topography of the bed of a water body. Along with morphology, which is the change in the topography of the bed with time, it is an important factor in the design of waterside levees because it influences the hydraulic loading that acts on the levee and its stability and geometry.

When commissioning a bathymetric survey consideration needs to be given to the objectives and deliverables of the survey, along with methods of positional control. Section 7.9.1.2 includes some discussion of these issues in relation to terrain surveying. The broad principles outlined in that section apply equally to a bathymetric survey. The bed of the water body may periodically be above water level or covered by shallow water in the intertidal zone or during low flow conditions. Under these conditions the bathymetry may be surveyed using techniques such as those summarised in Section 7.9.1.4. This section primarily considers over water methods.

The scope and form of a bathymetric survey needs to take account of the scale of the project and the known dynamics of the bed and water body. The indicative forms that a bathymetric survey and factors to consider are summarised in Table 7.123.

Table 7.123 Indicative scope for bathymetric surveys and factors to consider

Size of project	Rivers	Coastal	Estuarine
Small	• ground survey crews wade in reaches of river, or operate from small boat, and collect depth data by rod and total station measurements or GPS • boats equipped with sub-bottom profilers and single channel seismic systems, in combination with multi-beam sonar and side-scan sonar • ground penetrating radar (Table 7.133) can produce profiles of strata and materials in river bed (non-saline waters) • area under observation limited to a single river reach at a time.	• shallow, stable waters can allow ground survey crews to perform elevation measurements similar to that for rivers • boats equipped with single-beam sonar • area of observation influenced by amount of time it takes to collect point data.	• shallow, stable waters can allow ground survey crews to perform elevation measurements similar to that for river. • boats equipped with single-beam sonar • area of observation influenced by amount of time it takes to collect point data.
	Data processing • data processing is usually completed after data collection, occasionally in real time • bathymetry map is an interpolation of the acquired elevation point data.		

Table 7.123 Indicative scope for bathymetric surveys and factors to consider (contd)

Medium	• shallow sub-bottom survey can be conducted with aid of portable sub-bottom profiler • boats equipped with sub-bottom profilers and single channel seismic systems, in combination with multi-beam sonar and side-scan sonar • ground penetrating radar can produce profiles of strata and materials in river bed (non-saline waters) • area of observation can be multiple river reaches • bed sub-layer geometry can be determined.	• produce profiles of the sea floor and its sub layers: boats equipped with sub-bottom profilers, single channel seismic systems, in combination with multi-beam sonar and side-scan sonar • area of observation is dependent on water depth. For beach areas, less area can be covered by boats. So, land survey speed is mostly dependent on productivity of ground surveys • deeper waters provide a large survey swath for boat mounted equipment to observe • bed sub-layer geometry can be determined • survey window often restricted by wave conditions.	• boats equipped with sub-bottom profilers, single channel seismic systems • area of observation can depend on geology of bed form, water salinity, and water depth • bed sub-layer geometry can be determined.
	Data processing • data processing can be done in real time • bathymetry map is an interpolation of profiles with accuracy dependent on density and number of passes conducted.		
Large	• combination of ground surveying techniques and aeroplanes equipped with LiDAR • for deeper rivers with homogenous bottoms, aeroplane surveys can be conducted with spectral sensing technology in conjunction with ground surveying • the boat surveying techniques described in the medium scope can be undertaken in conjunction with ground surveys or with aerial techniques or both • several reaches with various depths can be surveyed simultaneously.	• aeroplanes equipped with LiDAR, photogrammetric equipment, or spectral sensors to take bathymetric measurements • autonomous underwater vessel launched vehicles that can increase the range of light weight, advanced versions of sub-bottom profilers, single channel seismic systems, and multi-beam sonar equipment • waters further from coastline and at greater depths can be surveyed but at higher costs.	• aeroplanes equipped with LiDAR can produce fast bathymetric surveys which can aid in determining behaviour in dynamic estuaries • airborne surveys using LiDAR are limited to estuaries with low turbulence. Therefore, the use of airborne LiDAR surveys in combination with boat and ground techniques may be used • turbulent waters can present a danger to ground surveyors or submerged equipment • larger scopes of study can be costly.
	Data processing • data processing done in real time • 3D geometry can be determined by integration methods programmed in the surveying equipment and with post-processing • geology and composition of bed material and sub-layers can be determined.		

7.9.3.1 Bathymetric survey methods

The bed of a water body may periodically be above water level or covered by shallow water in the intertidal zone or during low flow conditions. A consideration in the selection of bathymetric survey methods, such as those detailed in Section 7.9.1.4, is important in determining whether land-based techniques can be used or whether water-based methods are required. Land-based topographic surveys methods are typically more accurate and less costly to obtain.

Table 7.124 presents details of some common methods of obtaining bathymetric data. Section 7.9.6.3 contains additional information on some of these techniques.

Table 7.124 Bathymetric survey methods over water

Method	Principles	Applications	Limitation
Ground survey (see also Section 7.9.1.4)	• survey crew equipped with GPS or total station equipment in shallow depths of waters to produce elevation maps • portable sub-bottom profilers exist that can facilitate bathymetry survey and assessment of sub-layer compositions • hand-held magnetometers can facilitate location of bed features.	• total station or kinematic GPS survey equipment can provide bathymetry data in shallow and slow water • small surveys may be used as preliminary study to more advanced/larger scale bathymetry studies.	• safety and limitations with increasing water depth and velocity. Impractical for long reaches • limited to river, estuarine environments, and coastal environments accessible by foot • the nature of the ground may not be suitable for passage by foot.
Single beam sonar (AKA echosounder/sub bottom profiler)	• active sonar transducers emit an acoustic signal or pulse of sound into the water • object in the path of the sound pulse return an 'echo' to the sonar transducer.	• sound waves need to originate in water • sonar units are mounted to submerged parts of the vessel • provides area coverage of a point directly below the vessel's path • method can be used in river and coastal studies • identification of layers beneath the apparent bed. Useful in assessing fluidity of bed and amount of material available to be moved during an event.	• several profiles of vessel pathways should be combined to provide a bathymetry plot, introducing interpolation errors • no indication of bed geology • sound wave speed is dependent on temperature, salinity and pressure • data collection can be a lengthy process.
Multi beam sonar (AKA multi beam echosounder)	• an array of single beam sonar pulses are emitted in a triangular swath that covers an area below the vessel's path • commercially available since the 1970s • additional facilities can include real time computation and data storage.	• provides dense coverage dependent on water depth and sampling frequency • angle of incidence of reflected waves provides information about seafloor geoacoustics, which is directly related to sediment grain size and compactness • useful in gathering information on environmental habitats.	• echo noise caused by insignificant objects lead to 'holes' in bed mapping • no description of bed geology at surface • size and vulnerability of the costly transducer makes it best suited to large rivers and open water • minimum water depth requirement about 2 m.

Table 7.124 Bathymetric survey methods over water (contd)

Side-scan sonar	• towed below water surface behind a vessel • specially shaped acoustic beam pulse at 90 degrees from towed path • each pulse provides an image of a strip typically 100 m wide directly below the instrument.	• seabed relief from a metre to a kilometre can be recorded continuously along the vessel's path • various energies of return signals provide information on roughness and hardness of bed surface material.	• most side-scan sonar system cannot provide depth information • for full bottom coverage, it needs to be used in conjunction with other forms of bathymetric soundings and sub-bottom profiler data • additional analysis, or ground truthing, can be done to determine the real nature of bed.
Spectral sensing/ analysis	• water depth is calculated based on radiation of optical waves reflected back to an airborne sensor from the water surface • requires observation of light exiting the entire water column at the air-water interface.	• measures density of one pixel • depth accuracy is highly variable but can be a minimum of 0.20 m • clear water with homogeneous bed and no overhanging vegetation • aerial or satellite imagery provided a variable area coverage for a given time.	• measuring depth is highly dependent on turbidity of water • not suitable for very shallow waters or where there are highly reflective bed materials • data about the water body should be gathered to calibrate the system first.
Photogrammetry	• camera mounted on aeroplane to point vertically down • multiple overlapping photos of ground taken along flight paths • the photos are used to create a digital elevation model (DEM), map, drawing or a 3D model of geometric bed features.	• measures density of one pixel to a depth accuracy of 0.20 m • clear water with homogeneous bed and no overhanging vegetation • aerial or satellite imagery can cover variable area sizes for a given time.	• poor in shallow water as reflected rays are within the infrared spectrum • use site specific relationships between depth and water colour, requiring other forms of survey to calibrate • errors due to changes in bed material, overhanging vegetation, surface waves and shadows.
LiDAR	• scanning laser pulses set towards bed layer at a steady rate • pulses consist of infrared wave length and a green wave length. Infrared wave is reflected by the air-water interface, green wave penetrates and reflects at the bed surface • distances each wave length pulse travels can be determined and hence the water depth.	• measures between 2 m × 2 m to 5 m × 5 m at a time, depth accuracy of 0.18 m to 0.35 m, horizontal accuracy of 1 m to 2.5 m • one hour to complete 70 km^2 • water depths from 0.5 m to 60 m depending on turbidity • on flat water surface the minimal measurable water depth is 0.41 m • suitable for use in rivers and ocean where there is no turbidity.	• not fully developed for shallow water research • not suitable for areas with overhanging vegetation • no description of bed geology at surface • ineffective in region of breaking waves.

7.9.4 Sediment survey methods

The characterisation of bed sediments (and transported sediments) below a water body is an essential element to understanding the morphological processes that could affect the performance of a levee over time. The nature of the bed deposit will determine whether it is resistant to erosion or how it will be transported.

- sediments containing clay exhibit cohesion and increased resistance to erosion

- finer sediments (such as fine sands, silts and clays) are transported in suspension (suspended load
- coarser sediments (such as gravels and coarse sands) are transported by rolling or creeping along the bed (bed load)
- the layering of bottom sediments can provide data on the depositional history of the bed sediments and an insight to armouring and depth-variation in erodibility.

Samples should be taken in order to determine the grain size distribution of the sediments (Section 7.8.3.1). They should be taken at a number of locations over a range of depths and times as the movement of sediment can change significantly (Table 7.125). Sampling should be extended outside of the immediate environs of the project to adequately understand the morphological processes of the immediate site and the environs.

Table 7.125 Sampling of sediments

Material sampled	Rivers	Coastal	Estuarine
Transported sediments (suspended or bed load)	• one or more locations during conditions of low and high discharge • samples required over time (months to years) to establish water-sediment discharge relationships	• difficult to measure as it is a very dynamic environment. Extensive sampling programme required • sampling required to develope profiles along the shoreline.	• difficult to measure as it is a very dynamic environment. Extensive sampling programme required.
Bed sediments (surface and at depth to establish sediment stratigraphy)	• every 1 to 10 km depending on the size of the river and variability of bed materials • at least three points on each cross-section and at defined points. 'Dead water' areas should be avoided.	• sampling is typically done on sections perpendicular to the shoreline at a spacing of about 1 km along the coastline • along the profile, samples may be taken at major changes in morphology (berm, swash zone, trough, bar crest) and then at each 3 m change in water depth.	• 1 to 5 km grid adapted to suit width of the estuary.

The size of the sample in terms of sediment concentration or the mass of sediment required for analysis is a function of the particle size and test method. Table 7.126 details some recommended sample sizes.

Table 7.126 Recommended sample size for determination of grain size distribution

Test method	Grain size range, (mm)	Sediment concentration, (mg/l)	Mass of sediment, (gm)
Sieve[1]	0.062–62	–	0.07–64 000[3]
Visual accumulation tube[2]	0.062–2.0	–	0.05–15.0
Pipet[2]	0.002–0.062	2000–5000	1.0–5.0
Bottom Withdrawal tube[2]	0.002–0.062	1000–3000	0.5–1.8
Hydrometer[2]	0.002–0.062	40 000	30.0–50.0

Notes

1 Measures physical diameter of grains only.
2 Measures sedimentation diameter, which includes grain shape and specific gravity effects.
3 See also Table 7.159 Sample sizes.

Two frequently used methods for separating sediments from the water in the sample are evaporation and filtration:

1. **Evaporation** is usually best for high concentrations of sediment (>2000 mg) but requires a correction if the dissolved solids concentration is high

2 **Filtration** is faster if the sample size is small and/or comprises relatively coarse grained particles (>62μm). It can be used for fine grained sediments but is slower.

7.9.4.1 Methods of sampling sediments transported in suspension

A number of methods have been developed to sample the fine sediments transported in suspension. The suitability of a sampling method varies depending upon the environment within which it is to be used (river, coastal or estuarine) and the size of the sediments. Most samplers used in low velocity environments, such as coastal or estuarine locations, are point or trap samplers that are oriented vertically and do not sample isokinetically. Under these conditions good practice would be to sample continuously through a tidal cycle at a number of locations to define temporal variations at each location. Some of the principal sampling methods are presented in Table 7.127.

Table 7.127 Methods of sampling suspended sediments

Method	Principles	Applications	Limitation
Depth–integrating samplers (general)	• sample of water sediment suspension taken as instrument is lowered to bed and raised to surface at a uniform rate.	• useful in unidirectional flows • depth averaged concentration, and depth integrated suspended sediment transport obtained • used in fluvial settings but can be used in estuaries.	• repeat sampling required at each vertical and horizontal section • particle size samples limited by size of nozzle • sediments not obtained at or close to bed due to position of intake nozzle.
USD-49 depth-integrating sampler	• the sampler is lowered at a uniform rate from the water surface to the bed, instantly reversed, and raised to the water surface • sampling nozzle (imperial equivalent of 6.4 mm, 4.8 mm or 3.2 mm dia) is pointed in direction of flow and collects sampling as it is lowered.	• suitable for unidirectional flow as intake nozzle samples in one direction • depth averaged concentration, and depth integrated suspended sediment transport obtained • used in fluvial but can be used in estuaries.	• collects a sample that is representative of only a small amount of time, when the sampler is lowered. This requires the collection of many samples to cover a representative timescale.
Point integrating samplers (general)	• a number of discrete water sediment samples taken over a range of depths or at the same depth over a measured interval of time.	• can be used for silt/ sand sediments in rivers depending on type of sampler used • provides data on sediment concentration, transport, and particle size • used in fluvial settings but can be used in estuaries.	• provides information only at a single location and depth • particle size samples limited by size of nozzle used • sediments not obtained at or close to bed due to position of intake nozzle.
Streamer traps	• traps are small towers fitted with mesh collection socks aligned with long-shore current • traps placed in a row perpendicular to the shoreline.	• traps designed for coastal surf zones • long-shore sediment quantity and type can be obtained.	• careful calibration of each trap should be conducted • not useful in determining seaward migration of sediments.
Delft bottle	• sampling bottle is moved through water at velocity equal to the local water velocity in order to obtain representative sample.	• suitable for steady flow conditions in rivers or streams • level of sample tube in water profile can be targeted • local average sediment transport is measured directly.	• davit or derrick is required to move equipment • large amount of material lost during hoisting and correction calculations needed • only sediments >100 µm captured.

Table 7.127 Methods of sampling suspended sediments (contd)

Pump filter samplers	• water sediment suspention is pumped through a mesh filter screen.	• effective in coastal and river environment requiring time-averaged concentrations • sampling in low silt environments to prevent filter blockage.	• intake velocity required is >0.8 of local current • sediment testing undertaken in laboratory • only sediments >50 μm captured.
Pump sampler	• water sediment suspension is pumped to determine sediment concentration at various elevations above the bed.	• point sediment transport assessed from pumped sample and current measurement at the same elevation • point transport rates integrated over depth to assess suspended sediment transport.	• pumping velocity should match the current velocity to prevent acceleration/ deceleration of flow into the sampler • sample collection and processing is labour intensive • pumping may destroy aggregates.
Optical backscatter sensor (OBS)	• high intensity infrared emitting diode illuminates a small volume of water • photodiodes measure light scattered back by sediment particles • measured voltage is proportional to total surface area of particles illuminated.	• point sediment transport assessed from OBS and current measurement at the same elevation • point transport rate integrated over depth to provide suspended sediment transport • suitable for sandy environments, including surf zones.	• results sensitive to grain size (particularly presence of fines) • requires calibration with local sediment sample.
Acoustic doppler current profiler (ADCP)	• sediment suspension is measured from the intensity of the backscattered sound • current velocity is measured from the doppler shift of sound waves reflected off particles in the water column.	• ADCP may be fixed or deployed off a vessel • many sensors are capable of profiling vertically to cover the water column • current and sediment concentration are combined to estimate suspended transport rate • bottom tracking capability is being explored to estimate bed load.	• the volume sampled increases with distance away from the sensor • the acoustic backscatter is sensitive to particle size and calibration is required with local sediments.
Ultrasound	• ultrasonic waves induced in water stream. Sand particles alter the frequency and amplitude of return signal.	• simultaneous measurement of velocity and concentration of sand particles in one or two directions • measurement taken at single point.	• method insensitive to silt particles • cannot be used where plunging waves generate bubbles.

7.9.4.2 Methods of sampling sediments transported along the bed

Sampling of materials transported along the bed during a flow event is problematic as the sampling system will disturb the hydraulic conditions that cause the movement of the sediment. It can also change dramatically with time and location during the same event. So, multiple samples are often required to fully understand how the sediment is being transported. For this reason the sampling regime is just as important as the sampling equipment, Table 7.128.

Table 7.128 Methods of sampling bed load sediments

Method	Principles	Applications	Limitation
Bottom (bed load) transport meter Arnhem	• wire mesh basket attached to a frame is pressed into the bed • dynamics of the frame cause coarse bed load to be collected at ambient flow rate.	• appropriate for particle sizes from 0.3 mm to 5.0 mm • bed load can be determined on site immediately after collection • use in steady flow where bed form migration is gradual.	• when bed form is longer than frame, randomised collection requires positional movement of frame • davit or derrick is required to move equipment.
Helley-Smith sampler	• works the same as bottom transport meter Arnhem but, nozzle on basket can be changed to correspond to the particle sizes to be sampled.	• several versions of sampler in terms of weight and size: heavier samplers are typically used for fast flowing and deeper rivers and streams.	• calibration required for the observed environment to achieve accurate results.
BL-84 bed-load sampler	• shape and design adapted from the Helley-Smith sampler but is a heavy version.	• operates in any depth of river where it can be properly placed in the bed and in water velocities up to 3 m/s • samples particle sizes between 1 mm and 38 mm.	• tether line required in velocities greater than 1.2 m/s
Delft Nile bed load and suspended load sampler	• as bottom transport meter Arnhem and Helley-Smith sampler but with lower errors from initial scooping and gap effect, where material passes under the sampler, achieved using a tension swing arm on the nozzle.	• as bottom transport meter Arnhem and Helley-Smith sampler • suitable for collection of particle sizes larger than 400 μm • simultaneous collection of suspended load through series of intake nozzles.	• calibration required for the observed environment to achieve accurate results • in order to avoid adverse effects on hydraulic coefficient, samples collected should only fill bag to 50 per cent.
Bed load tracking	• a 1D or 2D echosounder is used vertically along a defined horizontal path to determine the height of bed load at different times. This results in a record of bed form migration.	• bed load transport rate calculated from echosounder data and bed form dimensions • dual frequency sounders measure vegetation/soft mud over bedrock.	• no physical sample • flow conditions must be steady with no external disturbance between passes • fluid composition needed for increased accuracy • complex bed formations make migration behaviours difficult to assess.
Pit and trough samplers	• a rectangular hole is excavated along a cross-section of a stream bed • trough hole spans the entire cross-section of the bed • pit hole dimensions are specified during sampler calibration • sampling by emptying of pit.	• collection of wide range of particles sizes controlled by mesh size above pit • sampling period defined by time or capacity of pit • suitable for ephemeral rivers or those having low base flows between transport events.	• permanently installed, expensive to build and sample at one location • lateral entry of sediment into the pits possible • troughs are more difficult to construct and operate but are less challenging to calibrate than pits.

Table 7.128 Methods of sampling bed load sediments (contd)

Vortex tube bed load samplers	• specially constructed weir generates vortex that carries bed load to a trap on side of channel • sediment sampled and returned downstream of the weir.	• effective on small gravel bed streams but sampling of a variety of grain sizes is possible • easy collections of samples as sediments are delivered to the side of the stream • diverted bed load is continuously weighed and sampled, and unloaded downstream.	• permanently installed, expensive to build and sample at one location • for meaningful data, recording should be taken throughout the transport event • sample size only limited by how fast it can be loaded and unloaded from observation area.

7.9.4.3 Methods of sampling bed material deposits

For levee studies, bed samples are typically obtained in order to determine the potential for sediment transport bed adjustments (degrading or aggrading). Undisturbed samples are not required for this purpose.

In some cases, specifically in a river environment, the bed may have become naturally armoured by coarser material. The armour and subsurface layers can be sampled either using a surface count or by collecting a 'grab' sample (Table 7.164):

- a **surface count** for an armour layer is conducted by measuring the medial axis (medial diameter) of a large number of surface pieces (typically >100 lumps per sampled site), then using statistics to determine size characteristics of the sample, such as median size
- a **'grab' sample** of an armour layer aims to collect material from a depth no deeper than about the medial axis of the maximum size particle in the bed.

For a sub-surface layer, the armour layer needs to be removed before undertaking either a surface count or a 'grab' sampling.

Bed deposits may periodically be above water level in the intertidal zone or during low flow conditions. Sampling in the dry is preferred because there is less opportunity for the fine fraction to be lost from the sample during collection. Samples from dry beds are typically collected manually with a shovel or scoop. Other land-based methods of intrusive investigation may be used (Section 7.9.7.5). However, special consideration will need to be given to access and egress due to the constraints imposed by tidal working and ground conditions, which may have a low bearing capacity.

Conventional methods of intrusive investigation can be used over water in conjunction with a floating platform. Methods specifically designed for obtaining bed deposits over water include the vibrocore sampler and grab sampler (Tables 7.148 and 7.164).

Additional information can be found in Table 7.129.

Further reading

Van Rijn (2007) *The manual sediment transport measurement in rivers, estuaries and coastal seas*

Gaeuman and Jacobson (2007) *Field assessment of alternative bed-load transport estimators. Journal of hydraulic engineering*

7.9.5 Stream and coastal gauging methods

Fundamental parameters needed to evaluate river, estuarine, or coastal loads for levees include water levels, quantities of discharge and currents. Various methods can be employed to measure these parameters as outlined in Table 7.129. It is necessary to obtain measurements for these parameters over an extended period of time to adequately determine statistical estimates of extreme magnitude events. So, it is preferable to implement a monitoring programme, which could extend over many decades.

Table 7.129 Stream and water-level measurements

Method	Principles	Applications/advantages	Limitation
Gauge height (river stage)			
Staff gauge	• vertical scale affixed to permanent mount in river used to measure water level.	• simple to install • most suitable where infrequent readings are required and at sites that have easy access.	• requires manual reading • can be difficult to maintain location, especially in high debris load or dynamic channel.
Pressure cells/ transducers	• measure change in hydrostatic pressure as water rises/falls to determine water level.	• can be automated to record and/or transmit data in real time. Less expensive to install than floats.	• can be difficult to maintain in turbid waters • barometric pressure correction is required • sensor can drift from true readings as tubes clog or accumulate condensation.
Float wells	• an encoder converts movement of a calibrated tape that is attached to a float that follows water level.	• simple concept and equipment • relatively simple to maintain sensor • can be automated to record and/or transmit data in real time.	• expensive to install • can be difficult to install and maintain gauge house/ structure • can freeze in cold climates.
Radar	• triangulation of radio wave to determine distance from the sensor to the surface of the water.	• non-contact method, simple to install where infrastructure crosses streams • can transmit data in real time.	• limited range of gauge height measurement possible between sensor and water level • range affects measurement accuracy • presence of ice may give incorrect readings.
Discharge (rivers)			
Fixed section	• fixed geometry permits use of a unique rating or empirical equation to calculate discharge.	• consistent results • most suitable in bed rock stream or locations where a fixed weir can be constructed.	• very few locations are suitable • high installation cost.
Conventional meter	• current meter is used to estimate velocity at various points across cross-section at different times/water levels • velocity transects are integrated across cross-section area to calculate composite discharge.	• straightforward method that has been used for many years • requires physical measurement in the field.	• depending on river size may take a long time to take individual measurements • typically taken from bridges, boats or cableways so safety can be an issue • can be expensive to install and maintain if cableways or specific equipment is required at site.
Acoustic doppler current profiler (ADCP) (mobile)	• uses triangulation between multiple sensor heads to determine Doppler shift of particles moving in the water column • continuously measures velocity and depth as moved • discharge is calculated by integrating depths, distance travelled and velocity measurements across the cross-section.	• newer technology that has been in use since early 1990s • much quicker to use than a conventional meter • provides 2D or 3D velocity field • requires physical measurement in the field.	• expensive to acquire equipment • can yield mixed results where stream has high suspended sediment load and/or high bed load • good practice is to take several transects then average to obtain representative discharge • same safety issues as for conventional meter but exposure is shorter in duration.

Table 7.129 Stream and water-level measurements (contd)

Acoustic doppler current profiler (ADCP)/ Acoustic doppler velocimeter (ADV) (fixed)	• ADCP or ADV sensor is permanently mounted in the channel section to scan either horizontally or vertically • continuously or intermittently measures velocity and discharge • calculation of discharge requires correlating of readings with index values.	• provides real time discharge/velocity data • staff are exposed to safety risks less frequently • can be automated to record and/or transmit data in real time.	• must establish site index values and verify consistency over time by repetitive physical measurements • generally suitable only where cross-section does not change significantly over time • can be expensive to install and maintain.
Tide height (coastal zones and estuaries)			
Tide gauge	• hydrodynamic low-pass filter using measured sea pressure.	• reliable and inexpensive • any environment and tidal range.	• biofouling • attached to an existing or independent structure.
Wave height (coastal zones and estuaries)			
ADCP	• ADCP placed on bed of water body or mounted to rigid structure • uses change in frequency due to Doppler shift to measure 2D or 3D currents that are converted to a water surface profile.	• reasonably non-intrusive in wave field • can provide 3D directional waves • data stored internally or transmitted to surface by telemetry • no limits on water depth for this application.	• difficulty in taking readings where waves are breaking or where there are bubbles in the water column • relatively expensive and high power consumption.
Buoy	• moored buoy measures heave only or heave, pitch, and roll using accelerometers, inclinometers and compasses to estimate water surface profile.	• good accuracy, long service life • data stored internally or transmitted to shore or satellite by telemetry • usually in deeper water.	• mooring affects response at higher frequencies • current-induced tilt • bias errors at low frequencies • low directional resolution • easily lost or damaged • subject to vandalism.
Pressure, u- and v-velocity gauge (PUV)	• subsurface instrument for measuring pressure, and u- and v-velocity components in the horizontal plane (ie x- and y-axes) that are converted to a water surface profile.	• reliable, and measures 2D unidirectional or 3D directional waves, tides and waves, mean current • small size, relatively inexpensive, low power consumption • data stored internally or transmitted to surface by cable.	• depth-limited as high frequencies attenuated by depth of water column • low directional resolution • current-induced Doppler effects cause shifting of wave frequency • biofouling.
Wave staff	• changes in capacitance or resistance of sensing length of wire that is short-circuited by changing water surface.	• simple, low cost, and accurate • any location where suitable mounting is available.	• intrusive, surface piercing • requires rigid mounting • biofouling.
Currents (coastal zones and estuaries)			
ADCP	• ADCP placed on bed of water body, mounted to rigid structure, tethered, or moving boat • uses change in frequency due to Doppler shift from pulsed beam of sound to measure 2D or 3D currents.	• reasonably non-intrusive • data stored internally or transmitted to surface by telemetry.	• difficulty in taking readings where waves are breaking or where there are bubbles in the water column • relatively expensive and high power consumption.

Table 7.129 Stream and water-level measurements (contd)

Ducted impeller	• mean current measured by reversible impeller at defined elevation in water column • impeller is free to weathervane with compass for direction.	• good accuracy • data stored internally.	• equipment has moving parts prone to biofouling • only mean currents measured.
Electromagnetic	• orthogonal components of current measured using Faraday effect at defined elevation in water column.	• no moving parts • data stored internally or transmitted to surface by telemetry.	• equipment relatively expensive and prone to biofouling • requires calibration, which is expensive to achieve.
Pressure, u- and v-velocity gauge (PUV)	• subsurface instrument for measuring pressure, and u- and v-velocity components in the horizontal (ie x- and y-axes) plane.	• small size, relatively inexpensive, low power consumption • data stored internally or transmitted to surface by cable.	• biofouling.

7.9.6 Geophysical and non- intrusive ground investigation methods

Characterisation of levees and their foundations, using a phased, integrated investigation approach largely based on surface geophysical and intrusive techniques, can lead to more reliable and cost-effective levee safety evaluations, remediation/strengthening and design programmes.

Geophysical investigation methods can provide valuable information during the feasibility phase of new levees and for the condition assessment and improvement works on existing levees. It can be used to characterise the foundation soils and the internal structure and properties of levees.

The key benefits of geophysical methods is that they allow relatively rapid screening of a site, providing information that can be used to target intrusive investigations and help to infer conditions or geological structure between probe or borehole locations.

Geophysical techniques applicable to levee investigation include a broad spectrum of airborne, onshore surface and borehole techniques, and water-based techniques deployed from a surface vessel. Other generally non-intrusive techniques include LiDAR surveying and aerial photography (incorporating including visible light and infrared), which can use surface features to infer ground conditions.

It is important to understand the capabilities of the different geophysical methods, so that they may be used to full advantage for subsurface investigations. Geophysical methods can be broadly described as ‘reconnaissance’ or ‘targeted’. For initial levee assessments, reconnaissance methods with a high data acquisition rate (continuous fast sampling and ground coverage) are generally favoured. Targeted geophysical methods generally have a more limited spatial coverage and are used to acquire more detailed additional information.

Box 7.34 gives some examples of the use of geophysics in levee investigations.

Box 7.34 Geophysics in levee investigation

The use of geophysics in the evaluation of levees has been trialled around the world and is well documented. Some examples are:

- the FloodProbe project addressed technologies for improved safety of the built environment in relation to flood events and produced a key report in 2012 on rapid and cost effective levee condition assessment methods: geophysics and remote sensing. It focused on urban areas and the use of electrical and electromagnetic methods
- the SAGEEP (Symposium on the Application of Geophysics to Engineering and Environmental Problems) is an internationally recognised, leading conference on the practical application of shallow geophysics. Proceedings of the SAGEEP conference are published annually by the Environmental and Engineering Geophysical Society (EEGS) and over the past decade or so a number of publications have described geophysical investigations of new and existing levees in several regions
- the US Army Engineer Research and Development Centre (ERDC) and the US Section of the International Boundary and Water Commission (USIBWC) performed a condition assessment of almost 800 km in the Rio Grande Valley between 2001 and 2003. The aim was to evaluate the ability of geophysical methods to screen for signs of distress in the levee and to guide the targeting of subsequent geotechnical investigations
- the Public Works Research Institute at Tsukuba in Japan in conjunction with other Japanese corporations and the Society of Exploration Geophysicists of Japan (SEGJ) has assessed more than 20 levees in Japan using integrated geophysical surveying methods
- the Kansas Geological Survey in conjunction with USIBWC, the US Geological Survey, ERDC and the USACE have evaluated the use of seismic techniques for the assessment and investigation for major levee systems in the USA
- in Germany, the proper application of geophysical techniques was evaluated by BAM Federal Institute, and others, as part of project DEISTRUKT in the Mulde and Elbe river systems
- other research initiatives have also been undertaken. These include:
  - ERINOH (France) National research project on internal erosion
  - FLOODSite (European Community funded project)
  - IMPACT (European Community funded project) investigation of extreme flood processes and uncertainty.

Considerations when implementing a geophysical phase of investigation

In order to understand and manage the geotechnical and hydrogeological risks associated with levee construction and maintenance, the following guidelines are suggested when implementing a geophysical investigation:

- geophysics is a specialist field of investigation and where its use is considered appropriate advice of a specialist should be obtained
- where the project requires the ongoing integration and interpretation of geophysical data, it is preferable to have a geophysical specialist as part of the project team
- interpretation of geophysical data should, where possible, be carried out with consideration of other relevant data including geotechnical, geological and hydrogeological information sources
- the geophysical methods used need to be appropriate to the site conditions, and the limitations that the site may impose on the effectiveness of the method should be recognised
- levee geometry and constituent materials may influence the geophysical approach
- where possible, forward numerical modelling can be used as part of any assessment to determine the likelihood of a technique or techniques being effective in meeting the investigation objectives for specific site conditions (this is subject to the known limitations of over-simplification and reliance on the quality of model input parameters)
- trial geophysical investigations of limited scope may be used to demonstrate 'fitness for purpose' of the methods and to assess their effectiveness in determining soil physical properties and ground conditions
- borehole geophysical methods can significantly enhance the volume of information derived from an intrusive investigation and can be linked to data from surface, water based or airborne geophysical methods.

Considerations when designing or developing a scope for geophysical investigations

Traditional investigations for levees and foundation soils have relied upon visual inspection, pre-existing site records, and intrusive geotechnical drilling, sampling, and laboratory analyses. Modern geophysical techniques appropriately phased as part of an integrated investigation can offer the potential to provide continuous, broad coverage of a site relatively quickly. The level of geophysical and other non-intrusive techniques deployed should be appropriate to the phase of the investigation.

The greatest benefit from geophysical techniques can usually be obtained by undertaking them at an early stage as part of a phased investigation.

Geophysics can derive vertical and lateral information to build a 3D ground model, which can show where soil conditions may be at variance with those of the more general background conditions, both within the body of the existing levee and the foundation soils. Such variations may be indicative of features that could affect the serviceability of the levee.

Geophysical data can allow a more effective use of intrusive investigations, by supporting decisions to adopt a wider spacing between points of investigation where ground conditions appear to be relatively uniform. It can also provide justification for more closely spaced, targeted intrusive investigation where ground conditions are indicated to be complex and critical.

However, the general requirements of country codes and standards on the spacing of intrusive investigations may need to be satisfied (Section 7.9.7.2). The information the geophysics provides can improve the interpolation of stratigraphic boundaries, structures and *in situ* properties between points of intrusive investigation, and likewise data from the intrusive investigation can improve the interpretation of the geophysical data. For this reason close integration of the geophysical data with all other data is necessary. The combined approach can be more effective than visual monitoring and discrete intrusive investigation alone in determining levee internal structure or ground conditions. Often an integrated geophysical and targeted intrusive investigation can provide significant additional site characterisation information at no greater cost and in less time than a traditional intrusive investigation.

Geophysical and other non-intrusive methods of investigation can provide data that can be used to assess ground conditions:

- below and adjacent to the levee (profile of levee foundation soils)
- below the adjacent water body (depth of water and profile of bed deposits)
- within the levee (internal structure of the levee).

Geophysical techniques are based upon physical principles and technology with certain limitations, which need to be fully understood by all involved parties when planning an investigation. It is preferable to use a number of geophysical methods, where possible in combination to improve ground model fidelity. Modern geophysical techniques can also be used to derive *in situ* geotechnical parameters (Section 7.8.1.5).

Geophysical methods can generally be adapted to survey site geometry. Many geophysical methods are suited to investigating long, linear levee structures along the principal longitudinal axis and some can be used, dependent on linear dimension, to investigate the transverse structure and characteristics of levees.

In general a phased approach will involve initial longitudinal profiles along the levee to determine lateral variability. Transverse profiles may then be acquired should further detailed information be required from a specific segment of the levee.

Many site specific factors need to be considered when scoping a geophysical investigation and a prescriptive approach should be avoided. Some countries have specific recommendation for the scope of non-intrusive investigations and country codes, standards and guidance documents should be consulted. In the absence of such information, Table 7.130 generalises the conceptual form that a geophysical investigation could take for levee applications.

Table 7.130 Example of a generalised form for geophysical investigations for levees

Application	Indicative survey plan for the geophysical investigation of levees
Condition assessment of existing levees	Longitudinal profiles or grids usually based on electrical or electromagnetic techniques but possibly including seismic, gravity or magnetic techniques, extending from the water body to about 20 m landward of the landside toe to determine stratigraphy, structure and ground properties. Airborne methods may be considered where linear dimensions to be investigated are sufficiently large and information on foundation condition is required.
New small levee foundations	Longitudinal profiles or grids usually based on seismic, electrical or electromagnetic techniques but possibly including gravity or magnetic techniques extending from the water body to about 20 m landward of the landside toe to determine stratigraphy, structure and ground properties. Borehole geophysics as required to reconcile intrusive and geophysical data.
New large levee foundation	Longitudinal profiles or grids usually based on seismic, electrical or electromagnetic techniques but possibly including gravity or magnetic techniques extending from the shoreline to about 40 m or more from the landward toe to determine stratigraphy, structure and ground properties. Borehole geophysics as required to reconcile intrusive and geophysical data. Airborne methods may be considered where linear dimensions to be investigated are sufficiently large.
Existing levees to be raised	Longitudinal profiles or grids usually based on seismic, electrical or electromagnetic techniques but possibly including gravity or magnetic techniques extending beyond the landward toe to gain an insight into the internal structure and properties of the levee. Borehole geophysics as required to reconcile intrusive and geophysical data.
River bed or foreshore profile	Longitudinal profiles based upon acoustic/seismic techniques on the marine side of the levee to gain an insight into bathymetry and shallow seabed/river bed conditions. Borehole geophysics as required to reconcile intrusive and geophysical data. Airborne methods should be considered where linear dimensions are sufficiently large.

Investigation of levees and/or their foundation soils range in scale from local to regional. This aerial variability should be reflected in the programme of investigation. An example of a generalised programme for a geophysical investigation is given in Table 7.131.

Table 7.131 Example of a generalised programme for geophysical investigations

Scale	Indicative programme for the geophysical investigation of levees
Site investigation, 50 m to 10 km linear dimension	• site-specific desk study, existing data review • integrated scope, possibly multi-phase geophysical-intrusive investigations • land and over water geophysical investigation, initial interpretation • targeted feasibility intrusive investigations with borehole geophysics, if appropriate, and *in situ* and laboratory testing • detailed intrusive investigations • integrated reporting (land/over water geophysics and intrusive information) including interpretation and recommendations.
Regional investigation, 10 km to 500 km+ linear dimension	• major regional desk study, existing data review • integrated scope, multi-phase geophysical-intrusive investigation strategy • airborne geophysical investigation, initial interpretation • site-specific land and over water feasibility geophysical investigation, initial data integration and interpretation • development of initial geological/geotechnical/hydrogeological model and priority sites for intrusive investigations • targeted regional borehole programme with wireline logging, borehole geophysics and *in situ* and laboratory testing • update of initial geological/geotechnical/hydrogeological model and priority sites for investigation • targeted, site-specific land and over water geophysical investigation, where required • detailed site-specific intrusive and sample testing programme, where required • integrated reporting (air/land/over water geophysics and intrusive information) including interpretation and recommendations.

Deliverables from a geophysical investigation

Geophysical investigations generally involve digital data acquisition with GPS for control of elevation and lateral position. It usually involves the measurement, recording, processing and presentation of a physical response to build a 1D to 3D representation of the ground that can be extended to 4D with time lapse investigation techniques, to detect changes in boundary or soil conditions in the foundation soils or within the levee structure. Data acquisition should be subject to acceptable norms of quality control as sophisticated processing software will not fix data quality issues. The information from a geophysical investigation can be:

- indirect determination of physical properties
- imaging and interpretation of subsurface geometry (including layering and discontinuities).

Many investigations combine both imaging and determination of physical properties in 1D (depth or distance profiles), 2D (maps in plan or distance-depth profiles) or 3D (area-depth volumes).

A generalised geophysical investigation has a number of core components:

- field data acquisition
- data conditioning (noise reduction) and processing
- data calibration (to known conditions from boreholes or CPT for example)
- numerical modelling
- qualitative and/or quantitative interpretations
- integration of other information for more reliable ground-model building.

Geophysical reports should fully detail all assumptions used when interpreting data.

7.9.6.1 Geophysical methods

This section comprises a reference of geophysical techniques to aid in the selection of possible methods to:

- assess ground or foundation conditions
- evaluate the internal structure of the levee from early feasibility investigations through to post-construction evaluation
- assess water depths and the nature of bed deposits below a water body.

The effectiveness of geophysical methods is limited by six controlling factors:

- depth of penetration
- vertical resolution
- lateral resolution
- signal to noise ratio
- contrasts in physical properties
- degree of correlation between the property measured by the geophysical technique and the nature and mechanical properties of the material.

The choice of technique(s) and adherence to good practice in planning, acquisition, processing and interpretation are also pivotal to the effectiveness of the investigations. Expert advice should be obtained in the use and selection of methods.

Geophysical methods applicable to levees fall into four key categories that are characterised by the general environment in which they are used:

- **above ground methods:**
  - surface methods deployed at the ground or levee surface

  - over water methods deployed on and below the water surface
  - airborne methods deployed from a fixed wing or helicopter platforms that overfly the site
- **below ground methods:**
  - wireline logging and borehole seismic methods deployed down boreholes, or some forms can be pushed directly into the ground (ie CPT rig).

Surface methods are considered to be primary methods. In general, surface techniques and in particular electrical and electromagnetic variants form the overwhelming majority of historical levee investigations in which geophysics has been used.

Developments in water-based, airborne and borehole-based geophysical techniques are considered to be secondary methods. The use of these technologies is likely to increase in the future, mirroring the growing role of engineering geophysics in geotechnical engineering and construction.

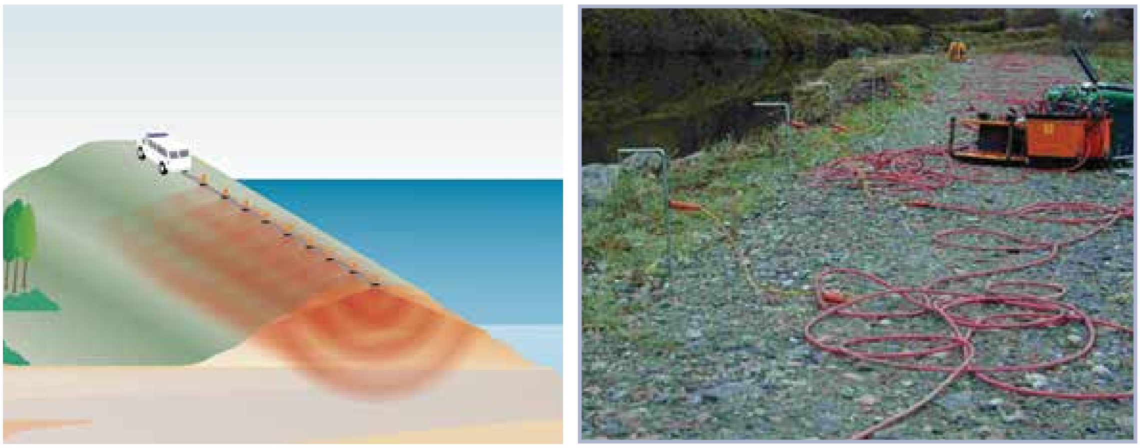
Borehole/wireline methods are included as they could be used when highly localised information is required such as the measurement of geophysical parameters down the axis and within the localised environment of the borehole. Some methods determine the ground conditions between boreholes and can provide images of properties such as seismic velocity in a co-planar sense between boreholes.

The following sections outline four key sub-environments for applying geophysical and non-intrusive investigation techniques to levees.

Each section is supported by a table, and for each method the measured property, where appropriate, is described. Guidance is provided as to which phase of a project the method is most likely to afford greatest benefit to, along with a description of the main objectives. Considerations specific to each method are listed along with key limitations to effective procurement of geophysical services and guidelines for productivity rates.

The most applicable techniques are listed at the top of each table in **bold text**, and what might be considered as non-routine techniques are *italicised* and listed at the bottom of each table.

7.9.6.2 Primary geophysical methods: surface



Figures 7.73 Surface methods: electrical resistivity tomography (courtesy Fugro)

Inazaki and Hayashi (2011) summarised the key suitability criteria for surface geophysical methods (Figure 7.73) for an investigation of levees or foundation soils. They stated that it should:

- be non-destructive and not damage the levee
- identify the physical properties that are helpful in evaluating the safety of levee systems
- be able to image shallow depths down to 20 m

- have sufficient resolution to identify an anomaly as small as 1 m
- provide a continuous profile along levees at an affordable cost
- be technically transparent and be widely applicable to levee surveys.

While appropriately selected geophysical methods generally meet these criteria, anomalies with smaller dimensions than 1 m can be detected including changes in material type, voiding and zones of seepage.

Surface geophysical methods involve the use of man-portable or vehicle-mounted instruments to obtain measurements at or very close to the ground surface. As such, access can be a key limitation in the case of thick vegetation or surface obstructions and it may be necessary to clear parts of the site to allow survey works to proceed.

Instruments are commonly linked to real time differential global positioning system (DGPS) for sub-metre positional accuracy. They can often be integrated with dGPS data from other geophysical techniques to form an efficient multi-sensor survey.

For some techniques, such as resistivity, electromagnetic and magnetic, site responses can be evaluated in the field shortly after acquisition of the data. For others, such as gravity and refraction, the data processing is best completed post data acquisition in the field base or office. The quality of all data can and should be evaluated in the field. Data are frequently processed to 2D maps and profiles and 3D volumes using powerful database processing and visualisation software. Intrusive data can be readily integrated with geophysical responses within a shared spatial framework.

For levees common applications include:

- define internal layering within levees and within the foundation soils
- support the targeting of follow on intrusive investigations
- determining zones of potential or actual levee seepage
- identify the presence of non-soil objects
- facilitate the derivation of *in situ* elastic geotechnical parameters of the levee and foundation soils in conjunction with borehole calibration.

It should be noted that despite the availability of a number of alternative technologies, most historical surface geophysical investigations of levees have been restricted to electromagnetic and electrical techniques. Table 7.132 is derived from Table 2.2 of the FloodProbe project (Royet, 2012) and is colour coded as a guide to the applicability of the most commonly used geophysical techniques for existing levees in the context of:

- zoning and structure determination
- weak spot and anomaly detection
- material property and condition identification.

Table 7.133 presents the primary surface geophysical methods applicable to levees and foundation soils.

Table 7.132 Guide to the application of surface geophysics techniques to existing levees (from Royet, 2012)

		Electrical resistivity tomography	*Electromagnetic induction*	*Surface wave seismic*	*Self potential*	*Seismic refraction*	*Ground penetrating radar*	*Microgravity*	*Magnetics*
Zoning and structure delineation	Horizontal zonation of levee into 'homogenous' blocks	Green	Green	Green	Red	Red	Yellow	Red	Yellow
	Vertical structure, layers, depth to foundation, water table	Green	Red	Yellow	Red	Green	Yellow	Red	Red
Weak spot and anomaly detection	Structural anomalies (eg breach repairs, transitions)	Green	Green	Green	Red	Red	Green	Red	Yellow
	Contact surfaces between layers of contrasting material or condition	Green	Red	Yellow	Red	Yellow	Yellow	Red	Red
	Cracking	Yellow	Red	Yellow	Yellow	Red	Yellow	Red	Red
	Voids, subsidence, foundation cavities	Yellow	Red	Green	Red	Red	Yellow	Green	Red
	Buried channels in foundation	Yellow	Red	Yellow	Red	Red	Red	Red	Red
	Seepage areas, potential erosion and piping	Green	Red	Yellow	Green	Red	Red	Yellow	Red
	Embedded manmade structures	Green	Green	Yellow	Red	Yellow	Green	Red	Yellow
	Buried metallic structures	Green	Green	Red	Yellow	Red	Green	Red	Green
Material properties and condition identification	Soil type, moisture content, clay content	Green	Green	Red	Red	Red	Red	Red	Yellow
	Soil geotechnical properties (porosity, consolidation, permeability)	Yellow	Red	Green	Yellow	Yellow	Red	Red	Red
	Monitoring of temporal changes (dry season/flood conditions)	Green	Yellow	Yellow	Green	Red	Yellow	Yellow	Red
	Seepage flow velocities	Red	Green	Red	Green	Red	Red	Red	Red

Key

Green	Recommended or preferred method
Yellow	Conditionally applicable method
Red	Generally not applicable

Table 7.133 Primary surface geophysical methods applicable to levees and foundation soil

Method	Deliverables	Suggested project phase	Geotechnical and hydrogeological objectives	Considerations	Limitations	Daily production rate
Primary methods are indicated in **bold** text Secondary methods are indicated in *italic* text	***Factors common to methods***					
	Profiles and maps of properties distribution	• reconnaissance • feasibility • design • post-construction.	• address potential failure mode • subsurface engineering properties • seepage pathway identification • changes in hydrogeology • archaeology.	• new or existing levee • access • potential complimentary methods to make results unique.	• utilities • acoustic noise • communication systems.	
Factors specific to methods						
Electrical resistivity tomography DC profiling/ERT and capacitively coupled (CCR)	Distance-depth plane profiles of subsurface resistivity distribution interpreted for layering, structure and anomalous electrical properties	• reconnaissance • feasibility • design • post construction.	• identify coarse materials • identify seepage zones.	• consider towed CCR array for large projects • tie interpretation to down-hole data where possible.	• conductive groundwater will mask material response • maximum investigation depth generally <20 m.	• 300m to 1 km (ERT with metal stakes.
Electro-magnetic induction Frequency domain	Interpreted distance profiles of subsurface electrical conductivity distribution	• reconnaissance • feasibility • post-construction.	• horizontal and vertical variation of electrical conductivity– changes in material types, detect buried pipes and other metallic structures • construction quality assurance (presence of exotic, non-soil construction materials).	• consider vehicle towed multi-sensor arrays for large projects • complementary methods include magnetic total field/gradiometry and ground penetrating radar.	• investigation depth, 1.5 m to 30 m dependent upon coil separation • loss of lateral resolution with increased penetration depth.	• 5 km to >10 km.
Surface wave seismic MASW, SASW, ReMi variants	Distance-depth plane profiles of subsurface shear wave velocity distribution derived from surface wave velocity interpreted for layering, structure and anomalous stiffness properties	• feasibility • design • post-construction.	• identify weak zones in existing structures or pre-construction surfaces • identify changes in material thickness and extent • identify extent of repair activities.	• consider towed arrays for large projects • tie interpretation to down-hole data where possible.	• investigation depth dependent upon frequency but generally <20 m • seismically/acoustically noisy sites.	• 500 m to 1 km.
Seismic refraction P-wave, S-wave and refraction tomography	Distance-depth plane profiles of subsurface velocity distribution interpreted for layering, structure and anomalous seismic properties	• reconnaissance • feasibility • design • post-construction.	• identify changes in founding materials by determination of compressional or shear wave velocity with depth, depth to base of levee.	• tie interpretation to borehole data where possible.	• most sites violate governing simple interpretation assumption.	• 300 m to 600 m (P-wave) 200 m to 500 m (S-wave).

Table 7.133 ***Primary surface geophysical methods applicable to levees and foundation soil (contd)***

Self-potential (SP)	Interpreted distance profiles of SP response indicating potential zones of leakage	• post-construction.	• identify seepage zones and pathways.	• Consider use of over-water SP using towed array systems	• not effective in electrically conductive saline environments. • Noise interference from metallic structures	• 2 km to 4 km.
Seismic reflection *P-wave, S-wave*	*Seismograms interpreted for layering, structure and anomalous seismic properties*	• *reconnaissance* • *feasibility* • *design* • *post-construction.*	• *stratigraphic and structural geologic interpretations within and in foundation of levees* • *depths to interfaces.*	• *tie interpretation to down-hole data where possible* • *consider multicomponent simultaneous P and S wave acquisition.*	• *near surface (30 m) requires higher seismic resolutions only possible with shorter wavelengths associated with S-waves.*	• 500 m to 1 km.
Ground penetrating radar (GPR)	*Radargrams interpreted for layering, structure and anomalous dielectric properties.*	• *feasibility* • *design* • *post-construction.*	• *stratification and structure in levees.*	• *consider towed arrays for large projects.*	• *very limited penetration in clay-rich materials or with brackish to saline pore fluids.*	• *>1 km (except for bi-static stepped measurement systems).*
Microgravity	*Maps or distance profiles of gravitational response, interpreted for density variations*	• *feasibility* • *design* • *post-construction.*	• *detect low-density zones and potential seepage pathways detect cavities.*	• *consider as a targeting rather than reconnaissance technique for specific applications such as shallow voiding.*	• *relatively slow technique – shallow targets require dense grids.*	• *50 to 150 data points.*
Magnetics	*Maps or distance profiles of gravitational response, interpreted for magnetic variations*	• *reconnaissance.*	• *detect UXO and other metallic objects (eg pipes).*	• *consider towed arrays for large projects.*	• *noise interference from metallic structures* • *depth limitations due to field decay with depth to object.*	• *>10 km.*

7.9.6.3 Secondary geophysical methods: over water

Figure 7.74 Over water methods – side-scan sonar 'towfish' (courtesy Fugro)

Various geophysical systems can be deployed from a vessel (Figure 7.74) with GPS positional control to:

- map the bottom surface and sub-bottom conditions, and bathymetry (interferometric and multi-beam sonar systems)
- map substrate type (side-scan sonar)
- map sediment stratigraphy and structure (seismic sub-bottom reflection profiling, refraction and surface wave systems).

However, water based equipment needs an adequate water depth in which to operate. Due to the nature of the levee adequate depth of water close to the levee or over the area to be investigated may only occur periodically, such as under tidal conditions. Where there is ease of access and a trafficable surface on foot or vehicle then some surface geophysical techniques can be used. As with surface geophysics, intrusive investigation and bottom photography can be used to validate the geophysical data.

For levees common applications include:

- bathymetry and morphology (deposition and scour)
- slope geometry
- surface sediment distribution
- deposition and scour
- sub-bottom stratigraphy and structure
- obstructions
- assessment of *in situ* small strain elastic geotechnical parameters (Table 7.60).

Typical over water geophysical methods applicable to levees and foundation soils are presented in Table 7.134.

Table 7.134 Over water geophysical methods applicable to levees and foundation soils

Method	Deliverables	Suggested project phase	Geotechnical and hydrogeological objectives	Considerations	Limitations	Daily production rate
Factors common to methods						
		• reconnaissance • feasibility • design.	• address potential failure mode • subsurface engineering properties • seepage pathway identification • changes in hydrogeology/archaeology.	• new or existing levee • access • potential complimentary method to make results unique.	• utilities • acoustic noise • communication systems • only in water filled channels.	
Factors specific to methods						
Continuous seismic profiling	Seismic profiles of subsurface stratigraphy and structure	• before or during new levee feasibility • large area reconnaissance of existing structures.	• assess changes in geologic and geotechnical properties • estimate removal volumes and best methods • identify potential seismic hazards • confirm seepage pathways • identify cultural resources.	• collect bathymetry data during seismic mobilisation.	• nearby noise sources (industrial, transport) • hard bottom and gas-rich sediments limits investigation depth.	• 15 km.
Continuous resistivity profiling	Resistivity profiles of subsurface stratigraphy and structure	• before or during new levee feasibility • large area reconnaissance of existing structures.	• assess changes in geologic and geotechnical properties • identify seepage pathways, identify contaminant discharge areas, confirm seismic hazards.	• collect bathymetry data during resistivity mobilisation • frequent water conductivity and temperature profiles, conduct forward modelling.	• buried utilities • groundwater discharge masking subsurface.	• 15 km.
Side-scan sonar	Acoustic images of seabed/ water bottom	• hydraulic and hydrology reconnaissance and feasibility studies, or when it can be combined with other mobilisation.	• channel bottom morphology • identify cultural resources and navigation hazards • surface sediment classification.	• frequency and range selection should be based on nature of target.	• very shallow water bodies • gas charged sediments • vegetation such as kelp.	• 15 km.
Magnetometer	Maps/profiles of magnetic response	• hydraulic and hydrology reconnaissance and feasibility studies, or when it can be combined with other mobilisation.	• identify cultural sources • identify buried utility cables and pipes • locate potential unexploded ordnance • locate navigation hazards • support broad area geological airborne geophysics efforts.	• often run with EM instruments.	• responds only to iron-bearing materials • thick metallic debris layers limit investigation depth and resolution.	• 20 km

Table 7.134 Over water geophysical methods applicable to levees and foundation soils (contd)

Frequency-domain EM	Maps/profiles of conductivity distribution	• hydraulic and hydrology reconnaissance and feasibility studies, or when it can be combined with other mobilisation.	• identify cultural sources • identify buried utility cables and pipes • locate potential unexploded ordnance • locate navigation hazards • support broad area geological airborne geophysics efforts.	• responds to all metallic items and minerals.	• investigation depth.	• 20 km
Time domain EM	Maps/profiles of conductivity distribution	• hydraulic and hydrology reconnaissance and feasibility studies, or when it can be combined with other mobilisation.	• identify cultural sources • identify buried utility cables and pipes • locate potential unexploded ordnance • locate navigation hazards • support broad area geological airborne geophysics efforts.	• consider boat and vehicle towed arrays for large projects • complimentary methods include magnetometer or terrain conductivity instruments	• investigation depth.	• 20 km
Marine refraction	Distance-depth plane profiles of subsurface compressional wave velocity distribution interpreted for layering, structure and anomalous seismic properties	• reconnaissance • feasibility • design • post-construction.	• identify changes in founding materials by determination of compressional wave velocity with depth.	• tie interpretation to borehole data where possible.	• Investigation depth <50 m • profile orientation.	• 10 km
Marine MASW	Distance-depth plane profiles of subsurface shear wave velocity distribution interpreted for layering, structure and anomalous seismic properties	• reconnaissance • feasibility • design • post-construction.	• identify changes in founding materials by determination of shear wave velocity with depth.	• tie interpretation to borehole data where possible. Integrate with density data to derive stiffness distribution.	• investigation depth <25 m • profile orientation.	• 5 km
SP measurement	Profiles of self potential interpreted for zones of high permeability/leakage	• post-construction.	• identify zones of leakage through levee.	• correlate with land based SP and resistivity measurements.	• fresh/brackish water only.	• 2 km

7.9.6.4 Secondary geophysical methods: airborne

Figure 7.75 Airborne methods – frequency domain EM measurement (courtesy Fugro)

Airborne geophysical investigations (Figure 7.75) for levee construction and evaluation are usually flown on multiple flight lines parallel to the levee or floodway, along the levee alignment, landside and waterside. Coverage is continuous over land and water on both sides of levees. Aviation regulations restrict flight over habitations, so coverage can be limited in urban environments, especially on the land side, where buildings and houses are too close.

The surveys are generally undertaken using a helicopter since the lower altitude, slower flying speed, and manoeuvrability improve coverage, resolution and safety. For most geophysical methods it is possible to cover more than 100 km of levee with multiple passes in a day. The data can be used to provide an overview of the entire levee scheme and gives guidance for planning detailed follow on non-intrusive and intrusive investigations.

Airborne techniques include penetrating methods (electromagnetic and magnetic) that measure the properties of soil and rock below the surface, and surface-sensitive methods that measure the nature or topography of the surface. The resolution of penetrating methods is limited by sensor height to about 15 m spatially for magnetic objects, and to a 50 m to 75 m diameter 'footprint' below the sensor for electromagnetic techniques.

Electromagnetic methods measure ground resistivity (or conductivity) of the levee foundation material in 3D. The most common targets are sandy paleochannels and measuring the distribution and thickness of the alluvial clays to assess the risks of under-seepage and uplift. Both electromagnetic and magnetic surveys can also detect buried infrastructure, including drain pipes, power lines etc. The power lines and pipelines can interfere locally with signals from the geology, and may limit depth of investigation of the EM.

Frequency-domain helicopter EM is generally the preferred method, as the high frequencies available give the best resolution in the top 50 m of the soil profile, and are generally less affected by interference from power lines than time-domain EM.

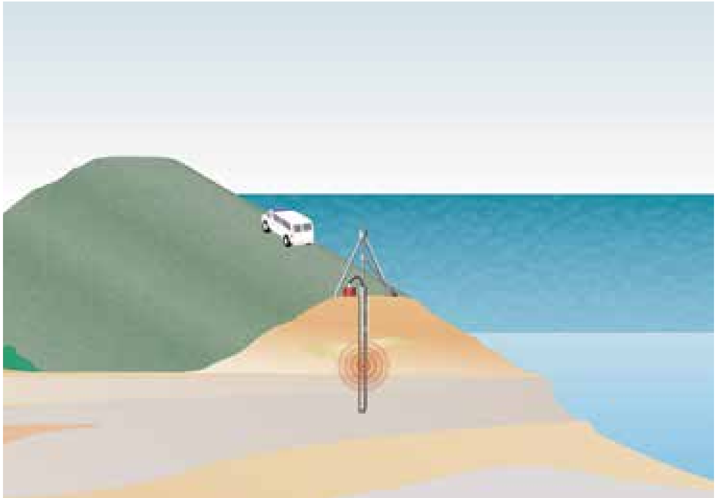
Thermal imaging methods can indicate seepage pathways and water discharges based on temperature contrast. Airborne LiDAR surveys provide high resolution, continuous topographic and surface infrastructure measurements for the levee and surrounding environment (Sections 7.9.1).

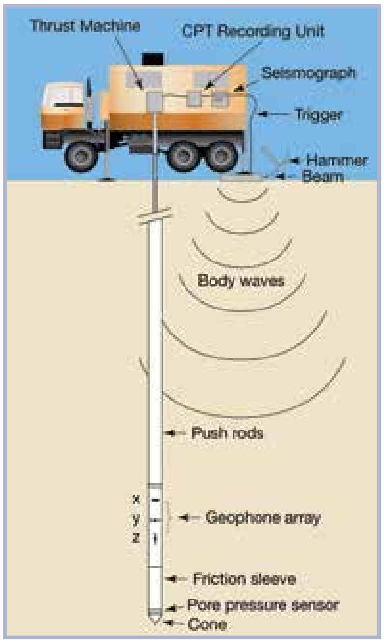
Table 7.135 provides typical airborne geophysical methods applicable to levees and foundation soils.

Table 7.135 Airborne geophysical methods applicable to levees and foundation soils

Method	Deliverables	Suggested project phase	Geotechnical and hydrogeological objectives	Considerations	Limitations	Daily production rate
Electro-magnetic	Conductivity or resistivity	• reconnaissance • feasibility • design • post-construction.	• address potential failure mode • subsurface engineering properties • seepage pathway identification, changes in hydrogeology infrastructure.	• new or existing levee • access potential complimentary method to make results unique. • no ground access needed • continuous coverage.	• no flight over habitations, busy roads • no 1:1 correlation of conductivity to soil type >30 m lateral resolution.	• 100+ km of levee.
Thermal infrared	Temperature	• feasibility, or when it can be combined with other similar methods.	• locate seepage pathway discharge zones.	• must be calibrated against ground measurements • data acquisition often at night to maximise thermal contrast.	• manmade thermal changes produce false positives.	• 50+ km of levee.
Magnetic	Magnetic susceptibility	• reconnaissance • post-construction.	• locate subsurface ferrous infrastructure.	• line interval • sensor altitude • must be calibrated against ground measurements.	• no flight over habitations, roads.	• 100+ km of levee.
LiDAR	Surface elevation, topography	• reconnaissance • design • post-construction.	• map levee height, ground surface height, subtle fault and subsidence features.	• requires land survey of control points for accurate positional data • best done during low water/no snow conditions, and minimal leaves on trees.	• snow, high water, and heavy foliage may limit data collection.	• 200+ km of levee.

7.9.6.5 Secondary geophysical methods: borehole (wireline) logging, borehole seismic and CPT





Figures 7.76 Borehole and CPT methods – schematic: seismic CPT (courtesy Fugro)

Borehole geophysical methods (wireline logging) involve the use of logging tools lowered into a borehole (Figure 7.76) on an armoured telemetry cable to determine the response of the ground along the borehole axis. Holes are commonly vertical for shallow applications. As such borehole techniques are usually used as part of a targeted investigation to confirm and further characterise anomalous or background responses detected by primary geophysical techniques.

Borehole seismic techniques can also be used to determine the elastic geotechnical properties of the soils axial to the borehole or within 2D panels or 3D volumes between boreholes. These techniques deploy seismic sources and seismic receivers in various configurations that are either a combination of surface and borehole operations, commonly known as 'down-hole' or 'up-hole', or are borehole based investigations, commonly known as 'cross-hole' or 'suspension logging'. Both P-wave (compression) and S-wave (shear) sources can be deployed and used to derive *in situ* estimates of elastic geotechnical parameters of levee materials in both vertical and horizontal planes when combined with density measurements from core or other forms of wireline logging. Borehole geophysical techniques between pairs of boreholes can determine seismic velocity distributions that can identify and delineate subsurface features such as zones of elevated porosity or low stiffness. This method could help identify the presence of point bar deposits (sandy) or paleochannels, which are filled with either high permeability sands and gravels or soft clays and peat.

For levees common applications include:

- evaluation of layering
- determination of the physical properties of levee materials, including density, clay content, water saturation
- evaluation of *in situ* small strain and elastic geotechnical parameters (Table 7.60).

Table 7.136 provides details of some borehole methods. There are three levels of applicability of different methodologies. These are distinguished as:

- **bold and underlined**: these are the four techniques that are always applicable and can be considered for feasibility, in support of surface surveys and for detailed targeted surveys. The induction and lateral resistivity techniques are complementary. Resistivity is preferred because of the control on the depth of investigation and the vertical resolution but it requires an unlined hole. Alternatively, where an unlined borehole is impractical to form due to the instability of the ground, a plastic lined borehole can be used for the induction techniques. Natural gamma logging can be used in any form or borehole, lined or unlined. The mechanical calliper is only usable if an open hole is available. Borehole diameter from a calliper allows all other borehole methods to be calibrated but also identifies weak horizons and intervals causing borehole breakout

- in **bold** are the techniques for providing ground truthing of the surface (sonic and density) and geotechnical properties (density, porosity) that can contextualise sampling for a feasibility investigation, provide indicators of high porosity zones and saline/freshwater intrusions
- in *italics* are the techniques that can be used where there are specific identified problems in a detailed survey as they require pre-identified pairs or sets of boreholes, or have been replaced by more sophisticated methods.

Table 7.136 Borehole geophysical methods applicable to levees and foundation layers

Probe name	Deliverables	Project phase (borehole availability and mobilisation may combine phases)	Geotechnical and hydro-geological objectives	Considerations	Required borehole conditions (limitations)
Natural gamma	Total natural gamma emissions	• reconnaissance • feasibility • detailed surveys.	• measure clay content and material boundaries • BH to BH correlation of stratigraphy and mineralogical variation.	• determine where electric logs and surface surveys indicate a conductive fluid instead of a conductive formation.	• no restriction.
Lateral resistivity (dual depth preferable)	Changes in resistivity focused laterally from BH	• reconnaissance • supports surface electrical surveys.	• identify material boundaries • deeper (0.5 m+) focused measurements reflect true resistivity to support and verify surface resistivity models • combine with natural gamma for 'lithological' interpretation.	• virtually equivalent to point data with high vertical resolution • comparison between shallow and deep resistivity • indicates permeability variations.	• fluid filled open hole.
Induction with pseudo-magnetic susceptibility (dual depth preferable)	Formation and interstitial fluid conductivity, and presence of magnetisable minerals such as magnetite or other iron species	• reconnaissance • support surface electrical survey.	• measure changes in clay content and/or dissolved solids • identify electrical and magnetic stratigraphic boundaries • calibration for surface electrical survey data models • identify saline intrusions.	• works well in conductive formation and water environments.	• open hole or PVC-cased borehole.
Mechanical calliper	Borehole diameter variations	• reconnaissance.	• verify open borehole integrity and casing bottom • identify weak horizons.	• open borehole diameter • calibration data for all other borehole logs.	• open hole.
Full waveform sonic (FWS)	Compressional, shear, and tube wave data using dual transmitter/receiver pairs	• feasibility • support surface seismic surveys.	• material boundaries, velocity.	• geo-mechanical property estimates assisted by *in situ* density log results • density velocity profile can be used to invert seismic data to acoustic impedance.	• fluid filled open hole.
Suspension logging	P- and S-wave velocities at pre-specified depth intervals	• feasibility.	• oriented shear wave data identifies anisotropy.	• correlate with uphole and downhole seismic, surface MASW and refraction data.	• fluid filled open hole or cased (not steel) and grouted hole • data may be noisy in poor borehole conditions or under low confining stress.

Table 7.136 ***Borehole geophysical methods applicable to levees and foundation layers (contd)***

Gamma-gamma density	Total electron density to calculate formation bulk density, porosity, moisture content with lithological control	• feasibility.	• assess density, large order porosity, and moisture content changes • estimate *in situ* geo-mechanical properties • identify failure plane, and low and high-density zones.	• measures higher porosity conditions better than the neutron porosity probe.	• no restrictions • calibrated in fluid filled boreholes.
Neutron porosity	Neutron response to water content using two or more detectors	• feasibility.	• assess small order porosity and moisture content changes • identify failure plane, and low and high-density zones.	• measures lower porosity conditions better than the gamma-gamma probe • less influenced by poor borehole conditions than gamma-gamma.	• no restriction.
Temperature logging	Measures borehole fluid temperature variation in rested borehole	• feasibility.	• identifies flows in and out of borehole.	• BH needs standby time of at least 12hrs.	• open fluid filled holes.
Flow logging	Measures variations in vertical flows within rested borehole	• feasibility.	• identifies fluid flows in and out from the borehole.	• vertical flow: impeller flowmeter, electromagnetic fm, heat pulse fm, tracer log • horizontal flow: colloidal flow log, tracer log.	• open fluid filled holes.
Acoustic televiewer	Time and amplitude 'map' of borehole. Showing fractures and texture	• mainly detailed survey.	• identify bedrock cavities faulting.	• a technique especially suited to rock boreholes where bedrock fractures and voids are to be identified.	• fluid filled open holes.
Guard resistivity	*Changes in resistivity over a limited vertical interval*	• *reconnaissance support.*	• *second best to lateral resistivity for absolute resistivity values avoiding borehole effect.*	• *suitable for conductive drilling fluids* • *good vertical resolution.*	• *fluid-filled open hole.*
Normal resistivity	*Changes in resistivity from an induced current across two or more electrodes*	• *reconnaissance* • *support surface electrical survey.*	• *identify material boundaries* • *near true formation resistivity to support and verify surface resistivity survey data models.*	• *works well in resistive formation and water environments* • *poor vertical resolution.*	• *fluid-filled open hole.*
Single point resistance (SPR)/ spontaneous potential (SP)	*SPR measures bulk resistance changes between the probe and a surface electrode.* *SP measures naturally induced voltage changes at geological contacts*	• *reconnaissance* • *support surface potential survey.*	• *identify material boundaries* • *complements natural gamma data* • *support natural potential surveys.*	• *water quality can mask or reverse formation response* • *surface conductivity can influence data quality* • *SPR has good vertical resolution but affected by borehole effects including mud masking.*	• *fluid-filled casing or open hole.*

Table 7.136 ***Borehole geophysical methods applicable to levees and foundation layers (contd)***

Vertical seismic profile/check shot survey	*Primary and shear wave travel time between surface source, and a down-hole geophone clamped against the casing or formation*	• *feasibility* • *supports surface seismic investigation.*	• *bulk in situ primary and shear wave velocity data needed to plan surface seismic surveys* • *direct measurement data and quality control on surface seismic data models.*	• *frequency compatible with surface seismic methods* • *steel casing response may be more easily removed than in the FWS data.*	• *no restrictions.*
Cross hole tomography	*2D image in plane of the boreholes*	• *detailed survey.*	• *resistivity and seismic tomography available* • *image of levee structure and water table/salinity.*	• *particularly suitable where access between boreholes is not available but significant features are suspected to be confined to the inter-borehole zone.*	• *limited separation of boreholes for resistivity tomography.*
Up-hole/down-hole seismic	*Surface to borehole P- and S-wave velocities*	• *detailed survey.*	• *as for seismic cone (below) provides velocity data to ground truth seismic surveys.*	• *consider suspension logging as an alternative method.*	• *loss of high frequencies may lead to poor velocity estimates.*

In addition to geophysical measurements in the borehole, four additional techniques are available through CPT (Table 7.137). The combination of geophysical measurement with CPT profiling allows the geophysical data to be directly comparable with the CPT and correlated with data from other forms of intrusive investigation, such as borehole, to provide a fully integrated investigation.

Natural gamma and conductivity cones provide the same data that is available from natural gamma and resistivity/induction tools in the borehole. The vertical resolution is very high and the gamma and conductivity measurements are not influenced by borehole fluids as they are pushed directly into the ground with the cone.

The seismic cone measures surface to cone velocities including shear velocity at predetermined intervals.

The magnetometer cone measures anomalous magnetic response in the vicinity of the cone sensor allowing the detection of ferrous objects such as UXO and services.

Table 7.137 Cone penetration geophysical methods applicable to levees and foundation layers

Probe name	Deliverables	Project phase	Geotechnical and hydro-geological objectives	Considerations	Limitations
Gamma cone	Total natural gamma emissions	• reconnaissance • feasibility.	• measure clay content and material boundaries • CPT to CPT and BH correlation of stratigraphy and mineralogical variation.	• allows correlation with BH data and identifies material changes relating to strength changes.	• stiff/dense materials may limit penetration.
Conductivity cone	Changes in soil resistivity	• reconnaissance • feasibility.	• identify material boundaries • deeper (0.5 m+) focused measurements reflect true resistivity to support and verify surface resistivity models.	• correlates with BH data and ground truths surface electrical surveys.	• stiff/dense materials may limit penetration.
Seismic cone	Surface to cone P and S-wave velocities	• detailed survey.	• velocity control of surface surveys. Major velocity boundaries.	• correlates with velocity derived from other methods.	• limited vertical resolution • stiff/dense materials may limit penetration.
Magnetometer cone	Depth log of magnetometer readings	• reconnaissance • detailed survey.	• detect buried metal objects, eg drums, UXO and depth of piles and sheet piles.	• deeper penetration than surface surveys, faster and cheaper than boreholes.	• detection radius limited to a few metres from probe.

Further reading

There are a number of books and publications, which will provide useful reading:

ASTM D6429–99 (2011) e1 *Standard guide for selecting surface geophysical methods*

Boukalová and Beneš (2008) *Application of GMS system in the Czech Republic – practical use of IMPACT, FLOODSite and GEMSTONE projects outcomes*

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7.9.7 Intrusive site investigation methods

Levees are predominately built on floodplains. The foundation soils can comprise a laterally and vertically variable sequence of soil types that can range from water bearing highly permeable sands, gravels and coarse silts/fine sands, to soft and very soft cohesive silts, clays and organic soils and peats. The internal structure of existing levees can be complex, reflecting periodic phases of raising and widening to contain ever larger flood events, or repairs to the earthwork following slope failure, breach or settlement. In order to evaluate their performance the spatial distribution and engineering properties of the soils forming the levee and foundation soils need to be defined. This can be achieved by intrusively penetrating the ground to recover soil samples for visual description and laboratory testing, allow *in situ* tests to be performed and to aid in the interpretation of geophysical and non-intrusive investigations.

This section aims to outline considerations in determining the distribution and depth of intrusive investigations, and the equipment and methods available. The methods included here are where the sampling method is separate from that used to advance the exploration hole. Not all items of equipment used for intrusive investigation are suitable for use in all possible ground conditions. Where a wide range of ground conditions are expected there may be a requirement to use a number of different intrusive techniques.

It is preferable that the methods to be used in an intrusive ground investigation are defined by a geotechnical engineer with relevant experience. Where other site issues are identified, such as contamination, UXO and archaeology that may also require intrusive investigations then an appropriate specialist should be consulted. The relevant specialist can assist in assessing what investigations are required and, if appropriate, how they can be integrated within the geotechnical intrusive investigations. For locations where access is difficult it may be preferable to consult a ground investigation contractor. They will be able to provide additional advice on the most appropriate method of obtaining the required information.

7.9.7.1 Selection of intrusive techniques

The techniques used for intrusive investigations should be appropriate to the ground conditions, the information required from the investigation and site access constraints. Some guidance on the capabilities of specific techniques is presented in Section 7.9.7.5. A summary of some forms of intrusive investigation techniques and the ground conditions where they are appropriate for use are given in Table 7.138. Some factors to consider when selecting a form of intrusive investigation are presented in Table 7.139.

Table 7.138 Summary of intrusive investigation techniques and ground conditions for which they are most appropriate (after Steeds et al, 2000)

Ground conditions	**Inspection pits: trial pits/trenches**	**Hand auger**	**Window sampling**	**Rotosonic drilling**	**Vibrocore**	**Cable percussive**	**Rotary drilling/ sampling**	**Soft ground rotary drilling**	**Cone penetration test**	**Dynamic probing**
Coarse and/or heterogeneous fill/made ground	A	B	B	A	U	B	B	U	B	B
Fine homogeneous fill/made ground	A	A	A	A	A	A	B	U	A	A
Unconsolidated sands	B	B	B	B	A	B	B	U	A	B
Unconsolidated gravel	A	B	B	B	U	B	B	U	B	B
Boulders and cobbles and Boulder clay	A	U	U	B	U	B	B	U	U	U
Soft sediments	A	B	A	B	A	A	A	A	A	A
Medium sediments	B	U	B	A	B	B	A	U	B	B
Strong sediment (sedimentary rocks – chalk, limestone dolomite, grit)	U	U	U	B	U	U	A	U	U	U
Metamorphic and igneous rocks	U	U	U	B	U	U	A	U	U	U

Notes

A = method should be suitable

B = method may be suitable depending on working method adopted

U = method is unlikely to be suitable

Table 7.139 Other factors to be considered in the selection of intrusive methods of investigation

Project and technical considerations	**Environmental factors**	**Operational considerations**
• objectives and phase of the investigation: reconnaissance, feasibility or design (Section 7.1) • budget constraints: unit costs and value for money • anticipated ground conditions: desk study and non-intrusive investigations (Sections 7.1 and 7.9.6) • capability of equipment: applicability and limitations • sample quality and purpose: visual description only or advanced laboratory tests • monitoring requirements: frequency and duration during and post fieldwork.	• past land use and potential for archaeology and contamination • UXO • requirements for aquifer protection measures • presence of sensitive habitats • invasive plant species • urban environment • noise and vibration from site operations • pollution from investigation plant: spillages of fuel, lubricant and hydraulic fluid • containment and disposal of soils/ cuttings and drilling fluids: a site waste management plan may be required.	• access to and within the site: poor or waterlogged ground, slopes, narrow crests • location: proximity to buildings, structures, services • overhead clearances: buildings, power lines • public access to the site • livestock • damage to existing structures/ earthworks/access routes by movement of plant and intrusive investigations • health and safety issues associated with working near or over water or on contaminated ground.

Some of the considerations presented in Table 7.139 are developed further in Section 7.9.7.8. One or more of these factors may preclude intrusive investigations by the method that is best suited to obtaining the information required. Under these circumstances it may be appropriate to hold discussions between the affected parties (overseeing body, ground investigation contractor and investigation designer) to establish an approach that is acceptable to all.

7.9.7.2 Spatial distribution of intrusive investigations

The spatial distribution of intrusive investigations needs be sufficient to provide data to adequately characterise the levee foundation and, in the case of an existing levee, its internal structure to a level that is appropriate to the phase and requirements of the project. There are no definitive rules for the spacings of intrusive investigations, but some country codes and standards do provide guidance. While the spacings detailed in Table 7.140 are applicable to the design phase, any intrusive investigations undertaken during the feasibility or reconnaissance phases may be more widely spaced. However, the general concept at targeting intrusive investigations as discussed in this section is still applicable.

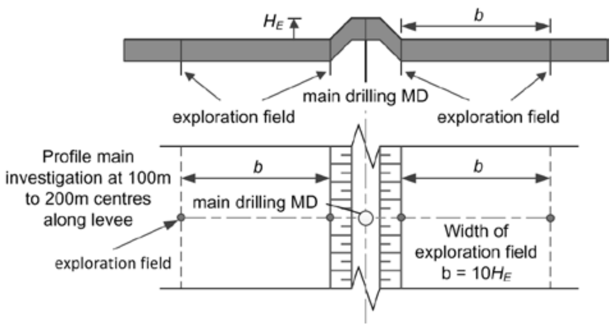
The selection of exploratory hole locations should not be driven by a concept of a fixed spacing but balanced with intelligently targeted locations against features identified by data captured within the CSM (Section 7.1.3), which may include data from non-intrusive and geophysical investigations. It is good practice to correlate the findings of the intrusive investigations with features captured in the CSM to understand their geotechnical context, implications to the project and to allow the likely extent to which the site as a whole will be affected by the features. In some cases it may be appropriate to sink two exploratory holes – one located within the feature and the second adjacent to the feature. In this way the nature and properties of the feature, relative to the background properties of the soil mass can be assessed. If a feature (such as a sand layer) is found in the foundation then the spacing of investigations should be adequate to fully define the feature in both depth and extent.

Assessment of the geotechnical performance of a levee tends to be based on a cross-section perpendicular to the existing/proposed alignment. With only one point of information on any cross-section the ground model and the assessments made based on the model will not be robust as the ground profile across the section can only be assumed to be uniform. Some estuarine and coastal levees can have a wide footprint, in particular where an extensive landward berm is required to mitigate the effects of uplift. More modest river levees may be sited in areas of locally complex geology or in built up areas where very variable anthropogenic deposits may be present. In both cases the assumption of a uniform ground profile across the width of the levee can be inappropriate. So, it is preferable for the investigations to explore the lateral variation in ground conditions, with additional intrusive investigation offset to the centreline of the levee. Additional offset intrusive investigations may also be justified at critical locations where ground conditions may differ from the background condition, near to other structures or at pinch points on the alignment. The benefits of additional intrusive investigations on the waterside of the levee, which may require investigations to be undertaken over water, may also need to be considered.

The internal structure of an existing levee is unlikely to be homogeneous (Section 3.1.4) and may be assessed by using geophysical techniques (Section 7.9.6) supported by intrusive investigations to validate the findings, or by undertaking intrusive investigations at a number of locations across the section. This may require mobile or hand-held equipment that can operate on the levee slopes as well as the crest, which may be narrow.

A number of country codes and standards offer some guidance on the spacing of investigations. Some of these are summarised in Table 7.140. Note that the location and spacing of intrusive investigations should be based on an understanding of the ground and how it will interact with the levee, and intrusive investigations should be targeted to resolve specific issues and explore the landform. The indiscriminate spacing of intrusive investigation at a fixed interval should be avoided.

Table 7.140 ***Guidance of spatial distribution of intrusive investigation from country codes and guides***

Code/guide	Spacing	Associated comment
Eurocode: BS EN 1997-2:2007	20 m to 200 m	Where ground conditions are relatively uniform or the ground is known to have sufficient strength and stiffness, wider spacing or fewer exploration holes may be adopted. In either case, this choice should be justified by local experience.
USACE (2000): EM 1110-2-1913)	60 m to 300 m	Closer spacing in problem areas and wider spacing in 'non-problem' areas, with intelligent targeting of exploration holes around geological features.
USACE (2008): Procedure REFP10L0. DOC	Generally at 300 m to 600 m horizontal spacing on waterside toe, crest and landside toe of levee, and on landside of levee up to 150 m away	The frequency of spacing should be tailored to the project.
GeoDelft (1991): CO-319830/20	Penetration testing at 50 m to 150 m, with boreholes at 200 m to 1000 m	
German Standard: DIN 19712:2013-01(2013)	≥ 100 m	Valid for geotechnical category 3 (which includes large and medium levees according to the German classification). Where ground conditions are relatively uniform or the ground is known to have sufficient strength and stiffness, wider spacing or fewer exploration holes may be adopted. Ground exploration has to be extended to both sides of the levee (waterside and landside). The sideward extension should be at least 10 × height of the protection works (levee, hard or temporary).
German guideline: DWA 507–1E (2013)	On average 100 m (in sketch 100 m to 200 m)	Exploration grid (spacing) has to be adapted depending on local situation. When applying geophysical exploration methods the spacing can be increased.  Spatial distribution of intrusive investigations (after DWA, 2013).

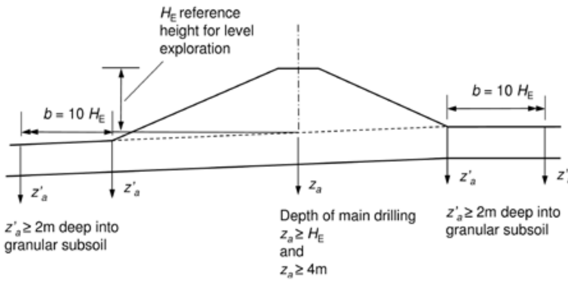
7.9.7.3 Depth of exploration holes

The depth to which the ground will be affected by the levee should be considered when assessing the depth of intrusive investigations. This is not just the zone affected by the self-weight of the levee, such as in the case of settlement or overall stability, but also how the levee and imposed hydraulic loads could combine to act to promote either failure or serviceability issues, for example, uplift and seepage.

It is good practice to include at least one borehole in an investigation that is deeper than might be required for purely geotechnical purposes, to prove the underlying geology. If the intrusive investigations do not extent to an appropriate depth the boundary conditions required for the design or condition assessment will not be fully defined and so the outcomes of any assessment or design could be flawed. There are no definitive rules for the depth of intrusive investigations, but some country codes and standards do provide guidance as shown in Table 7.141.

1 2 3 4 5 6 7 8 9 10

Table 7.141 Guidance of depth of intrusive investigation from country codes and guides

Code/guide	Depth	Associated comment
Eurocode: BS EN 1997-2:2007 (with reference to linear embankment type structures)	0.8 to 1.2 times the height of the levee or at least 6 m	The depth of investigation shall extend to all strata that will affect the project or are affected by the construction
USACE (2000): EM 1110-2-1913)		Where permeable or soft materials are encountered, the exploration holes should extend through the permeable material into low permeable material or through soft material into firm material
USACE (2008): Procedure REFP10L0. DOC	Three times the levee height with at least one every 1500 m to the bases of the aquifer	**For seepage:** investigations through the crest should extent to the base of the pervious layer while those on the waterside and landside should extend below base of impervious layer. **For settlement:** extend to depth of competent material.
DIN 19712:2013-01 (2013)		Deep enough to include all layers that will be affected by the structure.
DWA-M 507-1E (2013)	**Below the levee:** ≥ 4 m and ≥ levee height. **Besides the levee:** ≥ 2m	 Depth of intrusive investigations (after DWA, 2013)

7.9.7.4 Development of site specific correlations

Where different methods of intrusive investigation and *in situ* testing are used within the same investigation or as part of successive phases of investigation, it is useful to include a number of targeted investigation points where all the principal methods used are undertaken (Table 7.51). This approach will allow measured parameters to be compared with the same parameter measured or assessed by a different form of testing (for example, compare laboratory tests with *in situ* tests or parameters inferred from index properties), and permit the development of site specific correlations with indirect methods of assessing soil properties, such as penetration sounding techniques (CPT, dynamic probes etc). The site specific correlations can then be used to infer geotechnical properties from the results of soundings at other locations over the site.

7.9.7.5 Intrusive investigation methods

Routine site investigation techniques can be subdivided into three main categories as presented in Table 7.142.

Table 7.142 Categories of intrusive investigation

Excavations	Boreholes	Probing
• inspection pits • trial pits • trial trenches.	• hand-held augers • dynamic (window/windowless) sampling • rotosonic drilling • vibrocoring • cable percussion • rotary sampling • soft ground rotary sampling.	• CPT • dynamic probing.

Some understanding of the methods, and the applications and limitations of the equipment is essential when scoping an intrusive investigation. Tables 7.143 to 7.153 provide a summary of the key aspects of commonly used methods, together with an indication of how the method is applicable to levees. Further reading related to these methods is presented at the end of the section.

Table 7.143 *Inspection pits*

Inspection pits	Method	Applications	Limitations
 *(courtesy Fugro)*	• hand excavated using electrically insulated tools • typically 0.4 m × 0.4 m by 1.2 m deep • one to two hours to excavate depending on obstructions/ pavement thickness.	• service clearance at exploratory hole locations together with cable avoidance tools • proving services or inspection of near surface structures • disturbed geotechnical and geoenvironmental sampling.	• depth typically limited to 1.0 m to 1.2 m without shoring • excavation limited in water bearing or very loose granular soils • breakers required where obstruction/ pavement is present.
Cost factor: low			
Relevance to levees			
• mitigation of the risk of encountering unknown services in advance of primary intrusive investigations.			

Table 7.144 *Trial pits/trenches*

Trial pits/trenches	Method	Applications	Limitations
 *(courtesy Fugro)*	• machine or hand excavated • trial pits typically 3 m long by 0.5 m to 1 m wide by up to about 5 m deep. Trenches may extend to tens of metres • 5 to 10 machine excavated trial pits to 5 m depth in a day, depending on sampling requirements.	• examination of soils *in situ* and obtain large bulk samples for testing • assessment of ease of excavation, stability and groundwater issues • disturbed geotechnical and geoenvironmental sampling • block samples.	• relatively level site with vehicle access required • benefits of depth > 5 m questionable due to instability/health and safety and low light levels at depth • potential adverse influence on existing/proposed infrastructure • excavation limited in water bearing or loose granular soils • breakers required where obstruction/pavement is present • pits should be treated as 'confined spaces'.
Cost factor: low, but shoring/ reinstatement costs may be high			
Relevance to levees			
• provides best opportunity to examine the characteristics of natural soils and made ground or the structure of existing levees. Excavation of trial pits in and/ or at the proposed sites of, levees should take into account their influence on the overall stability of the earthworks. Backfill should ensure that the performance of the levee is not impeded.			

Table 7.145 Hand-held augers

Hand-held augers	Method	Applications	Limitations
 *(courtesy Fugro)* **Cost factor:** low	• light, portable hand-held equipment • rotated or driven into ground by hand • typically 50 mm to 200 mm dia • range of augers available to suit different soils • some methods limited to <1.25 m depth, others may reach 5 m and 8 m • 20 m to 50 m per day achievable.	• rudimentary soil examination and profiling • obtain small disturbed samples • suitable for sensitive locations (eg near noise-sensitive livestock) and for inaccessible sites • suited to very soft to firm clays and granular soil above water table • installation of standpipes.	• not suitable for unstable soils • difficult to progress in stiff clays and gravelly soils • penetration depth limited and small soil samples obtained • production rates affected by operator's fitness • small soil samples obtained.
Relevance to levees			
• useful technique for use in preliminary investigations of proposed levee sites, particularly at sites of limited access. Can be used to pre-bore through desiccated crust over very soft/soft clay to facilitate installation of penetration shear vane.			

Table 7.146 Dynamic (window/windowless) sampling

Dynamic (window/windowless) sampling	Method	Applications	Limitations
 *(courtesy Archway Engineering Ltd)* **Cost factor:** low	• light, portable hand-held or chassis mounted equipment that drives sampling tubes of varying diameter/length into ground to recover a near-continuous soil sample • 35 mm to 100 mm dia • depth 8 m to 15 m • 25 m to 30 m per day achievable • working area 2 m × 4 m required with 3 m to 4 m clear head room.	• sites where unrestricted vehicle access can be gained • tracked mounted and hand-held units may be deployed on sloping/rough terrain or on the crest of levees • obtain disturbed and intact samples • permits SPTs and borehole vane testing • best suited to firm to stiff clay • installation of simple monitoring wells.	• not suitable for granular, very coarse grained soils, very dense or hard soils or made ground containing very coarse or resilient material • poor core recovery in loose granular and very soft soils • vibration and noise associated with the driving process.
Relevance to levees			
• suitable for preliminary investigations of levees, soil profiling and basic sampling and *in situ* testing. Smaller tracked units can be deployed in rough terrain or on the narrow crests of levees. Often combined with dynamic probing.			

Table 7.147 Rotosonic sonic drilling

Rotosonic drilling	Method	Applications	Limitations
 *(courtesy Fugro)* **Cost factor:** medium	• wheeled or tracked mounted equipment which vibrate sampling tubes to recover a near-continuous soil sample • 150 mm to 250 mm dia • depth 100 m+ depending on plant • >75 m per day achievable with large rigs • working area 2 m × 6m required with 4 m to 6.5 m clear head room.	• sites where unrestricted vehicle access can be gained. Tracked mounted rigs may be deployed on sloping/ rough terrain or on the crest of levees • near-continuous disturbed and intact samples • permits SPTs and borehole vane testing • rapid penetration rates • suited to most soil types including dense granular soils and made ground • permits installation of simple monitoring wells.	• limited sample recovery possible in loose granular and very soft soils • vibration and noise associated with the driving process • the use of a 'dry drilling method' can 'bake' soils rendering them unsuitable for testing • some sonic rigs do not have the capacity to insert casing.
Relevance to levees			
• suitable for preliminary investigations of levees, soil profiling, basic sampling/*in situ* testing. Smaller tracked units can be deployed in rough terrain or on the narrow crests of levees. Often the only method of penetrating and obtaining continuous samples of coarse granular deposits.			

Table 7.148 Vibrocore

Vibrocore	Method	Applications	Limitations
 *(courtesy Fugro)* **Cost factor:** medium to high depending on size of survey vessel	• sea/river bed sampling tool deployed from a survey vessel • the vibrocore frame/ vibrator and sampler is lowered to sea/river bed using a suitable deck mounted crane or winch • near-continuous sediment sample obtained • 75 mm and 102 mm dia • three to five 6 m samples per day depending on sea state/bed conditions.	• investigation of sea and river sediments • obtain disturbed and intact geotechnical and geoenvironmental samples.	• not suitable for granular soils that are very coarse grained or very dense, or hard soils • poor core recovery in loose granular and very soft soils • careful selection of vessel required in tidal and intertidal zones • not suitable for use in very shallow water.
Relevance to levees			
• suitable for the investigation of river and sea bed sediments adjacent to sites of proposed or existing levees.			

Table 7.149 Cable percussion (shell and auger) boring

Cable percussion (shell and auger) boring	Method	Applications	Limitations
 *(courtesy Fugro)* **Cost factor:** medium	• heavy tools are repeatedly raised and dropped using a winch to cut, chisel or bail out the soil within the borehole • in unstable soils the borehole is supported by sectional threaded steel casing 150 mm to 300 mm dia • depth 40 m to 60 m+ • 10 m to 20 m per day achievable • collapsible rig is towed behind a suitable vehicle • working area 2 m × 6 m and 7 m clear head room.	• sites where unrestricted vehicle access can be gained and that are relatively level • suitable for all natural superficial soils and weathered rock • can obtain all standard forms of soil samples • all standard *in situ* tests can be undertaken • all standard forms of monitoring well can be constructed in the completed borehole.	• slow progress rates in coarse granular/dense/ hard strata and bedrock • addition of water may be required to penetrate some soils above the water table • drilling disturbance in gravels, silts/sands, very soft clays and weathered rocks may affect the quality of samples and in- situ tests • vibration and noise • difficult to set a rig up on or near sloping ground • control/disposal of arisings.
Relevance to levees			
• a versatile boring technique able to penetrate, sample and test most ground conditions encountered at the sites of levees. Able to install many types of monitoring instruments.			

Table 7.150 Rotary drilling

Rotary drilling	Method	Applications	Limitations
 *(courtesy Fugro)* **Cost factor:** medium to high	• 'multi-function' rotary/sampling rigs that can operate in various modes to suit soil/rock conditions • in soft ground borehole is advanced using various auger systems or driven tube sampling • in hard clays and rock borehole advanced using rotary coring systems • depth 30 m to 50 m (smaller plant) • 10 m to 20 m per day • working area 2 m × 6 m required with 4 m to 5 m clear head room (small plant).	• sites where unrestricted vehicle access can be gained and that are relatively level • some tracked units can operate with slope gradients <30 degrees • suitable for use in most soils from clays to sands or firm to hard clays and rock (in coring mode) • installation of standard monitoring instrumentation • depending on operating mode, most standard forms of sampling and *in situ* tests may be taken.	• performance depends on operating mode adopted • difficulties may occur if made ground, water bearing cohesionless soils or dense, very coarse granular soils are encountered. 'Liquefiable' material may be drawn into hollow stem augers • use of solid stem and helical auger systems will provide disturbed samples only • difficult to install aquifer protection measures • disposal of arisings.
Relevance to levees			
• versatile boring technique able to penetrate, sample and test most ground conditions encountered at the sites of levees. Able to install simple groundwater and ground gas monitoring instruments.			

Table 7.151 Soft ground rotary drilling

Soft ground rotary drilling	Method	Applications	Limitations
Core barrel modified for soft ground drilling  *(courtesy S N Wersching)*	• borehole advanced by rotary drilling with modified core barrels, which incorporate a cross-cut plate on the full diameter • fully contained re-circulated water flush with low exit velocity to remove cuttings and support sides of uncased borehole • depth <20 m • 10 m to 15 m per day • working area to suit form of rotary rig used.	• minimal disturbance of soft sensitive soils ahead of the borehole through elimination of percussive action or water jetting effects inherent with other boring techniques • usually used in combination with piston samplers to obtain a near continuous soil sample • often carried out in conjunction with CPTs.	• only applicable to very soft or soft clays/silts • boreholes cannot be advanced past obstructions or non-cohesive deposits. Other forms of drilling are required where these are encountered • the technique will not by itself guarantee high quality samples and it should be considered in combination with other factors (Section 7.9.8.1).
Cost factor: medium to high			
Relevance to levees			
• levees are often constructed on very soft and soft clays. The method is capable of taking very high quality samples for laboratory testing.			

Table 7.152 Cone penetration tests (CPT)

Cone penetration tests (CPT)	Method	Applications	Limitations
Lorry mounted CPT rig  *(courtesy Fugro)*	• rapid probing tool using an instrumented electric cone pushed directly into ground from surface using a hydraulic penetrometer rig • the penetrometer rig may be mounted on trucks or crawler units, long reach backhoe excavator booms and marine jack-up platforms • 100 m to 150 m per day.	• preliminary soil profiling • data may be used for semi-empirical design methods • derivation of soil parameters using empirical or site specific correlations • piezocones may be used to assess groundwater profile, permeability and soil characteristics • minimal disturbance of sensitive soft soils or water bearing silts/ sands.	• difficult to penetrate coarse granular fill and natural deposits, and rock • the characteristics of some soils and weathered rock are difficult to interpret without control boreholes • does not permit sampling of strata unless used in parallel with a Mostap sampler (Table 7.174).
Cost factor: low, but medium or high if deployed over water			
Relevance to levees			
• as it avoids the disturbance of the ground associated with boring and sampling, it is useful in the characterisation of alluvial clays and silts and water bearing sands. Can be carried out from all terrain plant and over water (Section 7.9.7.6).			

Table 7.153 Dynamic probing

Probing/dynamic probing	Method	Applications	Limitations
 *(courtesy Geotechnical Engineering)* **Cost factor:** low	• steel rod (lead length fitted with a cone-shaped driving shoe) driven or pushed into the ground. The number of blows/ energy required to advance the rod is recorded • generally mounted on tracked or wheeled units • hand-held equipment also available (Mackintosh Probe, TRL Probe) • 20 m to 50 m per day.	• sites where unrestricted vehicle access can be gained and that are relatively level • tracked units can be used in sloping or rough terrain or on crest of levees • simple monitoring wells can be constructed in the completed probe hole • the data from DPL – DPSH tests may be used to derive soil parameters.	• soil being tested cannot be identified • unreliable results in soils containing occasional cobbles or boulders • limited penetration in very dense and hard formations • difficult to penetrate near surface obstructions • dense gravels can grip onto the rods making extraction difficult • vibration and noise.
Relevance to levees			
• suitable for preliminary investigations of levees and soil profiling. Smaller tracked units can be deployed in rough terrain or on the narrow crests of levees.			

7.9.7.6 Intrusive investigations in locations of difficult access

Investigations for existing levees or the sites of new levees may require intrusive investigations to be undertaken over water-logged or marshy ground, on inclined ground, such as the side slopes of the levee, or where there is restricted access and limited space for manoeuvring, such as on very soft foreshore deposits or narrow crests. Some specialist vehicles and equipment are available to undertake investigations in these conditions. However, access to some remote locations may be very difficult, so consideration may have to be given to the use of hand-held portable equipment or using helicopters to lift the equipment to the site.

At some sites intrusive investigations may be required over water or within the intertidal zone. This may require the use of floating pontoons, survey vessels and ships, hovercraft or jack-up drilling platforms.

The additional health and safety requirement, high cost and limited availability of specialised drilling equipment, all terrain plant and plant capable of working over water should be borne in mind during the design of the intrusive investigation.

Where vehicles and plant are required to traffic across river floodplains, adjacent to unprotected riverbanks and along levees, it is good practice to prepare a risk assessment in advance of the investigation. Factors that may be addressed by the risk assessment are, among others:

- the presence and extent of marshy or waterlogged ground
- inclined ground – some wheeled vehicles may not be able to traffic on sloping ground, particularly when it is wet, even shallow slopes of 1 in 20 can cause a problem
- riverbank stability – many are actively eroding and unstable
- remoteness of the site – an evacuation plan may be required
- trafficability of the access route – consider the damage that vehicles could cause
- tidal and storm influenced water levels, which may overtop the levee, restricting access to within a defined period over the tidal cycle.

Various solutions are available, which allow the formation of exploratory holes in areas of difficult access (Tables 7.154 and 7.155). A specialist ground investigation contractor with early involvement will be able to identify where such equipment is considered essential.

Table 7.154 Summary of terrain and types of equipment suitable for gaining access relative to the phase of investigation

Equipment – technique – plant	*Soft or water logged ground*		*Rugged terrain/ access route*		*Confined or restricted access routes*		*Confined or restricted exploration sites*		*Shallow inland waterways, lakes, marsh and mudflats*		*Rivers and estuaries*	
	Reconnaissance SI	*Detailed SI*	*Reconnaissance SI*	*Detailed SI*	*Reconnaissance SI*	*Detailed SI*	*Reconnaissance SI*	*Detailed SI*	*Reconnaissance SI*	*Detailed SI*	*Reconnaissance SI*	*Detailed SI*
Hand-held auger	✓	?	✓	?	✓	?	✓	?				
Hand-held window sampler	✓	?	✓	?	✓	?	✓	?				
Hand-held penetrometer	✓	?	✓	?	✓	?	✓	?				
Mini-track mounted window sampler/dynamic probe	?	?	✓	✓	✓	✓	✓	✓				
Mini-track mounted CPT	?	?	✓	✓	✓	✓	✓	✓				
Window sampler/dynamic probe equipment mounted on low ground pressure tracked chassis	✓	✓	✓	✓	?	?	?	?				
CPT/MOSTAP/vane equipment mounted on low ground pressure track mounted	✓	✓	✓	✓	?	?	?	?				
Mini-track-mounted multi-purpose soft ground rig/rotary rigs	?	?	✓	✓	✓	✓	✓	✓				
Modular cable percussion rigs that may be disassembled into their component parts and carried by hand or on all-terrain vehicles to the drill site	✓	✓	✓	✓	✓	✓	✓	✓				
Conventional drilling plant used in conjunction with temporary access roadways	✓	✓	✓	✓	?	?	?	?				
CPT equipment attached to boom of long reach excavator	✓	✓	✓	✓	✓	✓	✓	✓	?	?		
Conventional rigs set up on scaffold platforms	?	?	?	?	–	–	✓	✓				
Mini track or skid-mounted conventional rigs 'craned' into position or carried by helicopters	?	?	✓	✓	✓	✓	✓	✓				
Mini-track-mounted multi-purpose soft ground rig/rotary rigs mounted on 'slope climbing' tracked chassis	–	–	✓	✓	?	?	?	?				
CPT equipment mounted on long reach excavators working at water's edge									✓	✓		
Hand-held auguring, window sampling, or probing carried out from mini drilling pontoons or small work vessels and hovercrafts									✓	?		
Mini-track mounted window sampler/dynamic probe carried out from mini drilling pontoons									✓	✓		
Mini-track mounted CPT carried out from mini jack-up drilling platform									✓	✓		
Small track or skid-mounted conventional rigs deployed on small drilling pontoons or mini jack-up drilling platforms									✓	✓		
Vibrocoring from small marine survey vessels											✓	?
Seabed CPTs deployed from marine survey vessels											✓	✓
CPTs/MOSTAP sampling/vanes carried out from jack-up drilling platform									?	?	✓	✓
Conventional drilling and sampling equipment deployed from modular pontoons and jack-up barges (subject to water depth, currents, weather etc)								?	✓	✓	✓	✓
Drill ships (very expensive)											✓	✓

Notes

✓ denotes technique may generally be used to good effect subject to favourable site conditions and/or soil profile

? denotes technique may be used but to limited effect and subject to site conditions and/or soil sequence

– denotes technique is not suitable

1 2 3 4 5 6 7 8 9 10

Table 7.155 Summary of methods, and types of equipment and plant capable of accessing difficult locations

Modular cable percussion rigs (courtesy Fugro)

Modular cable percussion rigs may be disassembled into their component parts and then carried by hand or on all-terrain vehicles to the drill site

Cable percussion rig on a scaffold platform (courtesy Fugro)

Conventional rigs may be set up on scaffold platforms to allow boreholes to be constructed on sloping ground. Suitably designed 'cantilevered' scaffold structures may allow boreholes to be carried out over water

Low ground bearing pressure crawler rigs (courtesy Fugro)

Multi-function rigs, rotary drilling rigs and percussive (window sampler/dynamic probing) rigs can be mounted on all terrain tractor units or low ground bearing pressure crawler units, which can traverse soft and/or inclined ground

Automatic levelling working platform and mast (courtesy Geotechnical Engineering Ltd)

Some drilling rigs are specially adapted to work on steep slopes up to 45°, such as levees. The working platform and drilling mast are automatically levelled in both planes to allow safe working. Various systems are used to anchor the drill unit to the ground during drilling

Low ground bearing pressure CPT rig (courtesy Fugro)

CPT systems can be mounted on a range of low ground bearing pressure tracked units to suit a range of ground and access conditions

Mini crawler mounted CPT rams (courtesy Fugro)

CPT ram sets can be attached to a mini crawler unit, which uses auger pickets to provide the reaction during penetration

CPT rams on long reach excavator (courtesy Lankelma)

A CPT ram sets can be mounted on a long reach excavators, which can be positioned on the crest of the levee or other stable ground, allow CPT sounding to be taken at adjacent locations were the access of conventional equipment would not be possible.

CPT rams on long reach excavator (courtesy Fugro)

Over water drilling platforms (a) floating pontoon, (b) jack-up rig, and (c) drilling ship (courtesy Fugro)

For over water work, drilling rigs can be mounted on purpose built modular barges, drill ships and platforms. The cost of these units and support vessels can be very high, particularly for work in deep water, tidal water, or where there are significant waves or current.

7.9.7.7 Backfilling and reinstatement

As levees are foremost water retaining structures, the backfilling of any exploration hole should be done in a manner that will not have an adverse effect on the performance of the levee. A poorly backfilled borehole may provide a pathway for near surface contaminants to migrate into an aquifer, or allow seepage or upward flow from underlying more permeable strata during a high water event. In addition there is the issue of ground subsidence and the hazard this presents to livestock and the general public.

For hand and machine excavated pits it is preferable to separate the materials during excavation so that they can be replaced and compacted in layers in the order they were excavated. Boreholes, which do not require installations, can be backfilled with a grout or other suitable material, such as bentonite or bentonite/cement pellets. Probe holes are not usually backfilled but there is merit in endeavouring to pour grout into the small void where a residual void is seen as critical to the performance of the levee.

Where installations are required in the boreholes their long-term influence on the performance of the levee should be considered. It may be necessary to return to site to cap, grout or remove them at a later date to ensure the integrity of the levee.

Grouts generally comprise a ‘pumpable’ mix of water, bentonite and cement. The strength of the grout can be designed similar to the surrounding ground by adjusting the proportions of cement and bentonite, and the water:cement ratio. For backfilling and grouting of installations see Section 7.9.9.11. For details of indicative grout mixes see Box 7.39.

7.9.7.8 Special requirements and considerations

Working close to water

Levees by their nature can be located close to a water body. Working close to water presents particular issues with intrusive investigations. These are:

- access to and setting up at exploratory hole locations
- potential damage to ecologically sensitive habitats
- pollution of the water body associated with vandalism, refuelling, lubricants, exhaust, discharge of silt, arisings, drill flush and backfill grout
- health and safety of operatives: working over water, drowning, welfare, PPE, working on sloping embankments and foreshores, soft or unstable ground, disease, dangers associated with lone working, dangers associated with wildlife, contamination and ground gas
- disposal of arisings and waste
- working close to the general public, livestock, or shipping.

Ground stability

During the walkover survey and site inspection, riverbanks and quay walls may appear to be stable, however the movement of plant, vibration from drilling activities and additional surcharge loads from spoil heaps and equipment may affect their stability. Care should be taken in locating exploratory holes, site compounds and access routes adjacent to water bodies.

Contamination

Urban river fronts may have been modified by humans over hundreds of years. Obstructions may be encountered in exploratory holes to a considerable depth. These may range from timber piles to outfall structures and infilled docks. Riverside developments such as docks and wharfs, and processing facilities and refineries, may contain high levels of contamination and due regard should be given to the selection of appropriate PPE.

Aquifer protection measures may be required to prevent pollution of aquifers by intrusive works. Measures may also be required to prevent cross contamination between boreholes. For example drilling and sampling equipment may require steam cleaning or jet washing before moving on to the next borehole location.

UXO

A UXO risk assessment should be carried out for sites located close to major cities and industrial plants located within areas previously affected by conflict, also existing and/or historical defence sites. Open countryside may have previously been the site of military activities.

Flora and fauna

Vegetated lake and riverbank habitats can contain a diverse and abundant community of flora and fauna. They may be important sites for fish reproduction and also provide a habitat for a number of protected species. It is essential that all practical measures are taken to reduce the impact on these habitats, particularly in the growing/breeding seasons. Disruption of the habitat may be caused by excessive noise and vibration, removal of vegetation or the illegal discharge of silt and contaminants into the water body.

An exclusion zone may be required around some protected and invasive species.

Biohazards

Health hazards from water borne contaminants could be present. These hazards may derive from overflow from sewage treatment plants, or from nearby agricultural or industrial premises, and may include the following:

- hepatitis – the virus is present in faeces, which can be present in water courses
- gastroenteritis – from ingestion of sewage bacteria
- cholera – an infectious gastroenteritis caused by the bacterium 'vibrio cholerae'
- blue green algae – an algae found in fresh water in the summer months, blooms grow on the surface, which is a blue-green colour, and can be toxic
- leptospirosis (Weil's disease) – a bacterial infection caused by rat urine.

Biogenic ground gas may be present in alluvial deposits and at elevated concentrations may represent a risk to site staff. Gas monitoring should be carried out at regular intervals during boring and smoking banned at the location of exploration holes.

Further reading

There are a number of books and publications, which will provide useful reading:

Butcher *et al* (1995) *Dynamic probing and its use in clay soils*

Clayton *et al* (1995) *Site investigation*

Hvorslev, M J (1949) *Subsurface exploration and sampling of soils for civil engineering purposes*

Lunne *et al* (1997) *Cone penetration testing in geotechnical practice*

Meigh (1987) **Cone penetration testing: methods and interpretation**

Steeds, J E, Slade, H J and Reed, W M (2000) ***Technical aspects of site investigation***

Stenzel and Meiser (1978) *Soil investigations by penetration testing according to DIN 4094*

USACE (2001) *Engineering geotechnical investigations*

US Department of the Interior, Bureau of Reclamation (1998) *Earth Manual. Part 1, third edition*

Standards

BS 5930:1999+A2:2010 *Code of practice for site investigations*

BS EN 1997-2:2007 *Eurocode 7 Geotechnical design. Ground investigation and testing*

DIN 4021:1990-10 *Ground exploration by excavation, boring and sampling*

7.9.8 Sampling methods

The majority of soil properties and engineering parameters used in the design and assessment of levees are derived from laboratory tests on samples obtained from intrusive investigations. So, the samples obtained need to be of sufficient quantity and quality to be representative of the *in situ* soils from which they are obtained. This section considers the factors affecting the selection of sample quality, size and frequency of sampling, and presents the applications and limitations of some common sampling methods used during intrusive investigations for levees. The methods presented represent techniques that are not the primary means of advancing the exploration hole.

In addition to soil samples, samples of the groundwater and water body may be required to evaluate chemical content as part of a contamination study and to assess the aggressiveness of the water to construction materials.

7.9.8.1 Selection of sampling techniques

The selection of sampling methods should reflect the type of soil to be sampled and the form of testing to be undertaken. Cohesive soils are more suited to being sampled by methods that result in limited sample disturbance and so produce higher quality samples. Soft clays that commonly form the foundation soils for levees require high quality samples for laboratory testing to assess their strength and consolidation characteristics. Non-cohesive soils are not suited to high quality sampling due to their lack of cohesion. However, the geotechnical industry has developed empirical correlations (Section 7.8.1.4) with soil characteristics derived from large scale laboratory or *in situ* tests. By contrast, disturbed or de-structured samples may be adequate as 'record samples' for the production of descriptive logs and for undertaking index tests.

The quality of a sample can be classified in different ways. In the USA, samples are often categorised very generally as disturbed and undisturbed, and high quality testing requires undisturbed samples. BS EN ISO 22475-1:2006 defines five sample quality classes, 1 to 5, where 1 is the best. By contrast, sampling methods have been divided into three sampling categories, A to C, where A is the highest quality. The general form that a sample may take and the correlation between sample class and sample category is presented in Table 7.156.

Table 7.156 Description of sample categories (after EN ISO 22475-1:2006)

Sample class	Typical sample forms and description of the sample category	
1	Open tube – intact samples	**Category A:** no to slight disturbance of soil structure during sampling or in handling. Moisture content and voids ratio correspond with *in situ*. No change in constituent and chemical composition has occurred
2		**Category B:** contains all the constituents of the *in situ* soils and retains the natural moisture content. General arrangement of the different soil layers can be identified but the structure of the soil has been disturbed
3		
4	From drilling tools – disturbed samples	Not within the sample categories. No indicative description provided in BS EN ISO 22475-1:2006
5		**Category C:** soil structure totally changed. General arrangement of soil layers modified so that *in situ* layers cannot be identified. Water content may not be representative of *in situ* soil

BS EN 1997-2:2007 provides indications of the minimum sample class required to establish particular soil properties. This is summarised in Table 7.157, together with the sample category.

Table 7.157 Sample class and category required for assessment of soil properties (after BS EN 1997-2:2007)

Soil properties and sample class	1	2	3	4	5
Unchanged soil property					
Particle (grain) size	*	*	*	*	
Water content	*	*	*		
Density, density index, permeability	*	*			
Compressibility, shear strength	*				
Property that can be determined					
Sequence of layers	*	*	*	*	*
Boundary of strata – broad	*	*	*	*	
Boundary of strata – fine	*	*			
Atterberg limits, particle density, organic content	*	*	*	*	
Water content	*	*	*		
Density, density index, porosity, permeability	*	*			
Compressibility, shear strength	*				
Sampling category according to BS EN ISO 22475-1:2006	A				
			B		
					C

The quality of a sample, and ultimately the specimen prepared from the sample for testing, is not solely a function of the sampling method. When attempting to obtain the highest quality samples in sensitive soils such as very soft to soft clays, the level of sample disturbance can be a function of a number of other related factors. Some of these are summarised in Table 7.158.

Table 7.158 Causes of sample disturbance and methods of minimising them

Factors affecting sample disturbance	Method of minimising sources of sample disturbance
Method of advancing the exploration hole	Aim to avoid percussive or vibratory methods of advancing the exploration hole in soft sensitive soils. Advance the exploration hole by rotary methods with low flush velocities, or augers, to limit disturbance at the base of the exploration hole
Method of sampling	Use large diameter thin wall samples (eg piston or Sherbrook sampler) or block samples Use push sampler methods rather than driven
Handling and transportation	Maintain samples vertical. Hand carry short distances over rough terrain from exploration hole to vehicle pick-up point, if necessary. Place samples in wooden or a similar container, with cushioning to avoid knocks and jolts. Place additional cushioning under sample containers. Transport at a speed appropriate to the terrain in a soft sprung vehicle
Specimen preparation	Prepare specimens for mechanical laboratory tests from the central section of the sample. Extruded samples in the same direction the samples as taken. Hand trim the specimen to the required size

All of these factors are important and a lack of due diligence in one element of the chain of events can reduce the quality of the test result.

Groundwater samples should be representative of the strata from which they are taken. By their nature, they are taken from the more permeable strata where seepage is encountered during the investigations, from standpipes installed as part of the investigation or from the open water body. This may be achieved by direct collection from the seepage in trial pits, having considered the stability of the sidewalls. In boreholes, casing may be required to prevent entry of water from strata higher in the borehole. Water in the borehole should be purged several times until consistent *in situ* monitored characteristics are recorded before a sample is taken. To preserve the integrity of the sample it should fill the inert container in which it is placed and be stored in the cold and dark, and tested promptly.

7.9.8.2 Sample size

The size and mass of the sample should reflect the nature of the material to be sampled, the sampling method and the tests to be performed in order to achieve test results that are representative of the *in situ* soil mass.

For open tube samples in cohesive soils, sample disturbance reduces with increasing sample diameter. The absolute minimum sample diameter for undisturbed specimens is 50 mm. The sample should ideally be equal to or larger than the intended test specimen. Larger samples allow smaller diameter specimens to be taken from the material within the central section of the sample, away from the zones of disturbance at either end, and the disturbed material around the circumference of the samples. Nominally 100 mm diameter samples are adequate for routine testing purposes but large diameter samples can be obtained and may be appropriate where very high quality, specialist testing is to be undertaken. Some laboratory equipment is directly compatible with the diameter of open tube samples, simplifying the transition from sampling to specimen preparation. While it is preferable to prepare samples for consolidation and effective stress testing at a smaller diameter than the sample size, undrained triaxial tests can generally be performed on specimens prepared at the full sample diameter.

For bulk samples the mass of soil required is a function of the test and maximum particle size. The masses presented in Table 7.159 are only intended as an indication.

Table 7.159 Mass of soil required for various laboratory tests (after Table 3, BS 5930:1999+A2:2010, Table 5, BS 1377-1:1990, Head (1984), and ASTM D 2487-11)

Purpose of sample	Soil classification	Mass of sample required (kg)
Soil identification, including Atterberg limits, sieve analysis[a], moisture content and sulfate content tests	Clay, silt, sand	1
	Fine and medium gravel	5
	Coarse gravel	30[a]
Compaction tests	Up to medium gravel	10 (25)
	Including some coarse gravel	50 (80)
Maximum and minimum density	Sand and fine and medium gravel	16
	Coarse gravel	30
Permeability	All	3.5–8
Erosion/dispersion	Clay, silt, sand	0.5[b]

Notes

[a] ASTM D 2487 provides further guidance on minimum dry weight of sample required based on maximum particle size for sieve analyses

[b] for soils with larger particle/grain sizes the mass should be about 100 times the mass of the largest particle

[c] for soil with particle/grain sizes greater then sand, the sample should be sufficient to give the stated mass of fine-grained material

() increased mass for soils susceptible to crushing during compaction

For water samples a nominal quantity of one litre is usually adequate for testing. However, where more stringent testing is needed, multiple samples may be required. For contamination testing the advice of a specialist should be obtained.

7.9.8.3 Sampling frequency

The sampling frequency should take into account the level of information already available, the technical objectives of the investigation and number of tests required to reasonably define the properties of the stratum. It should also consider the size of the investigation, the vertical variability of the ground profile, the anticipated levee failure and deterioration mode, and the nature of the works to be undertaken.

It is preferable to have the samples spaced closer together within the depth range where there is greatest variability in the properties of the ground and the influence of the imposed load is most pronounced. Where levees are founded on soft clays with a desiccated crust, the greatest variability in soil properties is likely to occur within the upper 5 m or so of the soil profile, which is also likely to include the zone of minimum undrained strength and greatest compressibility. Continuous sampling could be justified within this zone. Below this level the undrained strength is likely to progressively increase with depth and the compressibility reduce. So, the sample spacing could be increased at depth. Where CPT soundings have previously been undertaken as part of an earlier phase of investigation adjacent to or close to the borehole, the results can be used to establish the sampling frequency and to identify any critical elevations within the soil profile where samples should be targeted.

For small investigations undertaken for minor remedial works on the levee and that require only one or two exploration holes with little or no pre-existing information on ground conditions, sampling could be undertaken at close centres, if not continuous or alternating continuously with *in situ* testing, to ensure the information captured by the investigation is maximised. In highly variable ground it may be necessary to sink two adjacent exploration holes – one for *in situ* testing and the second for continuous sampling.

It is preferable to take samples within the zone of an *in situ* test for material identification purposes and to allow empirical modification factors to be applied to the results. The results can also be compared with established empirical correlations derived from classification tests (Section 7.8).

Routine samples are usually taken at each change in stratum and then at set intervals, not usually more than 3 m (BS EN1997-2:2007) but 1 m to 1.5 m is more usual.

7.9.8.4 Sample labelling, handling, transportation and storage

The labelling, handling, transportation and storage of samples are important factors if details of its origins and condition immediately after sampling are to be preserved.

As soon as a sample has been recovered and preserved it should be numbered, recorded, labelled and, if appropriate, marked to indicate the top of the sample. The label may record:

- project name
- exploration hole number
- sampling date
- sample type and category
- depth or depth range.

The measures adopted for sample preservation and transportation can be a function of the sample category, with category A samples requiring the greatest attention. The general principle is that samples may need to be sealed within a container to preserve the moisture content and to protect it from vibration and extremes of temperature to prevent disturbance. Soil and water samples required for chemical testing may need to be preserved in dark conditions and at low temperature but not frozen. Indications of the measures that may be required are detailed in Table 7.160, together with those previously considered in Table 7.158.

Table 7.160 Sample category, preservation and transportation requirements (from BS EN ISO 22475-1:2006)

<table>
<tr><th>Sample categories</th><th>Preservation requirements</th><th>Transportation requirements</th></tr>
<tr><td>A</td><td rowspan="2">Preservation of moisture content:
• place a plastic bag tightly around the sample to the exclusion of air and seal with ties
• tube samples: seal ends with microcrystalline wax. Fill ends with packer material and place on end caps
• block type samples: wrap in foil or film (or muslin) and coat in several layers of wax.</td><td>Protection of samples from vibration, shocks and extreme temperatures:
• place samples in sturdy boxes containing cushioning material (ie sawdust, rubber, polystyrene, urethane foam, bubble wrap) to prevent movement, protect from external jolts and provide insulation.</td></tr>
<tr><td>B</td><td>Protection of sample containers from failure/breakage and maintain preservation of moisture content</td></tr>
<tr><td>C</td><td>Place in water tight container to the exclusion of air if preservation of moisture content is required</td><td>Place in any clean container to suit available method of transport</td></tr>
</table>

Samples should be stored so that they are safe and secure, and protected from knocks and bumps. Unnecessary handling should be avoided. The storage environment should match that of the ground, which may be +6°C to +12°C at a humidity of 85 to 100 per cent in a temperate climate.

7.9.8.5 Sampling methods

Levee investigations may typically involve one of two broad categories of soil sampling – bulk and open tube samples, and groundwater samples. The typical methods of intrusive investigation by which the samples can be obtained are summarised in Table 7.161. Each of the sample types is then further described in a series of tables as shown.

1 2 3 4 5 6 7 8 9 10

Table 7.161 Principal samples types and associated intrusive investigation methods

Sample categorise		Type of sample	Table number	Form of intrusive investigation for sampling		
				Pits	*Boreholes*	*Direct penetration*
Bulk		Small disturbed	**7.162**	*	*	
		Bulk disturbed	**7.163**	*	*	
		Grab samples	**7.164**			*
		Block samples	**7.165**	*		
Open tube	Thick wall – driven	Split spoon (SPT)	**7.166**		*	
		UT100 samplers	**7.167**		*	
		U100 samplers	**7.168**		*	
	Thin wall – pushed	Shelby	**7.169**		*	
		Piston	**7.170**		*	*
		Pitcher sampler	**7.171**		*	
		Laval	**7.172**		*	
	Others	Sherbrook	**7.173**		*	
		Mostap	**7.174**		*	*
Groundwater[1]			**7.175**	*	*	*

Note

1 Groundwater samples may also be obtained from bodies of open water and surface seepages.

In order to choose an appropriate sampling method, there is a need to be aware of their applications and limitations. Tables 7.162 to 7.175 provide a summary of the key features of commonly used sampling methods. Documents for futher reading are included at the end of the section.

Bulk samples

Bulk samples can range from a small disturbed sample to a high quality block sample. Disturbed bulk samples are often obtained from open excavations and boreholes, as a quick low cost method to determine soil classification. They also often provide samples for soil classification tests and are used by the logging engineer in the field to classify the soils.

Table 7.162 Small disturbed samples

Small disturbed samples		Method	Applications	Limitations
 *(courtesy Soil Property Testing Ltd)*		• collected from open face or spoil of excavation, test pits, cutting shoes from open tube samples, split spoon of SPT • size 1 kg/1 lt • samples placed in small plastic bags or filling 1 lt plastic tubs.	• fine grained soils – classification test (PSD, Atterberg limits and, when in and filling a sealed tub, moisture content) and soil identification • coarse grained soils – soil identification.	• disturbed sample and can only be used to assess a limited range of soil properties • unrepresentative of coarse grained soils with larger particle sizes due to limited sample size.
Quality class	**4/5**			
Sampling category	C			
Relevance to levees				
• routine sampling for soil identification and index classification.				

Table 7.163 Bulk disturbed samples

Bulk disturbed samples	Method	Applications	Limitations
 *(courtesy Soil Property Testing Ltd)* **Quality class**: 4/5 **Sampling category**: C	• collected from open face or spoil of excavations, test pits, drill or boring cuttings • size to suit testing, multiple samples required where large particle sizes are present • samples placed in large thick bags or large sealable containers.	• essentially any test requiring a large disturbed sample • fine grained soils – classification (PSD, Atterberg limits) and compaction related tests • coarse grained soils – classification (PSD), compaction related tests, shear box, permeability.	• disturbed sample and can only be used to assess a limited range of soil properties • use for moisture content only if sample fills the sealable container.
Relevance to levees			
• routine sampling for earthworks assessment and PSD in coarse grained soils.			

Table 7.164 Grab samples

Grab samples	Method	Applications	Limitations
 *(courtesy Fugro)* **Quality class**: 5 **Sampling category**: C	• lowered through the water. Steel clamshells close on reaching bed and grab a surface sample • volumes range from a few litres to a cubic metre and can operate in water depths up to 200 m where sampling rate could be three to four per hour.	• sampling the bed of water bodies • in rivers a small grab sampler may be used where bed is mud and sands, and can be deployed from riverbanks or overbridges.	• not suitable where bed deposits contain a high proportion of very coarse material • clays and dense granular deposits require a larger hydraulic grab.
Relevance to levees			
• sampling of surface bed deposits for morphological studies and assessment of PSD.			

Table 7.165 Block sampling

Block sampling	Method	Applications	Limitations
 *(courtsey Geotechnical Consulting Group)* **Quality class**: 1 **Sampling category**: A	• collected from test pits by cutting and trimming a pedestal of soil. Seal surface with wax and muslin. Place sturdy box over sample, pack voids and cut base from *in situ* soil • size to suit.	• tests requiring larger high quality samples • fissured or sheared soils where known orientation is required during test.	• limited to shallow depths above groundwater • soil needs to have sufficient cohesion to allow it to be trimmed in the field • expensive • disturbance due to stress relief if carried out in hand dug and/or shored pits.
Relevance to levees			
• testing shear strength on levee slopes with significant desiccation or shrinkage cracks, or soils containing pre-existing shear surface.			

Open tube

A variety of sampling methods can be used in boreholes and their applications vary depending upon the nature of the subsurface material, quality of sample needed and type of drilling equipment. Samples in boreholes are typically classified as driven or push samplers and rotary cut or core samplers.

Table 7.166 Split spoon (standard penetration test, SPT)

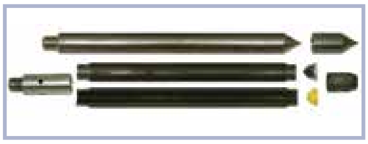
<table>
<tr><th colspan="2">Split spoon (standard penetration test, SPT)</th><th>Method</th><th>Applications</th><th>Limitations</th></tr>
<tr><td colspan="2">
(courtesy Archway Engineering (UK) Ltd)</td><td rowspan="3">• dynamically driven into base of borehole. Driving blows recorded
• sampler and sample split longitudinally to examine and collect disturbed samples
• size: 35 mm diameter samples, 450 mm long split barrel.</td><td rowspan="3">• rugged and can recover samples in most soil types, including dense sands and stiff clays
• blow counts correlated with soil properties.</td><td rowspan="3">• high area ratio (>20 per cent), sample not high quality
• debris in base of borehole will enter sampler
• low sampler recovery ratio
• obstructed by very coarse material
• unsuitable in medium/coarse gravels.</td></tr>
<tr><td>Quality class</td><td>4/5</td></tr>
<tr><td>Sampling category</td><td>B</td></tr>
<tr><td colspan="5">*Relevance to levees*</td></tr>
<tr><td colspan="5">• can be used to obtain a small sample of cohesive or non-cohesive soil
• SPT *N* value can be correlated to sample strength and density for slope stability analyses
• split spoon may be used to investigate levee foundation material and identify locations for high quality undisturbed samples.</td></tr>
</table>

Table 7.167 UT100

<table>
<tr><th colspan="2">UT100 sampler</th><th>Method</th><th>Applications</th><th>Limitations</th></tr>
<tr><td colspan="2">
(courtesy Fugro)</td><td rowspan="3">• dynamically driven into base of borehole. Driving blows recorded
• area ratio: 15 per cent
• size: 100 mm diameter by 450 mm long.</td><td rowspan="3">• soft to stiff clays with limited coarse material
• strength and compressibility tests
• sampling medium dense sands above the water table to provide as Category B sample.</td><td rowspan="3">• more expensive than U100
• debris in base of borehole will enter sampler
• may be damaged in soils containing coarse particles, weathered rock or extremely high strength clays.</td></tr>
<tr><td>Quality class</td><td>1/2</td></tr>
<tr><td>Sampling category</td><td>A</td></tr>
<tr><td colspan="5">*Relevance to levees*</td></tr>
<tr><td colspan="5">• assessing properties of levee and foundation soils where they comprise soft to firm clays.</td></tr>
</table>

Table 7.168 U100

U100 sampler		Method	Applications	Limitations
 *(courtesy Archway Engineering (UK) Ltd)*		• dynamically driven into base of borehole. Driving blows recorded • can contain rigid plastic liner • area ratio: 25 to 30 per cent (47 per cent with liner) • size: 100 mm diameter by 450 mm • can include a core catcher.	• insensitive soft to stiff clay • where a robust open tube sampler is required • suitable for sands and gravel above the water table.	• sample disturbance due to high area ratio and dynamic installation • debris in base of borehole will enter sampler • can give low sample recovery ratio • used with caution for strength and compressibility tests.
Quality class	3–5			
Sampling category	B			
Relevance to levees				
• this sampling technique can be used to obtain samples of firm to stiff cohesive and stony cohesive soils for laboratory testing.				

Table 7.169 Shelby tube

Shelby tube sampler		Method	Applications	Limitations
 *(courtesy Fugro)*		• a thin walled sample tube, typically with a bevelled edge to allow for cutting of soil during sampling • sampler attached to the drill rod and pushed hydraulically in base of borehole • 50 mm to 125 mm diameter × 0.9 m to 1.4 m long • area ratio (10 per cent).	• soft to stiff clays where high quality samples are required for testing or ground profiling by taking continuous samples • strength and compressibility tests.	• easily damaged in hard or dense materials • expensive due to additional care and time involved in retrieving the samples • piston samplers (Table 7.170) perform better in very soft/ soft or sensitive soils.
Quality class	1/2			
Sampling category	A			
Relevance to levees				
• used for retrieving undisturbed samples in cohesive material forming the levee or foundation soils for visual description, strength, permeability and compressibility tests.				

Table 7.170 *Piston sampler*

Piston sampler		**Method**	**Applications**	**Limitations**
 *(courtesy Geonor)*		• pushed into base of borehole (rotary drilled) in smooth action • piston fixed in position • area ratio: seven per cent • standard tube is 100 mm diameter (range 54 mm to 250 mm) × 1 m • aluminium or stainless steel.	• very soft to soft sensitive clays, firm clay, silt and amorphous peat • good sample retention • strength and compressibility tests.	• expensive relative to other open tube samples • maintenance of sampling equipment (ie piston seal, cutting edge and roundness of tube section) • good post sampling handling of tubes needed.
Quality class	1/2			
Sampling category	A			
Relevance to levees				
• used for retrieving undisturbed samples in very soft/soft or sensitive cohesive material forming the levee or foundation soils for visual description (profiling), strength, permeability and compressibility tests.				

Table 7.171 *Pitcher sampler*

Pitcher sampler		**Method**	**Applications**	**Limitations**
 *(courtesy URS)*		• rotary drilled in base of borehole • inner spring loaded thin walled sampling tube retracts when hard material encountered and extends in soft ground to protect samples • 60 mm to 150 mm diameter × 0.9 m long.	• sampling in most soils and suitable for stiff clays or dense materials that can damage other thin walled samplers • good sample retention • strength and compressibility tests • soil with alternate hard and soft layers.	• expensive relative to other open tube samples • prone to operational errors • not suitable for gravels or loose sands • tubes may be damaged in interbedded coarse granular and cohesive deposits.
Quality class	1 to 4			
Sampling category	A/B			
Relevance to levees				
• used for retrieving samples from cohesive and some non-cohesive levee and foundation material.				

Table 7.172 Laval sampler

<table>
<tr><th colspan="2">Laval sampler</th><th>Method</th><th>Applications</th><th>Limitations</th></tr>
<tr><td colspan="2">
(courtesy David Height)</td><td rowspan="3">• pushed into base of 400 mm diameter rotary drilled borehole in smooth action
• sample tube over cored by integral rotary cutter with mud flush
• extruded on site and sub samples waxed
• area ratio: five per cent
• tube 200 mm diameter by 600 mm long.</td><td rowspan="3">• soft and sensitive clays
• quality almost comparable with block sampling
• good sample retention
• strength and compressibility tests.</td><td rowspan="3">• expensive and slow process cannot be justified for routine investigation
• specialist operator needed
• good post sampling handling of subsamples required
• not suitable in cohesive soils with high gravel content.</td></tr>
<tr><td>Quality class</td><td>1/2</td></tr>
<tr><td>Sampling category</td><td>A</td></tr>
<tr><td colspan="5">Relevance to levees</td></tr>
<tr><td colspan="5">• used for retrieving undisturbed samples from cohesive levee and foundation material particularly suited for soft and sensitive clays for strength, permeability and compressibility tests.</td></tr>
</table>

Other forms of sampler

A number of other sampling methods are available to address specialised sampling needs or to deal with site specific constraints. Details of some of these sampling methods are provided below.

Table 7.173 Sherbrook sampler

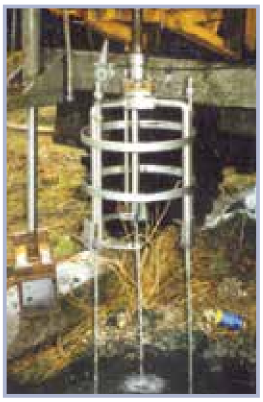
<table>
<tr><th colspan="2">Sherbrook sampler</th><th>Method</th><th>Applications</th><th>Limitations</th></tr>
<tr><td colspan="2">
(courtesy David Height)</td><td rowspan="3">• rotary drilled borehole 400 mm diameter supported by mud
• 350 mm high by 250 mm diameter soil cylinder cut by rotating blades
• horizontal rotating blades cut sample free from in situ soil
• sample waxed.</td><td rowspan="3">• soft and sensitive clays
• quality comparable with block sampling
• no coarse material in soil
• strength and compressibility tests.</td><td rowspan="3">• very expensive relative to other methods
• sampling cycle takes three hours, including packing for transportation
• specialist operator needed
• good post sampling handling needed
• not suitable for cohesive soils with high gravel content.</td></tr>
<tr><td>Quality class</td><td>1</td></tr>
<tr><td>Sampling category</td><td>A</td></tr>
<tr><td colspan="5">Relevance to levees</td></tr>
<tr><td colspan="5">• suitable for collecting larger diameter high quality undisturbed samples for strength, compressibility and permeability tests. These samples can also be used for conducting EFA (erosion function apparatus) tests for estimating erodibility of soil samples.</td></tr>
</table>

Table 7.174 Mostap sampler

Mostap sampler	Method	Applications	Limitations
 *(courtesy Geonor)* **Quality class** 2/3 **Sampling category** B	• CPT rig used to penetrate sampler • end cone prevents entry of soil during penetration • cone unlocked at sample depth • soil enters tube during penetration as cone retracts • sample collected in stockinet in liner tube • 35 mm and 65 mm diameter × 1.0 m, 1.5 m, and 2 m long.	• no borehole needed • samples taken at targeted depths • correlations with adjacent CPT profile • index and chemical testing.	• difficulty penetrating coarse dense granular soils and stiff clays • penetration stopped by obstructions • smaller diameter samples are usually of lower class and larger diameter samples have difficulty penetrating dense soils.
Relevance to levees			
• provides physical samples for testing and profiling of the ground, or at targeted locations, as a benchmark reference for adjacent CPT soundings and to clarify anomalous CPT data as part of a feasibility investigation.			

Groundwater samples

Samples of groundwater may be required during the course of an investigation for chemical analysis.

Table 7.175 Groundwater samples

Water sampler	Method	Applications	Limitations
 *(courtesy Soil Property Testing Ltd)* **Quality class** Not applicable **Sampling category** Not applicable	• bailer or pump sample from borehole/well. or CPT using BAT piezometer system • extract water until parameters stabilise • 40 ml vial for volatile organic compound or 1 lt for other tests • use laboratory cleaned inert containers • sample stored at ≤4°C in the dark • test within days.	• determination of levels of liquid and water soluble contaminates • levels of water soluble chemicals that are aggressive to construction materials • measurements of *in situ* temperature and conductivity data may help differentiate between seepage and groundwater.	• can be difficult to achieve representative sample • installation of monitoring well required to target strata for longer term monitoring of contamination • decontamination of sampling equipment needed to avoid cross-contamination • requires advance co-ordination with laboratory.
Relevance to levees			
• assess whether chemicals in the groundwater or adjacent water body will result in accelerated deterioration of structures within the levee and associated works • determine levels of potentially mobile contaminates in the ground and the influence that the levee may have on their movement.			

Further reading

There are a number of books and publications, which will provide useful reading:

Acker (1974) *Basic procedures for soil sampling and core drilling*

Arnold (1993) *Flachbohrtechnik*

Australian Drilling Industry Training Committee Ltd (ed) (1997) *Drilling – the manual of methods, applications and management*

Chugh (1992) *High technology in drilling and exploration*

Clayton *et al* (1995) *Site investigation*

Hvorslev (1949) *Subsurface exploration and sampling of soils for civil engineering purposes*

USACE (2000) *Engineering and design – design and construction of levees*

USACE (2001) *Engineering geotechnical investigations*

Standards

ASTM D7015-07 (2007) *Standard practices for obtaining intact block (cubical and cylindrical) samples of soils*

ASTM D1586-11 (2011) *Standard test method for standard penetration test (SPT) and split-barrel sampling of soils*

ASTM D2487-11 (2011) *Standard practice for classification of soils for engineering purposes (unified soil classification system)*

ASTM D1587-08 (2012) *e1 Standard practice for thin-walled tube sampling of soils for geotechnical purposes*

BS EN ISO 22475-1:2006 *Geotechnical investigation and testing. Sampling methods and groundwater measurements. Technical principles for execution*

ISO 10381-1:2002, ISO 10381-2:2002, ISO 10381-3:2001, ISO 10381-4:2003, ISO 10381-5:2005, ISO 10381-6:2009, ISO 10381-7:2005, ISO 10381-8:2006 *Soil quality – sampling*

ISO 5667-1:2006 *Water quality – sampling – Part 1: Guidance on the design of sampling programmes and sampling techniques*

7.9.9 Field instrumentation and monitoring

The construction or improvement of a levee or the application of direct or hydraulic loads will cause a change in boundary conditions to which the soils will respond. There are situations where it is advantageous to quantify these changes to better understand the:

- condition of the levee
- interaction of the levee with the environs
- provide construction control
- validate the design and the assumptions
- assess or validate the ground characteristics.

The common ground responses measured in the case of levees are:

- displacements (lateral and vertical)
- pore water pressures.

There are a number of forms of instrumentation available that will quantify these responses. Details of some of these are presented in Section 7.9.9.6.

Other soil and water related measurements may include:

- total soil stresses
- seepage discharge
- soil/groundwater temperature.

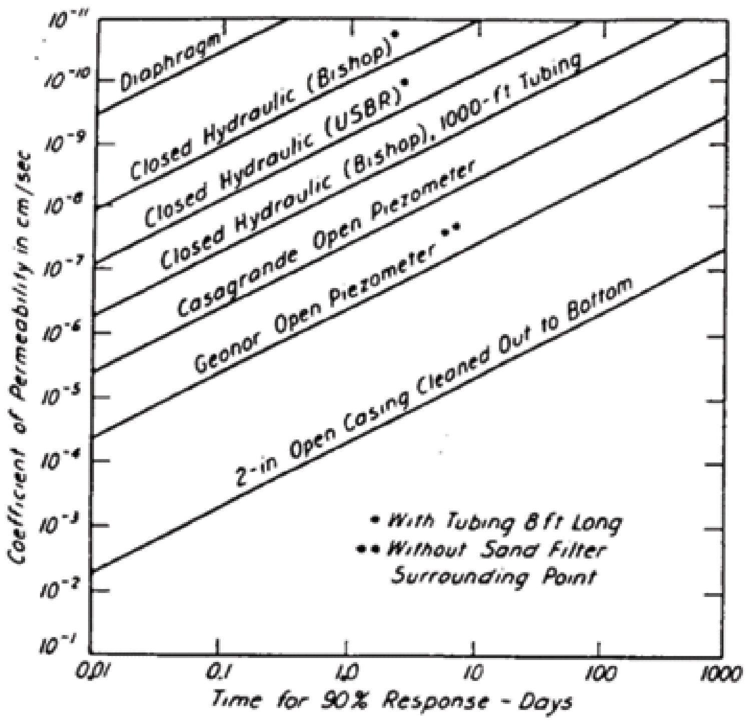
While the primary aim of this section is to consider methods of monitoring ground movements, and water elevations and pressures, levees can include structural elements that form part of the flood defence, such as crest walls. Where there is evidence of distress or when they are under hydraulic load or as part of a programme of routine condition assessment, it may be desirable to monitor the settlement, horizontal displacement and/or tilt of these structures. Some information on instrumentation that can be used to achieve this is also presented in Section 7.9.9.6.

7.9.9.1 Considerations in the selection of instrumentation

The instrumentation of levees may be used to monitor the response of the ground and levee during construction, to aid in the evaluation of the current condition and monitor long-term performance. Instrumentation can be used to monitor a specific response or a collection of interdependent responses. Some of these are detailed in Tables 7.44 and 7.45. When selecting instrumentation the anticipated rate of change and magnitude of the response needs to be compatible with the capabilities of the instrument. Some of the factors to consider in relation to the monitoring displacements and pore water pressures are summarised in Table 7.176.

Table 7.176 Some factors to considered when select instrumentation to monitor displacements and pore water pressures

Response	**Factors to be considered**
Displacements	• assess anticipated amount of displacement and use to design a robust system that will not be damaged by movements caused by settlement or lateral displacements • compressibility and lateral flexibility of tubing should be compatible with anticipated compression and lateral displacement of the soil profile • where excessive settlement is anticipated include telescoping sections that have shear pin connectors, to allow the tubing to accommodate large settlements without buckling or breaking.
Pore water pressure	• response range should be compatible with anticipated magnitude of pore water pressures. In general, expect lower pressure resolution (eg less precision) with increase in response range • response time may need to be compatible with the actual rate of change of pore water pressure in the field. Response time is a function of the type of piezometer and permeability of the soil (see figure below). Piezometers can be broadly classified into those that require either large or small volumes of water to flow into the instrument to come into equilibrium with a change in pressure. The figure below provides a broad indication of the repose times of a number of piezometers • the piezometer filter tip can be is installed in a sand pack, which may be sealed above and below with bentonite so that the monitored response is confined to a defined soil horizon. Alternately, current thinking is that where electronic vibrating wire piezometers are used, the sand pack and bentonite seals can be eliminated in favour of simply grouting the piezometers in place. Section 7.9.9.11 (Mikkelsen and Green, 2003, Contreras *et al*, 2008, Weber 2009, and Simeoni *et al*, 2011) • in high organic soils ground gases enter the piezometer causing it to become unsaturated and less responsive. The selection of high air entry filters can reduce the ease with which gas can enter the piezometer. Some piezometer can be de-aired if required, post installation.



Typical response times for various piezometers as a function of the permeability of the soils in which the piezometer is installed (from USACE, 1999)

Caution

Stand piezometer filters (ceramic, plastic) can have permeability of the order of 3×10^{-4}m/s. This needs to be considered in the context of the permeability of the soil in which it is installed as it could limit the reposed rate in high permeability soils. Similarly, consideration needs to be given to the permeability of the material used to form the 'sand cell' in which the piezometer tip is installed, relative to that of the surrounding soil.

Some other factors to consider when designing an instrumentation programme are:

- use of manual data logger or telemetric means of taking readings. The selection of the method may be dependent upon overall cost, the purpose for which the data is required and the speed of reaction needed in response to the data, the frequency with which readings are required, remoteness of the site, number of instruments, and whether there are concurrent site activities. Further discussion on data collection, transmission and management are presented in Section 7.9.9.8
- where cabling and piping pass through areas where large displacement are anticipated, such as under a new levee, sufficient slack should be provided. This may include zigzagging them in the base of the cables/pipes trench
- where grout is used during the installation of instrumentation, the long-terms strength of the grout should be compatible with that of the surrounding soil so that it does not have an adverse influence on the performance of the instrument (Section 7.9.9.11)
- knowledge of the ground profile, its properties, likely interaction with any external loads and an understanding of the potential failure and deterioration modes will help with the identification of what responses can be monitored and the locations of the responsive element of the instruments. The responses of the instruments can be correlated with the soil profile and compared with predicted behaviour. If necessary the geotechnical parameters can be re-evaluated on the basis of the field observations. (Section 7.7.4)
- it is preferable to include some redundancy in an instrumentation system so that principal responses are recorded by at least two different forms of instrumentation. This will provide a gross error check on the data and can mitigate against the loss of data should an instrument need to be repaired or reinstalled. Some information on combining instruments is included in Table 7.177
- installation of the instrumentation should be carefully planned and executed, or else it is possible that the instrumentation installation could jeopardise the integrity of the levee itself. The installation should not create vulnerabilities in the levee system. Future abandonment of the instrumentation installation should be planned before, whether the abandonment is caused by planned (eg its only used for short-term monitoring, or is beyond its useful lifespan) or unplanned (eg becomes non-functional or needs to be eliminated due to budget cuts in a monitoring program)
- when planning the nature of the instrumentation and associated monitoring programme, the life cycle of the levee system should be considered to account for possible extreme loading and rare events, as well as more common typical loading conditions. For example, most vibrating wire piezometers have a specified nominal range, but can typically operate over short durations at ranges usually much greater (often twice) the nominal working range.

Table 7.177 presents some typical applications with comments on installation details, and how they could be combined with other forms of instrument to achieve a fuller understanding of the response of the levee and foundation soils. It also provides information on building some redundancy within the system for common form of instrumentation.

Table 7.177 Some applications and considerations when selecting common forms of instrumentation

Instrument	Applications and considerations
Surface markers	• monitor settlement and lateral displacement of final or existing ground surface • locate adjacent to inclinometer to provide cross check of surface lateral displacement and monitor settlement at levee toe • locate adjacent to rod settlement gauge on final surface of levee to monitor post-construction settlement of fill material • locate at points intermediate between inclinometers/rod settlement gauges/ extensometers to confirm consistency of soil behaviour along the levee section.
Rod settlement gauge, subsurface settlement point and settlement cell	• settlement of location at reference depth, usually original ground surface, as fill material is placed • can be located adjacent to extensometer with a plate magnet on original ground level to provide cross check on settlement • mark up height above base plate in 1 m divisions with intermediate 0.5 m marks on rod or pipe for visual check of thickness of fill placed (excluding settlement cell).
Extensometer	• settlements at depth in a soil profile (target – spider magnets), and at existing ground surface and within fill material during placement (target – plate magnets) • locate target magnets at boundaries between soil types or changes in properties within a soil type, to isolate the settlement within layers of consistent properties • use intermediate target magnets if soil layer is thick and relatively uniform • on small projects the access tube could be perforated and wrapped with filter fabric to form a standpipe piezometer.
Inclinometer	• orthogonal lateral displacements at depth in soil profile • one axis to be aligned along direction of maximum anticipated displacement • larger diameter extensometer target magnets can be installed around inclinometer casing to form a combined instrument to monitor vertical settlement as well as lateral displacement • on small projects access tube could be perforated and wrapped with filter fabric to form a standpipe piezometer.
Piezometers – high water intake volume (observation well and standpipe piezometer)	• steady state groundwater levels in low permeability soils as a long response time is required • transient groundwater levels in high permeability soils as there is a more rapid response time • monitor response of groundwater in high permeability soils with variation in hydraulic load. Assess lag effects. Data may be used to validate soil permability using transient flow seepage models.
Piezometer – low water intake volume (twin tube hydraulic, pneumatic and vibrating wire)	• transient groundwater levels in low permeability soils as more rapid response time • cluster multiple piezometers adjacent to an extensometer. Locate piezometers at mid-point between adjacent magnets to measure consolidation characteristics of layer. Where settlements are large allowance needs to be made for the increase in pore water pressure due to settlement of the piezometer tip.
Weir box	• various weir configurations can be used to capture seepage discharge • seepage rate and turbidity can be monitored and evaluated.

An example of how some of these principles have been applied during the construction of a trial section of levee is presented in Box 7.35.

Box 7.35 ***Case study showing layout of instrumentation for a trial section of new levee***

Author: Richard Brooks

Client: Environment Agency (UK)

Engineer: CH2M Hill

Principal contractor: Team Van Oord (May Gurney)

Subcontractors: Fugro Engineering Services Ltd

Location: Steart Peninsula, at the mouth of the River Parrett in North Somerset, UK

Background

The project required the construction of some 4 km of levee over soft ground as part of a habitat creation scheme. There were a number of geotechnical uncertainties with regard to the proposed scheme and an instrument section of trial levee was constructed to resolve these issues. This case study illustrates the arrangement of instrumentation implemented to monitor the performance of a trial section of levee.

Trial section of levee

A plan and cross-section of the trial section is shown in Figures 7.77 and 7.78. One full instrumented cross-section (B-B) was installed. On cross-section A-A the subsurface instrumentation was omitted due to changes in the performance requirement of the levee.

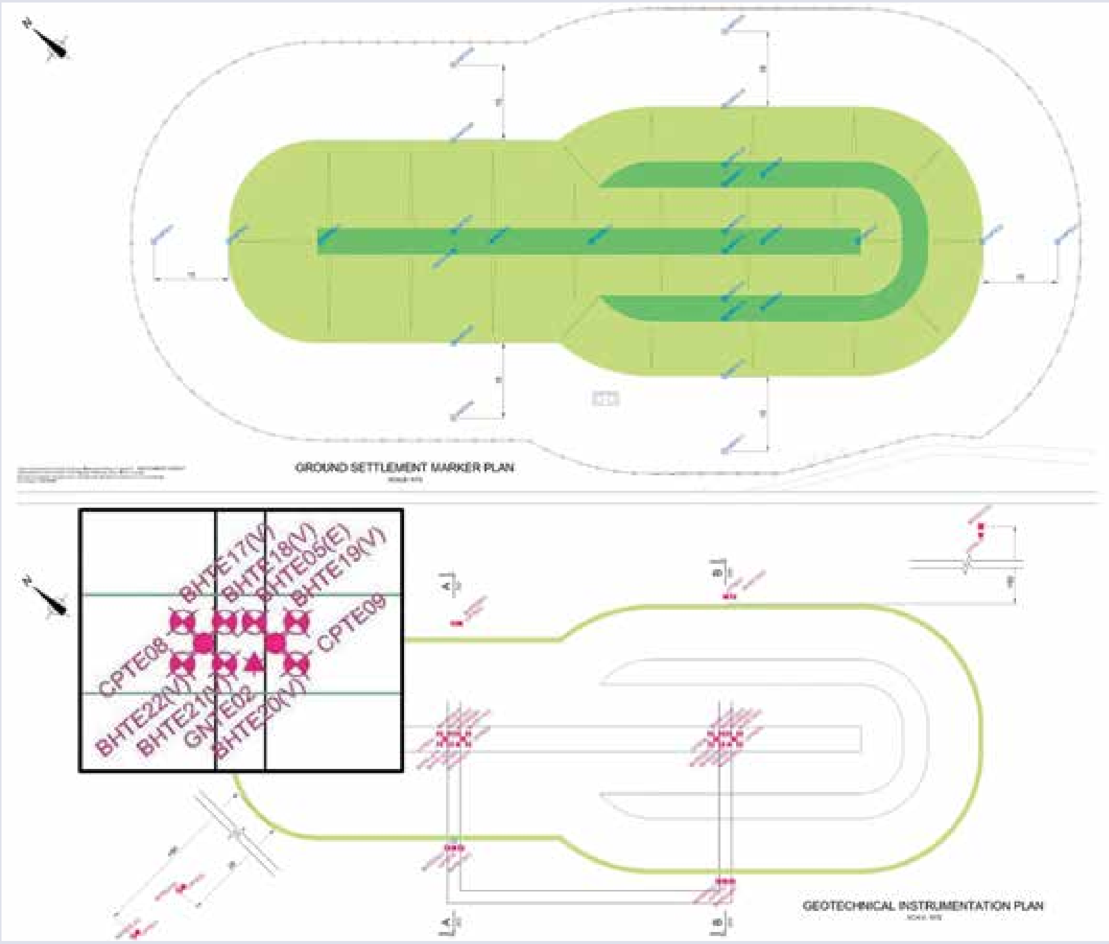


Figure 7.77 ***Plan location of instrumentation in trial section of levee – surface and subsurface instrumentation***

Figure 7.77 shows the relative position of the surface displacement markers on the embankment, adjacent to the toe and at change points on the final profile, with rod settlement gauges on the centreline and on each berm. They are offset from the section lines by a few metres to avoid clashing with the subsurface instrumentation.

Box 7.35 ***Case study showing layout of instrumentation for a trial section of new levee (contd)***

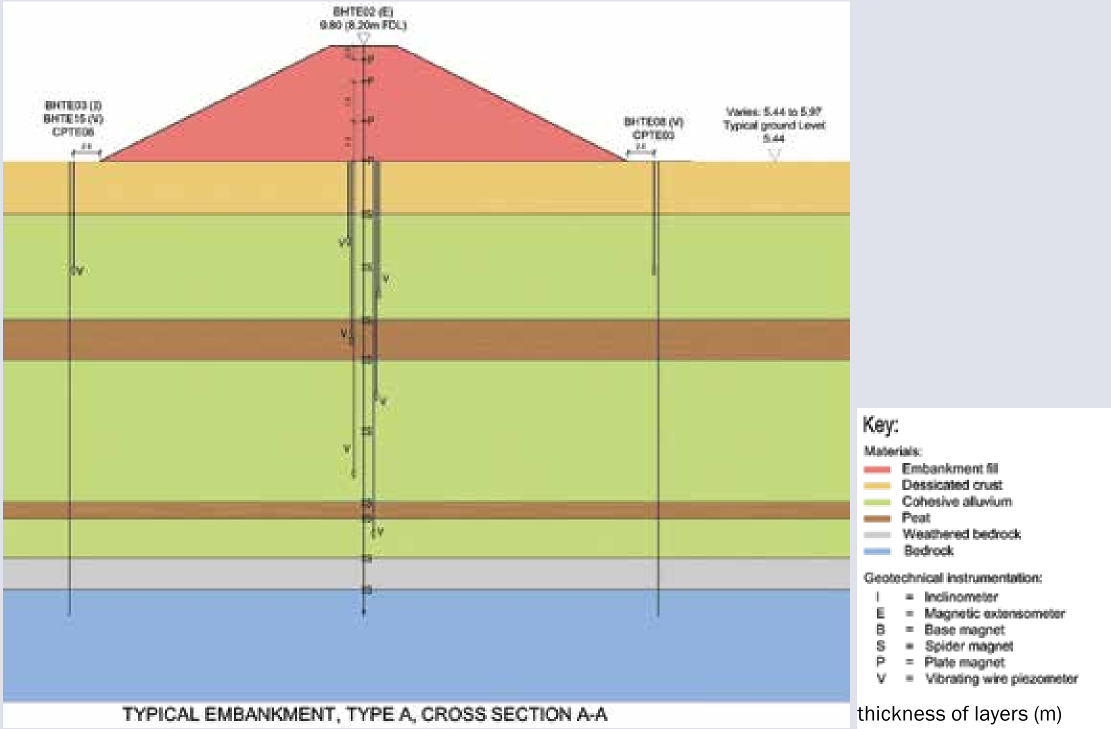


Figure 7.78 ***Indicative ground profile and relative elevation of instrumentation in trial section of levee – subsurface instrumentation***

Figure 7.77 (lower) shows the relative plan position of the subsurface instrumentation, extensometers (E), inclinometers (I) and piezometers (V), with an enlarged plan of the centreline instrumentation cluster. Figure 7.78 shows the elevation of the extensometer magnets and piezometers relative to the ground profile, along with the position of the inclinometer. Both the inclinometers and extensometer extend into the underlying bedrock to provide a stable reference.

7.9.9.2 Installation records

Installation record sheets for each instrument capture details of the installation, which will aid in the future interpretation of the data. The information to be recorded may include:

- manufacturer literature, purchase order receipts, calibration records etc
- instrument type and identification number
- existing ground level at the time of installation
- planned location in plan, depth and elevation
- planned orientation
- planned lengths, widths, diameters, depth and volumes of backfill
- plant and equipment used, including diameter and depth of associated exploration hole
- where appropriate, measurements or readings required during the installation process (ie any required to demonstrate the correct functioning of the instrument)
- a simplified log of ground conditions and relevant *in situ* test results (ie SPT *N* values)
- type of backfill used and design strength of grout
- as-built location in plan, depth and elevation
- as-built orientation
- as-built lengths, widths, diameters, depths and volumes of backfill
- weather conditions
- notes on problems encountered, delays, unusual features of the installation, and any events that may have a bearing on instrument behaviour
- a record of commissioning information and readings
- any colour coding used for wiring or pipes.

7.9.9.3 Baseline and monitoring readings

Subsurface instrumentation tends to be more complex than simple surface instruments. Where appropriate, calibration data for the instrumentation should be made available before installation and its correct functioning demonstrated before and after installation.

Where absolute changes in displacement or groundwater levels are required they need to be related to a fixed datum with an agreed elevation and co-ordinates. Many levees are located on soft compressible ground, which can be unstable. For long-term reliable monitoring it may be necessary to install a deep datum away from the influence of site activities, such as construction. The base of the datum is fixed into stronger soils at depth and the extension rod passes up through and is isolated from movements in the poor unstable ground. Some factors to consider when taking readings are given in Table 7.178.

Table 7.178 Some factors to consider when taking readings

Factor	Considerations
Readings can take time to stabilise following installation of the instrument	• piezometers require time for the movement of groundwater to occur around the instrument before the pre-installation pore water pressure is re-established. This is dependent on the permeability of the soil and the type of piezometer. Under steady state conditions a series of readings can be taken over time to demonstrate that these conditions have been established. Under transient conditions, when a boundary condition is variable (ie affected by tidal or fluvial events) the response should be compatible, and repeatable with cycles in the transient event. Corresponding changes in the boundary conditions should also be recorded • instrumentation that is grouted into a borehole may be free to move slightly until the grout has gained sufficient strength to hold the instrumentation in place.
Establish a baseline reading against which subsequent readings are compared	• where readings are taken manually, either optically or electronically, there is scope for human error. The whole monitoring process could be repeated at least three times in succession. If the results are shown to be repeatable within acceptable limits then the average of the results can be taken as the baseline reading. In a transient environment the base readings should be taken in quick succession to minimise the effect of the change in the boundary condition. The changes in the boundary condition should be recorded so that it can be correlated with any change in the series of baseline readings • results taken remotely are subject to errors that may be introduced in the communications system (both hardware and software). It is recommended, when possible, that baseline readings be recorded manually using a data collection system at the instrumentation system location, and compared with values broadcast through the wireless (or wired) telemetry system • the collective baseline data from all available instrumentation should be reviewed and evaluated to ensure that it is logical and consistent with known site conditions. Any anomalies should be reassessed to confirm the correct functioning of the instrument.
Repeatability and variability in readings	• at the start of a monitoring programme the rate at which the response changes to a variation in the boundary conditions may not be well defined and the degree to which the result will be affected by the repeatability of the reading process will be unknown. So, readings could initially be undertaken more frequently until the trends and background variability in readings for each instrument are clearly established.
Changes in boundary conditions	• where incremental changes in boundary conditions occur, such as during the placement of layers of fill when constructing or improving a levee, or the application of a static load on a levee, good practice would be for readings to be taken immediately before and after the change in load and then at a diminishing frequency (eg daily for three days, every three days for the next nine days and then weekly) up until the next incremental change in the boundary condition • where cyclic changes in the boundary condition occur, such as over a tidal event, readings could initially be taken at around high and low water to assess the gross response to the event. Where the response is considered to be significant then frequent readings, such as hourly, could be taken over the full cycle of the event • in both cases the change in the boundary condition should also be recorded (fill thickness, intensity of static load, tide level etc) to allow the response to be correlated with the change in boundary condition.

7.9.9.4 Instrumentation readings and records

When manual readings are taken there are usually other associated readings taken to contextualise the data and aid in the interpretation, such as fill elevation and time. When taking a reading it is good practice to perform site checks of the value, where practical, at the time by comparing the new record with the previous value. If an anomalous reading is identified that differs from the expected value or trend, then a further reading can be taken at the time to validate the result. Should the second result still be anomalous then further investigation may be justified to establish the cause. The data can also be prepared in an electronic form to an agreed format, which will facilitate data processing and interpretation in the office. In addition to the instrumentation data, and time and weather data, some typical examples of the records that could be taken when manually reading some common forms of instrumentation are present in Table 7.179.

Table 7.179 Typical information presented on a data record sheet

Instrument	Data required
Displacement/ settlement marker	• co-ordinates or distance from fixed point • elevation of top of marker • change in absolute plan position from fixed point relative to baseline and previous reading • change in elevation of top of marker relative to baseline and previous reading.
Rod settlement gauge, subsurface settlement point and settlement cell	• elevation of top of rod (excluding settlement cell) • original ground level at gauge location • elevation of ground adjacent to gauge or above settlement cell • total thickness of fill placed • length of rod, included extensions (excluding settlement cell) • elevation of base plate or settlement cell • settlement of base plate or settlement cell relative to baseline and previous reading.
Magnetic extensometer	• elevation of top of access tube • elevation of ground adjacent to access tube • total thickness of fill placed • depth of each magnet from top of tube (reading taken as probe is both lowered and raised in the tube) • elevation of each magnet • settlement of each magnet relative to baseline and previous reading.
Inclinometer	• co-ordinates or distance from fixed point • elevation of top of access tube • total thickness of fill placed • elevation of ground adjacent to access tube • record of deflection values from base upwards on both axis (x and y) and repeat with the probe turned through 180° to evaluate face errors • horizontal movement profile of access tube relative to baseline and previous reading • where probe readings are recorded electronically, the file name should be reported.
Observation well and standpipe piezometers	• depth to water from top of tube • elevation of top of tube • elevation of ground adjacent to standpipe • total thickness of fill placed • elevation of piezometric surface • change in groundwater table elevation relative to baseline and previous reading.
Twin tube hydraulic, pneumatic and vibrating wire piezometer	• water pressure readings • elevation of ground adjacent to standpipe • total thickness of fill placed • elevation of piezometer tip as-built • estimated or measured settlement of piezometer tip • elevation of water head adjusted to take account of settlement of piezometer tip and, if required, barometric pressure • change in water head relative to baseline and previous reading.

7.9.9.5 Visual presentation of instrumentation data

Vast amounts of data can be generated from instrumentation. These data are usually more readily evaluated visually, in the form of a graphical plot. The change in the instrument response will be a reaction to a change in some factor, for example placement of fill, consolidation of the soil or changes in external water level. Where there are related variables acting at the same location they could be presented as combined plots on one sheet to allow a visual evaluation of their interdependence.

To aid in the direct visual assessment and comparison of results from other instruments it is useful to adopt a common scale for the axis (time, elevation, depth, pressure etc).

Some typical examples of data plots that could be prepared for common forms of instrumentation are presented in Table 7.180.

Table 7.180 Typical instrumentation data plots

Instrument	Typical data plots
Displacement marker	• deflection since taking baseline reading on critical axis relative to levee geometry v time • displacement profile in x-y space since taking baseline reading.
Settlement marker	• settlement vs. time.
Rod settlement gauge, subsurface settlement points and settlement cell	• thickness of fill and settlement of gauge plate or anchorage vs. time.
Magnetic extensometer	• thickness of fill and settlement of each target magnet vs. time.
Inclinometer	• profile of horizontal movement of access tube relative to baseline reading vs. depth • thickness of fill and maximum horizontal movement relative to baseline readings vs. time • tilt angle/rotation of access tube relative to baseline reading vs. depth.
Observation well and piezometers Twin tube hydraulic, pneumatic and vibrating wire piezometer	• elevation of piezometric surface vs. time • elevation of piezometric surface vs. elevation of external water level • thickness of fill and change in water head vs. time.
Other plots	• settlement (s) at centreline on the y-axis vs. horizontal movement at embankment toe (δ) normalised against centreline settlement (ie [(δ/s] on the x-axis, Figure 7.57) • change in water head from baseline reading (or excess pore water pressure) vs. applied load from fill (see the figure in Table 7.56).

7.9.9.6 Types of instrumentation

This section provides a description of the various types of instrumentation available that are applicable to levees. The primary focus is on displacements and pore water pressures.

The forms of instrumentation available are considered in terms of:

- what the instrumentation is used to measure
- how the response is measured
- how measurements are taken, transmitted and stored.

Conventional instrumentation and monitoring techniques are presented first. These are mature, widely-used, and very useful forms of instrumentation.

Advances in electronics and communication systems continue to be adapted to meet the needs of instrumentation industry. Some of which are potentially beneficial for monitoring levees. These new technologies will be briefly discussed and a case history illustrating their application is presented.

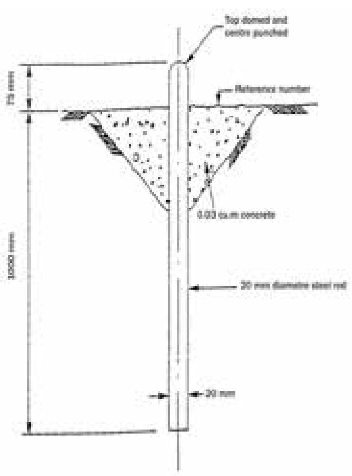
Displacement and deformation

The simplest form of monitoring at any stage during the levee life cycle is visual inspections done on a routine or periodic basis (Section 5.4). The parameters that can be assessed are relatively limited, since they are based on visual observations of the levee system, surface features and the adjacent environment. These inpsections document the visual behaviour and changes in the surface of the levee system over time. Critical observations are often made during a flood event when the levee is heavily loaded. One of the fundamental drawbacks to observational monitoring is that the period between observations can be large and it is limited to surface features and responses.

Instrumentation installed to monitor displacements and deformations is useful in that it provides quantitative evidence for settlement, incipient slope instability, sliding of the levee etc, which may all be indications that there could be a problem with the integrity of a levee. Section 7.7.2 considers some monitoring requirements in relation to failure and deterioration modes. During the construction of levees on soft foundations, the monitoring of displacements and pore water pressures may be used to evaluate the onset of instability (Section 7.7.4).

Typical methods used to monitor displacements and deformations include the survey methods outlined in Section 7.9.1 and the following form of instrumentation detailed in Tables 7.181 to 7.187.

Table 7.181 Surface displacement/settlement markers

<table>
<tr><td>What it measures</td><td colspan="3">Vertical movement (total settlement) and lateral displacement at a single surface point.</td></tr>
<tr><td>How it measures</td><td colspan="3">Measurements taken periodically (manually) by survey techniques.</td></tr>
<tr><td>General information</td><td>Method</td><td>Application</td><td>Limitations</td></tr>
<tr><td>

(courtesy CH2M Hill)</td><td>• steel rod driven into ground and surrounded with concrete for additional stability
• top of rod may be domed, counter sunk or has a metal cap to provide survey reference point
• locate at position away from direct interference by site activities or provided protection
• simple design, installation and method of monitoring.</td><td>• monitoring of surface settlements and horizontal displacements on the levee profile and adjacent ground
• variants may be embedded into structures associated with levee to monitor settlements and horizontal displacements.</td><td>• readings taken at discrete time intervals
• accuracy and precision dependent on survey method
• may be disturbed by site or public activities. Disturbance may not be visually evident at time of survey
• location should be accessible for optical survey methods.</td></tr>
<tr><td colspan="4">Relevance to levees</td></tr>
<tr><td colspan="4">• only applicable to measuring total surface settlement and lateral displacement at a location where no new fill will be placed.</td></tr>
</table>

Table 7.182 Rod settlement gauge

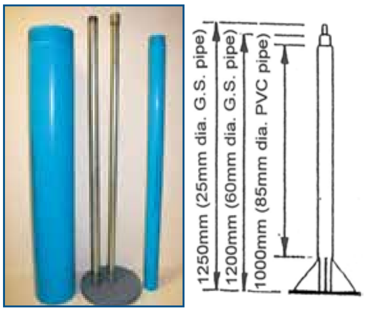
What it measures	Absolute vertical movement (total settlement) at a single location and reference depth		
How it measures	Measurements taken periodically (manually) by survey techniques		
General information	**Method**	**Application**	**Limitations**
 *(courtesy Geosense and CH2M Hill)*	• steel base plate (0.5 m to 1 m square) with centrally attached rod/pipe (typically steel) • placed on ground before filling • pipe sections extended as fill is raised • level survey readings taken on pipe head and adjacent ground.	• measurement of fill thickness, which can aid in the assessment of the fill quantity placed • measurement of settlement below base plate, usually original ground level.	• easily damaged by earth moving plant • pipe should remain vertical • hand operated plant used to compact fill around the gauge • readings taken at discrete time intervals • accuracy and precision dependent on survey method.
Relevance to levees			
• only applicable where new fill is to be placed above an existing surface, and so is not appropriate for existing levees except in the case where an existing levee is to be raised or widened with additional fill material.			

Table 7.183 Subsurface settlement point: Borros anchor, spiral or fixed-foot anchor

What it measures	Absolute vertical movement (total settlement) at a single location and reference depth		
How it measures	Measurements taken periodically (manually) by survey techniques		
General information	**Method**	**Application**	**Limitations**
 *(courtesy Geosense)*	• installed just below base of borehole and can be extended during filling • an inner riser pipe(s) extend through outer pipe to surface • survey reading taken on pipe head and adjacent ground • inner pipe is anchored in soil using either 'Borros' type anchor – three extendable prongs, helical screw anchor, or grouted anchor (fixed foot) • simplicity of design and method of monitoring.	• monitor settlement at depth within the levee of underlying foundations soils • unlike the rod settlement gauge, it can be retrofitted to a target depth during or post-construction • an array of pipes can be anchored at different depths to measure differences in settlement and compressibility throughout a soil profile.	• easily damaged by construction activities • pipe section should be maintained vertical • hand operated compaction plant should be used to compact fill around the pipe • readings taken at discrete time intervals by manual survey techniques • accuracy and precision dependent on survey method.
Relevance to levees			
• used to monitor settlement within and below an existing levee or during construction works			

Note

A similar device (settlement extensometer) employs a special sensor head installed at the top of the unit to measure the relative displacement between the head and the anchored end of the rod. Multiple rods can be housed in the same unit to measure relative displacements between the anchor head (typically installed at the ground surface) and rods anchored at various depths. Useful in measuring variations in settlement over depth where the compressibility of subsurface materials changes.

1 2 3 4 5 6 7 8 9 10

Table 7.184 Settlement cell

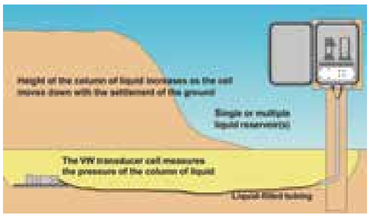
What it measures	Vertical settlement at a single location on a surface to be covered by fill material		
How it measures	Pressure head in a the cell connecting by a liquid filled tube to an above ground measurement reservoir		
General information	**Method**	**Application**	**Limitations**
 *(courtesy Geosense)*	• transducer measures pressure head of liquid column in the cell • reservoir located above ground outside influence of the zone of settlement • change in pressure head used to determine settlement • cell and tube can be protected by installing in shallow trench.	• measurement of settlement below the cell (usually original ground level) but no direct survey required • where access is restricted during or post-construction, as reservoir can be located outside of work area or where there could be post-construction health and safety issues.	• depending on application and accuracy required, changes in temperature and atmospheric pressure may need to be taken into account and a correction applied to the reading.
Relevance to levees			
• similar to a rod settlement gauge or point but can be monitored remote from the measurement location.			

Table 7.185 Magnetic extensometer

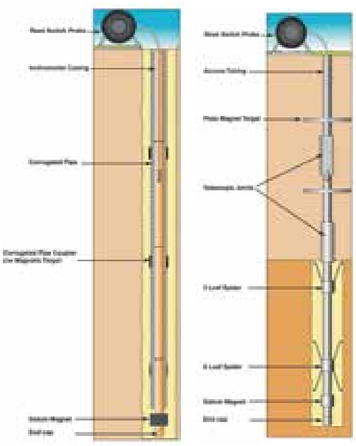
What it measures	Vertical settlement at multiple locations through the soil profile		
How it measures	Depth measurements taken by inserting a probe down the tube, which responds to targets that move vertically with the soil column		
General information	**Method**	**Application**	**Limitations**
 *(courtesy Geosense)*	• can take two forms: (a) corrugated sleeve around casing with in-built sensing rings at uniform spacing, or (b) plastic pipe with telescoping couplings and independent spider magnets installed at required depths • installed and grouted into borehole • bottom of tube (base magnet) located below compressible layers • additional sections and magnets added to extend tubing up as fill is raised.	• determination of settlements throughout a soil profile and compression or vertical strain between target magnets • larger diameter sensing rings and target magnets can be installed around inclinometer (Table 7.186) casing to form a combined instrument • related to site elevation by surveying top of pipe.	• easily damaged by construction activities • hand operated compaction plant should be used to compact fill around the instrument • readings taken at discrete time intervals by direct, manual profiling by field staff • grout strength should be compatible with the strength of the surrounding soil.
Relevance to levees			
• where a significant thickness of new fill is to be placed above an existing surface in the case of new levees or where an existing levee is to be raised, widened or repaired • monitoring and quantifying ongoing vertical settlement at depth in an existing levee.			

Table 7.186 ***Inclinometers***

What it measures	Orthogonal lateral displacements at multiple locations through the soil profile		
How it measures	Measures inclination of casing using a wheeled probe that runs along guides in the casing and has a fixed gauge length. Readings are taken at fixed depth intervals, usually equal to the gauge length of the probe, and integrated over the full depth of the casing		
General information	**Method**	**Application**	**Limitations**
 *(courtesy Geosense)*	• casing has two sets of opposed wheel guides, which maintain the probe orientation • readings taken along orthogonal guides at fixed intervals from bottom of casing • casing installed and grouted into borehole • bottom of casing installed to depth where no movement is anticipated (fixed) • readout unit may store data that can be downloaded to a computer for processing.	• used to monitor orthogonal lateral displacements, usually along and perpendicular to the levee alignment, within and at the toe of levee • data can show potential zones of lateral movement and deformation, which may provide an indication of the development of slip surfaces/ failure planes, as well as monitoring rate of horizontal movement.	• casing easily damaged by construction activities • access pipe section should be maintained vertical with wheel guides aligned along axis of measurement • hand operated compaction plant should be used to compact fill around casing • readings taken at discrete time intervals by manual profiling by field staff • grout strength should be compatible with the strength of the surrounding soil • aluminium cases can corrode in aggressive soils (peat).
Relevance to levees			
• lateral displacement at the toe of the levee where a significant thickness of fill is to be placed, in the case of a new levee, or where an existing levee is to be significantly raised or widened to aid in monitoring stability • monitor the lateral (creep) displacement in an existing levee.			

Table 7.187 ***Tiltmeter***

What it measures	Rotation about a single axis at a single location		
How it measures	Measures change in tilt of a rigid structure		
General information	**Method**	**Application**	**Limitations**
Tiltmeter and readout  *Tiltmeter installed on bracket affixed to concrete flood wall*  *(courtesy Scott Raschke)*	• there is a diversity of sensor types to accommodate a wide range of tilts and precisions • a weatherproof housing is securely attached to the structure • readout unit is used to obtain data • system can be configured to automatically transmit data to a remote location.	• used to monitor the change in tilt or rotation of a structure • data can be acquired at the location of the installation • if structure is 'flexible' such that tilt varies up the structure, a continuous profile can be obtained by installing multiple tiltmeters. Alternately, consider attaching an inclinometer casing to the face of the structure.	• only measures the tilt at a single location, unlike an inclinometer, which captures the change in vertically over an entire profile • when installing a tiltmeter (or inclinometer casing) on an exposed surface, consideration should be given to the potential for physical impacts and the aggressiveness of the weather and other environmental variables.
Relevance to levees			
• can be installed on a levee structure (such as a flood wall) to monitor rotation.			

Pore water (piezometric) pressure

Pore water pressures are relevant to evaluating the stability of a levee, both during construction or when improvement works are being undertaken or during periods of high hydraulic load. Instrumentation installed to monitor the pore water pressures can be used to assess a number of factors, as well as determining the response of the levee and foundation soils to change in the hydraulic load and precipitation. These are:

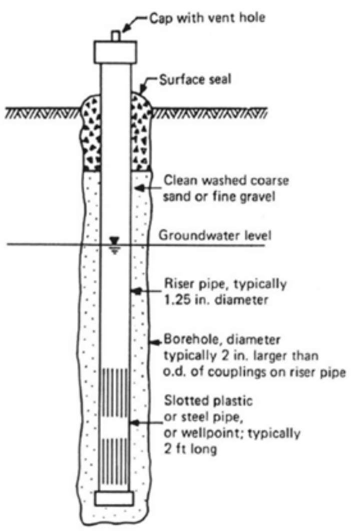
- define the groundwater flow regime
- understanding seepage through and under levees
- quantifying uplift pressures on the landside during period of hydraulic load (Figure 8.84 and Section 9.7)
- implementing construction rate control when undertaking levee works of soft, compressible soils (Section 7.7.4).

Typical methods used to evaluate pore water pressures are presented in Tables 7.188 to 7.192.

Caution

This section and Box 7.38 makes reference to installing piezometers in a fully grouted borehole, ie the piezometer tip is encased in the grout rather than being placed in a sand cell between bentonite plugs, which seal the sand cell within a defined section of the ground profile. Careful consideration needs to be given as to the appropriateness of the fully grouted technique to the prevailing ground conditions and whether the desired outcomes of the monitoring programme will be achieved.

Table 7.188 *Observation well*

<table>
<tr><td>What it measures</td><td colspan="3">Static water level measured in a slotted pipe installed in a borehole</td></tr>
<tr><td>How it measures</td><td colspan="3">Measurements taken by inserting a tethered water level indicator (dipmeter) down the riser pipe, which responds on encountering water</td></tr>
<tr><td>General information</td><td>Method</td><td>Application</td><td>Limitations</td></tr>
<tr><td>

(courtesy USACE)</td><td>installed in a borehole
perforated pipe installed in a sand-filled zone placed in borehole around the pipe
static water level determined by lowering an electronic sensor attached on a calibrated cable down the pipe. The surface unit makes an audible tone when water is encountered.</td><td>simple design and method of monitoring
steady state groundwater levels
transient groundwater levels in high permeability soils
monitoring the response of groundwater in high permeability soils to variations in hydraulic load.</td><td>can create a vertical connection between strata
water level may not be representative of static water table if, for example, a perched water table is encountered
easily damaged by construction activities
readings taken at discrete time intervals by field staff
slow response time in low permeability soils.</td></tr>
<tr><td colspan="4">Relevance to levees</td></tr>
<tr><td colspan="4">simplest form of groundwater level monitoring system. Not recommended in general for anything other than short-term monitoring in uniform, relatively high permeability soils.</td></tr>
</table>

Table 7.189 Standpipe piezometer

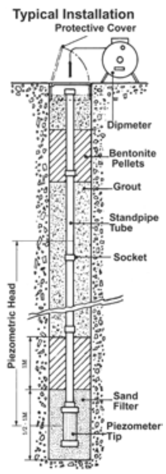
What it measures	Static water level at a discrete depth defined by the extent of the response zone formed by a sand pocket in which the tip is installed		
How it measures	Measurements taken by inserting a tethered water level indicator (dipmeter) down the riser pipe, which responds on encountering water		
General information	**Method**	**Application**	**Limitations**
Ceramic piezometer tip  ***Drive-in piezometer tip***  ***Schematic of piezometer installation in borehole***  *(courtesy Geosense)*	• installed in a borehole • filter (typically ceramic or plastic) attached to PVC riser (standpipe) • filter installed within sand pocket and sealed above and, if necessary, below with a bentonite seal to limit the response zone • bentonite seals eliminate vertical connectivity problem associated with observation wells • variant form has protected filter and steel riser allowing it to be driven or pushed in soft ground with no sand cell. • static water level determined by inserting a water level indicator as per observation well (Table 7.188).	• simple design and method of monitoring • steady state groundwater levels • transient groundwater levels in high permeability soils • monitoring the response of groundwater in high permeability soils to variations in hydraulic load • drive in variant can be cheap and quick to install for an initial assessment.	• easily damaged by construction activities • readings taken at discrete time intervals by field staff • slow response time in low permeability soils • response time in high permeability soils can be controlled by filter or sand cell permeability as these can be lower than the soil in which it is installed • risk of hydraulic 'short circuits' between sand cells where multiple installations are included in one borehole • driving variant can be damaged and response zone assumed to be over filter length.
Relevance to levees			
• provides measurement of pore water pressure over a predetermined discrete depth interval at one location. Multiple installations at various depths may be required to assess *in situ* pore water pressure distribution.			

Table 7.190 Twin-tube hydraulic piezometer

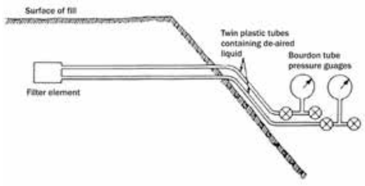
What it measures	Pore pressure at the location of the filter element or over discrete depth interval defined by the extent of the response zone formed by a sand pocket in which the tip is installed		
How it measures	Porous filter element connected to two tubes, with each end attached to a Bourdon tube pressure gauge, manometer or electrical pressure gauge		
General information	**Method**	**Application**	**Limitations**
 *(courtesy USACE)*	• tubes are filled with liquid • if no gas present, pressure gauge readings will be identical on both gauges • if pressures not equal, gas is present in liquid and system must be flushed with de-aired liquid.	• place in boreholes or levee fill during construction • relatively rapid response to change in pore water pressure in low permeability soils • robust and simple construction, which can readily be maintained • long-term monitoring.	• in cold conditions use antifreeze or protect from cold • use robust tubing that will not degrade and leak over time • periodic flushing may be required to de-air the liquid, particularly in organic soils • response time extended where there is gas in the liquid.
Relevance to levees			
• can be installed in borehole to monitor pore water pressure in foundation soil or within the levee fill material during construction. Suitable for long-term monitoring programmes.			

Table 7.191 Pneumatic piezometer

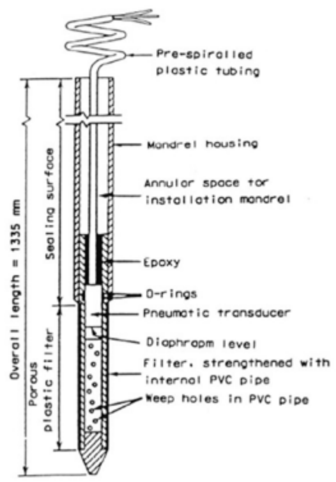
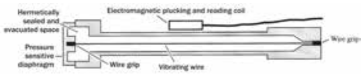
What it measures	Pore pressure at the location of the filter element or over discrete depth interval defined by the extent of the response zone formed by a sand pocket in which the tip is installed		
How it measures	Flexible diaphragm above filter with pneumatic transducer to measure pore pressure		
General information	**Method**	**Application**	**Limitations**
 *(courtesy USACE)*	• can be installed in boreholes or pushed into soft ground • probe includes filter, diaphragm, and transducer • borehole installations can be fully grouted in place or installed within a sand filter with bentonite seal (Section 7.9.9.11 and Box 7.38) • relatively simple operating principle. Only moving part is the flexible diaphragm in the pneumatic transducer.	• can be installed in soft foundation soils to monitor pore pressures during construction • relatively rapid response to change in pore water pressure in low permeability soils.	• readings taken manually at the point of installation • may need to 'purge tubing' if there are erratic readings caused by moisture or debris in tubing • filter cannot be de-aired following installation.
Relevance to levees			
• can be used to monitor pore pressure for evaluating seepage and responses of soft foundation soils to construction activities.			

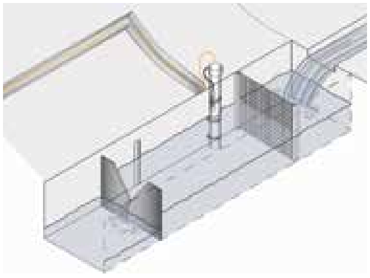
Table 7.192 Vibrating wire piezometer

What it measures	Static and transient water level/pressures at a discrete depth interval defined by the extent of the response zone formed by a sand pocket in which the tip is installed		
How it measures	Vibration frequency of a wire, which is electronically plucked and the frequency monitored. Changes in frequency correspond with a change in pore water pressure. Measurements taken using a readout unit or data logger connected to transducer through a data cable. Compatible with telemetric systems of data transmission		
General information	**Method**	**Application**	**Limitations**
Vibrating wire piezometer transducer  *Vibration wire piezometer*  *(courtesy Geosence)*	• probe includes filter and vibrating wire transducer that converts water pressure to a signal • standard vibrating wire piezometers cannot be de-aired but variant is available that can be de-aired • borehole installation can be fully grouted in place (see Section 7.9.9.11) or installed with sand filter and bentonite seal • for 'fully grouted' system, probe located at defined depth by attaching to a sacrificial grout pipe • readings can be taken automatically using a data logger.	• cables can be buried so data readings can be taken at a central accessible location away from the works • compatible with telemetric systems of data transmission • measurement of transient pore water pressures in low permeability soils as rapid response time can be achieved • in fully grouted system, multiple piezometers can be installed by attaching a number at various depths to a sacrificial grout pipe.	• need to consider interference that can affect signal for long cable runs • in fully grouted system, design of grout needs to be compatible with the permeability of the ground • ensuring long-term saturation of the filter can be problematic in organic soils prone to forming ground gas, which will affect the response time. In these circumstance, consider variant that can be de-aired.
Relevance to levees			
• provides pore water pressure at a single depth/location. In fully grouted system a string of 'nested' piezometers can be installed in one borehole to monitor the profile of pore water pressure.			

Seepage discharge

The evaluation of seepage may be desirable as it can affect the performance of the levee. The extent of the effects will depend upon both the seepage rate and turbidity of the water. The latter provides an indication of the amount of material that may be removed from or is migrating through the levee or foundation soils. Monitoring seepage may be as simple as collecting it in a container of known volume, and timing how long it takes to fill the container. The turbidity can also be monitored visually for a qualitative assessment of changes. Seepage rate and turbidity can also be evaluated by diverting the flow to pass over a simple weir bulkhead (Table 7.193).

Table 7.193 Weir bulkhead

What it measures	Seepage rate. The water collected can be used to visually evaluate changes in turbidity		
How it measures	Based on hydraulic principles and the geometry of the weir bulkhead		
General information	**Method**	**Application**	**Limitations**
V-notch weir bulkhead  *(courtesy itmsoil.com)*	• flow is diverted over a weir with a specific geometry • flow depth over weir measured and discharge rate evaluated based on hydraulics principles and geometry of weir.	• relatively simple, reliable, and inexpensive system.	• need to be able to adequately divert flow to the weir • possibility that some turbidity may be due to erosion of diversion channel • requires observer at monitoring location • experienced staff required to monitor seepage, which may develop into piping, possibly leading to levee failure.
Relevance to levees			
• used to evaluate seepage rate and visually observe turbidity.			

Load and total stress

Instrumentation installed to monitor load and total stress, particularly when coupled with an understanding of the pore water pressure distribution, can provide a more complete picture of stresses in a levee and foundation soils. This information can be useful in evaluating the stability of a levee, both new and existing. Typical methods used to evaluate load and total stresses include:

- contact earth pressure cells
- push-in pressure cells.

Pressure cells are not commonly used in levees and manufacturers' information should be consulted if they are considered necessary.

Temperature

Temperature monitoring on levees may be beneficial for several reasons:

- it can cause damage or affect the performance of structural, electrical and mechanical systems. This can include freezing of gates and valves, heave due to frost, and damage to concrete due to excessive heat generation during curing
- it also affects the operation of transducers used in instrumentation systems, so it may be necessary to monitor the temperature of the installation to ensure it is operating within the allowable environmental limits of the instrument
- ground temperatures can be useful in locating seepage paths, because the water flow will affect ground temperatures. Seepage temperature may also be useful in tracing the source of the seepage.

Typical methods used to evaluate soil/groundwater temperature include:

- thermal imaging
- temperature probes
- distributed monitoring systems.

7.9.9.7 New and evolving instrumentation and monitoring technologies

Advances in electronics and related technologies, particularly communications, have led to new developments in instrumentation, and the ability to automate data collection and transmission. While new technologies are usually initially more expensive than older forms, costs are typically offset due to the need for less maintenance, and ease of use and automation. Also, costs typically fall as demand rises and manufacturing technologies mature. Often the increased instrumentation hardware costs are offset by increased efficiency, resilience, and reduced monitoring costs associated with automation in data collection and transmission. There has also been an increased emphasis on integrating various types of instrumentation, such as inclinometers, piezometers and temperature sensors or fibre optic based systems, into a single installation cluster or unit. The integration of various sensor systems provides a diverse set of data, which may result in better information at reduced cost. While widespread adoption of these systems has not yet taken place, these smart or intelligent levee instrumentation and monitoring systems show great potential and have slowly begun to be implemented not only on experimental levee test sections, but also for flood risk reduction on a limited number of levees in several countries. These systems promise real time monitoring of data to assess the performance of levees, and aid in the identification of potential failures before they occur, through the comparison of the data with predefined trigger levels. In this way they can provide a more timely early warning, allowing the implementation of emergency actions or develop targeted strategies for flood fighting.

It can be difficult to decide where to install traditional discrete sensors to provide a robust levee monitoring system. There are, however, exceptions where serviceability issues are evident from a visual inspection. Traditional discrete instrumentation can have a limited application for routine levee monitoring. This is primarily due to the limited zone of soil around them that influences their response and their inability to gather broader information of the behaviour of the soil mass as a whole, such as would be obtained from the installation of an integrated instrumentation system (Section 7.9.9.9). Distributed sensors, such as fibre optic systems (Section 7.9.9.10) also provide a much greater spatial coverage of the levee through the installation of a single continuous fibre optic cable. The benefit of wider data coverage is that it greatly improves confidence in identifying locations where failure is more likely to occur.

As the cost of instrumentation decreases due to improvements in technology and the integration of different types of sensors into one instrument, coupled with easily implementable techniques for real time remote data acquisition and transmission, the benefits of the new systems may begin to outweigh their cost.

7.9.9.8 Advancements in data collection, transmission and management

Retrieval of data from traditional, discrete instrumentation systems may require manual collection, recording, reduction, and interpretation. This approach has many drawbacks (it can be time consuming, introduce errors in transcribing data values, etc.), which can have significant implications depending on the objective of the instrumentation programme and the location of the project. Mobilisation of personnel to the site where the instrumentation is located may be time consuming and costly, forcing a reduction in the reading frequency to below that which would be optimal and greatly diminishing the benefits to the project.

Advances in solid state electronics and communication technologies have proved beneficial for real time remote monitoring of instrumentation. These systems often rely on communications over the internet or mobile phone network, which have become (more or less) ubiquitous. Many of the monitoring systems are proprietary in nature, with different manufacturers providing products and systems that are similar. However, there is a significant trend for manufactures to implement open, non-proprietary systems.

Data from the instrumentation may be stored on servers connected to the internet (eg the Cloud) which can be accessed from a networked computer. Software applications for mobile phones and tablet devices are also becoming more frequently available. The development of these software applications with mobile devices, as well as browser based software applications has made data visualisation and management more user friendly and accessible. Many systems provide convenient ways to interact with the data. Trigger levels and alarm can be set if the data reaches a predefined limit in some performance parameter.

7.9.9.9 'Discrete' monitoring network – Micro-Electro-Mechanical (MEMS) instrumentation

Advances are being made with new discrete multi-sensor systems (integrated instrumentation), which may be deployed in arrays to increase spatial coverage and reduce the limitations of non-distributed systems. These new sensor systems are often referred to as micro-electro-mechanical systems (MEMS), which is the technology of miniaturised mechanical and electromechanical elements that are fabricated into a single device (Desrosiers *et al*, 2012).

The new systems have benefited from a reduction in cost and size associated with recent technological advances. In addition to the miniaturisation of the sensors, other advances include:

- incorporation of multiple sensors into a single unit that can be installed to measure a variety of parameters (temperature, pressure, strain etc) at a single location, which improves the usefulness of the data and reduces installation costs
- inclusion of microprocessors into the sensor so that digital data streams can be obtained directly from the sensor without an external analogue to digital signal converter (ADSC)
- availability of continuous, real time data
- development of new software tools for data management, improving the usefulness of the data for levee monitoring
- the ability to implement a 'daisy chain' of sensors in a series arrangement, often connected by a single cable.

The framework of integrating sensor systems with information technology (IT) components to perform data management (both hardware and software systems) has been described as intelligent 'cyberphysical' systems (Kambar *et al*, 2012). An outline of two proprietary systems is presented in Box 7.36.

Box 7.36 Examples of proprietary MEMS

Measurand ShapeAccelArray (SAA) consists of 300 mm or 500 mm long segments connected in series by flexible joints to form a continuous coil or rope up to 100 m long. Each segment includes three MEMS accelerometers and digital temperature sensors. The system provides a 3D profile of ground deformation, temperature, and acceleration (vibration) in real time, both vertically and horizontally, depending on the axis of installation. The system can also be combined with piezometers that use a shared communication protocol.

Source: www.measurandgeotechnical.com/

GEOBEADS system (alert solutions) combines multiple sensor node units on a single serial bus cable. Each node unit can contain sensor elements to monitor pore water pressure, inclination and temperature. The system can also be deployed in a vertical or horizontal orientation. The measurement frequency can be set by the user.

Figure 7.79

Measurand ShapeAccelArray (SAA) (courtesy Measurand Inc)

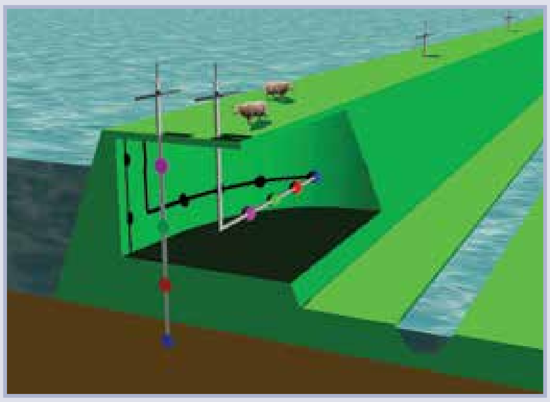


Figure 7.80

A GeoBeads sensor system issue (courtesy alert solutions BV)

Source: www.urbanflood.eu

7.9.9.10 'Distributed' monitoring – fibre optic instrumentation

Advances in communications technologies, which increasingly rely on fibre optic cable for data transmission, have led to the rapid development and commercialisation of fibre optical systems. Due to massive increases in production quantity and quality, prices have dramatically fallen. In a fibre optic cable, pulses of light travel along a transparent core made of glass (silica) or plastic, which is contained in an outer cladding layer. The cladding has a lower index of refraction that traps the light within the core, causing it to be reflected continuously along the core. Another outer covering or jacket protects the cable from damage. The optical fibre core is typically between 10 and 1000 microns in diameter, depending upon the material used (eg glass versus plastic) and the application. Light may be transmitted for up to about a thousand metres, but will eventually degrade due to impurities in the glass. If necessary, signals can be boosted to make up for the losses if extended lengths are required.

The fibre optic cable can operate as a sensor since the light intensity and the scattering of light within the cable is directly affected by many external factors, which are of interest when monitoring a levee. These factors typically include strain, temperature, humidity and pressure. They can be used to form a distributed sensor along the levee, which will monitor the change in these factors and their locations along the length of a single cable. The spatial resolution is typically of the order of a metre. Further, different wavelengths can be used so that the sensor can be multiplexed to monitor different parameters. This gives them a significant advantage over traditional discrete instrumentation systems that can only measure a single parameter at a single location and depth (or response zone), unless multiple sensors are employed, which would increase costs significantly in comparison with a distributed fibre optic system.

The most common applications of a fibre optics sensor in levees are the measurement of strain and temperature. Strain is useful in detecting movement, which could indicate settlement due either to the natural consolidation of the foundation soils or as a result of the formation of voids due to piping, animal burrows etc as well as movements due to the initiation of slope failure. Temperature variation has often been used as an indicator of seepage. When water is present but there is no movement (seepage), heat transfer is slow and is governed by conduction only. When water is moving through the soil, heat transfer is quicker and is controlled by advection from the flow of water. This difference enables seepage areas to be identified based on temperature measurements.

Pilot studies have been undertaken with fibre optic systems to examine their effectiveness in providing real time monitoring of levees. They can be integrated into both existing and new levees. For new levees, the fibre optic cable can be placed during construction to form a continuous distributed network along the levee at different elevations as shown in Figure 7.81. Integration of a similar networked fibre optic system can be retrofitted by installing the cable into shallow slit trenches within the levee.

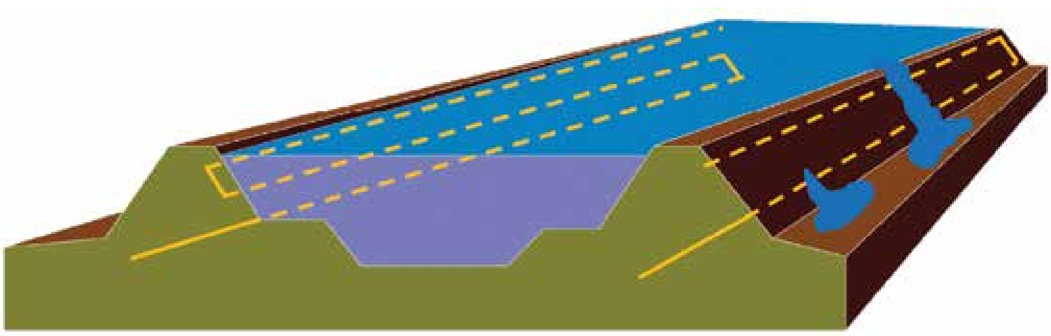


Figure 7.81 Distributed fibre optic sensor (depicted in orange) installed along a levee (after Inaudi and Church, 2011)

The fibre optic cable can also be incorporated into geosynthetic materials used to reinforce levees. A proprietary example of this is presented in Box 7.37.

Box 7.37 ***Example of intelligent geotextile incorporating fibre optic cables***

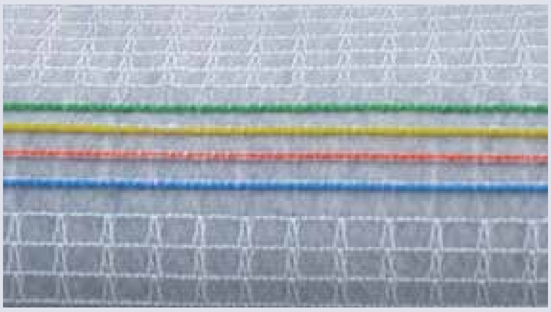


Figure 7.82

Geotextile sensor with four fibre optic cables (two for temperature and two for strain measurement) (courtesy TenCate Geosynthetics)

Manufacturers are developing 'intelligent geotextiles' in which the fibre optic cable is woven into the material as it is manufactured. The fibre optics can measure strain in the geotextile as low as 0.02 per cent, as well as temperature to a precision of 0.1°C, to a spatial resolution of between 0.5 m and 1.0 m. The complete system is composed of the embedded fibre optic geotextile, data acquisition and logging system, as well as alert and reporting software.

7.9.9.11 Backfill of instrumentation locations

Some forms of instrumentation can be placed during construction of new levees (settlement cells, twin-tube hydraulic piezometers, fibre optic and similar sensors) or on the existing ground surface (rod settlement gauges) or on existing structures (tiltmeters). If instruments are placed during construction, care should be taken not to damage them and to avoid forming a weak zone through inadequate compaction around the instrument or because of other construction issues. Instrumentation for existing levees can sometimes be placed in shallow excavations that can be backfilled with materials similar in composition to the existing levee, and to a degree of compaction equal to or better than the existing levee.

Where instruments are to be installed in boreholes, the first issue to address is mitigating the potential for hydraulic fracturing (Section 7.7.3) the levee or foundation soils, which could adversely affect the performance of the levee. The second issue is to fill the borehole with material to ensure the proper function of the instrumentation while maintaining the serviceability of the levee.

Cement bentonite grouts are commonly used to backfill boreholes. Manufacturers typically have recommended trial mixes based on the type of instrumentation and ground conditions. The objective is to typically produce a grout mix that will not shrink or excessively settle, and match the engineering properties of the soils (permeability, stiffness etc) in which the instruments are installed, as closely as possible.

It may be preferable to avoid using cement bentonite or cement based (liquid) grouts where boreholes are located close to groundwater abstraction wells and high permeability strata close to water courses where grout losses can be high and migrate into them. Commercially available bentonite or bentonite/cement pellets can be poured into the borehole and are hydrated by the groundwater.

Other components of the backfilled borehole can include sand packs around the piezometer filter, which provide a direct pervious contact between the soil strata and the piezometer. Bentonite pellets or chips can be used to provide a tight seal between the sand pack and the remainder of the grouted borehole. As discussed previously, however, recent literature questions if sand packs and seals are necessary and suggests that fully grouted piezometers can function adequately (see Box 7.38).

Cement (or fly-ash) is typically added to reduce the expansion of the bentonite, and the stiffness of the grout can be controlled by increasing the amount of cement, thereby altering the water-cement ratio. The grout can be mixed in the field, either with specialised mixing equipment, or simply in an open container. A drill rig pump can be used to circulate and mix the water with the cement and bentonite. Once the water and cement are mixed to a water-cement ratio that will provide desired strength/ stiffness, the bentonite is added. The more bentonite that is added, the lower the grout permeability will be. The objective is to obtain a consistency that is pumpable through a relatively small diameter tremie pipe and flowable so that it will conform to and fill the voids between the instrumentation and walls of the borehole.

Backfilling and grouting of instrumentation, and backfilling of an open borehole, should be done carefully and with great attention to detail. Again, grouting could cause hydrofracture of the levee or foundation soils. Only staff with a great deal of knowledge and experience, working with a knowledgeable and experienced contractor, should be responsible for design of mixes and directing of field work.

Box 7.38 illustrates a fully grouted piezometer system. Box 7.39 provides some information on the cement-bentonite mix proportion that could be used.

Box 7.38 Installation of piezometers

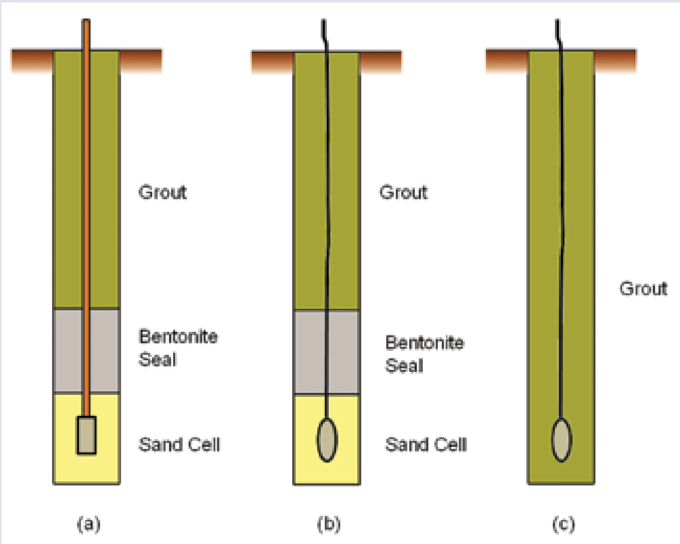


Figure 7.83

Traditional standpipe piezometer with sand pack (a), vibrating wire piezometer with sand pack (b), and fully-grouted vibrating wire piezometer (c) (after Contreras et al, 2008)

The use of a sealed sand pack system has also been traditionally employed when using an electronic vibrating wire piezometer as well. However, recent studies (Mikkelsen and Green, 2003, Contreras *et al*, 2008, Weber 2009, and Simeoni *et al*, 2011) point to the success of eliminating the sand pack and bentonite seal and simply grouting the piezometer in place. This method of installation is much more expedient, particularly when including multiple nested piezometers in a single borehole. Selection of the installation method should be left to the discretion of an experienced instrumentation specialist.

Box 7.39 Typical cement-bentonite grout mix designs

To ensure that the results of instrumentation data are representative of the surrounding soil mass there is a need for the grout strength to be comparable with that of the soil. Where there is doubt, trial batches of grout can be prepared and tested. Some typical grout mix proportions are presented in Tables 7.194 and 7.195.

Table 7.194 Two typical cement-bentonite grout mix designs (from Mikkelsen, 2002)

Application	Medium to hard soils	Soft soils
Material	Ratio by weight	Ratio by weight
Water	2.5	6.6
Portland cement	1.0	1.0
Bentonite	0.3	0.4
Notes	28 day compressive strength 350kPa and modulus 68MPa	28 day compressive strength 30kPa

Table 7.195 Typical grout mixes used to backfill boreholes

Water/ Cement ratio	Water (litres)	Bentonite (kg)	Cement (kg)	7 day confined shear strength (triaxial) (kPa)	28 day confined shear strength (triaxial) (kPa)	28 day unconfined compressive strength cube test (kPa)
2:1	100	5	50	180	592	1184
2:1	100	7	50	312	652	1304
3:1	100	7	33	112	316	632
3:1	100	5	33	52	184	368
4:1	100	5	25	8	136	272
4:1	100	7	25	20	136	272
5:1	100	7	20	8	92	184
5:1	100	5	20	48	44	88
5:1	100	6	20	30	100	200
3:1	100	6	33	40	250	500
8:1	100	5	12.5	25	70	140
11:1	100	6	9	14	37	74

Caution

In selecting the mix strength consideration needs to be given to what the instrument is measuring. A specific issue is the installation of an extensometer. If it is to function effectively the grout should have a compressibility that is approximately equal to that of the surrounding soil. In soft or very soft clay and peat the long-term strengths reported in Box 7.39 may be too strong, resulting in the extensometer being encased in a relative incompressible cement bentonite 'pile', which will not match the behaviour of the adjacent ground. Under these circumstance consideration could be given to performing grout trials with cement:bentonite ratios by weight initially in the range 1:4 to 1:10, with sufficient water to achieve a creamy pumpable mix.

An example of a field trial using the technologies discussed in Sections 7.9.9.8 to 7.9.9.10 is presented in Box 7.40.

Box 7.40 ***An example of a field trial using new and evolving technologies, IJkdijk (Smart Calibration Dike) Testing Facility, the Netherlands***

The IJkdijk (Smart Calibration Dike) Testing Facility is a full-scale experimental facility in the Netherlands. It was established to develop state of the art inspection and monitoring techniques for improving the ability to predict and understand the performance and failure modes of levee systems, and to develop what are sometimes referred to as 'smart levees' (Van *et al*, 2009, Kolewijn *et al*, 2012, and Kamber *et al*, 2012). The impetus for the creation of the facility were several incidents, including one in 1995, involving concerns over the performance of a levee system during extreme flooding (but which did not fail), and another in 2003, which involved sudden failure of a levee during a relatively dry season (Koelewijn *et al*, 2012). The goal is to develop 'smart levee' systems that can be used to monitor and predict the performance of levees under various loading conditions and potential failure modes. Several different types of tests have been performed at the IJkdijk facility using a wide variety of sensor and instrumentation technologies. The details of the testing previously performed and results obtained, as well as future proposed testing, are summarised in Table 7.196.

Table 7.196 ***Various tests performed at the IJkdijk facility (Koelewijn et al, 2012)***

Year/type	Type of test/failure mode	Sensors	Outcome/result
2007 4 m high earthen levee	Wave overtopping	• pore pressures • humidity • temperature • acoustic.	A much higher resistance to erosion than expected from current design guidelines.
2008 6 m high, 100 m long earthen levee	Slope failure induced by a controlled sequence of events: • excavation of a ditch at the levee toe • inundation of the dike's sand core • application of load on the levee crest.	• pore pressure piezometers • infrared cameras • three different fibre optic systems (measuring strain, temperature, and acceleration) • two types of inclinometers • extensometers • geophones • hydrophones • inverted pendulums • liquid level settlement sensors • humidity sensors • laser scanning equipment.	The dike failed on the third day of the test, however, the sensor data revealed the location of the eventual failure location on the first day. The sensor data indicated catastrophic failure well before visible signs of failure were observed.
2009 3.5 m high, 15 m wide, 15 m long clay levee over a 3 m thick sand layer	Failure by piping by increasing the hydraulic head on one side of the levee	• grid of 8 × 15 pore pressuremeters • fibre optic cables (strain and temperature) • infrared cameras • self-potential measurements • hydrophones • flow meters.	Failure was induced by collapse of progressive piping at the dike toe. Pore pressuremeters identified the growth of the internal erosion and piping. Similar results were achieved with the fibre optic temperature measurements, although interpretation of the data was more complicated.
2012 (to be completed) 90 m long levee consisting of four unique segments	Simultaneous failure by several mechanisms including piping, slope stability, and fluidisation	• in development.	The goal is to evaluate the predictive power of a sensor system using data processing and information processing.
2013 (to be completed) To be determined but larger than previous levees	Failure by liquefaction	• in development.	The goal is to understand the process of liquefaction failure and testing sensor systems to evaluate onset of liquefaction.

Box 7.40 ***An example of a field trial using new and evolving technologies, IJkdijk (Smart Calibration Dike) Testing Facility, the Netherlands (contd)***

Figure 7.84
The IJkdijk 2008 test (a) levee before failure and (b) levee after failure (courtesy IJkdijk)

Further reading

There are a number of good references are available which discuss these instrumentation and their applications in more detail (see References).

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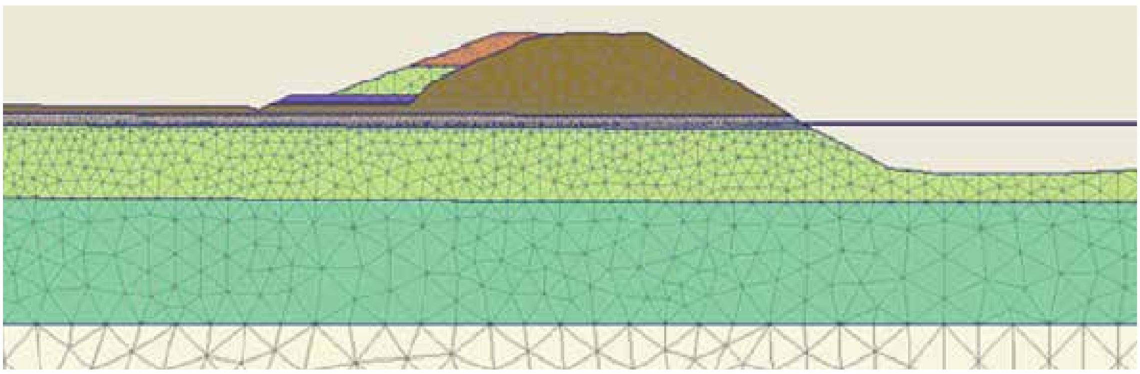
8 Physical processes and tools for levee assessment and design

Courtesy E Durand



Courtesy E Durand

CHAPTER 8 CONTENTS

8 PHYSICAL PROCESSES AND TOOLS FOR LEVEE ASSESSMENT AND DESIGN

Chapter 8 details the morphological, hydraulic and geotechnical analysis tools needed to assess performance of a levee.

Key inputs from other chapters

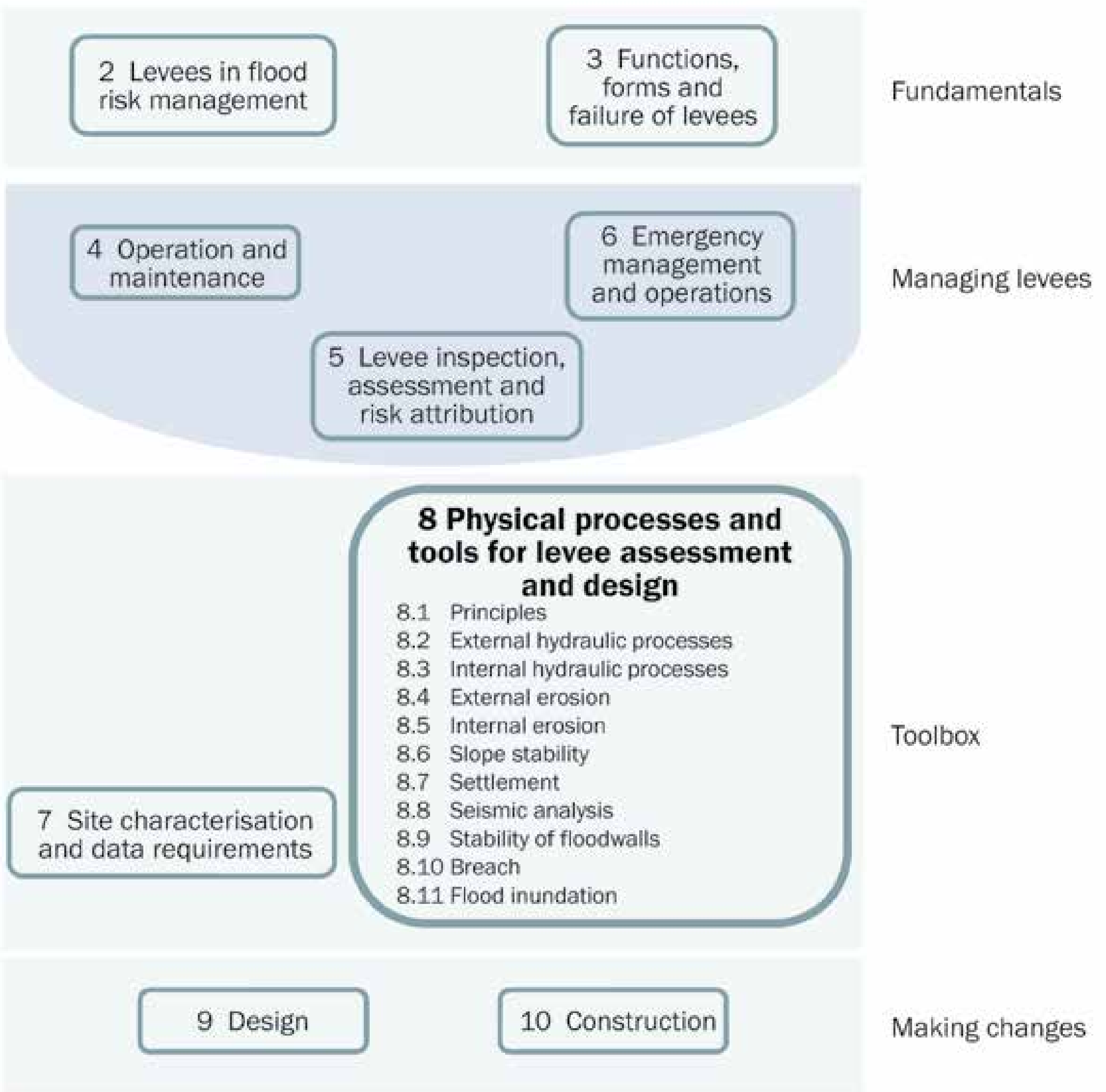
- Chapter 3 ⇨ **functions, forms and failure mechanisms**
- Chapter 5 ⇨ **requirements for analysis**
- Chapter 7 ⇨ **morphological, hydraulic and geotechnical parameters**
- Chapter 9 ⇨ **requirements for analysis.**

Key outputs to other chapters

- **Tools for levee assessment, design and construction** ⇨ Chapters 5, 9 and 10

Note: The reader should **revisit Chapters 2 and 3** throughout the levee life cycle for a reminder of important issues.

This flow chart shows where to find information in the chapter and how it relates to other chapters. Use it in combination with the contents page to navigate the manual.



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CHAPTER CONTENTS AND TARGET USER

This chapter is divided into 11 sections, and serves as a toolbox, which gives a thorough analysis of levee performance. The chapter first discusses the external and internal hydraulic processes that impose loading on the levee, through to the basic failure mechanisms of external erosion, internal erosion and slope instability. Additionally, the chapter addresses several critical issues including settlement, seismic loading, and stability of flood walls. The chapter concludes with a treatment of breaching and inundation modelling.

Principles

Section 8.1 introduces the key principles discussed in the chapter, and issues relating to levee analysis. It also provides links with other chapters.

External hydraulic processes

Section 8.2 describes the multiple hydraulic processes that impose loading on a levee. These processes include wave run-up and overtopping, overflow, and scour in river channels, on beaches and in front of coastal levees.

Internal hydraulic processes

Section 8.3 details multiple hydraulic processes that occur within the levee, which can lead to deterioration and damage. These processes include seepage and pore pressure and their impacts.

External erosion

Section 8.4 describes the principles and concepts of external erosion. The section details resistance to erosion from grass systems and other erosion resistant systems. Resistance of protection systems to erosion due to currents and waves is also presented.

Internal erosion

Section 8.5 discusses the principles of internal erosion including backward erosion, concentrated leak erosion, suffusion, contact erosion, as well as use and stability of filters.

Slope stability

Section 8.6 details methods to analyse slope stability from simplified methods, design charts, limit equilibrium methods, limit analysis approaches, and stress deformation analysis.

Settlement

Section 8.7 presents the principles of settlement analysis, assumptions and approximations, settlement calculation, verification of settlement prediction and use of finite element methods (FEM).

Seismic analysis

Section 8.8 details the analysis of a levee from seismic loading. This includes the governing parameters, slope stability with inclusion of dynamics, crest settlement, and liquefaction.

Stability of flood walls

Section 8.9 provides information related to the analysis of flood wall stability from hydraulic forces acting on flood walls. Analysis for stability of T-walls and I-walls is further detailed in the section.

Breach

Section 8.10 aids in understanding the breaching process, determining the parameters necessary for performing a breach analysis, and details methods for modelling breach growth.

Flood inundation

Section 8.11 provides information related to the end users of inundation modelling, parameters and data requirements, types of inundation models, modelling approaches, model outputs, and treatment of uncertainties.

8.1 PRINCIPLES

8.1.1 Links to other parts of the handbook

Levee assessment and design are often complex steps in a project since these require multidisciplinary engineering approaches, especially for levee stability assessment (for existing levees or for a new levee) where geotechnical and hydraulic issues are intimately linked and interact very strongly. For such projects, designers and engineers are often specialised in one discipline, so levee projects require integrated management (Chapter 5).

In a new levee project, for all phases of the project from feasibility to design, after the levee alignment has been selected (Chapter 9) and the site properly characterised (Chapter 7), engineers analyse levee stability with regard to various physical processes. These physical processes form a levee failure scenario (Chapter 3) and their analysis ensures the levee will be stable for all stages of construction (Chapter 10), and for all loads and hydraulic situations defined in the design process (Chapter 9) to reach a protection level chosen by owners or stakeholders (Chapter 2).

For stakeholders, engineers and designers, Chapter 8 can be considered a 'tool box', which details several actual methods to analyse stability for various physical processes that could lead to levee deterioration or failure as defined in Chapter 3. Each of these individual mechanisms is treated from the simplest to the more complex existing approaches so that the proper analysis model may be found at each stage of the project. The experimental and theoretical bases of the approaches are briefly described and the advantages and disadvantages of each method are discussed in terms of conditions of use and accuracy of the results. The different approaches are expressed, following a gradual complexity, through rule of thumbs, empirical formulae, analytical models or numerical methods.

8.1.2 Analyses issues for levees

In a simple way, levees could be defined as civil works projects designed to resist 'hydraulic' loading. The assessment of levees includes several aspects related to geotechnical, structural, and hydraulic domains. Figure 8.1 gives an illustration of those issues. For levees, both disciplines are intimately linked and even if there are purely geotechnical stability situations to analyse, the main critical situations for levees often depend on external hydraulic steady or transient loads.

Levees are built in river or coastal settings that continually change. The movement of water over and through the landscape shapes and forms the stream channel network or beach/dune complex as it interacts with the geologic formations that form the landscape. The levee will alter this interaction, sometimes in a negative way.

Stream flow path and planform variability can have significant consequences if not allowed for in the evaluation of levees. Lateral instability can change the angle or point of attack for a river on a nearby levee, possibly accentuating local and contraction scour and inducing bank instability. This instability may have direct and indirect impacts on the levee. For example, direct impacts may be the result of increased stream bank height that leads to slope failure, which extends through the levee embankment. Indirect impacts may result where changed stream alignment alters currents toward the levee embankment.

Changes in stream or beach planform may be gradual or the result of a single flood event. The directions and magnitudes of such changes are difficult to predict. However, it is essential to assess potential planform changes and how they relate to the levee to ensure its successful performance over the design life. Bank erosion and lateral migration analyses should be carried out, for example, to identify the erosion potential of the foundation near to or at the levee. The importance of lateral instability assessments is emphasised by Graham (1983) and Simon (1994) who observed that increased discharge in rivers leads to changes in channel width in preference to depth in unconstrained sections. According to Simon (1995), width adjustment processes may represent the dominant mode of adjustment in coarse-grained streams with cohesionless banks.

1 2 3 4 5 6 7 8 9 10

Lateral instability in rivers can be in the form of general channel bank erosion, bend scour, channel widening, and channel shift. General channel bank erosion can result from erosion by current flow, the action of waves generated by wind, or human weathering mechanisms such as freeze-thaw and desiccation, seepage effects, surface runoff, and mass failure mechanisms. There is potential for significant change in both coastal and riverine settings, which can result in substantial changes in planforms and profiles for example.

Tools for estimating meteorological, morphologic, hydrologic and hydraulic loads are given in Chapter 7. These loads are the drivers in changed boundary conditions. Boundary characteristics including bed material size and composition interact with these loads over time. Chapter 7 also describes methods for assessing the interaction between the boundary and the movement of water over the land surface (whether river channels or coastal features). Sediment transport, also covered in Chapter 7, describes methods for assessing long-term system response. Localised erosion and scour that may occur near or at the levee embankment are addressed in Section 8.2.

Whether assessing the condition of an existing levee or designing a new levee, the interaction between the water, the landscape and the levee embankment typically requires an iterative process using site condition information and hydraulic modelling tools (found in Chapter 7), local boundary condition calculations (Chapter 8), and any constraints defined for the system (Chapter 5 for existing levees or Chapter 9 for new levees).

The principle relationships between relevant boundary conditions associated with assessing watershed and stream characteristics and hydraulic loads for levee projects are shown in Figure 8.1. The diagram also indicates relevant design parameters that should be determined.

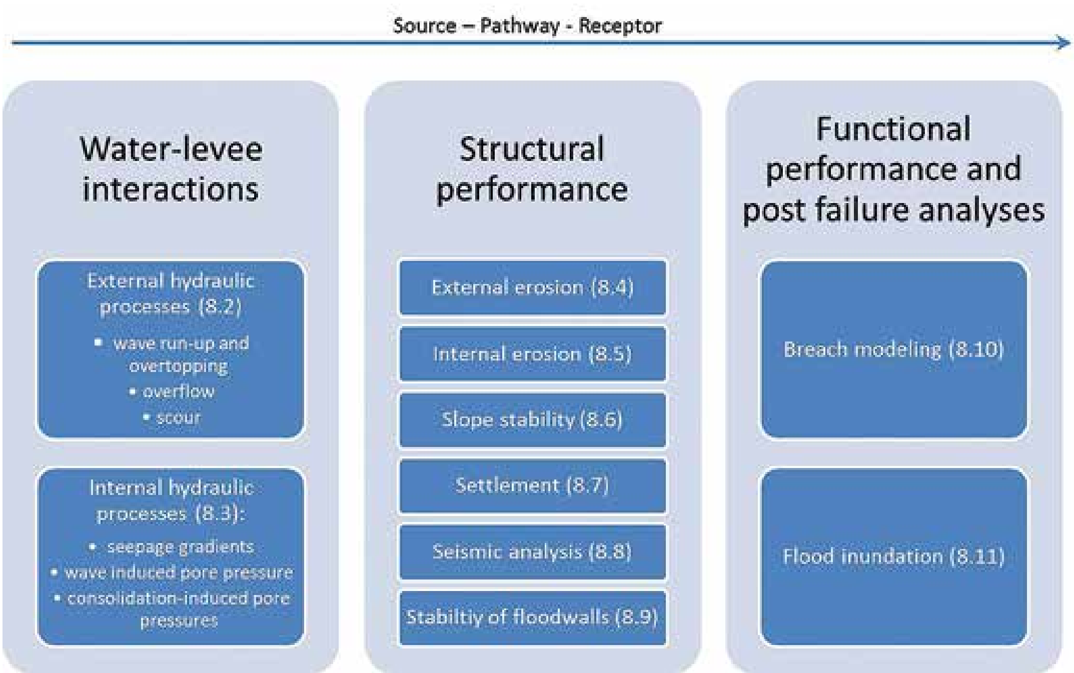


Figure 8.1 ***Geotechnical and hydraulic issues for levee section analysis***

8.1.3 Links with other chapters

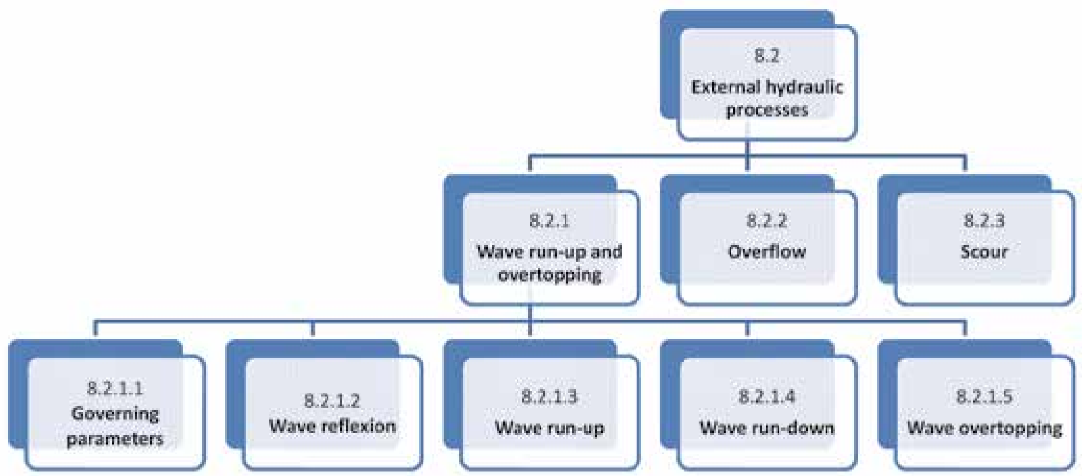
Chapter 8 is designed as a toolbox, using data mainly derived from processes described in Chapter 7. However, Chapter 8 can be useful for several adjustments required during design process (Chapter 9) or construction process of a levee (Chapter 10). Table 8.1 lists the main data required for the calculation process and relation with other chapters.

Table 8.1 Data used during calculation process

Type of data	Description of data and its use in construction	Chapter refs
Geometry of levee and its environment	**Description:** topographic and bathymetric data **Use:** required to determinate typical cross-sections of the levee and the suite of models used for stability analysis	7
Design loads	**Description:** external loads (geotechnical, hydraulic, permanent or transient loads etc) and their evolution versus time (hydrograms) need to be defined **Use:** required for determinate levee induced loads as internal loads in order to perform the levee stability analysis	7
Design situations	**Description:** hydraulic situations (depending on choices made by the levee owner or stakeholders in relation to the protection level objectives) **Use:** required to determinate levee internal loads and perform levee stability analysis	5, 7, 9
Feasibility, project or design site analysis and characterisation reports	**Description:** site characterisation during feasibility, project or design phases **Use:** during calculation phases, data are used to define geometric and geotechnical models. Affects the choice of methods related to processes that have to be studied	7, 9
Project programme (schedule)	**Description:** project programme (schedule) **Use:** affects design choices and construction methods and then calculation steps (for example, necessity of settlement acceleration techniques or soil reinforcement techniques)	5, 9
Construction phases	**Description:** project phases of construction of levee **Use:** affects calculation steps (intermediate steps etc)	10
As-built drawings	**Description:** documentation of levee constructed condition including changes to design, site conditions and constructed work **Use:** provides written records for local specific calculations	10

8.2 EXTERNAL HYDRAULIC PROCESSES

There are several external hydraulic processes that have to be accounted for in the proper design or assessment of a levee. These processes will be detailed in this section and include wave reflection, wave run-up and run-down, wave overtopping, overflow and scour. The layout of this section is shown in the following flow chart.



8.2.1 Wave run-up and overtopping

8.2.1.1 Governing parameters

Many hydraulic and structural responses on levees depend on the form and severity of wave action before, and after it reaches the levee. Design methods generally use empirical equations or graphs based originally on results of hydraulic model tests. These relate the required response (eg wave run-up level, or limiting armour mass) to parameters describing the incident wave conditions (height, period, wavelength), and the structure geometry (water depth, slope angle of the structure). These are generally grouped to form dimensionless parameters that have physical meaning, some of which are summarised as follows:

Wave steepness

Wave steepness s_0 (-) is a parameter defined to integrate the influence of the wave period. It is defined as the ratio of wave height to wave length:

$$s_0 = \frac{H_s}{L_0} = \frac{2\pi}{g}\frac{H_s}{T^2} \tag{8.1}$$

where:

g = gravitational constant (9.81 m/s²)
H_s = significant local wave height (m)
T = wave period (s)
L_0 = deep-water wavelength (m)

In random wave terms, mean and peak wave steepness are introduced, which are defined to consider the mean wave period T_m and the peak wave period T_p respectively in Equation 8.1. It is worth noting that this definition is not itself complete as both H and L can vary with position and depth (in turn varying with water and sea/lake/river bed level).

Surf similarity or Iribarren number

Wave breaking on a slope, whether approach or revetment slopes, can be categorised by a parameter known as the surf similarity parameter or Iribarren number, ξ (-). It is defined as:

$$\xi = \frac{\tan\alpha}{\sqrt{s_0}} \tag{8.2}$$

where α is the slope angle of the structure (°).

As for wave steepness, this parameter can be adapted by substituting, s_{0m} or s_{0p} to s_0 to obtain surf similarity related to mean (ξ_m) or peak (ξ_p) waves.

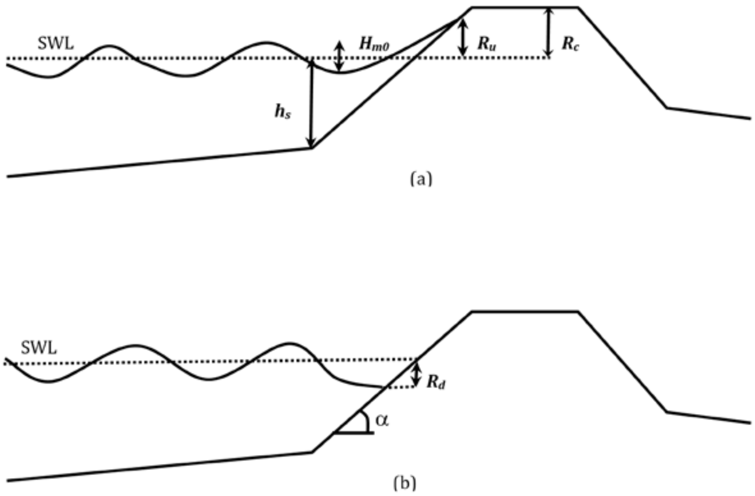
Relative water depth

Many wave processes depend on the water depth at the toe of the structure h (m), not as an absolute value, but when related to the waves. The most useful wave parameter here is generally the wave length, usually given as L_{0m} or L_{0p}. The relative water depth may then be expressed as h/L_{0m} or h/L_{0p}.

Structure geometry

One of the most important responses of a levee or seawall is the overtopping performance given by the proportion of waves overtopping, the mean overtopping discharge per unit length of defence, or the coefficient of wave transmission. Each of these depends on the crest elevation above the still water level (SWL). This structure freeboard, R_c (m), is often related to the incident wave height as R_c/H_{si}.

The other controlling parameter is the waterside slope angle. Levee slopes of 1:1 (difficult to achieve in practice) or 1:2 (cot α = 2.0) have very similar run-up/overtopping performance. Overtopping falls quite rapidly as the slope is reduced to 1:4 (cot α = 4.0) or beyond.



Note

R_C = freeeboard, R_u = wave run-up, R_d = wave run-down, H_{m0} = wave height at the toe of the levee, h_s = water depth of the toe of the levee as regards to the still water level (SWL)

Figure 8.2 Definition of run-up (a), and run-down (b) situations

8.2.1.2 Wave reflection

All waves encountering any structure will reflect (at least in part) back from that structure. The most useful measure of wave reflection performance is the reflection coefficient, C_r , defined by the ratio of reflected H_r (m) to incident H_i (m) wave heights:

$$C_r = \frac{H_r}{H_i} \tag{8.3}$$

For vertical walls, reflections from plain vertical walls generally fall in $C_r \approx 0.85$ to 0.9, with relatively little influence of incident wave height or period. On typical levees, some wave energy will be dissipated on the slope, so C_r will be less, and this reduction will be greatest for shallower slopes. Reflections may also be reduced by roughness and/or porosity on the levee/revetment surface.

Upperbound estimation of reflection ratio

A very simple method describes an upper bound to these results:

$$C_r = 0.79 + 0.11\frac{R_c}{H_s} \qquad \text{for } R_c/H_s < 1.0 \tag{8.4}$$

$$C_r = 0.90 \qquad \text{for } R_c/H_s \geq 1.0 \tag{8.5}$$

Seelig formula

Reflections from smooth or armoured slopes may be described by a simple formula derived by Seelig (1983) and adapted by Allsop (1990) for random waves.

$$C_r = \frac{a\,\xi_m^2}{b + \xi_m^2} \tag{8.6}$$

where a and b are constant parameters depending on surface roughness and permeability. Allsop and Channel (1989) derived coefficients for smooth and armoured slopes, with wave conditions in the ranges $0.004 < s_m < 0.052$, and $0.6 < H_s/\Delta D_{n50} < 1.9$ where Δ is the relative buoyant density and D_{n50} the median nominal diameter. Some typical values are given in Table 8.2 and represented in Figure 8.3.

Table 8.2 *Values of the coefficient a and b in equation*

Slope type	*a*	*b*
Smooth	0.96	4.80
Armourstone – two layers	0.64	8.85
Armourstone – one layer	0.64	7.22

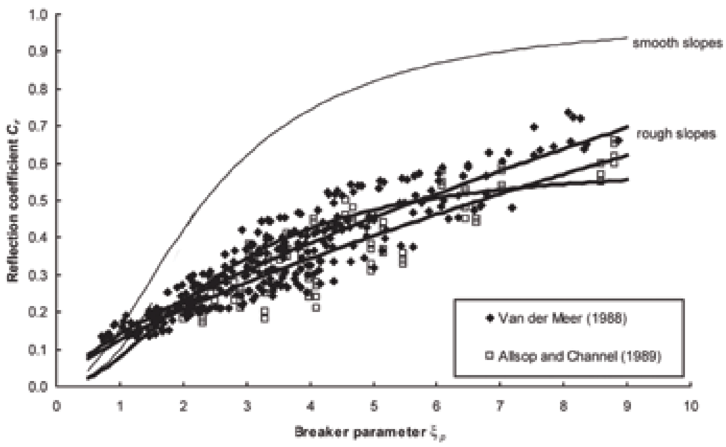


Figure 8.3 *Wave reflections for slopes (CIRIA; CUR; CETMEF, 2007)*

8.2.1.3 Wave run-up

The process of wave up-rush on a slope will reach a run-up level at its upward extent. Run-up levels (R_u) are defined vertically relative to the static water level used for those calculations(Figure 8.2). The run-up level most closely associated with setting levee crest levels is the two per cent exceedance level, $R_{u2\%}$. The run-down level at the same exceedance level, $R_{d98\%}$, may be useful in determining the lower extent of armouring on the levee/revetment face. For structures where methods are not available to estimate overtopping, estimates of extreme wave run-up level(s) may be required. Wave run-up depends primarily on the structure slope angle, and the incident wave steepness. Two methods for estimating run-up are presented. One method is based solely on geometry of the slopes (Box 8.1), while the other is based on surf similarity.

Box 8.1 *Geometrical methods for run-up calculation*

Wassing formula (1957)

For many years, the Netherlands used a simple formula for estimating irregular wave run-up for milder slopes verifying $\tan\alpha \leq 1/3$ (Wassing, 1957):

$$\boxed{R_{u,2\%} = 8\,H_{1/3}\,\tan\alpha} \tag{8.7}$$

where α is the slope angle of the structure (°) as previously defined and $H_{1/3}$ is average of the highest one-third of the waves.

Saville method (1958)

It is one of the most widely-used methods for predicting run-up over complex geometries (Saville, 1957). It is based on the preliminary definition of a hypothetical average or 'effective' slope β of the entire active surf zone, extending between the wave break point and the run-up limit.

$$\boxed{\tan\beta = \frac{R_u + h_b}{X_{Ru} + X_b}} \tag{8.8}$$

where X_b is the horizontal distance from the shoreline to the breakpoint (m), and h_b is the incipient breaking depth (m).

Box 8.1 ***Geometrical methods for run-up calculation (contd)***

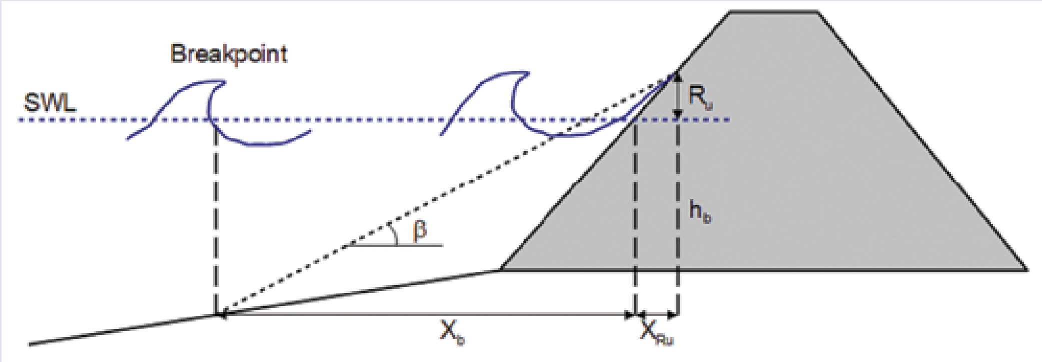


Figure 8.4 ***Definition of effective slope for idealised beach profiles***

Combining the slope equation with Hunt's formula:

$$\boxed{R_u = \tan\beta\sqrt{H_0 L_0}} \quad (8.9)$$

where L_0 is the deep-water wave length and H_0 the deep-water wave height. For general application over arbitrary beach geometries, the run-up is estimated iteratively by following the procedure:

1 A run-up limit is assumed.
2 An average slope is calculated from the break point to the assumed run-up limit.
3 Run-up is estimated using an average slope in empirical design curves.
4 Calculated run-up is compared to the initially assumed value.

This general procedure is time-consuming. However, in the case of a known uniform slope, the problem is simplified and it exists as an analytical solution:

$$\boxed{R_u = \frac{\tan\beta}{2}\left(X_b - \sqrt{H_0 L_0}\right)\left[\sqrt{1 + \frac{4\,h_b\sqrt{H_0 L_0}}{\tan\beta\,\left(X_b - \sqrt{H_0 L_0}\right)^2}} - 1\right]} \quad (8.10)$$

This approach is applicable to smooth slopes (following Table 8.3). Generalisation of this approach to rough slopes may be done by considering a reduction factor γ_f = 0.67.

Table 8.3 ***Run-up estimation for smooth slopes, from (CFBR, 2012)***

Slope	**1/3**	**1/2.5**	**1/2**
H_0/L_0 = 0.1	1.15	1.40	1.90
H_0/L_0 = 0.08	1.37	1.64	2.00
H_0/L_0 = 0.07	1.49	1.73	2.00

Surf similarity methods

The relative run-up level, R_u/H_s may be related to the peak surf similarity parameter or Iribarren number ξ. Taking into account the influence of berms (γ_b), of slope roughness (γ_f) and wave obliquity (γ_β) the relative run-up level may take the general form:

$$\frac{R_{u,p}}{H_s} = \gamma_b\,\gamma_f\,\gamma_\beta\,(A\,\xi + B) \quad (8.11)$$

where A and B are fitting coefficients depending on slope permeability and target probability p for run-up estimation (%). This formula may be completed by an upper bound fit to the data.

Hunt's method (1959)

For surging regular waves on plane, impermeable slopes, Hunt recommended the following equation:

$$R_{u,2\%} = 2.3\,H_s\,\xi_0 \quad (8.12)$$

Ahrens method (1981)

This method accounts for wind-induced waves following a Rayleigh distribution. The run-up level for a probability of exceedance p, $R_{u,p}$, may be calculated using the following equation:

$$R_{u,p} = 0.67\, H_s\, \xi_0^{0.51} \sqrt{\frac{-\ln p}{2}} \tag{8.13}$$

EurOtop method (Pullen *et al*, 2007)

For example, run-up level at two per cent exceedance level, given by $R_{u2\%}$ may be calculated using equations from Pullen *et al* (2007):

$$R_{u,2\%} = \gamma_f\, \gamma_\beta\, H_{m0}\, \min\left[1.65\, \gamma_b\, \xi_{m-1.0}\, ;\, 4.00 - \frac{1.50}{\sqrt{\xi_{m-1.0}}}\right] \tag{8.14}$$

where:

$R_{u,2\%}$ = wave run-up height exceeded by two per cent of incoming waves (m)
H_{m0} = spectral significant wave height (m)
γ_b = influence factor for a berm (-)
γ_f = influence factor for roughness on the slope (-)
γ_β = influence factor for oblique wave attack (-)
$\xi_{m-1,0}$ = surf similarity parameter (-)

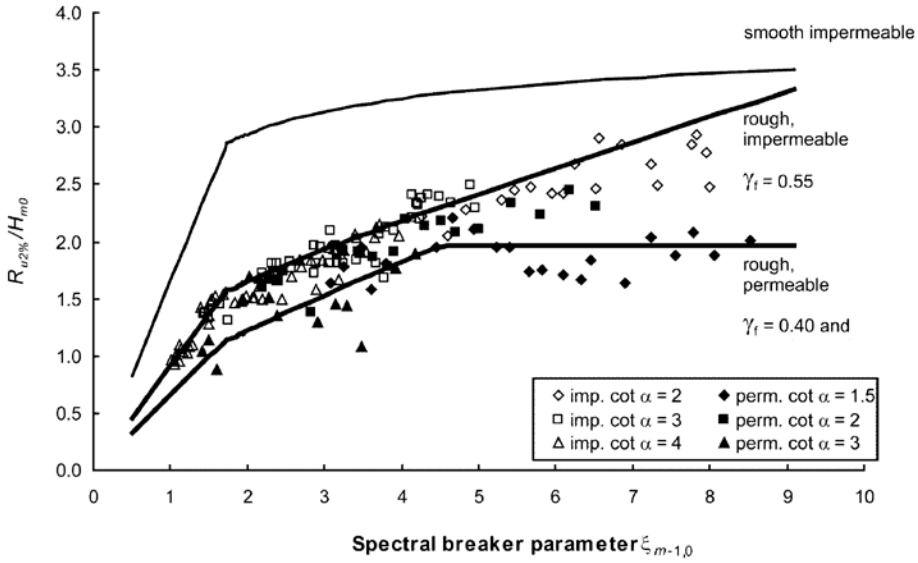


Figure 8.5 ***Relative run-up on rock slopes (permeable or impermeable core), compared to smooth impermeable slopes (from CIRIA; CUR; CETMEF, 2007)***

Pohl method (1997)

The surf parameter is also used for the classification of breaking behaviour and breaker types. For small surf parameters breaking waves (spilling, plunging, surging) can be expected whereas for $\xi_{m-1,0} > 2$ to 3 nonbreaking waves (reflection) are typical. As in a wave spectrum a wide spread of wave parameters may be included as there are both breaking and nonbreaking waves influencing the run-up process in the transition zone. This was taken into account by the formula (Pohl 1997, Pohl and Heyer 2005, Figure 8.6):

$$R_{u,2\%} = P\, R^{nb}_{u,2\%} + (1 - P)\, R^{b}_{u,2\%} \tag{8.15}$$

with

$P = 1 - exp(\xi_0/3.6)^{2.25}$

This formula considers the statistical $R_{u,2\%}$ run-up height to consist of a fraction P of nonbreaking waves and a fraction $1–P$ of breaking waves. In other words P is assumed to be the probability that no breaking takes place and $1–P$ is the breaking probability.

For example, the run-up of 'breaking waves' on smooth slopes may be calculated by means of the Hunt/Battjes formula:

$$R^{b}_{u,x\%} = k_r \, k_x \sqrt{H_m L_m} \, \tan\alpha \tag{8.16}$$

Using k_r = 1.0 on smooth slopes and k_x = 2.23 as a dimensionless parameter for the run-up exceedance probability of two per cent yields, with $H_m = 0.63H_s$:

$$R^{b}_{u,x\%} = 1.77 \, H_{m0} \, \xi_{m-1.0} \tag{8.17}$$

For 'nonbreaking waves', the run-up may be calculated as:

$$R^{nb}_{u,x\%} = 1.89 \, H_{m0} \sqrt{\frac{\pi}{2\alpha}} \tag{8.18}$$

This yields almost identical results for breaking waves ($\xi_{m-1,0}$<2). In the transition zone this gives a local maximum for the normalised run-up $R_{2\%}/H_{m0}$ at $\xi_{m-1,0} \approx 3$. The weakness of other approaches, that the results either tend to infinity with growing (breaking) or dropping (nonbreaking) $\xi_{m-1,0}$ or that different formulae have to be used for different ranges of validity, could be overcome with this approach (Figure 8.6). For large $\xi_{m-1,0}$ (nonbreaking, vertical wall) the $R_{2\%}/H_{m0}$ curve by Pohl und Heyer (2005) goes asymptotically towards the value of $R_{2\%}/H_{m0} \rightarrow 2$, which stands for full reflection and is known as standing wave (clapotis), from theory.

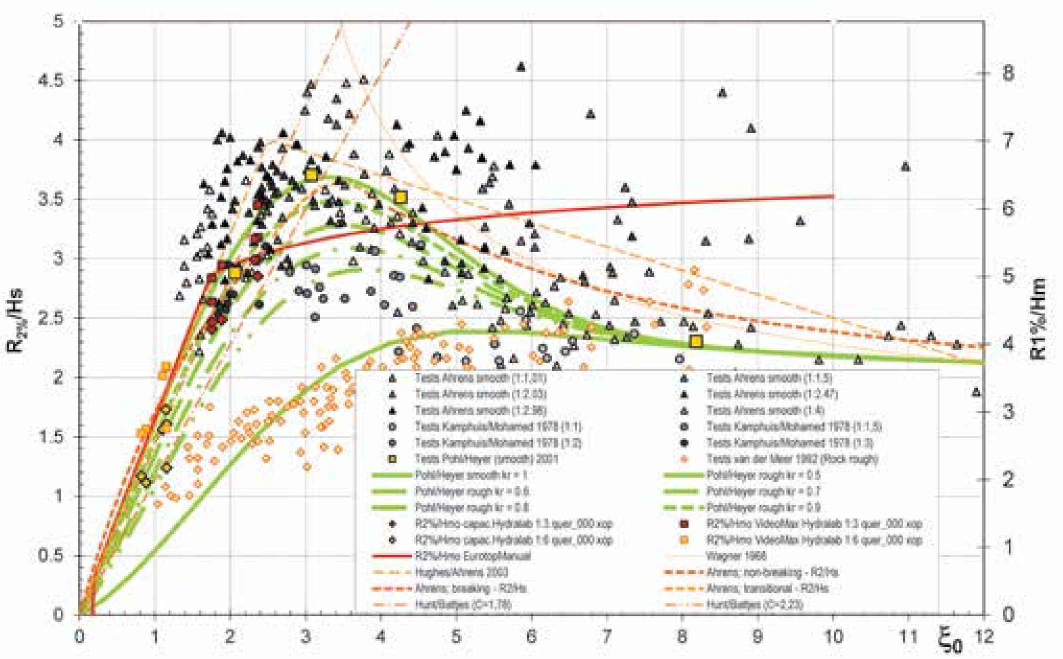


Figure 8.6 Normalised wave run-up on smooth and rough slopes (Pohl et al, 2012)

Van der Meer method (2002)

$$\frac{R_{u,2\%}}{H_{m0}} = \gamma_f \, \gamma_\beta \, \gamma_b \, \gamma_p \begin{cases} 1.77 \, \xi_{m0} & 0.50 \leq \gamma_b \, \xi_{m0} \leq 1.80 \\ 4.30 - \dfrac{1.60}{\sqrt{\xi_{m0}}} & 1.80 \leq \gamma_b \, \xi_{m0} \end{cases} \tag{8.19}$$

where γ_p is an influence factor for structure permeability (-).

For the consideration of further influences on the wave run-up the coefficients γ_b for the influence of a berm, γ_f for the slope roughness, and γ_β for oblique incident waves are used.

Van Gent formula (2000 and 2001)

An alternate form of the run-up equation was recommended by Melby (2012) after evaluating several popular empirical methods for predicting wave run-up on structures and beaches. Melby recommended the run-up equation by Van Gent (2000 and 2001) as the best predictor for impermeable coastal structures such as levees. It gives potentially smaller values than Pullen *et al* (2007) and is defined as:

$$\frac{R_{u,2\%}}{H_{m0}} = \gamma_f \gamma_\beta \gamma_b \gamma_p \begin{Bmatrix} 1.35\xi_{m-1.0} & \xi_{m-1.0} \leq 1.7 \\ 4.7 - 4.1/\xi_{m-1.0} & \xi_{m-1.0} \geq 1.7 \end{Bmatrix} \quad (8.20)$$

Influence of slope roughness

Different values of γ_f are suggested by Pullen *et al* (2007), and a few examples are listed in Table 8.4.

Table 8.4 Examples of influence factor accounting roughness on the slope

Type of armour	Reduction factor, γ_f
Smooth concrete/asphalt	1.0
Concrete with roughness elements	0.7–0.95
Grass slope	0.9–1.0
One layer rock armour	0.55–0.6
Two layers rock armour	0.50–0.55

Influence of wave obliquity

The angle of wave attack, β (°), is defined as the angle between the direction of propagation of waves and the axis perpendicular to the structure (for normal wave attack: $\beta = 0°$). There are many approaches existing for the estimation of γ_β for the oblique wave approach. The coefficient γ_β is defined as the quotient of normalised run-up height with incident wave angle $\beta \neq 0°$ and the normalised run-up height for straight approaching $\beta = 0°$. This can be calculated using the equations by Wagner and Bürger:

$$\gamma_\beta = 0.35 + 0.65 \cos \beta \quad (8.21)$$

and by de Waal and van der Meer for a short-crested sea:

$$\gamma_\beta = 1 - 0.022\,|\beta| \quad (8.22)$$

Particularly for a very oblique wave approach ($\beta \rightarrow \pm 90°$) the limiting values are partly not plausible, so the application of these equations should be limited to angles $\rightarrow \beta < |\pm 50°|$.

Influence of berms

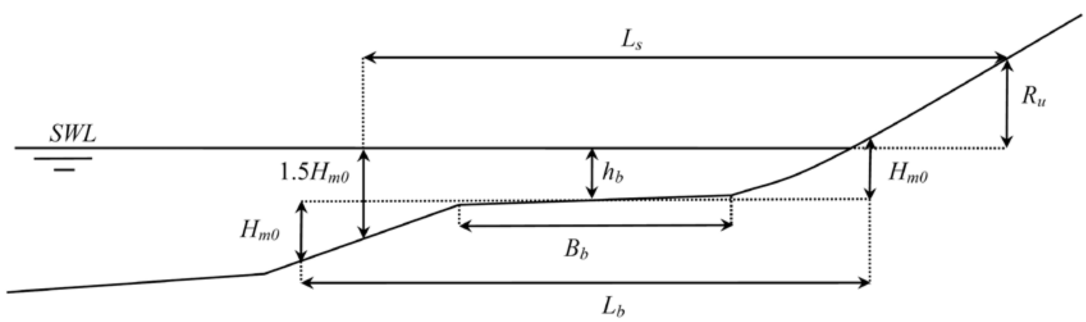


Figure 8.7 Definition for geometrical parameters of the berm

The berm influence area (Figure 8.7) is defined by the horizontal distance L_s (m) between the point corresponding to the level $SWL–1.5H_{m0}$ and the level $SWL+R_u$. The first step consists of defining the representative slope. For bermed slopes, it may be estimated as:

$$\tan\alpha = \frac{R_u + 1.5H_{m0}}{L_s - B_b} \tag{8.23}$$

For slopes with complex profiles, using an average structure slope given by:

$$\tan\alpha = \frac{4\,H_{m0}}{L} \tag{8.24}$$

where L is the horizontal distance between points on the levee at 2 H_{m0} above and 2 H_{m0} below the still water line.

So, the surf similarity parameter may be defined as $\xi_{m-1,0}$. The berm reduction factor is determined by the expression:

$$0.6 \leq \gamma_b = 1 - \frac{B_b}{L_b}(1 - k_h) \leq 1.0 \tag{8.25}$$

with k_h defined according to the following equation:

$$k_h = \frac{1}{2}\left[1 - \cos\left(\frac{\pi\,h_b}{x}\right)\right] \tag{8.26}$$

where:

x = R_u if berm level is above SWL or $2H_{m0}$ if berm level is below SWL

k_h = 1 if berm is outside influence area

H_b = the incipient breaking depth (m)

8.2.1.4 Wave run-down

Run-down is usually not as important as run-up, which can lead to overtopping, but both give an idea of total water excursion on a slope. Run-down is usually larger on impermeable slopes (ie concrete or reveted) as the water cannot percolate into the bottom as it does for permeable slopes (ie grassy or dirt). According to van der Meer (1988), wave period and bottom slope angles also have an effect on run-down. So, depending on the wave and slope characteristics, there may be a possibility of erosion on the slope due to the run-down velocity on the levee.

8.2.1.5 Wave overtopping

For coastal or lake seawalls/levees, the hydraulic response of most concern is wave overtopping, commonly expressed by the mean overtopping discharge per unit length along the defence q (Box 8.2), but sometimes as the number or percentage of incident waves overtopping the crest, $N_{wo\%}$. Noting that wave heights are distributed randomly, it will be seen that most practical levees on a sea or lake shore may experience some wave overtopping under extreme conditions. So, calculations of wave run-up levels are generally less useful in design than overtopping discharges. The simple method developed by Owen (1980) is described in Box 8.3.

Box 8.2 Wave overtopping on coastal flood embankments (from Hewlett et al, 1987)

Conventional coastal engineering practice is to adopt an embankment crest level and profile, which limits the mean overtopping discharge intensity at design upslope wave and water level conditions to a maximum acceptable value. Mean overtopping discharge intensity has been determined by laboratory tests for regular embankment profiles under various upslope wave and water level conditions.

There are no universally accepted values for maximum allowable mean overtopping discharge intensity for coastal defences. Goda (1971) recommends the following maximum values of mean overtopping discharge intensity, $\overline{q}$, for stability of grassed and paved protection to the crest and downslope face of coastal flood embankments:

	$\overline{q}$ (m^3/s/m)
Crest and downslope paved	0.05
Crest paved, downslope grassed	0.02
Crest and downslope grassed only	0.005

Box 8.3 *Simple method of assessing overtopping (from Owen, 1980)*

Around the UK, many rural seawalls have a simple cross-section, with slopes of 1:2–1:4. The overtopping performance of these structures under random waves was studied intensively in the late 1970s. Overtopping discharges under random waves were related to freeboard R_c, and wave parameters H_s and T_m. The prediction method developed by Owen (1980 and 1982) relates dimensionless parameters Q* and R* by an exponential equation with a roughness coefficient, r, and coefficients *A* and *B* for each slope angle:

$$Q^* = A \exp\left(-B\frac{R^*}{r}\right) \tag{8.27}$$

where:

$Q^* = q/(gT_m H_s)$

$R^* = R_c/T_m(gH_s)^{0.5}$

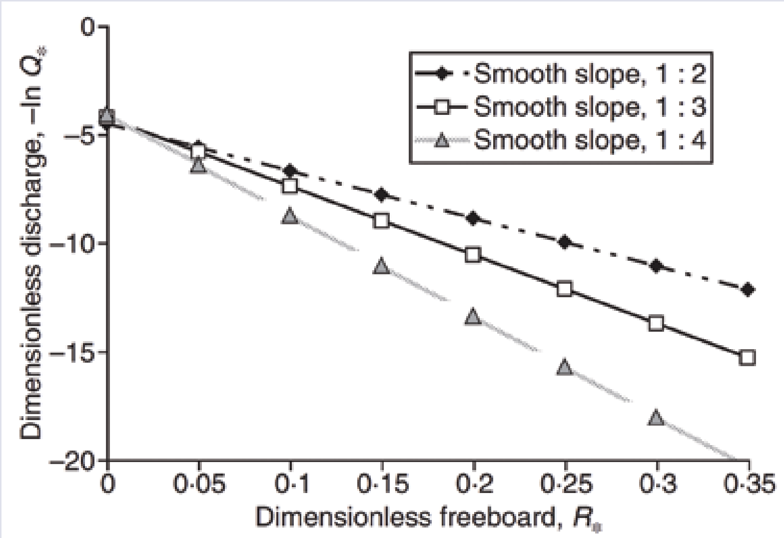


Figure 8.8 ***Overtopping for simple slopes (after Allsop et al, 2005)***

and values of coefficients *A* and *B* are:

Slope	*A*	*B*
1: 1.0	0.0079	20.1
1: 1.5	0.0102	20.1
1: 2.0	0.0125	22.1
1: 3.0	0.0163	31.9
1: 4.0	0.0192	47.0

The form of Equation 8.27 is illustrated in Figure 8.8 where *Q** is plotted against *R**. For levees with particularly small relative freeboards and/or large wave heights, the prediction lines come together at one point, indicating that the slope angle no longer has much influence in controlling overtopping. At this point, the slope is said to be ‘drowned out’. Over the normal range of freeboards, the discharge characteristics for slopes 1:1, 1:1.5 and 1:2 are similar, but overtopping reduces significantly for slopes shallower than 1:2.

Owen’s method (1980) was developed initially for smooth slopes only, but the use of the roughness factor, *r*, allowed its use for rough, and even armoured slopes. The main advantages of Owen’s method are its simplicity, and availability of data to support particular coefficients. The disadvantages are that the method was not explicitly developed for armoured slopes, the coefficient *r* is not always constant, and values of r have not been measured for some types of armour. The range of validity of this approach generally considered is $0.05 < R^* < 0.3$. Other approaches have been developed for configurations or armour not covered by Owen’s original analysis:

1. Use Owen’s method (1980) and coefficients *A* and *B* with values of *r* derived from tests with the appropriate armour and slope geometry.
2. Use Owen’s general equation, but with new values of *A* and *B* derived for similar section geometry and armouring, and with *r* = 1

Overtopping configurations

There are four configurations of overtopping (Figure 8.9) that can affect levees:

- wave-only overtopping with positive freeboard
- wave-only overtopping with zero freeboard
- surge-only overflow with negative freeboard
- combined surge and wave overtopping with negative freeboard.

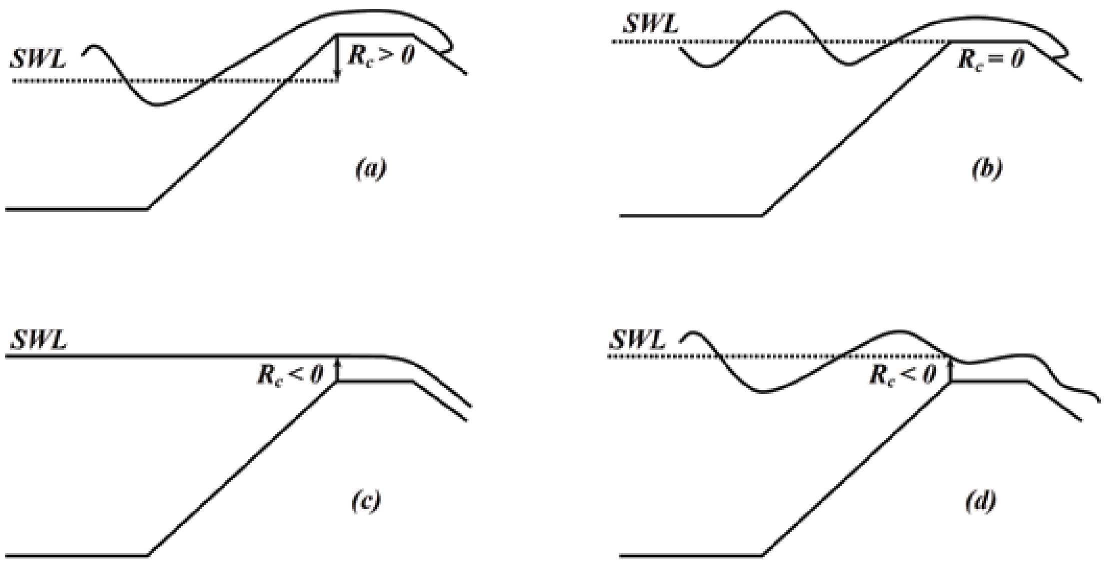


Figure 8.9 Four types of overtopping on levees: wave overtopping for positive freeboard (a), wave overtopping for zero freeboard (b), overflow for negative freeboard (c), overflow and overtopping for negative freeboard (d) (from Pullen et al, 2007)

Wave-only overtopping with positive freeboard

Van der Meer (2002) and Pullen *et al* (2007) revised the average wave overtopping discharge q_w developed by van der Meer and Janssen (1995) for probabilistic design. In cases where heavy breaking is present (ie $\xi_{m-1.0} > 5.0$), long waves influence the predictions leading to underestimation of wave overtopping. When $\xi_{m-1.0} > 7.0$, the following equation should be used for wave-only overtopping with positive freeboard:

$$Q = \frac{q_w}{\sqrt{g\,H_{m0}^3}} = \begin{cases} \dfrac{0.067\,\gamma_b\,\xi_{m-1.0}}{\sqrt{\tan\alpha}} \exp\left(-\left\{\dfrac{4.75\,R_c}{\gamma_f\gamma_b\gamma_\beta H_{m0}\xi_{m-1.0}}\right\}\right) & \xi_{m-1.0} < 5.0 \\ 10\exp\left(-\left\{\dfrac{R_c}{\gamma_f\gamma_b H_{m0}\,(0.33+0.22\xi_{m-1.0})}\right\}\right) & \xi_{m-1.0} > 7.0 \end{cases} \tag{8.28}$$

Use linear interpolation between these two equations for breaking waves $5 < \xi_{m-1.0} < 7$. The following equation is the maximum value that the dimensionless average wave overtopping discharge Q should not exceed.

$$Q < 0.2\,\exp\left(-\left\{\frac{2.60\,R_c}{\gamma_f\gamma_b H_{m0}}\right\}\right) \tag{8.29}$$

Wave-only overtopping with zero freeboard

Schüttrumpf (2001) and Schüttrumpf *et al* (2001) derived equations for average wave overtopping discharge q_w based on model tests with smooth slopes between 1:3 and 1:6. His results are also presented in Pullen *et al* (2007) for overtopping resistant levees when the water level comes close to the crest as:

$$Q = \frac{q_w}{\sqrt{g\,H_{m0}^3}} = \begin{cases} 0.0537\,\xi_{m-1.0} & \xi_{m-1.0} < 2.0 \\ 0.136 - \dfrac{0.226}{\xi_{m-1.0}^3} & \xi_{m-1.0} \geq 2.0 \end{cases} \tag{8.30}$$

Surge-only overflow with negative freeboard

If the water level is higher than the crest, then overtopping can be modelled as flow over a broad-crested weir as described for open channel flow (Henderson, 1966). The surge-only overflow discharge q_s is defined as:

$$q_s = 0.5443\sqrt{g\,|-R_c^3|} \approx 0.6\sqrt{g\,|-R_c^3|} \tag{8.31}$$

where R_c is the negative relative crest height or overflow depth (ie difference between surge elevation and levee crest elevation). The second half of this equation is the approximation used by Pullen *et al* (2007).

Combined surge and wave overtopping with negative freeboard

The last form of levee overtopping is the combined wave and surge overtopping. In this condition, both the wave only and the surge only conditions occur together. Every wave has the possibility of overtopping the levee and the peak instantaneous discharge can be several times the value of the steady overflow discharge. The overtopping flow is unsteady in time and spatially non-uniform. Pullen *et al* (2007) suggests an approximation of the average combined wave and surge discharge q_{ws} for $\xi_{m-1.0}$ <2.0 as a superposition of the wave only with zero freeboard and surge only with negative freeboard equations given as:

$$\frac{q_{ws}}{\sqrt{gH_{m0}^3}} = 0.034 + 0.53\left(\frac{-R_c}{H_{m0}}\right)^{1.58} \qquad R_c < 0 \tag{8.32}$$

Hughes and Nadal (2008) conducted laboratory experiments of a trapezoidal levee at a 1:25 scale. Their experiments covered 27 overtopping conditions consisting of three water levels above crest elevation and nine irregular wave height and period combinations. They developed a new empirical equation that expresses the average overtopping discharge per unit length along the levee crest q_{ws} as a function of negative freeboard and incident energy-based significant wave height. The new equation fits the data very well. It was compared with Schüttrumpf *et al* (2001) and Reeve *et al* (2008) overtopping equations and gave lower overtopping rates, but following the same trends. Their combined overtopping q_{ws} is given as:

$$q_{ws} \equiv q_w + q_s = 0.0537\xi_{m-1.0}\sqrt{gH_{m0}^3} + 0.5443\sqrt{g|-R_c^3|} \qquad \xi_{m-1.0} < 2.0 \tag{8.33}$$

Note that R_c has to be entered as a negative number to ensure that the quantity in brackets is positive.

Landward slope erosion potential

The toe of the landward slope is the most common location for the initiation of erosion. The flow accelerates to reach supercritical and proceeds down slope until it reaches the base of the slope where a hydraulic jump develops. Erosion occurs due to the high velocities and turbulence under the hydraulic jump. The erosion typically advances upslope as a headcut develops.

In flow conditions typical of surge only overtopping, the flow becomes steady as a balance is reached between the water momentum and the frictional resistance of the slope material. The Manning equation for the steady flow velocity v_b is defined as:

$$v_b = \left(\frac{\sqrt{\sin\beta}}{n}\right)^{0.6} q_s^{0.4} \tag{8.34}$$

where:

β = the landward slope angle

q_s = steady critical discharge

n = Manning's coefficient

Hewlett *et al* (1987) recommended $n = 0.03$ for slopes of 1:10, decreasing linearly to $n = 0.02$ for 1:3 slopes and steeper. Since the Manning equation was derived for mild slopes, this equation is not strictly valid for steep slopes and significant air entrainment.

The flow down a landward slope for combined wave and surge overtopping is unsteady and more difficult to analyse. Hughes and Nadal (2008) developed an expression for the mean flow thickness h_b and flow velocity v_b on the landward slope as:

$$h_b = 0.4 \left(\frac{q_{ws}^2}{g \sin \beta} \right)^{1/3} \tag{8.35}$$

$$v_b = 2.5 \left(q_{ws}\, g \sin \beta \right)^{1/3} \tag{8.36}$$

Strictly speaking, these equations are only valid for the 1:3 slope and roughness used in the experiments until further research validates the results. In general, the friction factors for grass-covered slopes should be similar to the laboratory roughness, but armoured slopes would have significantly higher roughness factors.

Wave overtopping at flood walls

Waves can overtop a vertical flood wall even when the storm surge elevation is below the top elevation of the wall as illustrated in Figure 8.10. That portion of the wave above the flood wall will tumble over the wall and plunge to the ground under the force of gravity. The quantity of water will vary in time, and the unsteady discharge will be a function of wave height, wave period, and surge elevation relative to the wall. Erosion of unprotected soil will occur as the waves cascade over the wall, but the unsteadiness of the process, coupled with the variation of impact point due to irregular waves, makes scour estimation difficult, if not impossible.

The hydrodynamics of this phenomenon are quite complex because a substantial portion of the incident wave is reflected by the flood wall, and the reflected wave will interact nonlinearly with the incident wave. So, a few simplifying assumptions are necessary for the approximation given here.

Assume the incident waves are reasonably approximated as shallow water waves. Also, assume the incident wave crest height reaches the flood wall without being modified by the reflected wave, ie there is no nonlinear interaction between the incident and reflected wave. Waves in deeper water are symmetrical about the still water level (SWL) with the vertical distance between the wave crest and SWL being the same as the vertical distance between the wave trough and SWL. However, in shallow water the wave crests become more peaked and the troughs become flatter, and the vertical distance between the wave crest and the SWL becomes proportionally larger. For this simple development, assume the distance of the wave crest above the SWL is 70 per cent of the wave height, H, as shown in Figure 8.10.

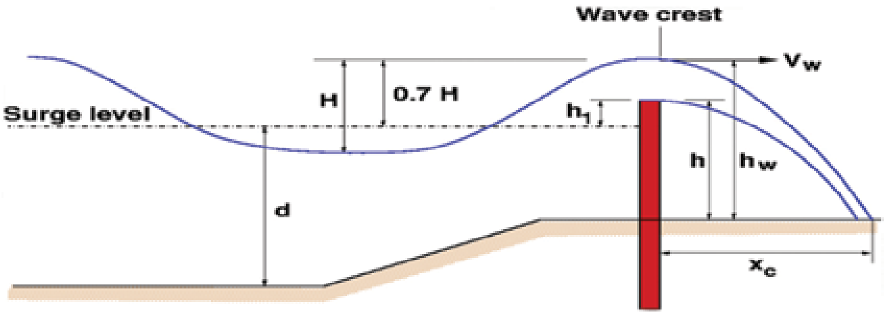


Figure 8.10 Definition sketch of wave overtopping flood wall (USACE, 2008)

1 2 3 4 5 6 7 8 9 10

As the wave crest passes over the flood wall, the orbital velocity of water particles at the free surface will be nearly the same as the wave celerity. Using the expression for wave celerity given by third-order theory for nonlinear, shallow water waves, the horizontal velocity V_w is given by:

$$V_w = \sqrt{g\,(d + H)} \tag{8.37}$$

where:

g = gravitational constant (9.81 m/s²)
d = water depth at the toe of the structure (m)
H = incident wave height (m)

Note that wave celerity is independent of wave period in shallow water, and instead depends only on water depth and wave height. The distance from the wall to where the plunging wave crest impacts the ground level is found using the formulae for an object in free fall having an initial horizontal velocity of V_w and falling a vertical distance h_w.

$$h_w = h + 0.7\,H + h_1 \tag{8.38}$$

where h is the vertical distance between the top of the flood wall and the ground level, h_1 (m, positive or negative depending on surge level relative to the top of the wall) is the distance between the top of the wall and the surge level. If the surge level is lower than the flood wall, h_1 is negative. When the surge overtops the flood wall, h_1 is positive. The vertical fall distance is a function of fall time and gravitation acceleration $h_w = gt^2/2$. So, the fall time for a water particle at the wave crest free surface to fall to the ground level is given by:

$$t_f = \sqrt{\frac{2\,h_w}{g}} \tag{8.39}$$

The horizontal distance traversed by the water particle during this freefall time is simply $x_C = V_w\,t_f$, so that combination of the above equations yields:

$$x_C = \sqrt{2\,(d + H)\,(h + 0.7\,H + h_1)} \tag{8.40}$$

Additional details can be found in USACE (2008). Details for calculating nappe trajectories under wave or surge conditions are similar to those presented in Section 8.2.2.4 for flow over a flood wall. In the presence of waves, the equations shown in Section 8.2.2.4 have to be adjusted to include a variable horizontal velocity produced by the oscillatory wave action. This results in an unsteady, time-dependent estimate of jet position and jet velocity. The force and plunge position will vary over the length of the wave cycle, and landward protection should be designed based on the maximum range of fluctuation over that cycle.

8.2.2 Overflow

Overflow occurs when water on one side of a levee is higher than the top of levee elevation at point(s) along the levee profile. Overflow most often results from the waterside (or coastal) water level being greater than the top of levee elevation. However, it is possible for interior runoff to cause the reverse effect. Overflow can be continuous for a period of time where a design flood level is exceeded, or it may be intermittent as in the case of waves. Overflow may occur for both earthen embankment levees and flood walls of various types. Consequences of overflow range from minor erosion of the landward levee slope to entire failure of the levee component due to progressive erosion that leads to a breach.

In levee analysis and design processes the ability of the levee section to resist erosive forces caused by overflow has to be checked. The potential for erosion depends on the peak flow velocity as well as the depth and duration of overflow. Analysis and design should assess the potential for erosion due to overflow even where overflow is not likely under expected service conditions.

All levee systems are subject to overflow because of natural phenomena. The probabilistic methods used and uncertainties in estimating the water level needed to set top of levee elevations lead to potential overflow. Even where low probability discharges are used for determining levee height, there is risk that a greater magnitude event may happen over the project life (Box 2.8).

Assessment of an existing levee has to consider hydrologic and hydraulic conditions of the watershed and their influence on water levels that may cause overflow. Section 7.3 describes hydrologic and hydraulic models to accomplish this analysis.

The designer of a new levee has to decide on the site conditions and the combination of extreme events under which overflow occurs. Section 9.3.5 describes design considerations for overflow. The main consideration affecting the design decision is the likely consequence of failure and, more importantly, of any effect on life, property and land downstream.

8.2.2.1 Overflow discharge

Overflow discharge is the amount of water transferred across a levee segment and is usually expressed as a unit discharge, q (m³/s/m). Unit discharge is a function of the height of water above the levee crest and physical characteristics of the levee crown and length of overflow section. Because levees are generally aligned parallel to a river's main flow direction overflow is similar to a lateral diversion. This means that flow over a levee is unsteady and gradually varied due to the slope of the water surface profile along the river (Figure 8.11). This is compounded further if there is a non-level crest elevation as in the case of levee overflow. The addition of waves adds additional complexity in unsteady flow conditions (Figure 8.12).

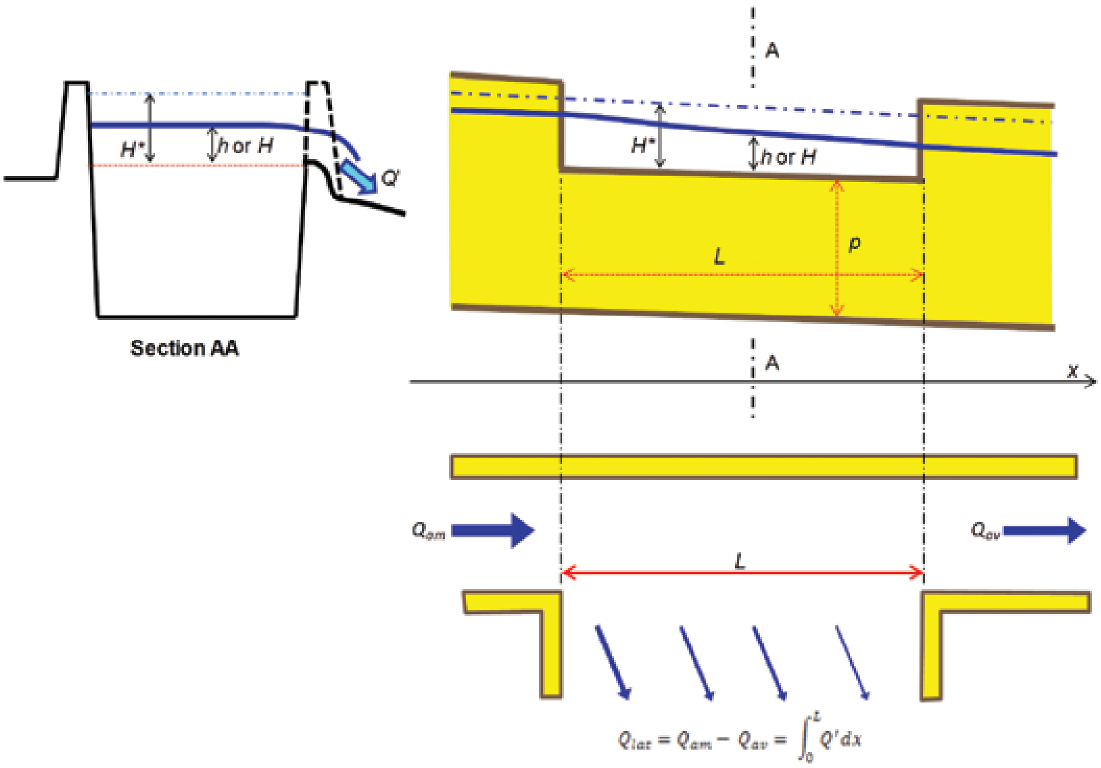


Figure 8.11 Illustration of gradually varied discharge over lateral overflow section (from Degoutte, 2012)

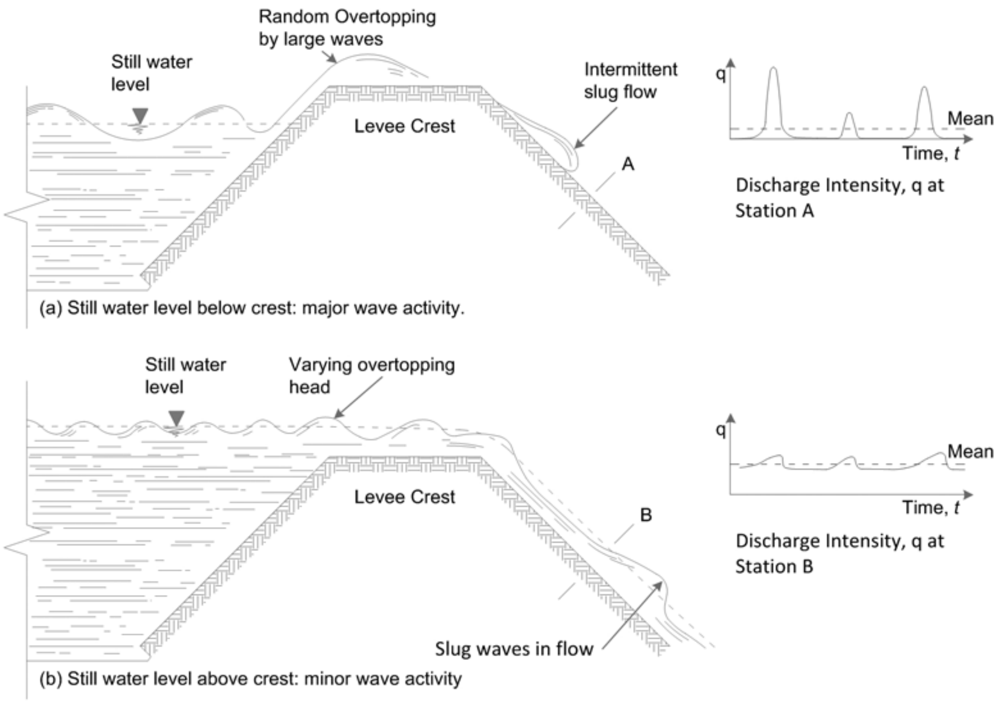


***Figure 8.12** Wave effects on creating unsteady overtopping discharge (from Hewlett et al, 1987)*

A simplified method for estimating uniform overflow discharge at a levee embankment cross-section is the standard broad-crested weir equation:

$$Q = C_d\, L\, H^{3/2} \tag{8.41}$$

$$q = C_d\, H^{3/2} \tag{8.42}$$

where Q (m³/s) is total discharge, q (m³/s/m) is unit discharge per length of overtopping section, C_d is a weir discharge coefficient, L (m) is the length of the overflow section, and H (m) is the head above the crest of the weir.

Basic approach

Assuming that the crest has a constant top elevation, the overflow is well approximated by the classic hydraulics problem of flow over a weir. Also, consider an additional head H_a corresponding to the velocity of approach V_a defined by:

$$H_a = \alpha \frac{V_a^2}{2\,g} \tag{8.43}$$

where α is the kinetic energy correction factor allowing non uniformity of velocity in the cross-section model. The linear discharge is given by:

$$q = \frac{2}{3}\, C\, \sqrt{2\,g}\, \left[H^{*3/2} - H_a^{3/2}\right] \tag{8.44}$$

where:

q	=	flow rate per unit length (m²/s)
C	=	flow coefficient (-)
g	=	gravitational constant (9.81 m/s²)
H	=	head over the crest (m)
H^*	=	$H+H_a$, equivalent head over the crest (m)

Experience suggests that typical values for C may range from 0.5 to 2.6 for levee overflowing situations. The lower value represents conditions where overflow is perpendicular, or nearly so, to the main channel flow direction. Higher values for C may be used when overtopping occurs on the convex side of bends where overflow is more closely aligned with the main channel flow direction (Figure 8.13).

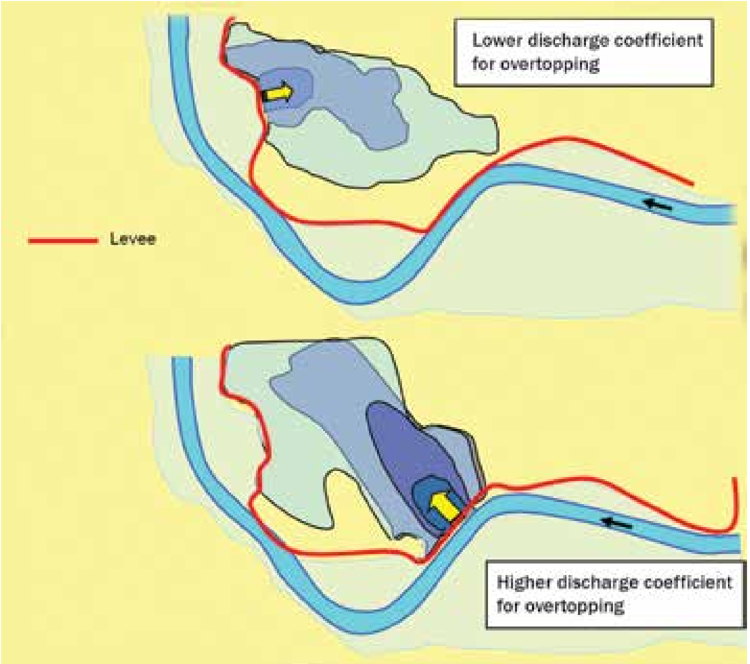


Figure 8.13 ***Angle of main channel current direction and its effect on overflow weir coefficient (from Degoutte, 2012***

The difficulty of this approach is that there is a longitudinal water surface slope along the river's length and levee crest elevations are rarely constant, both resulting varying head along the length of levee that is subject to overflow. Estimated overflow rates determined from the simplified approach should be compared to numerical model results obtained during site characterisation (Section 7.3), or from detailed numerical models of the overflow or spillway segment. Numerical model results obtained from site characterisation are required to determine the overflowing head, H (m). Unsteady model results also provide the overtopping discharges (Q m^3/s or q m^3/s/m) and heads that occur at various time steps through the flood hydrograph.

Hager procedure

Hager (1987) developed a procedure for calculating a value for weir coefficient C_d to be used with the standard weir Equation 8.41 as:

$$C_d = \frac{3}{5}\sqrt{g}\,C_0\sqrt{\frac{1-W}{3-2y-W}}\left\{1-(\beta+s_0)\sqrt{\frac{3(1-y)}{y-W}}\right\} \tag{8.45}$$

with:

W = $p/(H_t+p)$, $y = (H+p)/(H^*+p)$

where:

H = height of water surface above the weir (m)
p = height of weir above the ground (m)
H^* = height of the energy gradeline above the weir = $H + H_a$ (m)
s_0 = average main channel bed slope (rd)
β = main channel contraction angle (0 if the weir is parallel to the main channel) (rd)
C_0 = f (weir shape), base discharge coefficient as shown in Table 8.5

The main channel contraction angle used in Equation 8.45 is shown graphically in Figure 8.14.

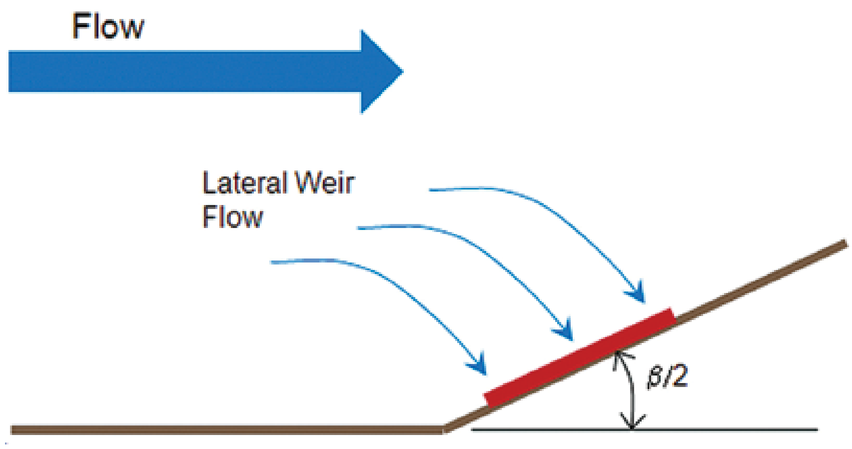


Figure 8.14 Angle β for calculation of Hager (1987) weir coefficient

Table 8.5 Values for C_0 (from Hager, 1987)

Weir type	Value of C_0
Sharp crested	1.0
Zero height	$\frac{8}{7}$
Broad crested (b = weir width in direction of flow)	$1 - \frac{2}{9\left[1+\left(\frac{H_t}{b}\right)^4\right]}$
Round or ogee crested (*r* = weir crest radius)	$\frac{\sqrt{3}}{2}\left[1+\frac{\frac{22}{81}\left(\frac{H_t}{r}\right)^2}{1+\frac{1}{2}\left(\frac{H_t}{r}\right)^2}\right]$

Hager's (1987) equation takes into consideration the effects of flow depth, approaching velocity, lateral outflow direction and side weir channel shape in determining a value for the coefficient of discharge.

Sharp-crested levees

A similar approach can be applied where overflow occurs at a flood wall where the crest is narrow. Flow over wall type structures creates a jet that does not remain in full contact with the landside face of the wall.

Although viscous and surface tension effects are usually of secondary importance, such effects cannot be entirely neglected when the flood wall width (B) is not negligible relative to the head (H). Values of C_o range from approximately 0.58 to 0.78. Empirical formulae may be able to assess this phenomenon.

Francis' formula

This is one of the most commonly used formulae for calculating discharge. The flow coefficient is expressed, excluding lateral contraction due to end effects, as:

$$C_d = 0.623 \tag{8.46}$$

Bazin's formula

Based on this formula, the flow coefficient is given by

$$C_d = 0.405 + \frac{0.003}{H + H_a} \tag{8.47}$$

Rehbock's formula

Based on this formula, the flow coefficient is given by:

$$C_d = 0.605 + 0.08\frac{H}{z} + \frac{0.001}{H} \tag{8.48}$$

where z is the crest height (m).

The principal concern for flow over a structural flood wall is the potential for scour where the overflow jet impinges on the landside of the structure (Section 8.2.2.4).

8.2.2.2 Spillways and fuse plugs

Design calculations take hydraulic models developed during site characterisation (Chapter 7) and expand them to assess spillway/fuse plug components of a levee system. Models developed during site characterisation include solution of weir flow equations and an assessment of the effects of overflow (if any occurs) on the flood hydrograph but do not typically optimise spillway or fuseplug design (Figure 8.15). Additional detail is added during design in order to evaluate the spillway/fuse plug. Results from the unsteady flow models are used to proportion spillway/fuse plug features. In some cases, physical models are used to evaluate spillway/fuse plug performance and to adjust the design obtained from numeric calculations.

In basic terms, flow hydrographs that describe current conditions in the watershed (developed during site characterisation in Chapter 7) are routed through the system with desired levee alignments to evaluate how the levee may alter the magnitude and timing of discharges. Figure 8.15 shows this as the 'hydrograph with spillway'. If necessary, to manage and control overflow due to this hydrograph, a purpose designed overflow section may be included as a part of the levee plan. The capacity of the overflow section is determined by the depth of water above the overflow section crest, the length of section that overflows, and the length of time that overflow occurs (Figure 8.15). Water diverted by overflow reduces the discharge rates in the main conveyance system (Figure 8.15). The effects of an adequately designed overflow section prevents the riverward stage from exceeding top of levee elevations along other parts of the levee. However, water levels on the landward side of the overflow section experience increased water levels as depicted in the lower stage hydrographs in Figure 8.15.

The weir equations (8.46 to 8.48) above represent the simplest case for calculating overflow discharge. The equations provide a reasonable estimate for overflow discharge when the levee embankment or spillway configuration is in the form of a weir. There are spillways that use various types of gates and even explosives to control and regulate flow into the spillway outlet. Where gates are used the discharge characteristics of the gates and associated structures and their operation will determine the amount of water that leaves the primary conveyance system and enters the landward area. Fuse plugs are segments of a levee embankment designed to a lower crest elevation to permit overflow. In some cases, fuse plugs incorporate provisions for erosion and eventual breach of the embankment. Analysis of fuse plug overflow sections is complex due to the largely unknown rate of breach development.

Detailed hydraulic design for spillways/fuse plugs are beyond the scope of the handbook. Specific approaches and methodology for spillway design can be found in Degoutte (2012) and USACE (1992). Spillway/fuse plug detailing for levee systems will typically involve an iterative process to achieve a balance in spillway performance and required spillway structural requirements with respect to unit discharges, frequency of use and resulting erosive forces.

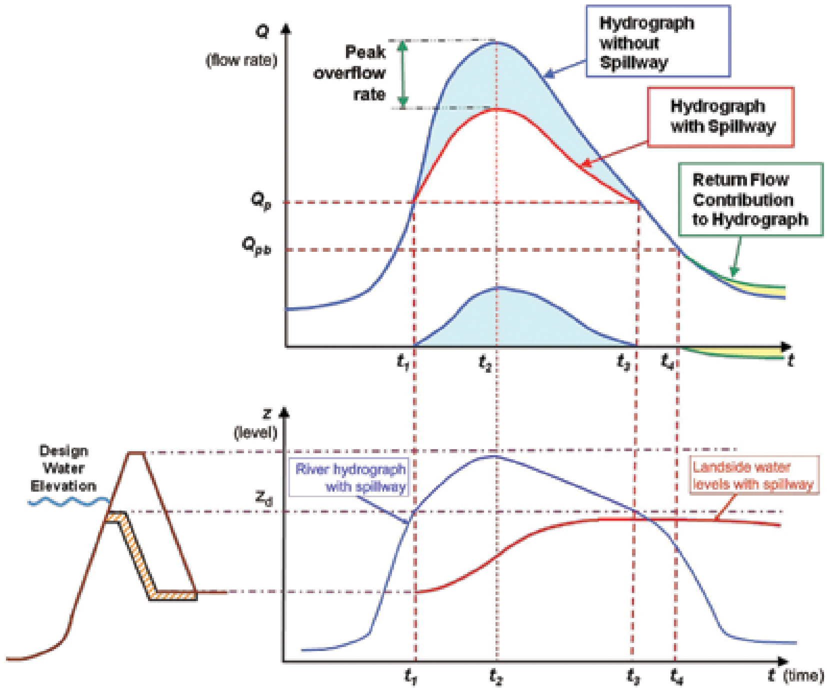


Figure 8.15 Effect of spillway/fuse plug on flood hydrograph (from Degoutte, 2012)

Blue shaded segments of hydrographs in Figure 8.15 are diverted by overflow at the spillway. The yellow segment represents where diverted water returns to the river. Depicted spillway hydrographs would be obtained if there is no limit on spillway (infinite storage volume downstream).

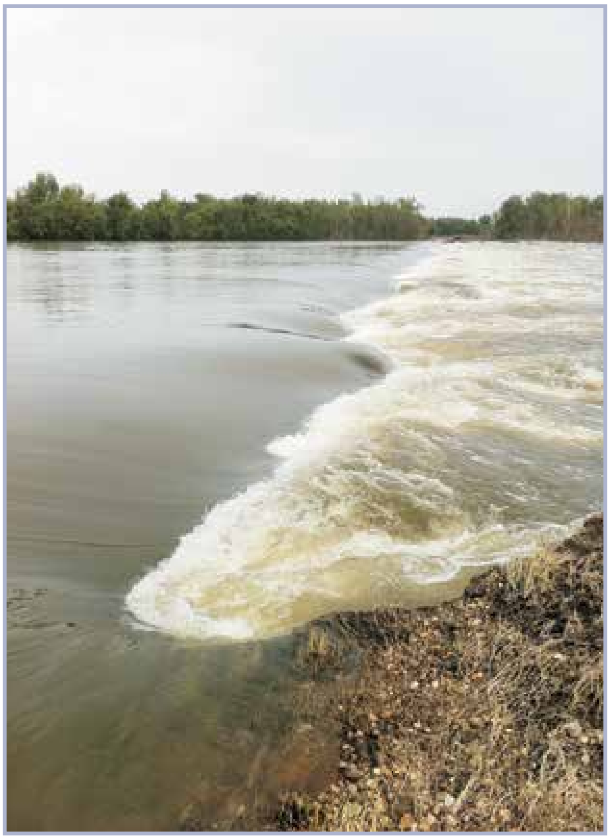
With intended overflow sections and with overtopping of embankments where wave activity on the waterside slope is limited (such as those associated with small lakes or river flood defences), threshold discharge conditions and design discharge are usually related to events with a defined probability of occurrence (or risk).

Where embankments are subject to substantial wave activity on the waterside slope (lakes, estuaries or large river systems with considerable wind fetch), overflow conditions are likely to be caused by a combination of extreme water level and wave action. In such cases, overflow discharge will fluctuate and the value of peak design discharge for protection measures is a matter of engineering judgment. Owing to the random nature of wind generated waves, the local peak discharge intensity, when a particular section of the embankment is overtopped by a large wave, could be between one and two orders of magnitude larger than the time-averaged mean discharge intensity (Figure 8.12).

8.2.2.3 Hydraulic performance of overflow spillways at levee embankments

Once overflow discharge and duration of overflow have been estimated (Section 7.3), the flow characteristics over the crest and along the landward face of the levee have to be calculated. First, critical depth (where Froude number, **Fr**, equals unity) (Section 7.3.6.1) is calculated for the overtopping discharge. Critical depth occurs at or very near the landward side of the levee crest.

Provided the landward slope is steep and the tailwater is low, the flow continues to accelerate until the normal depth is reached. Normal depth can be calculated using an iterative solution of the Manning's equation shown in Equation 7.17 using the estimated overflow discharge (Equations 8.46 or 8.47). As tailwater increases, the location of the jump moves further up the landward slope until the crest is submerged (Figure 8.16). Once normal depth is achieved for a given overflow discharge, flow continues down the levee slope at this state (depth and velocity) until there is a change in slope or downstream water levels begin to increase. At this point the flow decelerates rapidly resulting in significant energy loss through a 'hydraulic jump' (Figure 8.17). At this point the landside area is fully inundated to nearly the same level as in the river. Figure 8.18 shows possible states of overflow at the landward toe of a levee.



Mississippi River: Birds Point-New Madrid Floodway activation during 2011 flood

Figure 8.16 *Water overflowing a levee with significant energy dissipation where accelerated flow interacts with tailwater the levee crown has been substantially eroded by the overflow (courtesy USACE)*

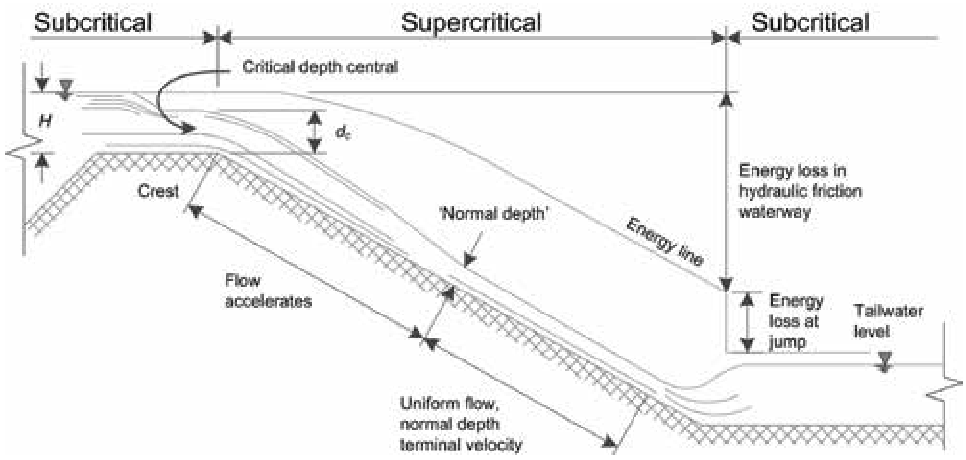


Figure 8.17 *Elevation showing flow states down face of levee or spillway due to overflow (Hewlett et al, 1987)*

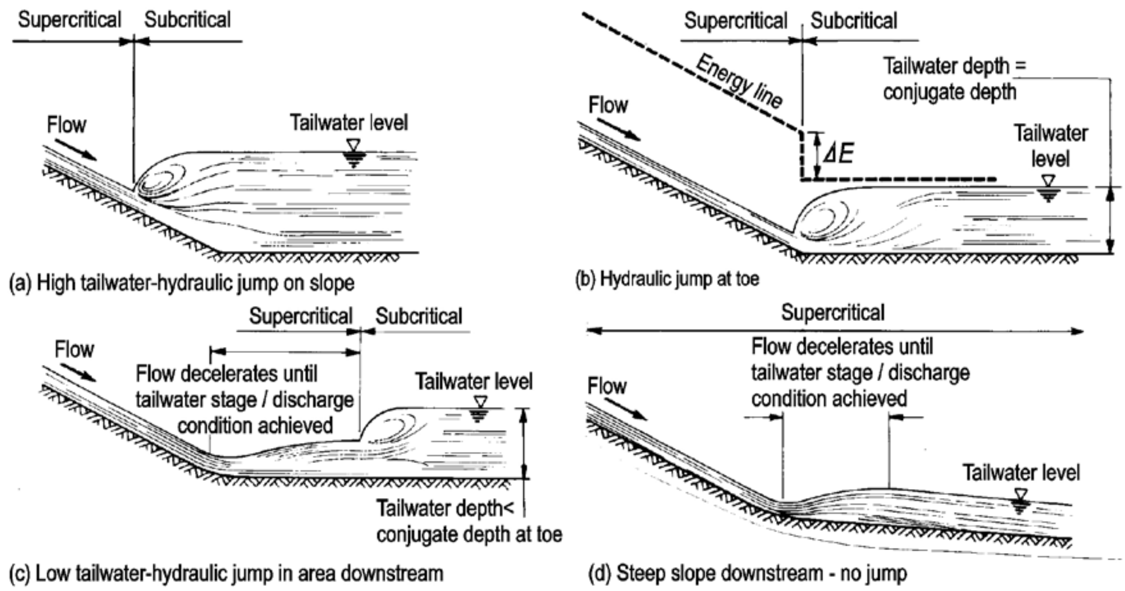


Figure 8.18 ***Different combinations of overflow near landward toe of levee (Hewlett et al, 1987)***

Two critical concerns for levee overflow are velocity of flow on the downstream levee slope (here downstream may be either the riverward or landside slope depending on the direction of overtopping) and the high turbulence and energy dissipation at the hydraulic jump. Velocity on the slope has been addressed through calculation of the normal depth using Equation 7.17. It becomes necessary to determine the dimensions of the hydraulic jump so that adequate protection measures may be designed. The reader is referred to standard hydraulics text books, notably Chow (1959) for full details of the hydraulic jump.

The amount of energy dissipated through a jump depends on the Froude number (**Fr**) of the upstream supercritical flow (see Box 8.4). The downstream depth required to fully form the jump can be calculated by:

$$y_2 = \frac{y_1}{2}\left(\sqrt{1+8Fr_1^2}-1\right) \tag{8.49}$$

where y is depth, subscript 1 denotes upstream conditions and subscript 2 denotes downstream conditions. For a fully formed jump, the jump length can be estimated by:

$$L_{jump} = 220\, y_1 \,\tanh\left[\frac{Fr_1 - 1}{22}\right] \tag{8.50}$$

where L_{jump} is in metres, subscript 1 denotes upstream conditions and tanh the hyperbolic tangent.

Tailwater depth should also be calculated using the Manning equation and this value compared to y_2 computed from Equation 8.53. If the calculated tailwater depth is less than y_2, then the jump will not fully develop. One measure to ensure full jump development is to extend the levee slope to a lower elevation so that the full y_2 depth is achieved before continuing downstream (Figure 8.19).

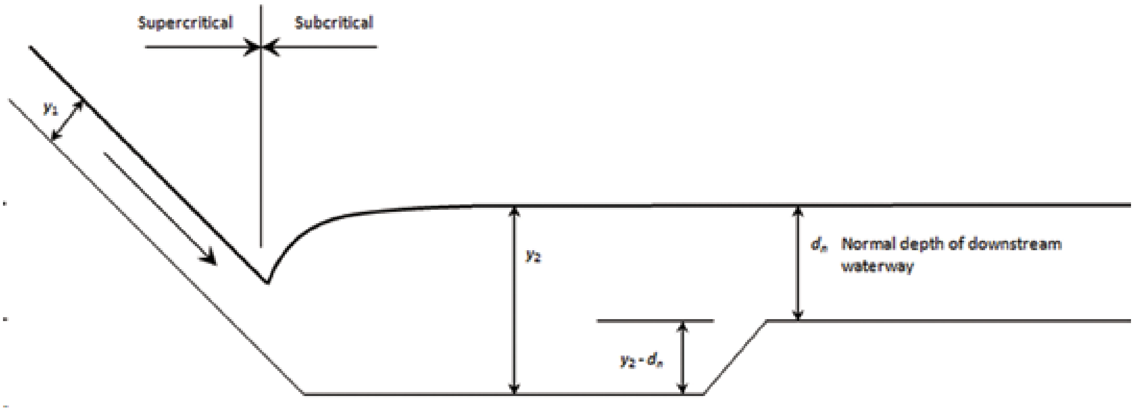


Figure 8.19 Measures to ensure full jump development where normal depth downstream may be limited

Box 8.4 Energy loss in a hydraulic jump (Hewlett et al, 1987)

For a given slope and roughness, Froude number, **Fr**, does not vary greatly with discharge. For example, in a hydraulically wide waterway, substituting Manning's equation for mean velocity of flow:

$$\boxed{Fr \cong \frac{V}{\sqrt{gh}} = \frac{h^{1/6}}{n}\sqrt{\frac{S}{g}}} \qquad (8.51)$$

Typical values of Froude number for grassed waterway applications (as for a grassed levee slope) are:

Slope, *S*	Fr
1V: 2.5H	5 to 6
1V: 5H	4 to 5
1V: 10H	3 to 4
1V: 25H	2 to 3
1V: 50H	2

Energy loss, ΔE, in a hydraulic jump is usually considered in relation to the specific energy, E, of the incoming flow. $\Delta E/E$ varies from about 65 per cent for **Fr**=7, to about 15 per cent for **Fr**=2. Below Froude numbers of about 2, the hydraulic jump is weak and relatively little energy dissipation occurs.

An example of issues to consider during the operation of a levee fuse plug or spillway, or when overtopping occurs is presented in Box 8.5.

Box 8.5 ***Subsurface flow (from Hewlett et al, 1987)***

During the operation of a levee fuse plug or spillway or when overtopping occurs, the flow field in the underlayer and/or the subsoil below the armour layer is determined by the hydraulic boundary conditions at the interface with the open channel above.

In uniform flow conditions, the hydrostatic head due to open channel flow in the waterway can give rise to:

1 Infiltration into the unsaturated subsoil.
2 Seepage flow parallel to the slope (with hydraulic gradient equal to the slope of the waterway)

Infiltration is determined by the infiltration rate at the open channel boundary (ie the armour layer) and seepage flow is governed by the permeability of the underlayer/subsoil.

The turbulent flow conditions in the waterway will give rise to dynamic fluctuations in water pressure at the boundary, but in general (and within the limitations of velocity recommended herein) subsurface flow is relatively steady, and its direction is into or parallel to the open channel boundary. Within the limitations of waterway flow velocity and subsoil composition recommended in Hewlett *et al* (1987), piping or entrainment of soil particles in the underlayer/subsoil by subsurface flow is therefore unlikely.

Conditions of subsurface flow during operation of a reinforced grass waterway has to be distinguished from those below an armour layer or any other surface that is subject to wave attack. With wave action, the hydraulic boundary conditions are unsteady and during part of each wave cycle the direction of subsurface flow is out of the open channel boundary. This cyclic 'pumping action' in the subsoil with repeatedly high exit gradients gives rise to onerous requirements for a filter that can:

1 Retain the subsoil particles from migration.[box lists to be styled]
2 Maintain a sufficiently high permeability throughout its service life to avoid excessive head loss through the filter with consequent failure by uplift.

Further information on subsurface flow and filter requirements associated with wave action and navigational waterways is given in ICE (1984), CIRIA CUR CETMEF (2007), and PIANC (1987).

8.2.2.4 Hydraulic performance of flood walls

Flood walls that might be subject to overflow by rising water should be designed with erosion protection on the protected (dry) side capable of resisting the force of the free-falling water jet. Figure 8.20 illustrates flow discharging over a flood wall and plunging (in this case) into standing water on the protected side of the flood wall. The plunging jet penetrates the water and creates large eddies that erode material from the unprotected soil surface. The same mechanism will scour bed material when there is no standing water on the protected side of the flood wall.

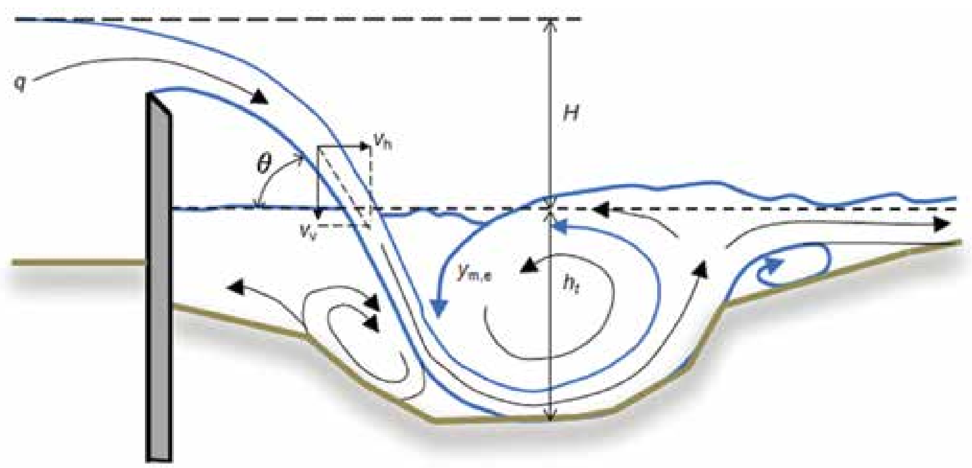


Figure 8.20 ***Scour hole formation by overtopping jet (from Hoffmans and Verheij, 1997)***

This scouring action removes material that may be providing critical lateral support pressure against the protected side of the vertical flood wall. Failure occurs if the remaining undamaged portion of the foundation adjacent to the wall cannot withstand either the shear force or the overturning moment exerted on the flood wall by the elevated water on the flood side of the wall.

The jet of water passing over the vertical flood wall has two surface profiles referred to as 'nappes' (meaning 'a continuous surface'). The lower nappe is closest to the backside of the flood wall, and the upper nappe is the extension of the flow free surface as it spills over the wall. The trajectories of the lower and upper nappes are given in most open channel flow literature (eg Chow, 1959, and Morris and Wiggert, 1963).

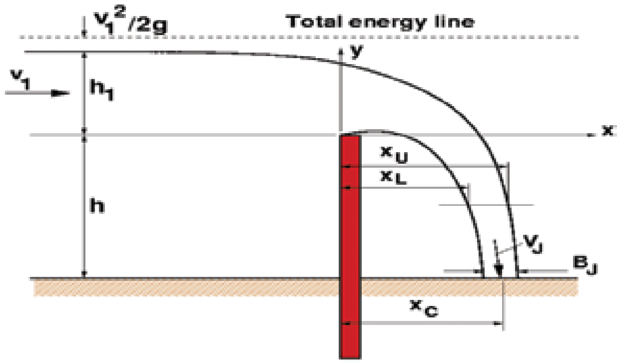


Figure 8.21 Flow over a flood wall approximated as a sharp-crested weir (USACE, 2008)

In dimensionless form, the equations are as follows with the x-y co-ordinate system as defined in Figure 8.21.

Lower nappe
$$\frac{y_L}{H^*} = A\left(\frac{x}{H^*}\right)^2 + B\left(\frac{x}{H^*}\right) + C \tag{8.52}$$

Upper nappe
$$\frac{y_U}{H^*} = \frac{y_L}{H^*} + D \tag{8.53}$$

where the parameters are defined as follows:

A = $-0.425+0.25G$
B = $0.4111-1.603G-(1.568G^2+0.892G+0.127)^{1/2}$
C = $0.150-0.45G$
D = $0.57-0.02[10\,(G-0.208)]^2 \exp\,[10(G-0.208)]$ with $G = V_a^{\,2}/2gH^*$

This yields equations for x_L and x_U as:

Lower nappe
$$\frac{x_L}{h_1} = \frac{-B-\sqrt{B^2-4\,A\,(C-y/h_1)}}{2\,A} \tag{8.54}$$

Upper nappe
$$\frac{x_U}{h_1} = \frac{-B-\sqrt{B^2-4\,A\,(C+D-y/h_1)}}{2\,A} \tag{8.55}$$

The distance to the centre of the jet at impact with the ground surface is the arithmetic average of x_L and x_U. The intersection points of the lower and upper nappes with the horizontal ground level on the landward side of the flood wall are found by setting $y=^-h$ in Equations 8.52 to 8.55. The horizontal width of the overflowing jet at impact is given by $B_x = x_U\,(y=-h) - x_L(y=-h)$.

If there is no venting, the air pressure in the space between the flood wall and lower nappe may become less than atmospheric as air is entrained into the jet during sustained overtopping. The decreased pressure will draw the plunging jet closer to the wall, however, this decrease in plunge point location away from the vertical wall is difficult to predict. This is likely not to be a problem because the scour protection will probably cover the entire region from the base of the wall out past the location of jet impact.

The overtopping jet impacts the ground at an angle less than vertical (which is given by -90° in the co-ordinate system defined in Figure 8.21). The jet entry angle is well approximated by the average of the angles of the lower and upper nappe profiles when they intersect the horizontal ground level. The entry angles of the nappe profiles are found by taking the derivative of Equations 8.52 and 8.53 and evaluating the result at $x=x_L$ and $x=x_U$, respectively, to get:

$$\theta_L = \arctan\left(\frac{dy}{dx}\right)_L = \arctan\left(\frac{2\,A\,x_L}{h_1} = B\right) \tag{8.56}$$

$$\theta_U = \arctan\left(\frac{dy}{dx}\right)_U = \arctan\left(\frac{2\,A\,x_U}{h_1} = B\right) \tag{8.57}$$

The jet entry angle is estimated as:

$$\theta_J = \frac{\theta_L + \theta_U}{2} \tag{8.58}$$

From geometric considerations the width of the impinging jet normal to the flow streamlines can be estimated with reasonable accuracy by the formula:

$$B_J = B_x \sin\left(-\theta_J\right) \tag{8.59}$$

Discharge over the flood wall remains constant for steady flow, and the discharge per unit length of the plunging jet at impact with the ground surface is given simply as the jet velocity parallel to the flow streamlines times the width of the jet normal to the flow. Therefore, the jet entry velocity can be estimated as:

$$V_J = \frac{q}{B_J} \tag{8.60}$$

Finally, the total force (thrust) exerted by the overtopping jet on the scour protection per unit length along the wall is given in inviscid jet theory (Milne and Thompson, 1960) as:

$$F_J = \rho\,B_J\,V_J^2 \tag{8.61}$$

where ρ is the water density.

This equation is an expression of the momentum flux of the jet, and the force is directed parallel to the jet streamlines.

The force of the overflow jet at impact creates high pressures because the jet width is narrow. The impact force given from Equation 8.61 can be resolved into vertical and horizontal components using the estimated jet entry angle from Equation 8.58. So, the apportioning of force between vertical and horizontal components will vary with overflow condition, and successful scour protection has to be able to resist the expected range of vertical and horizontal forces. For high discharges over low walls, the jet entry angles are far from vertical, and the water after impact will retain a substantial horizontal velocity as it flows down the protected side of the earthen levee.

Depending on the elevation of the adjacent land on the protected side of the flood wall, there may be standing water at the base of the wall. The impact force of an overflow jet will be dissipated to some degree as it enters the standing water but it still retains sufficient force to erode unprotected foundation soil. Scour protection that relies on self-weight for stability will be less stable when submerged, and the overflow jet may be able to dislodge submerged components of the protection. The highly turbulent conditions that exist in the plunge area make estimation of scour extent and depth difficult. It is necessary to use multi-dimensional numerical models with capability to simulate an erodible boundary or physical models should be used. Use of cohesive materials typical for levee construction further complicates estimating the rate and extent of erosion in the situation depicted in Figure 8.21. Current good practice is to provide continuous paving that incorporates a structural design so that the paving can withstand the expected impact forces from the jet.

8.2.3 Scour in river channels

This section will provide information related to evaluating scour in river channels and the relationship to a levee system. The flow of subsections is shown in Figure 8.22.

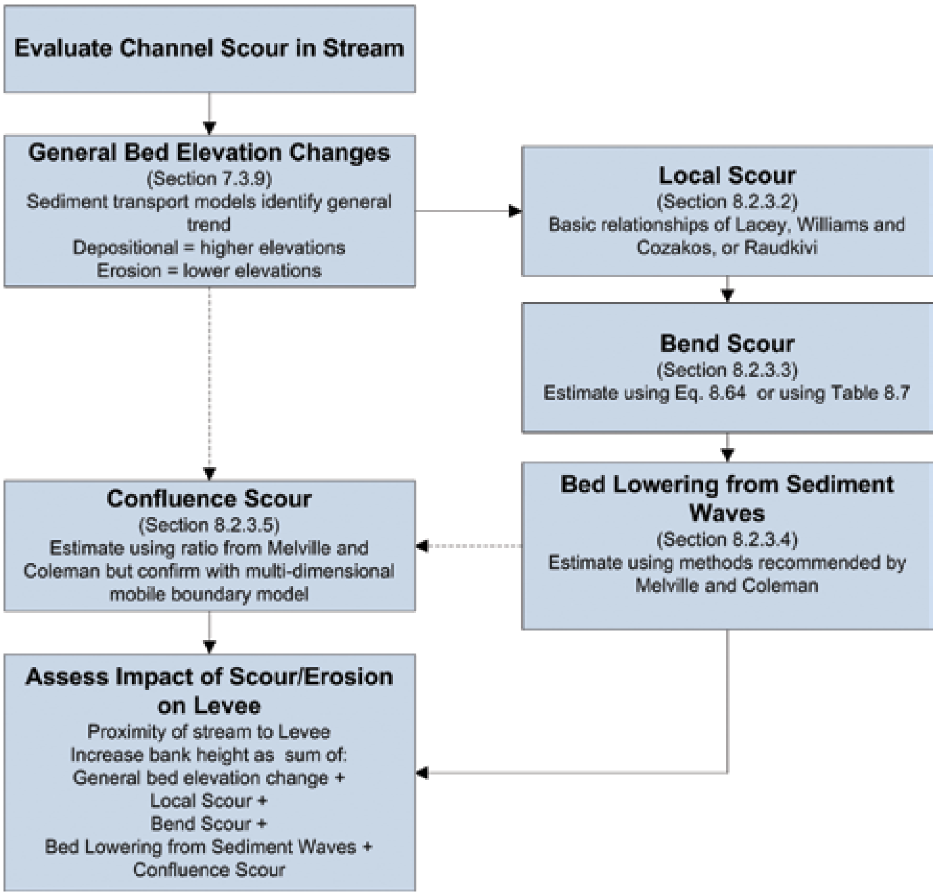


Figure 8.22 Basic approach to evaluating scour

8.2.3.1 General

Introduction of levees into a stream system will produce a system response. Evaluation of long-term channel stability was discussed in Section 7.3. Part of this analysis involves use of sediment transport models to estimate deposition or erosion trends in the stream over the project life. The analysis provides an indication of the amount and rate of vertical change that could be expected to occur. Adjustment of the stream bed elevation over time influences the design water level as described in Section 7.3. It is also necessary to assess how long-term (over the project life) trends in bed elevation may impact levee stability. When levees are set back from the stream channel any downward change predicted by sediment transport studies should pose no threat to levee structural integrity. This is not true when levees are close to the stream top bank. In this case, general erosion depths have to be included in the slope stability analysis. Adjustments in the levee alignment and/or embankment slopes may be required if calculated erosion depths create a bank height that is unstable.

Stream channels are not always straight with regular cross-section geometry. There is considerable variability, particularly in natural channels. Part of this variability is a result of shifts in the channel alignment and cross-section in response to the various boundary conditions that exist in the watershed. This introduces a requirement to evaluate both general trends in bed elevation change and local influences due to thalweg shifts and the presence of bends. Confluence scour occurs where two channels combine, and it should be considered as necessary.

8.2.3.2 Local scour

While not as apparent as a shift in channel location for braided streams, a shift in the thalweg will alter local bed elevations and can change the point and/or angle of attack for a flow. This can lead to markedly increased scour at the bank, which in turn may result in bank failure and increased threat to a levee located near the main stream channel. So, it is necessary to assess local scour depths in the vicinity of the levee alignment so that appropriate protection measures can be included to ensure long-term levee integrity.

For flow around a bend, the interaction between the vertical gradient of streamwise velocity and the curvature of primary flow creates secondary currents. These secondary currents lead to larger flow depths, velocities and shear stresses along the outside of the bend, and so increased deepening at the toe of the outside bank. The position of the greatest depth in a bend is affected by changes in flow characteristics and channel-forming processes, flow variability, and bank conditions. The general observations shown in Table 8.6 apply.

Table 8.6 *Influences on local scour depth at bends*

Condition	Bend scour is principally a function of:
Abrupt change in flow direction/sharp bend angles	• degree of direction change
Eroding bends/migrating bank on outside	• bank material.

Once average bed elevations have been assessed at a site (Section 7.3), allowance needs to be made for the effect of variations in bed elevation across the site on local scour depths. The critical consideration for levees involves stream types that migrate over time. For meandering channels, estimates of bend scour will allow for lowered bed elevations due to the presence of the thalweg in the bend.

Lacey (1930) remarks that stable reaches of rivers frequently present a semi-elliptical cross-section. Lacey gives the relationship that for a truly semi-elliptical section the maximum flow depth, y_{max} is given by multiplying mean depth by 1.27. If a channel has a constricted width, Lacey indicates that y_{max} is equal to the mean depth.

In estimating design scour depths for protecting levee toes, Williams and Cozakos (1994) allow for thalweg formation based on the Lacey relationship of 25 per cent of flow depth for straight reaches. Raudkivi (1990) suggest that maximum channel depth equals 1.69R or 1.58y where R is the hydraulic radius and y is obtained by dividing cross-sectional area by channel top width if the channel is assumed to be of a cosine cross-section.

8.2.3.3 Bend scour

Lacey (1930) considered different classes of cross-section, for a semi-elliptical cross-section shape, that may develop in a river for varying bend radius of curvature. Lacey used a relationship where a constant wetted perimeter and cross-section area existed for various degrees of lateral adjustment on the channel boundary due to increasing scour depths (Figure 8.23). Lacey summarises the influence of bend curvature on maximum flow depth as shown in Table 8.7 where y_{bs} is maximum flow depth in the bend and R is the hydraulic radius for the channel only. Neil (1973) provided coefficients as recommended by the Indian Roads Congress (1966).

Table 8.7 *Some coefficients relating bend curvature and maximum flow depth for a cross-section*

Degree of bend curvature	Lacey (1930), y_{bs}/R	Neill (1973), y_{bs}/R
Greatly constricted	1.00	–
Straight	1.27	1.25
Moderate bend	1.50	1.50
Severe bend	1.75	1.75
Right-angled bend	2.00	2.00
At cliffs and walls	–	2.25

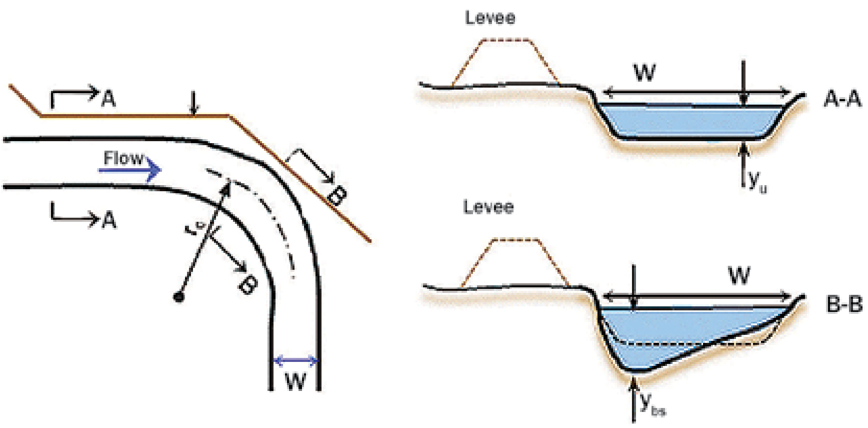


Figure 8.23 Bend scour and variables (from Melville and Coleman, 2000)

Various investigators include an allowance for the influence of bend angle on maximum bend flow depth (Galay *et al*, 1987, Apmann, 1972, Thorne, 1988, Thorne *et al*, 1995, Thorne and Abt, 1993, and Maynord and Hubbard, 1993). Thorne (1988) used data from 70 bends along the Red River between Arkansas and Louisiana in the USA to develop a relationship between y_u and y_{bs} as:

$$\frac{y_{bs}}{y_u} = 2.07 - 0.19 \ln\left[\left(\frac{r_c}{W} - 2\right)\right] \tag{8.62}$$

for r_c/W >2 where y_u is the average flow depth (Area, A/Width, W) in the channel upstream of the bend.

Thorne *et al* (1995) includes a comparison of flume data and data for 257 bends on natural rivers, which varied widely in type and size, located in different physiographic regions and different parts of the world. The dataset included maximum flow depths of a few centimetres in the flumes up to about 17 m to cover all but the world's principal rivers. Equation 8.62 was found to be in reasonable agreement with the larger dataset with the majority of predictions falling between +30 per cent to -25 per cent of observed values.

Based on bend-scour data from the Mississippi River, USACE (1994) presents a 'safe' design curve for maximum bend flow depths of:

$$\frac{y_{bs}}{y_u} = 3.37 - 0.66 \ln\left[\left(\frac{r_c}{W}\right)\right] \tag{8.63}$$

The USACE equation is designated a 'safe' design curve because only five per cent of data used to derive the curve fall above predicted values. Maynord (1996) expressed concern that Equation 8.63 is conservative for the vast majority of measured data, particularly for relatively small streams. Incorporating channel aspect ratio into the expressions for bend scour, regression analyses of the Thorne and Abt (1993) and Maynord and Hubbard (1993) data yields, for 1.5 <(r_c/W) <10 and 20 <(W/y_u) <125:

$$\frac{y_{bs}}{y_u} = 1.80 - 0.051 \ln\left[\left(\frac{r_c}{W}\right)\right] + 0.0084\left(\frac{W}{y_u}\right) \tag{8.64}$$

Maynord (1996) suggests that the preceding empirical methods are valid up until there is significant interaction between main channel flow and overbank flow. Recommended use where overbank flow conditions exist is limited to where overbank depths are less than 20 per cent of the main channel depth, y_u.

Melville and Coleman (2000) recommend use of Equation 8.64 to estimate bend scour. Alternative methods described in Equations 8.62 and 8.63 may also be used if appropriate. Use of these equations together with coarse indications from Table 8.7 may be used as a guideline in estimating bend scour for use in slope stability analysis and in developing bank stabilisation requirements associated with levees. These equations may also be used where the low flow channel is sinuous within a larger channel cross-section.

8.2.3.4 Bed lowering from sediment waves

For flood flows associated with levee performance, sediment waves will be migrating through the river channel. The magnitudes of these waves influence bank height because wave troughs momentarily and locally lower bed elevations as the sediment waves propagate through a reach (Figure 8.24).

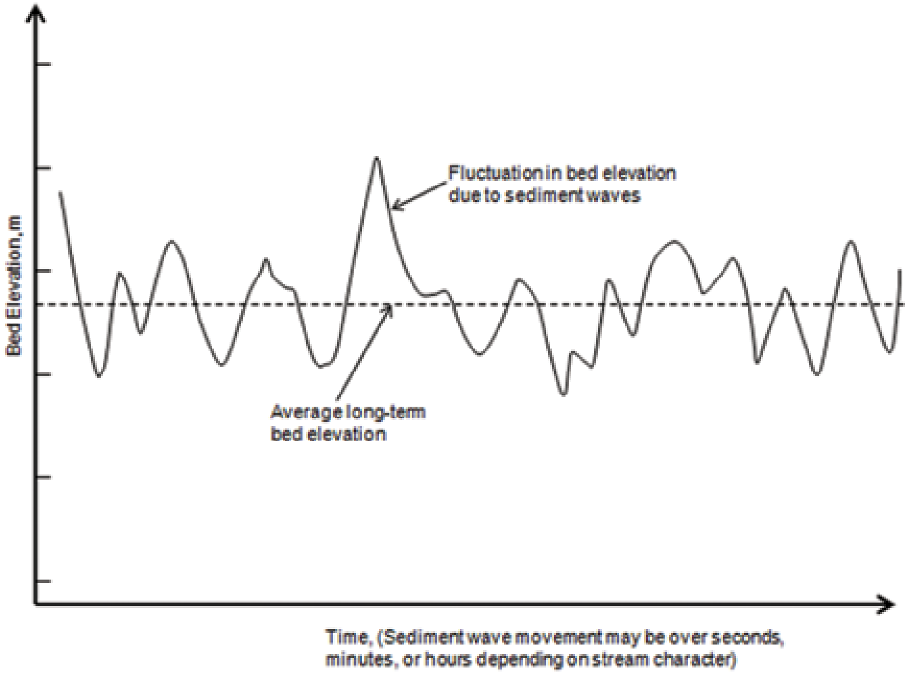


Figure 8.24 ***Development of additional depth due to sediment waves along river channel (Arneson, 2012)***

Bed profiles typical of sand bed streams are commonly classified as flat bed, ripples, dunes, transition bed, antidunes, and chutes and pools (ASCE, 1966). The presence of coarser bed materials influences bed movement tendencies, generally suppressing the amplitude (height) of sediment waves as compared to beds composed entirely of sand. In rivers with gravel beds, bed form migration occurs primarily as the movement of gravel bars or waves down the stream. Bars are large depositional features that generally occur in meandering or braided channels. Bar migration can reduce channel waterway area and redirect flows, possibly resulting in increased scour owing to flows concentrating at the bank.

Sediment wave prediction is a two-stage process that requires estimating the type of bed form then its height. Available methods predict the types of bed profiles for sand bed streams based upon various combinations of flow strength and sediment characteristics. The topic is quite complex and the reader is referred to references for predictive equations and additional information (Simons and Richardson, 1966, van Rijn, 1984, Julien and Klaassen, 1995, Hey *et al*, 1982, Yalin, 1964 and 1992, Ikeda, 1984, Nordin and Algert, 1965, Shen *et al*, 1969, Raudkivi, 1990, Coleman, 1991, Coleman and Melville, 1994, Chang, 1988, and Williams and Cozakos, 1994). Although there are one or two exceptions, the empirically developed equations estimate average magnitude of bed forms at equilibrium conditions. There is significant scatter in the data used to develop the equations, and the principle source of concern is maximum bed form size as the sediment waves pass through the stream.

While average bed form height may be useful, the most significant issue for levee assessment and design is the maximum height, or the condition that yields the greatest scour depth. Yalin (1964) used experimental data and theory to project that the maximum dune height should not exceed one-sixth of the flow depth. Nordin and Algert (1965) suggested that $y/3$ is more appropriate for average maximum dune height, particularly where 3D bed profiles occur. Neill (1973) indicated that maximum dune heights for migrating dunes in natural alluvial streams can be up to half the flow depth. With respect to design of levee toe protection, Williams and Cozakos (1994) adopted $y/3$ as the design scour depth for bed form migration. Melville and Coleman (2000) suggest that peak flow depth due to bed form migration past a site, y_{ws}, can be estimated as:

$$y_{ws} = \max\left[1.5\,y\,;\,y + h_{ws}/2\right] \tag{8.65}$$

where y (m) is the flow depth without bed forms and h_{ws} (m) is the maximum bed form height determined from predictor equations. Melville and Coleman emphasise that judgement has to be exercised in using Equation 8.65 as the first equation (y_{ws}=1.5y) may produce unrealistically large scour depths.

Because of the complexity and interpretative nature of evaluating bed forms, predictor equations are not presented in this handbook. The reader should refer to Melville and Coleman (2000) or other references for details on the individual methods. Engineers with experience of river mechanics should be consulted for analysis required to estimate sediment wave characteristics needed to support analysis and design efforts for levees.

Melville and Coleman (2000) point out that Raudkivi (1990) observed that dunes formed when bed material consisted of a broad grading of sediment sizes were very different than when formed in uniformly graded sediment. They also state that the presence of a high concentration of colloidal materials in the flow affects bed forms by delaying their development, causing the transition to flatbed sooner than flows without suspended clays.

8.2.3.5 Confluence scour

Flow typically meets at the centreline of the junction, plunges toward the channel bed, and then returns to the surface towards the sides of the channel where two streams converge. This flow pattern results in helicoidal secondary currents that produce a deep scour hole with steep sides. Confluence scour can be of significant concern for levees located along braided channel systems. Braided systems can undergo rapid shifts in channel position resulting in the confluence of individual channels of a river rapidly moving towards a levee. The additional depth in the confluence increases total bank height, and slope stability analysis has to address the potential for exceeding a stable bank height. The addition of a levee in close proximity to a channel makes this situation more critical.

There is little agreement in the literature on principle parameters that influence confluence scour. In general, principle factors include confluence angle, flow rates, flow depths, channel slope, bed material size, bed-material transport rate, concentration of suspended sediments and type of channels involved. Melville and Coleman (2000) cite observations by Ashmore and Parker (1983) that indicate naturally occurring confluence angles are typically about 100°. Melville and Coleman used this angle to predict a maximum value of y_{cs}/y=5.34. Chow (1959), however, suggests that owing to the complexity of confluence scour there is no way to generalise the phenomenon and that model studies are the only feasible way to develop estimates of scour depth. The recommended approach for evaluating confluence scour is to develop numerical sediment transport models capable of simulating 2D development of the bed in the confluence region. For highly complex areas a physical model study may be warranted.

Good practice for levee design is to avoid placing levees in close proximity to confluences. In the case of braided channels, levees should be located well outside the zone of potential channel migration. Protective measures should be included in the levee project to guard against threats imposed by channel shifts and rapid changes in confluence locations where this is not feasible. Protection can be in the form of revetment along stream banks or other stream bank stabilisation measures. Melville and Coleman (2000) provide further details on contraction scour.

8.2.4 Scour of beaches in front of coastal levees

This section details scour of beaches in front of coastal levees following the approach shown graphically in Figure 8.25.

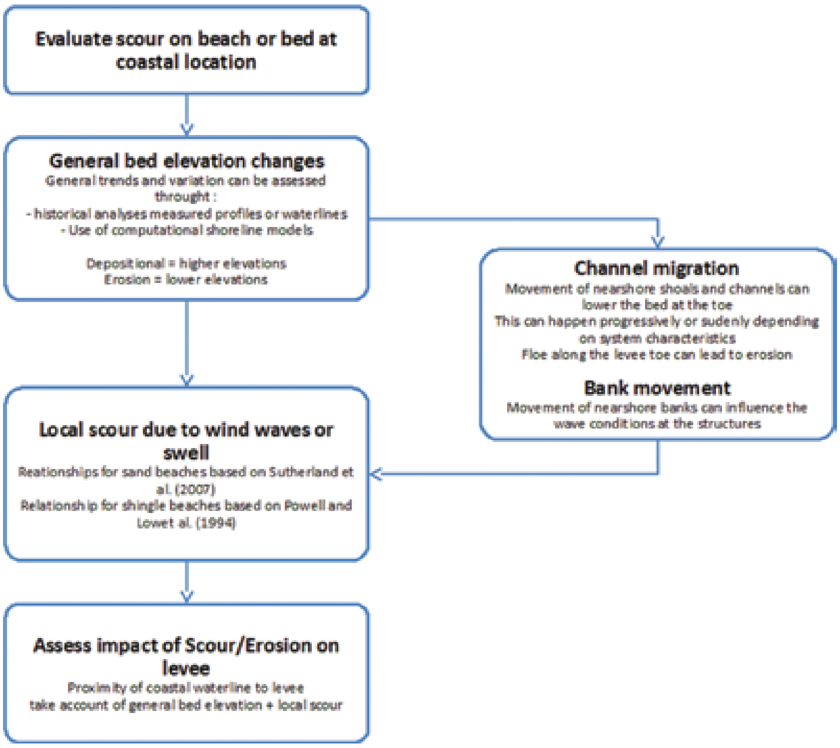


Figure 8.25 ***Basic approach to evaluating scour at the coast***

8.2.4.1 General

The introduction of levees on an open coast will produce a system response where the toe of the structure is submerged. The degree of response will depend on characteristics of the structure including front face slope and roughness. The vulnerability to scour is governed by the water depth at the toe due to tidal variation, wave set-up at the shoreline, storm induced and possibly seasonal changes to normally occurring water levels. The nearshore and beach profile shape as well as sediment composition will also influence the response. The forcing of scour depends on the wave height and period of waves reaching the levee. The information in this section draws on results from laboratory research and field evidence at seawalls (Sutherland *et al*, 2006, Sutherland *et al*, 2007, and Wallis *et al*, 2009).

There are two sets of analyses that are required to evaluate the amount of vertical change that could be expected to occur in front of a levee at a coastal site. These are:

- predicting the lowering of beach levels
- predicting sediment scour at the toe of the wall. (Note localised sediment scour at the toe of the levee is a different physical process to beach lowering, although partly dependent upon that of broader scale beach lowering).

The approach that is adopted for the toe scour prediction is as detailed below. Methods are provided for scour on sand beaches and shingle beaches in front of vertical walls. Commentary is provided on how to relate the results to sloping walls as would be found on levees.

8.2.4.2 Predicting beach lowering

The performance of a beach largely depends on the volume of material present and the limits to its plan and profile changes – influenced particularly by sediment control structures within it (eg groynes, sills, breakwaters). Where there is a continuing net loss of sediment, then the lack of beach recovery is an issue. In general, pressure on the integrity of the structure can result from depletion in the volume of the beach through increased longshore and/or cross shore transport of beach sediment, or, a reduction in supply of sediment onto the beach front. Beach levels are constantly changing, and trends of depletion or deposition are generally gradual (long-term), however significant erosion and lowering can occur during 'one-off' storm events.

The variations in beach levels occur in a range of timescales from one tide or storm to annual events, and are the accumulation of residual changes in level that occur during each tide or storm. It is common to find beach levels lower during the storm season (eg winter, monsoon, cyclone, and hurricane) due to the higher occurrence of increased levels of wave energy. It also follows that where there is a periodic signal of storm events the beach levels may show a greater variation about their seasonal mean during the storm season.

A range of advanced linear and nonlinear data analysis methods can be used to evaluate the long-term behaviour of beaches (Larson *et al*, 2003, and HR Wallingford 2008c). Data-based analysis will become more powerful as the amount of regularly sampled and accurate data collected, stored and managed by organised regional coastal observatories and other agencies increases. The evaluation of profile data may be supplemented by the results from process-based numerical models of cross-shore beach evolution (eg van Rijn *et al*, 2003).

One dilemma the engineer faces is what prediction 'horizon' can be expected when extrapolating beach level time series data. Analysis of beach monitoring data from Lincolnshire, UK (HR Wallingford, 2008a, and Sutherland *et al*, 2007) illustrates that the predictive ability of a straight line fit from more than 10 years of data are limited to a few years beyond the end of the dataset. However, this should be sufficient for the purposes of supporting annual inspection combined with predictive modelling. An indicative per annum allowance for beach lowering based on data provides a guide to potential beach lowering rates and informs the design and maintenance of coastal defences. The indicative allowances for beach lowering can be applied in the same way as, say, indicative allowances for sea level rise. Indeed, each site should be treated individually to determine the general context for the levee under consideration as this may also be influenced by nearshore banks and channels, which will affect waves and currents. Channels in open embayments, inlets and estuaries that move so as to run adjacent to the toes of levees can cause erosion.

8.2.4.3 Predicting sand bed scour due to waves

The development of toe scour is a dynamic process, highly dependent on the water level at the wall and the incident wave conditions. In areas of varying tidal range and wave climate, the development of a scour hole will be an episodic process with periods of erosion followed by infilling, and perhaps even general accretion of the bed (Powell and Lowe, 1994). So, the scour hole itself may be a short-lived feature with no obvious evidence of its extent, or perhaps even its existence after a storm has declined and infilling has taken place as the tide recedes. This means that the profile seen before and after the storm may be quite similar in consecutive beach profiles taken at low water. There is a need to be able to predict the maximum depth of the scour hole during storms, as well as the more widespread and longer-term processes that cause the lowering of beach/shore-platforms. This is an important factor to take into account at the design stage of a structure, and in its operational life to fully understand risks to integrity of the levee and plan for timely remedial action to be undertaken when required.

As storm event scour is frequently short-lived, a programme of annual or seasonal beach profile monitoring is unlikely to capture a major scour event but can indicate the way in which the beach is evolving and record seasonal variations at the seawall. Indeed, the evidence supplied by data from scour monitors (Sutherland *et al*, 2006) suggests that a significant amount of a scour hole can fill in within a few hours of the peak of a storm. So, even regular beach profiling with a spacing of a few weeks, supported by profiles collected within a day or two of each large storm may not capture the transient phenomenon of toe scour in the field. The combined evaluation of beach level trends and scour prediction is an appropriate way forward.

One rule of thumb for vertical seawalls has been that the maximum scour depth is equivalent to the (unbroken) significant wave height H_s. Whitehouse (1998) and CERC (1984) suggested the depth of scour may be equal to the maximum unbroken wave height H_{max} (ie $1.8H_s$). As an improvement on this

(Sutherland *et al*, 2007, and HR Wallingford, 2008b) recommended the use of a conservative predictor of scour depths, which may be used in the absence of site-specific information on beach slope. It is reproduced as Equation 8.66 with H_s as the commonly used scaling parameter for predicting scour depth:

$$\frac{S_{tmax}}{H_s} = 4.5\,\mathrm{e}^{-8\pi(h_t/L_m+0.01)}\left[1-\mathrm{e}^{-6\pi(h_t/L_m+0.01)}\right] \tag{8.66}$$

for -0.013 ≤ H_t/L_m ≤ 0.18 and where:

S_{tmax} = maximum toe scour depth at a vertical wall (m)
H_s = the deep water (unbroken) significant wave height (m)
h_t = water depth above the sediment level at the toe of the wall (m)
L_m = $gT_m^{\ 2}/2\pi$ the linear theory wavelength based on acceleration due to gravity g (default assumption of 9.81 m/s²) and mean wave period T_m (s)

The equation is plotted with data in Figure 8.26. When this equation was tested by validating laboratory tests with field data from two UK sites, Blackpool (vertical wall) and South Bourne (sloping wall), it was found that the field data generally had lower scour depths than the laboratory data. This is believed to have been caused by the fact that wave height, wave period and scour depth were only measured at a single tidal state in the laboratory. The field data was collected in situations with constantly varying water levels and wave heights. However, the upper limits of the field observations confirm the laboratory data and envelope curve of Equation 8.66 – even with a sloping wall.

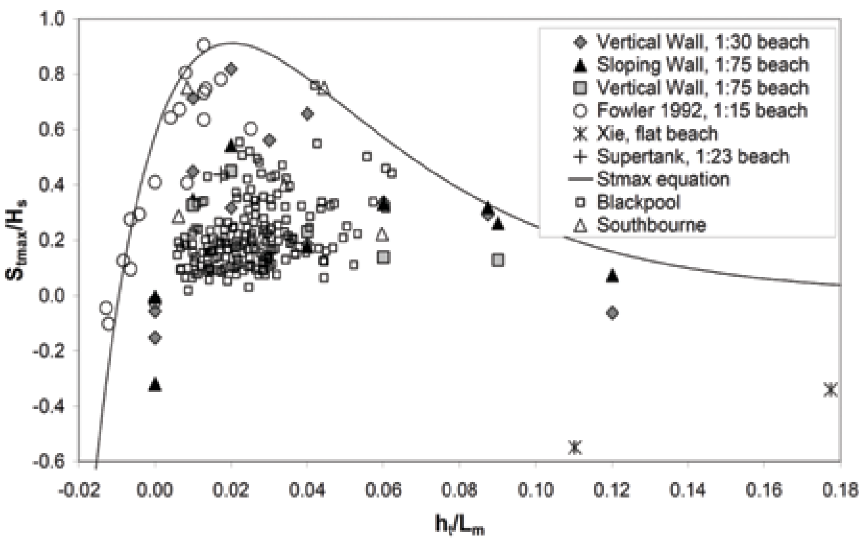


Figure 8.26 ***Envelope to scour predictor. Equation 8.66: laboratory data and field data (after Sutherland et al, 2007)***

It can be seen from Figure 8.26 that the scour depth is always less than H_s, and that the peak scour depth occurs for relative water depths (H_t/L_m) of around 0.01 to 0.02 and that the scour depth reduces for shallower and deeper water.

In situations where the beach slope is known then an alternative empirical equation for the depth of scour at the toe of a vertical wall developed using the laboratory data in Figure 8.26 can be used (HR Wallingford, 2008b, and Sutherland *et al*, 2007). HR Wallingford (2008b) showed that the relative toe scour depth can be given with a beach slope dependency by Equation 8.67:

$$\frac{S_t}{H_s} = 6.8\,(0.207\,\ln\alpha + 1.51)\,\mathrm{e}^{-11.7\pi h_t^*/L_m}\left[1-\mathrm{e}^{-6\pi h_t^*/L_m}\right] - 0.137 \tag{8.67}$$

for -0.04 ≤ H_t/L_m ≤ 0.12 and where:

S_t = the scour depth at the toe of the structure (m)
H_s = the deep water (unbroken) significant wave height (m)
α = the beach slope (radians)
h_t^* = the water depth above the sediment level at the toe of the wall (m) including effect of wave set-up calculated using the equation of Holman and Sallenger (1985) where h_t/L_m ≤ 0

Hollman and Sallenger's (1985) expression for the maximum set-up, η_{max}, that would occur on a natural beach is given in Equation 8.68, where both the wave height and wavelength (in the Iribarren number, *Ir* or ζ_m) are calculated in deep water but the beach slope is calculated at breaking:

$$\eta_{max} = 0.45\, H_s\, Ir = 0.45\, \tan\alpha\, \sqrt{H_s\, L_p} \tag{8.68}$$

In the derivation of the scour predictor, Equation 8.68 was only applied for cases where $h_t/L_m \leq 0$ as the set-up is a maximum at the shoreline and decreases to the breaker line, where set-down will occur. In practice there will be an interaction between the incident and reflected waves so parameterisations of set-up derived for the open coast may not be particularly accurate in front of a structure.

Equation 8.66 was derived from tests with normally-incident irregular waves and beach slopes of 1:15, 1:30 and 1:75. The equation predicts maximum scour depth reducing with decreasing beach slope as seen in the laboratory data.

Equation 8.67 is plotted with the measured data in Figure 8.27, where 'O 1:N' and 'P 1:N' are the observed and predicted scour depths with a beach slope of 1:N (with N = 15, 30 or 75) respectively. The equation predicts the highest toe scour depths relatively well. There are relatively low errors for the high relative scour depths, which are likely to be the most important, while the largest errors in the predictions occur for negative observed scour depths (ie accretion at the toe of the structure). However, these cases may be relatively unimportant, at least as far as the stability of a structure is concerned.

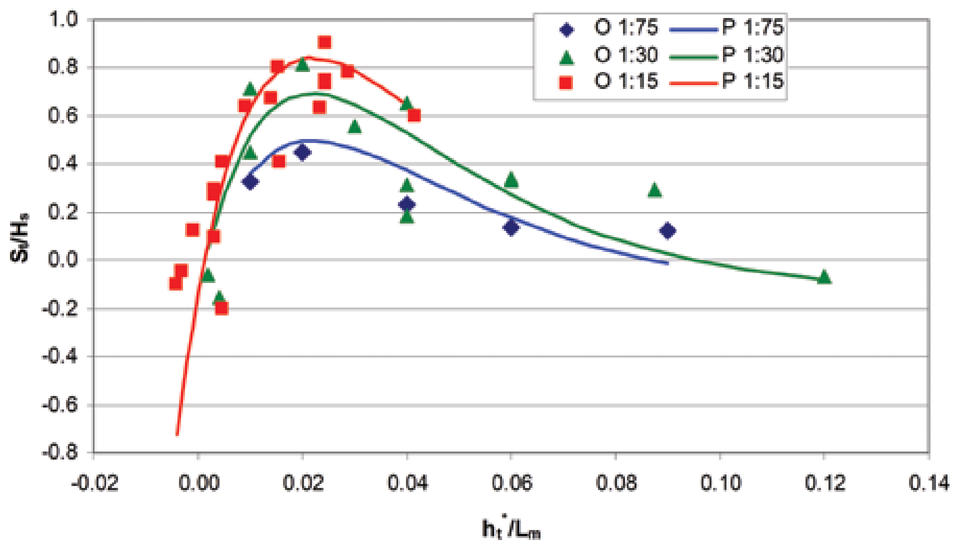


Figure 8.27 ***Measured and predicted (Equation 8.67) relative toe scour depths as a function of relative toe depth in sand (Sutherland et al, 2007)***

Both Equations 8.66 and 8.67 predict the scour after 3000 waves (ie 6.7 hours for an eight second period wave) and a correction has to be used to predict scour for time intervals other than 3000 waves.

8.2.4.4 Prediction of toe scour at vertical seawalls with shingle beaches

Scour depths in shingle beaches can be predicted using the parametric plot of Powell and Lowe (1994) reproduced as Figure 8.28. This was based on an extensive set of laboratory tests conducted with normally-incident irregular waves that broke on a 1:7 slope shingle beach, with a vertical impermeable seawall. The maximum scour predicted was $1.5H_s$. The method is valid for beach sediment in the range 5 mm $< d_{50} <$ 30 mm (modelled at 1:17 scale).

Figure 8.28 shows contours of S_{3000}/H_s plotted on a graph with axes of relative water depth, h_t/H_s and relative wave steepness, H_s/L_m, where:

h_t/H_s = the relative water depth

h_t = the initial water depth above the sediment level at the toe of the wall (m)

H_s = the extreme deep water (unbroken) wave height (m)
H_s/L_m = the wave steepness
L_m = the mean wavelength of the unbroken wave (using $T^2g/2\pi$) (m)
S_{3000} = the scour depth after 3000 waves.

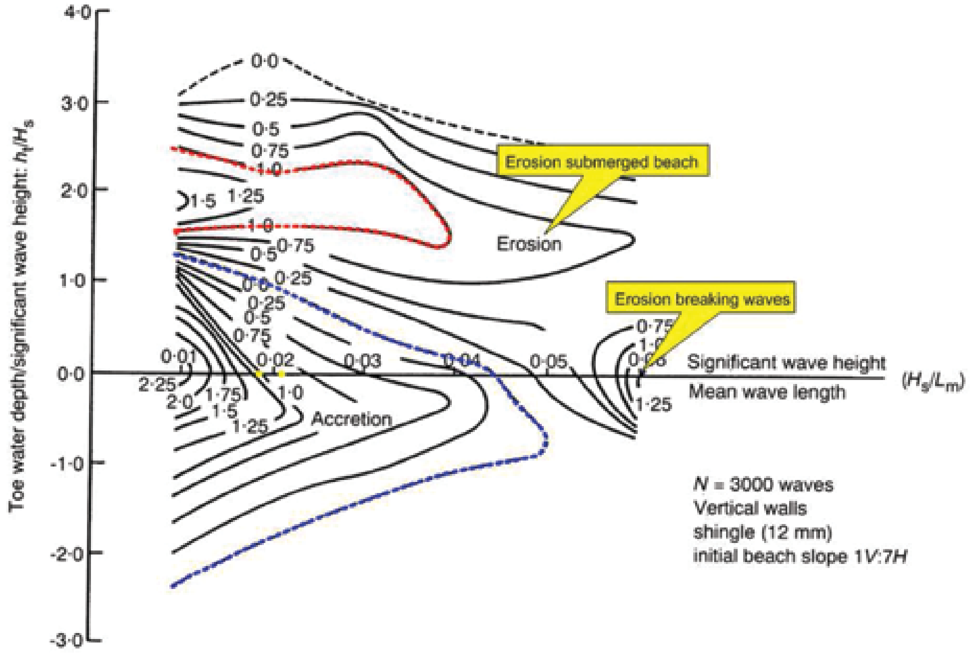


Figure 8.28 ***Prediction diagram for scour (erosion) and accretion at vertical seawalls with shingle beaches – contours of dimensionless scour depth S_{3000}/H_s (from Powell and Lowe, 1994)***

To select the worst possible scour, look at the dimensionless scour values for all h_t/H_s values below the maximum relative water depth, corresponding to the wave steepness, H_s/L_m and select the greatest relative scour height, which can exceed H_s. The plot gives the scour after 3000 waves, so a correction has to be used to predict scour for time intervals other than 3000 waves.

8.2.4.5 Effect of sloping front face on scour

The effect of a sloping wall on scour depths has been investigated by several authors, including:

- Sutherland *et al* (2006) compared the maximum scour depths and the toe scour depth at a 1:2 (27° above horizontal) sloping impermeable wall to those at a vertical impermeable wall for four different offshore wave conditions and water depths with H_{si}/h_t = 0.5 to 1.0, where H_{si} is the incident significant wave height and h_t the toe water depth. The results are shown in Figure 8.29 and show no systematic reduction in scour depth with wave height. In these cases the down-rush from the highest waves was reaching the seabed in some cases, which caused scour to occur.

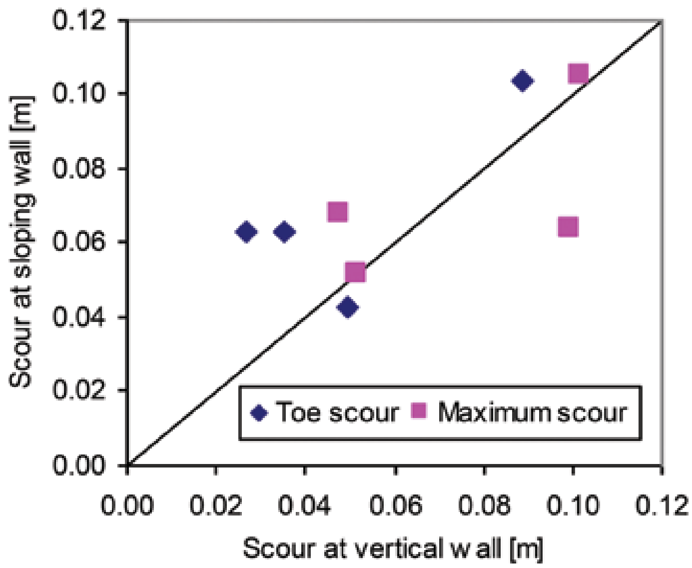


Figure 8.29 ***Comparison of laboratory measurements of scour depths in sand at a 1:2 sloping wall and at a vertical wall for the same offshore wave conditions (Sutherland et al, 2006)***

- Sumer and Fredsøe (2002) (Figure 7.17) quantified the effect of wall slope in the nonbreaking wave case (0.05>*d*/L>0.2) and showed that scour was reduced by about 80 per cent or 60 per cent for wall slopes of 30° and 40° respectively above horizontal (compared to the scour from a vertical wall). This is for the situation where the toe of the structure is always submerged and the bed in front of the structure is initially flat and horizontal
- Powell (1987) noted that for impermeable sloping structures of 1:1.5 to 1:2 there was no significant reduction in scour depth compared to that at a vertical wall. However, reducing the slope of an impermeable structure to 1:3 reduced the scour hole depth by 25 to 50 per cent. Powell also noted that rock armour revetments generally showed less susceptibility to local scour and may even show accretion
- Powell and Lowe (1994) showed a reduction in scour depth of almost 65 per cent in a shingle beach when a vertical wall was replaced by a sloping wall of 1:1.25. The scour depth was reduced by about 80 per cent for a 1:2 slope and there was accretion at the structure toe for a 1:3 slope. A rubble mound coastal defence showed no scour at its toe.

In shallow water the depth of scour is controlled by waves breaking on the wall and turbulence reaching the seabed. Under these circumstances the effect of reducing the seawall slope can be insignificant. It is only when water depths at the toe of the structure are sufficient to prevent turbulence reaching the seabed that a systematic reduction in scour depths with wave height can be expected. Moreover, for a sloping seawall, there is a phase shift on wave reflection (Sutherland and O'Donoghue, 1998) so the position of deepest scour may change to be away from the toe of the wall.

8.2.4.6 Storm duration

The duration of the wave/water level conditions is also an important control on toe scour development. Scour is not an instantaneous process – the trough deepens over a number of waves. Powell and Lowe (1994) demonstrated how scour in shingle develops until a quasi-equilibrium is obtained within about 3000 waves. It was noted that there was rapid initial scour that declined exponentially towards the equilibrium depth.

Similar trends are also apparent for sand beaches, though results from model studies (McDougal *et al*, 1996) suggest slower scour hole development, with equilibrium unlikely to be achieved within a realistic storm/water level duration. The experimental tests of Sutherland *et al* (2007) indicated that the average timescale of the scour was such that 95 per cent of the equilibrium scour depth would be reached after about 2500 waves, although there was considerable scatter in the timescales derived. For typical storm mean wave periods of six to eight seconds, this would take between about four and 5.5 hours to achieve.

The use of Equation 8.69 is recommended for predicting potential scour depths in the field. If the environmental conditions are expected to last for less than 3000 wave periods, the expected scour depth may be reduced by a factor determined from Equation 8.69.

$$S(t) = S_e\left(1 - \mathrm{e}^{-t/T_s}\right) \qquad (8.69)$$

where:

$S(t)$ = scour depth at time t (m)

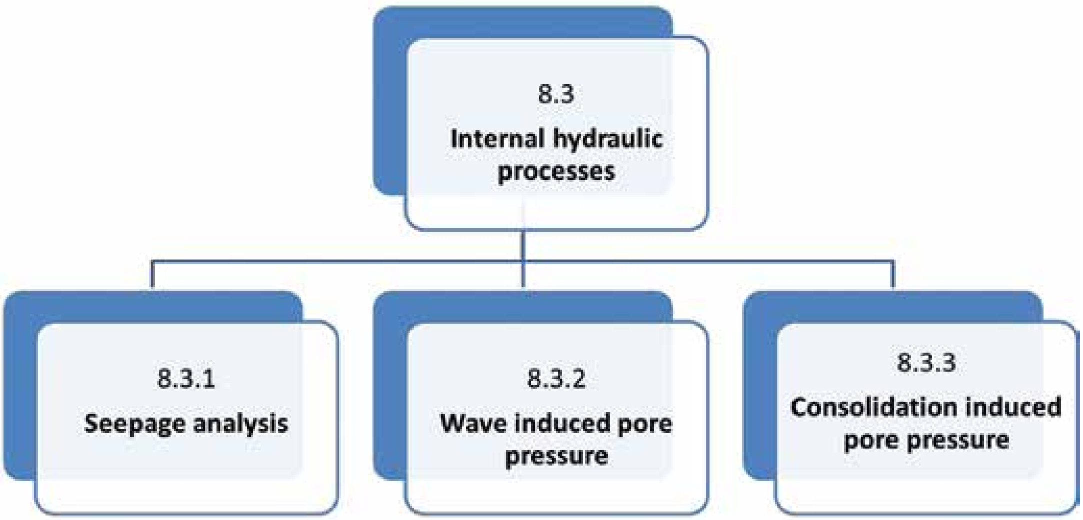
t = time since start of scour process (s)

S_e = equilibrium scour depth (m)

T_s = timescale for scour (s)

McDougal *et al* (1996) suggests T_s = 3100T, with T the wave period. Xie (1981) suggested that for fine sand in suspension the equilibrium scour depth would be reached in 6500 to 7500 wave periods for $H/L > 0.02$ and in 7500 to 10000 wave periods for $H/L < 0.02$.

8.3 INTERNAL HYDRAULIC PROCESSES



Hydraulic and mechanic actions may induce water flows and pore pressure fields within the levee and its foundation. Failure modes are influenced by the pore pressures and flow distributions and variation during time. All levees are subjected to internal flows as a result of either steady or transient external hydraulic conditions, and are a function of levee and foundation materials. Under hydraulic loading, seepage can occur either through the levee (through-seepage) or in its foundation (under-seepage). This phenomenon is accounted for in a levee stability assessment because pore water pressures and flows have a strong influence on deterioration and failure modes such as internal erosion (Section 8.5), slope stability (Section 8.6), and settlement (Section 8.7).

In this section, two main types of actions will be distinguished as shown in the section flow chart:

1 **Stationary hydraulic actions,** eg slow varying water level regarding the drainage characteristics of the soil

2 **Non-stationary hydraulic actions,** eg waves, which change rapidly regarding the drainage characteristics of the soil.

8.3.1 Stationary seepage analysis

8.3.1.1 Introduction

Seepage is governed by hydraulic laws initially developed for saturated soils. One difficulty for levees in opposition to dams is that materials (of levee body or foundation) are often totally or partially unsaturated when a flood event occurs and simple models or methods to study seepage are then not strictly applicable. However, they are often used in a first phase of studies because they are safer for stability analysis. Newer finite element programs include complex models to take into account unsaturated soil flow laws. These routines to perform partially saturated seepage analyses require additional inputs that are not very well known in practice.

The aim of a seepage study for levee design or analysis is to determine the following elements that could be used in stability analysis and for designing specific seepage control solutions:

- the phreatic line
- internal pore pressures that could occur in levee material or its foundation
- exit gradient
- seepage flow rate.

8.3.1.2 Basic hydraulic laws

Hydraulic head: Bernoulli's law and gradient

When a structure is subject to hydraulic head and for almost all geotechnical structures (and then for earthen levees and their foundations), flow of groundwater through a saturated soil is governed by Darcy's law:

$$q = A\,k\,i \qquad (8.70)$$

where:

q = volumetric flow rate (m^3/s)
A = cross-sectional area of flow (m^2)
k = Darcy's coefficient of permeability or hydraulic (m/s)
i = hydraulic gradient in the direction of flow (-)

The hydraulic gradient i is defined as the rate of total hydraulic head dh (m) with distance dx (m) along the direction of flow, defined as follows.

$$\overrightarrow{i} = -\overrightarrow{grad}\,h \qquad (8.71)$$

Box 8.6 gives a definition of hydraulic head, line of seepage and seepage surface.

1 2 3 4 5 6 7 8 9 10

Box 8.6 Definition of hydraulic head, line of seepage and seepage surface

In saturated soil, Bernoulli's Equation 8.72 enables to have the total hydraulic head *h* in each point *M* of the levee:

$$\boxed{h = z + \frac{u}{\rho_w g} + \frac{v^2}{2g}} \tag{8.72}$$

The flow velocity *v* in soil is generally very slow (<1 m/s). So, the velocity head (quadratic term equivalent to kinematic energy) can be neglected in most cases and then the following simplified equation can be used:

$$\boxed{u = z + \frac{u}{\rho_w g}} \tag{8.73}$$

where:

h	=	hydraulic head (m)
z	=	altitude of considered point related to reference plan (m)
u	=	internal pore pressure (kN/m^2)
v	=	flow velocity (m/s^2)
ρ_w	=	water volumetric mass (kN/m^3)
g	=	gravity acceleration (9.81 m/s^2)

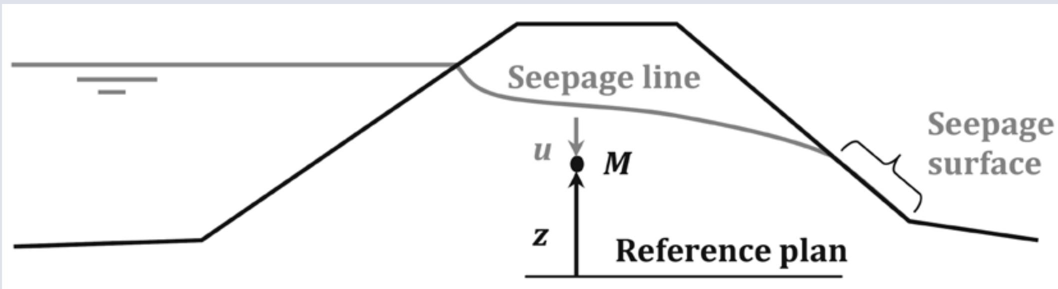


Figure 8.30 Phreatic line and surface of seepage in a levee cross-section with steady stage water level

In saturated soils, Darcy's law is valid under certain conditions. Firstly, in very low permeability soil such as highly plastic clay, flow cannot occur under a high threshold hydraulic gradient. Soil is then considered to be impervious (range of permeability in 7.8.3). Secondly, at very high flow rate, it has been recognised that Darcy's Law does not hold because flow is turbulent and no longer laminar (Chugaev, 1971). Regarding average diameter of soil particles, boundary between laminar and turbulent flow can be determined using Reynolds number (Box 8.7).

Under conditions of partial saturation, the flow is in a transient state and is time dependent. Darcy's law can no longer be strictly applied. However, it can be useful to apply Darcy's law in conditions where it is not strictly valid, to have in a first step of levee design an approximation (often by excess) of flow rate, flow velocity etc.

Box 8.7 ***Boundary between laminar and turbulent flow using Reynolds number***

The Reynolds number *R* is a dimensionless number that expresses the ratio of internal flow force to viscous force:

$$\boxed{R = \frac{v\,D\,\rho}{\mu}} \tag{8.74}$$

where:

v = true flow velocity (m/s)

D = average diameter of soil particles (m)

ρ = fluid density (kN/m^3)

μ = kinematic viscosity of fluid (kN/m/s)

The critical value of *R* at which the flow in soil changes from laminar to turbulent has been determined to range from one to 12 (Chugaev, 1971). For a water temperature of 20°C, ρ = 9.982 kN/m^3 and $\mu = 1.002 \times 10^{-5}$ kN/m/s, Figure 8.31 shows the upper boundary of validity of Darcy's law (laminar flow). Then, depending on the discharge (flow) velocity *v*, it is assumed that Darcy's law is applicable for silts through medium sands.

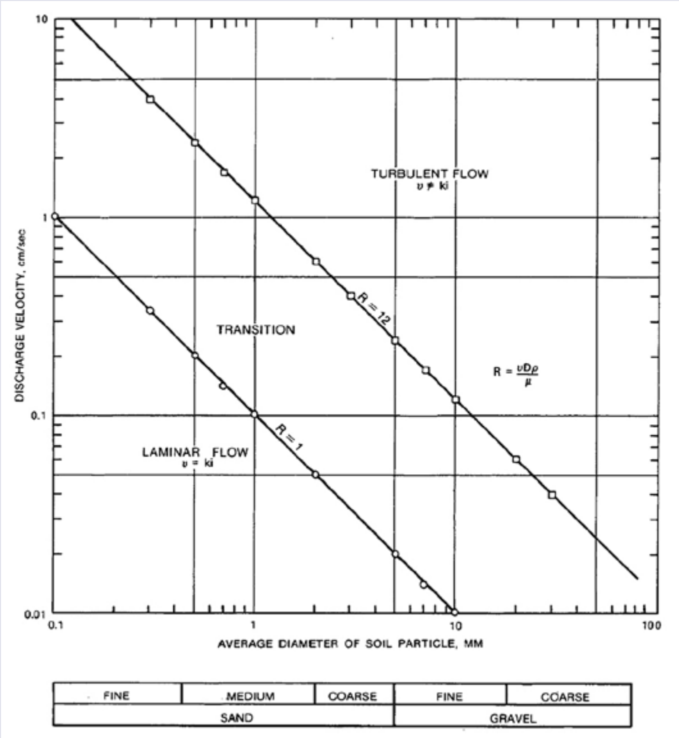


Figure 8.31 ***Boundary between laminar and turbulent flow in using Reynolds number and limit of Darcy (USACE, 1993)***

Flow velocity, seepage velocity and flow strength

For levees, if the duration of the flood is sufficiently long to impact the hydraulic conductivity of material (Case b, Figure 8.34), internal flow and seepage can occur. According to Darcy's law (Equation 8.70), the groundwater flow velocity (discharge flow velocity) is given by the following equation:

$$\overrightarrow{v_D} = k\,\overrightarrow{i} = -k\,\overrightarrow{grad}\,h \tag{8.75}$$

Equation 8.76 gives the relation between both velocities for a soil of porosity n ($0 < n < 1$, or a void index e):

Note

The discharge velocity is not the true velocity of the flow through the pores: the true seepage velocity v_t exceeds discharge velocity (which corresponds to an average laminar flow path through the soil as shown in Figure 8.32).

$$v_t = \frac{v_D}{n} = \frac{1+e}{e} v_D \tag{8.76}$$

Figure 8.32 ***Concepts of flow paths through a soil column (USACE, 1993)***

In a saturated soil, the flow velocity creates a flow density force on grains as presented in Figure 8.33. This force is given by Equation 8.77. This can initiate instabilities, primarily at the seepage exit point, like internal erosion of soil or shallow surface slope instabilities (Section 8.4) and then lead to important deteriorations or levee failure.

$$\vec{f} = \gamma_w \vec{i} \tag{8.77}$$

where:

γ_w = water volumetric mass (kN/m^3)
i = hydraulic gradient (-)

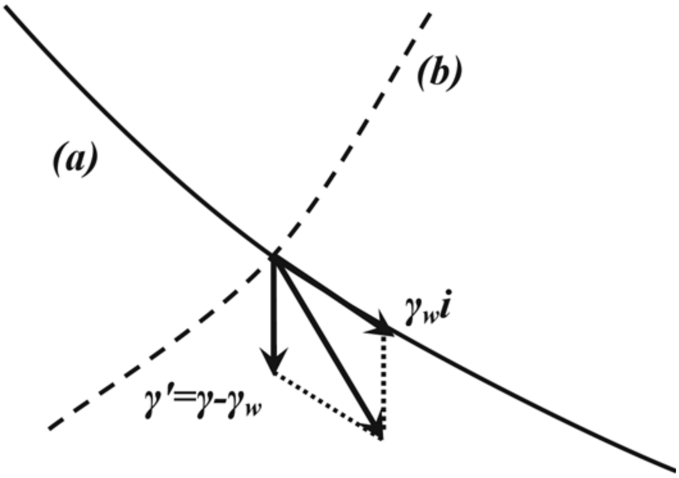


Figure 8.33 ***Hydraulic flow forces on grain in saturated soils due to flow gradient, current line (a) and equipotential line (b)***

8.3.1.3 Permeability and anisotropic permeability effects on levee saturation

The permeability (also called hydraulic conductivity) is one of the main parameters influencing seepage. In natural soils or built earthen structures such as levees, this parameter is quite difficult to obtain and is not equivalent in all directions (anisotropy of permeability). The main parameters influencing permeability of soils are the nature of soils (deposition modes), sizes and forms of particles, contents of fine elements, properties of seepage fluids (viscosity regarding to temperatures) and degree of saturation of soils. More information can be found in USACE (1993) and CFBR (2010). *In situ* and laboratory devices and tests to measure permeability (k), anisotropy (i) are described in Chapter 7 (Section 7.8.3). Table 7.111 provides typical values of hydraulic conductivity (permeability) for different types of soils.

Figure 8.34 shows the influence of permeability on levee saturation during a flood event. In Case a, the permeability is low enough that the levee is only partially saturated and seepage will not occur during a flood event. In contrast, Case b, shows the permeability is large enough to lead to full levee saturation producing seepage during a flood event. Landward slope instabilities and internal erosion can then occur.

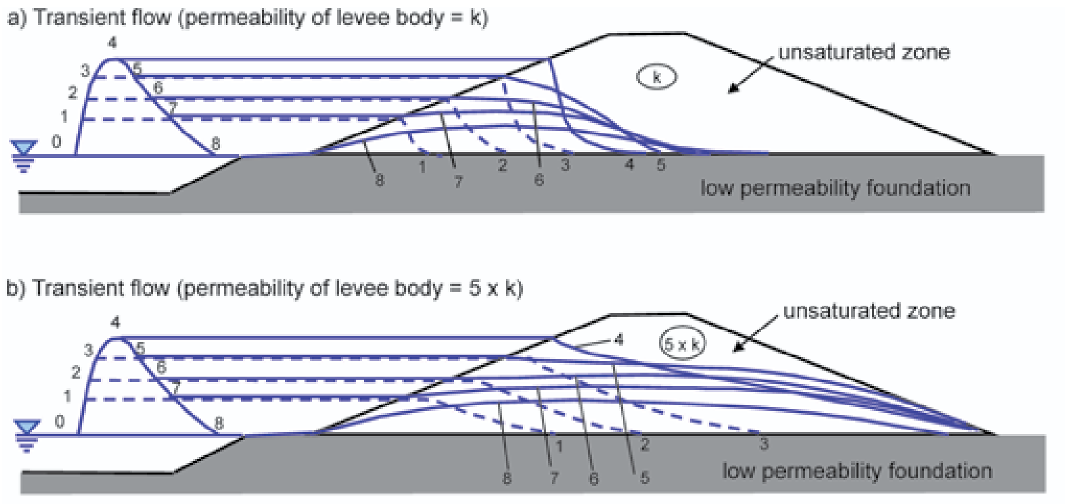


Figure 8.34 ***Effect of permeability on levee saturation during transient states of flooding situation (adapted from German guideline)***

Hydraulic conductivity is generally anisotropic, ie conductivity in vertical (k_v) and horizontal (k_h) directions are different. In natural soils, horizontal conductivity is generally greater than vertical conductivity (from 10 times in clay material to 100 times and more in fine layered soils), resulting from the deposition modes of soils. For built earthen structures as levees, this is often also the case because of construction of the levee by layers placed horizontally. Note that for the upper layer, cracks in silty or clayey soils can lead to a vertical permeability greater than the horizontal permeability. Figure 8.35 shows the effect of anisotropy on flow network. If the anisotropic rate is too large, seepage occurs on the landward slope.

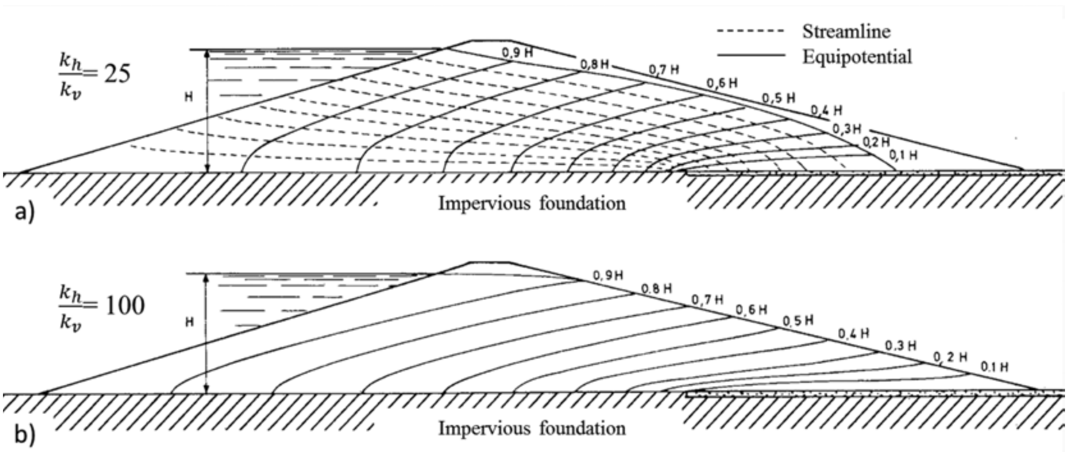


Figure 8.35 ***Impact of anisotropy of permeability on flow network for a permeable levee built on impervious foundation and for a steady state situation (after Josseaume, 1970)***

8.3.1.4 Determination of phreatic, flow and equipotential lines

For levee stability analysis and design, the flood event leads to several transient hydraulic situations. However, even in transient situations, it is easier and often safer to analyse the levee considering design water levels (Sections 7.3.5 to 7.3.9) in a permanent state (realising that these design situations do not strictly reflect reality). To do so, the determination of phreatic line is necessary and represents one of the first steps of modelling. Figure 8.36 gives an illustration of an approximation that can be done when considering permanent state instead of transient state. Note that for levee stability analysis (slope stability, internal erosion etc), considering permanent state water level (Case b, Figure 8.36) is often a safe approach because a higher internal phreatic line is taken into account in the design process.

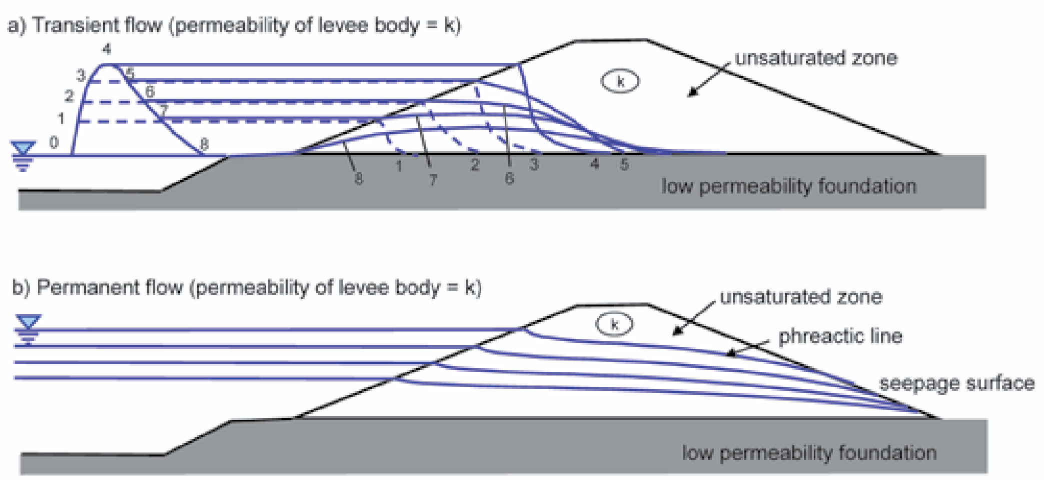


Figure 8.36 ***Comparison of saturation state during transient states of a flooding situation (a) and permanent state (b) considering same water level (after German guideline)***

The first step for a seepage analysis is to determine the position of the phreatic line, which is a hydraulic boundary condition for the flow network. Several methods are presently available to define the saturation line in an earthen structure including geometrical, analytical, and numerical methods. Simple methods such as the graphical methods determine this position with sufficient precision to perform initial calculations (Figure 8.37 in Box 8.8). Analytical methods, such as the segment method, are often empirical.

Box 8.8 ***Usual graphical methods for determining saturation line position***

Several authors proposed solutions to determine position of phreatic line and exit surface of seepage. These simplified methods are often used and give approximate but sufficient solutions. Kozeny shows that for a homogenous undrained earthen dam, the saturation line through the levee could be approximated with a parabolic line as defined on Figure 8.37 below. Several equations are proposed in Table 8.8.

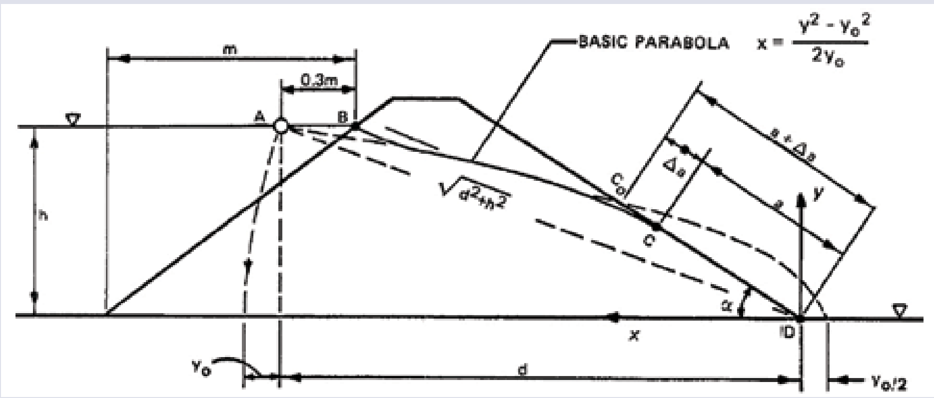


Figure 8.37 ***Phreatic line determination methods – terminology (USACE, 1993)***

Table 8.8 ***Equations for phreatic line determination (USACE, 1993)***

α (°)	**Methods**	**Equations**
< 30	Schaffernak	$a = \dfrac{d}{\cos\alpha} - \sqrt{\dfrac{d^2}{\cos^2\alpha} - \dfrac{h^2}{\sin^2\alpha}}$
	Van Iterson	$q = k\ a\ \sin\alpha \tan\alpha$

Box 8.8 Usual graphical methods for determining saturation line position

≤ 90	Casagrande	$a = s_0 - \sqrt{s_0^2 - \frac{h^2}{\sin^2\alpha}}$ with $s_0 = \sqrt{d^2 + h^2}$ if $\alpha \leq 60°$ or $s_0 = (\widehat{AC} + \overline{CD})$ if $60° \leq \alpha \leq 90$
180	Kozeny	$a_0 = \frac{y_0}{2} = \frac{1}{2}\left(\sqrt{d^2 - h^2} - d\right)$ $q = 2\,k\,a_0 = k\,y_0$
30 to 180	Casagrande	Determine (a + Δa) as the intersection of the basic parabol a and levee slope. Then determine Δa from C value on figure (a) $q = k\,a\,\sin^2\alpha$ or $q = k\,y_0 = k\left(\sqrt{d^2 - h^2} - d\right)$

Numerical methods are commonly included in finite element software, but it is important for readers to appreciate that these methods use complex equations to resolve flow through porous material. Attention should be paid to the assumptions and limits for use of each software program, and the analyst should validate these complex methods even if the project is not complex. Readers can find more details on methods in USACE (1993).

When saturation line is determined, current and equipotential lines can be graphically obtained considering boundary conditions:

- river or sea face of levee is an equipotential line
- saturation line and contact line between impervious layer are both current lines
- equipotential line and current lines are perpendicular
- hydraulic pressure u along phreatic line is null so hydraulic head along this line is due to altitude.

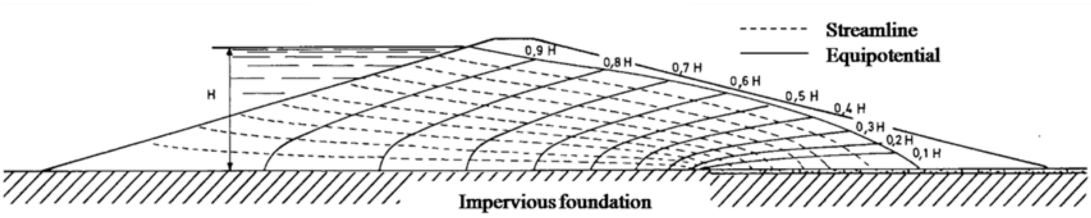


Figure 8.38 Example of flow net construction in an earthen levee on impervious foundation (adapted from BLR, 1970)

Then, flow net construction enables the estimation of total discharge q considering that, on each current line, Equation 8.70 can be applied.

8.3.1.5 Internal pore pressure

When the flow net is known and described, it is easy to determine internal pore pressure for each point as shown in Figure 8.39. Using terminology of this figure, M_0 and M_1 are on the same equipotential line so the internal pore pressure at M_0 expresses:

$$u\,(M_0) = \gamma_w\,[z\,(M_1) - z\,(M_0)] \tag{8.78}$$

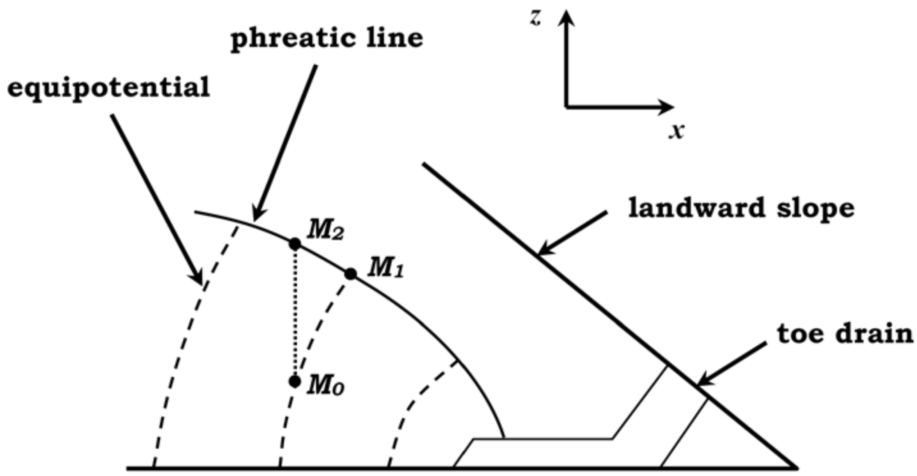


Figure 8.39 Example of internal pore pressure determination using flow net for a levee with toe drain (after Rolley et al, 1977)

For earthen levees, if the line of seepage is too high in the levee landward slope, it can initiate landward slope deterioration and instability. Design control systems are available (as toe drain etc) to control seepage. Description of such controls is given in Chapters 9 and 10. Such systems, designed to collect seepage flow passing through an embankment or its foundation has to follow criteria to be efficient for drainage but also to prevent material transport from one soil layer to another. These filter criteria are detailed in Chapter 9.

Hydraulic forces, excessive gradients or flow velocity through a levee or its foundation and the resulting excessive internal pore pressure are responsible for deterioration processes such as internal erosion (Section 8.5), and slope instabilities, hydraulic cracking, heave and uplift (Section 8.6).

8.3.1.6 Exit gradients

For levee diagnosis, specific design (interfaces with drainage systems) or complex structures (levees with embedded structures), it can be necessary to evaluate local exit gradients. For levee slope stability at the landward toe, most soil mechanics textbooks state that exit gradient should not be greater than one. However, considering earthen structures, factor of safety for critical exit gradient are recommended according to the soil's nature. Details on critical exit gradient are given in Section 8.5.

In flow-net and seepage analysis, if flow is unidirectional, the exit gradient $i_e = dh/dx$ (-) is determined between the last two successive equipotential lines at the landward toe. For a levee, the flow is generally not unidirectional (and vertical) but inclined with regard to the horizontal plane. Then, the exit gradient can be determined by Equation 8.79, knowing the exit velocity orientation as shown in Figure 8.40.

$$i_e = \frac{v}{k_s} = v\frac{k_h \sin^2 \alpha + k_v \cos^2 \alpha}{k_h k_v} \qquad (8.79)$$

where:

k_s	=	is the soil conductivity in $\vec{v}$ direction (m/s)
α	=	the angle between $\vec{v}$ and the horizontal plane (°)
k_h	=	horizontal hydraulic conductivity (m/s)
k_v	=	vertical hydraulic conductivity (m/s)

It will be necessary to use a numerical program that enables the calculation of local velocities. Figure 8.40 shows an example of a levee flow network with local velocities.

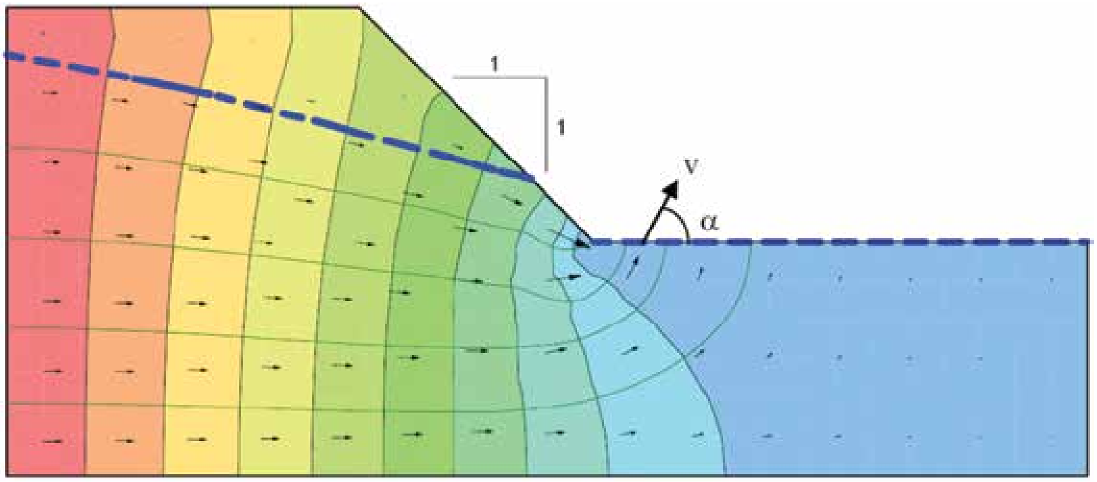


Figure 8.40 *Example of levee flow network and local exit velocity orientation (after Mishraand Singh, 2005)*

8.3.1.7 Numerical models for seepage analysis

For complex structures or for levee design in transient state, it is assumed that the use of 'piezometric lines' to determine pore water pressures can be incorrect (and unsafe) when there is a significant anisotropy of permeability and when vertical flows exist. Then, the use of numerical models, mostly based on FEM, is generally more rigorous and computations are rapid. However, they are more complex to use and require data based on additional sophisticated specific tests or specialised technical experience. Caution should be given that a result can always be obtained from the numerical models, which may not be based on valid data. It is then highly recommended to validate results with rapid simplified calculations to get an understanding of the order of magnitude of each parameter.

However, for complex levee design or critical analysis, it could be necessary to use specific geotechnical software that can take into account internal flow in porous media. Further points that should be noted (CFBR, 2010) are:

- elastic and perfectly plastic behaviour laws with Mohr-Coulomb criteria should be adopted
- construction stages to initiate effective stress in soil need to be modelled
- interstitial pore pressure, gradients, flows (saturated or unsaturated), and seepage should be taken into account
- interfaces between soil and rigid structures should be modelled
- for fine soil, consolidation should be taken into account.

Currently, several software programs (eg Seep/W, Plaxis and PlaxFlow, Cesar LCPC) enable engineers to study seepage using FEM of earthen structures. Each program has its own limits and the analyst should read the user manual to be familiar with these limitations. An example of FEM is shown in Box 8.9.

Note

For certain programs, the results defining material pore pressures during a flood event can be coupled with classical 2D stability programs, eg Talren V4 with Plaxis or Slope with Seep/W.

Box 8.9 ***Example of application of FEM for levee stability diagnosis and design***

On the River Loire, a general campaign of levee reinforcement began in the 1980s. Several techniques had been employed to ensure stability during flood events, but the most used was an enlargement of levee cross-section with embankment fill put on a drainage granular system (called 'drained carpet'). The geometry of the reinforcement enables lower slopes and containment of the phreatic line inside the levee, producing a better slope stability factor of safety.

At the same time, sandy and granular Loire sediments were extracted, external erosion occurred and the riverbed decreased by 2 m or 3 m locally. Instability then occurred on the riverside slopes and needed to be stabilised. For such design, an FEM was used to appreciate actual stability factor of safety and test different ways to reinforce the levees. Plaxis V8 and Plaxflow were used by the contractor to model the levee (Figures 8.41 and 8.42) during transient state of flood event.

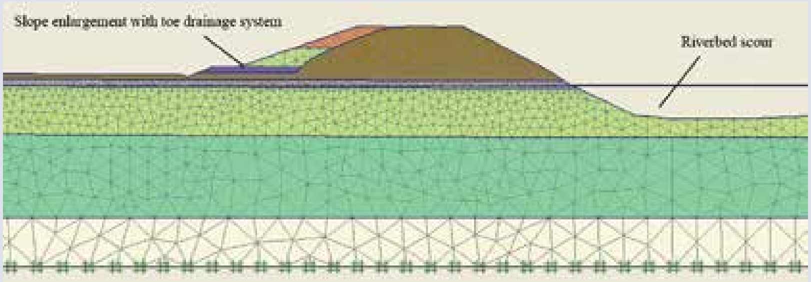


Figure 8.41 ***FEM of the River Loire's levee to study seepage and slope stability during permanent and transient state (flood event), Guilly, France***

Pore pressures, and flow velocities were considered at the landward toe to conduct stability analyses (slope stability, uplift, internal erosion etc) and design reinforcements.

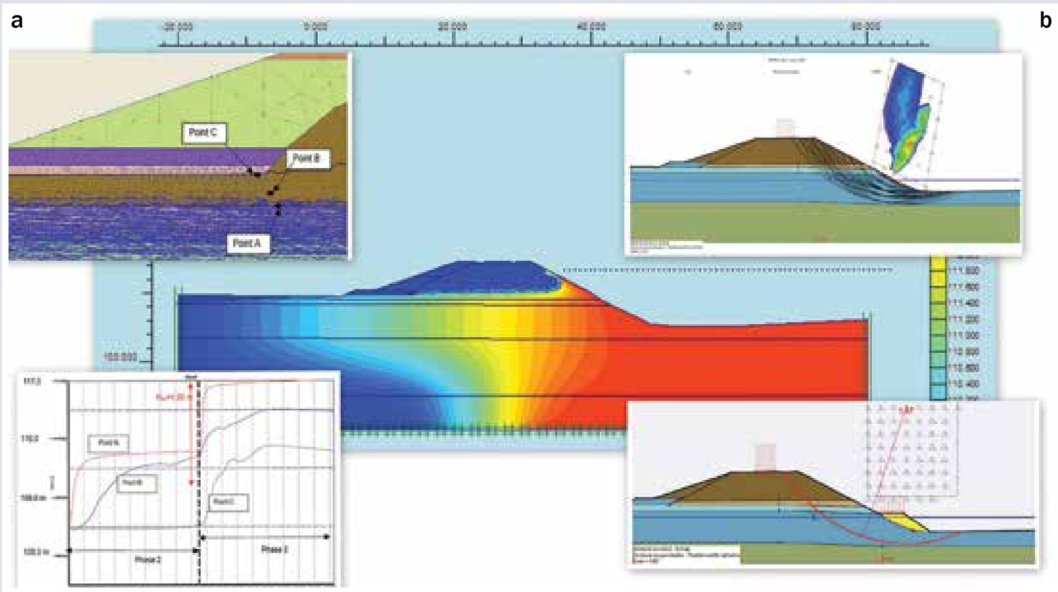


Figure 8.42 ***Example of a levee FEM used in transient state for appreciate saturation state (in the middle), pore pressures, active groundwater head and exit gradients during and after a 48 hours water elevation (flood event) (a) and slopes stability before and after reinforcement (b)***

8.3.2 Wave induced pore pressures

The specific effect of waves on internal pore water pressure lies in the fact that the hydraulic action varies quickly with time. The pore pressure response depends on the phreatic level imposed (Section 8.3.1), but also on two strain components of the soil under wave loading: elastic volume strain of soils skeleton and/or pore water and plastic volume strain of the soil skeleton (irreversible variation of the pore volume).

8.3.2.1 Pore pressure due to elastic strain

Variation of pore pressure results in effective stress variation and consequently variation of the pore volume due to the compression of the soil skeleton. This phenomenon produces water flow in and out at a rate governed by the soil permeability. When the rate of pressure changes along the external boundary, the flow rate becomes too quick in relation to the soil permeability, so the soil is no longer fully drained and the pore water pressure may progressively increase. This mechanism is characterised by a phase lag in the propagation of the cyclic phenomenon. A simplified analysis (assuming the incompressibility of water) may be performed based on the determination of an elastic timescale T_{el} (s) defined as follows (CIRIA; CUR; CETMEF, 2007):

$$T_{el} = \pi L^2 \frac{\gamma_w m_{ve}}{k} \tag{8.80}$$

where:

L	=	is a distance of penetration through the soil (m)
k	=	the soil permeability (m/s²)
m_{ve}	=	the elastic coefficient of volume change of the soil (-)

When considering the period of loading T (s), the ratio $T_{el}/T << 1$ corresponds to a negligible elastic storage and the load may be considered as quasi-stationary. However, if $T_{el}/T >> 1$, the elastic storage is important and the generated pore pressure increase has to be taken into account in the stability analyses.

8.3.2.2 Pore pressure due to plastic strain

Pore volume change may also be caused by dilatancy and contraction. Cyclic shear loading in loose soils may have a tendency to contract but in cases where the soil permeability is too small in relation to the period of external loading, the densification of the soil may be partly prevented by the pore fluid. The result of this phenomenon is a generation of excess pore water pressure within the soil, which increases at each load cycle (each wave). The characteristic timescale, T_{pl}, may be defined as follows (CIRIA; CUR; CETMEF, 2007):

$$T_{pl} = \frac{L^2}{N} \frac{\gamma_b \, m_{ve}}{k} \tag{8.81}$$

where:

L	=	the length over which the wave induced shear stress is important (m)
γ_b	=	the bulk unit weight of the dry soil (kN/m³)
k	=	the soil permeability (m/s²)

For example, the number of stress cycles for annulment of effective stress N may be determined in laboratory tests as a function of shear stress ratio and density index.

For practical application, 1D models are available (Ishihara and Yamazaki, 1984). The results of these models have to be taken into account in stability analyses.

An example of wave induced pore pressures that result in cyclic shear stresses in the soil is given in Box 8.10.

Box 8.10 ***Wave-induced cyclic shear stresses (from Ishihara and Yamazaki, 1984)***

The differential loading on the floor caused by the pressure wave induces a cyclic shear stress loading in the underlying soil. These stresses may cause significant deformations and even failures due to liquefaction phenomenon. The most common method for wave-induced liquefaction assessment was developed by Ishihara and Yamazaki (1984) and may be summarised as follows.

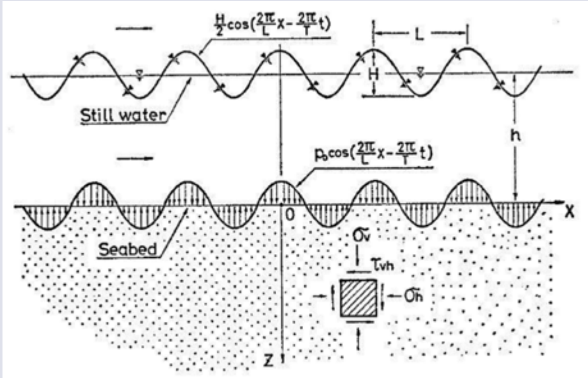


Figure 8.43 ***Definition of notations for wave-induced shear stress (from Ishihara and Yamazaki, 1984)***

Water waves propagating are considered to consist of an infinite number of wave trains having a constant amplitude and wavelength. Passage of such waves creates harmonic pressure waves on the seafloor. The stresses induced in the seabed are therefore analysed applying a sinusoidal changing load on the infinite horizontal surface. It can be shown that the cyclic stress ratio equals to:

$$\frac{\tau_{vh}}{\sigma_{v\prime}} = \left(\frac{\tau_{vh}}{\sigma_{v\prime}}\right)_{z=0} e^{-2\pi z/L} \qquad (8.82)$$

where:

τ_{vh} = amplitude of the shear stress (kPa)

σ_v' = vertical effective overburden pressure (kPa)

z = depth into the soil from the mud line (m)

The cyclic stress ratio at the mud line is expressed by:

$$\left(\frac{\tau_{vh}}{\sigma_{v\prime}}\right)_{z=0} = \frac{\pi\gamma_w H_0}{\gamma' L_0 \sin h\left(\frac{2\pi h}{L}\right)\sqrt{\tan h\left(\frac{2\pi h}{L}\right)\left(1+\frac{4\pi h/L}{\sin h(4\pi h/L)}\right)}} \leq \frac{\pi}{7}\frac{\gamma_w}{\gamma'}\frac{\sin h\left(\frac{2\pi h}{L}\right)}{\cos h^2\left(\frac{2\pi h}{L}\right)} \qquad (8.83)$$

where:

γ' = submerged unit weight (kN/m^3)

h = water depth (m)

H_0 = wave height in deep water condition (m)

L_0 = wavelength in deep water condition (m)

L = wavelength of the wave train where the water depth is *h* (m)

Note that equation 8.83 constrains the wave steepness to a value below a critical value as expressed by the inequality at the end of the equation.

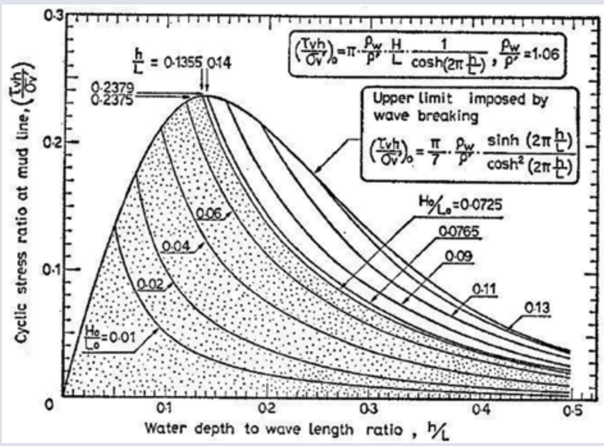


Figure 8.44 ***Estimation of cyclic stress ratio at mud line (from Ishihara and Yamazaki, 1984)***

The cyclic stress ratio calculated is then compared to the cyclic stress ratio causing liquefaction and cyclic mobility in which the continuous rotation of principal stress directions is considered. The procedure is described in detail in Ishihara and Yamazaki (1984).

8.3.3 Consolidation induced pore pressure

It has been shown (Skempton and Bjerrum, 1957, and Henkel, 1959) that a relationship may be established between spherical and deviatoric consolidation stress increments according to Equation 8.84:

$$\triangle u\,(t) = B\,(\triangle\sigma_{oct} + A\triangle\tau_{oct})\,[1 - U\,(t)] \tag{8.84}$$

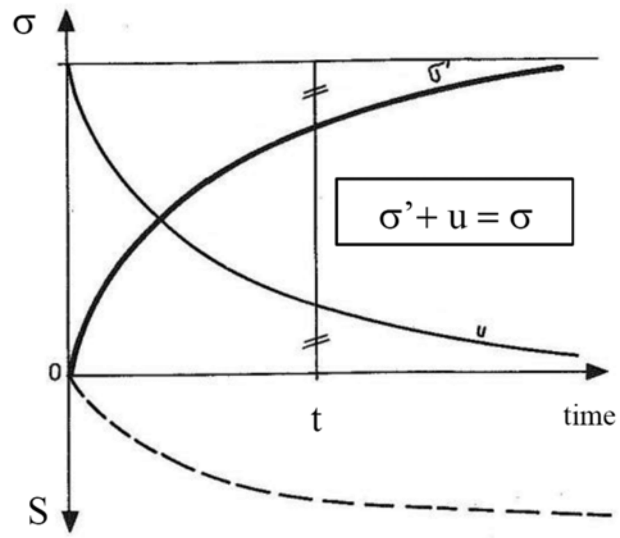
where B and A are pore pressure parameters (Section 7.8.3) depending on the degree of saturation and the compressibility of the soil skeleton, U(t) is the consolidation ratio at time t (Section 8.7.2). For normally saturated consolidated soils B is generally taken equal to one.

Except when the factor of safety of the slope is low, the part of the pore water pressure induced by shear deformations (coefficient A) is negligible and the horizontal earth pressure may be taken equal to the at-rest one K_0 (-). Under these assumptions, it is possible to express the pore pressure ratio r_u in terms of the incremental vertical load:

$$r_u\,(t) = \frac{\triangle u\,(t)}{\triangle\sigma_v} = \frac{1 + 2K_0}{3}\,[1 - U\,(t)] \tag{8.85}$$

This formula may be useful for determination of pore water pressure implementation in slope stability analyses (Section 8.6) during construction phases.

As illustrated in Figure 8.45, when a load is applied on a saturated low permeability soil, the pressure $\Delta\sigma$ (total stress) is firstly supported by the soils interstitial water that is uncompressible. The excess pore pressure Δu becomes quasi instantaneously equal to load pressure. If that load is maintained constant, a time dependant compression phase begins known as the primary consolidation phase. It corresponds to a period where water goes out of the soil and excess pore pressure Δu decreases.

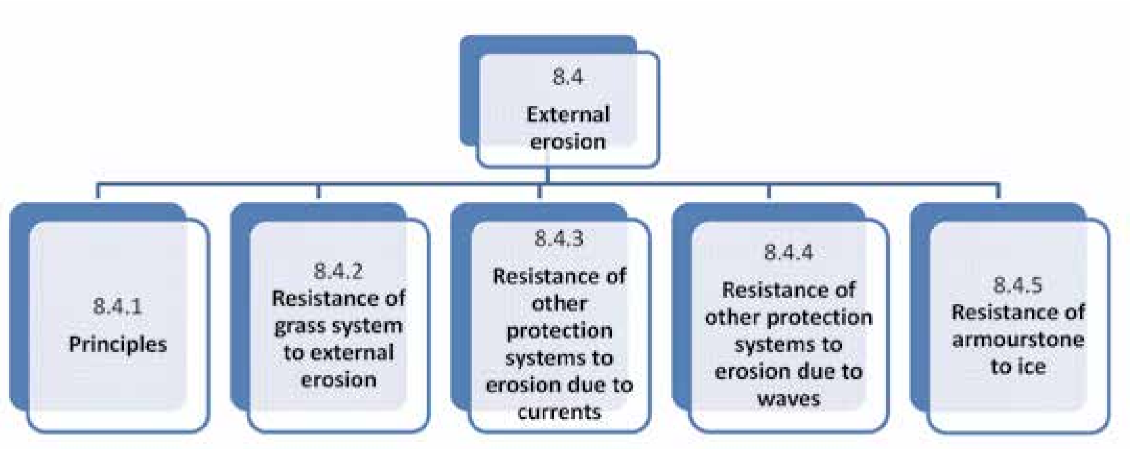


Notes

σ = total stress, σ' = effective stress (bold line), u = pore pressure, s = settlement (dotted line)

Figure 8.45 ***Soils primary consolidation phase. Settlement and excess pore pressure evolution (after Philipponnat and Hubert, 2003)***

8.4 EXTERNAL EROSION



8.4.1 Principles

In addition to the hydrostatic and hydrodynamic forces that act on the levee structure, the movement of water over the surface of the levee has to be considered. Hydraulic interactions associated with wave and current action on levees have been previously described in Section 8.2. It is necessary to consider the influence of these interactions on the levee to ensure its integrity and long-term stability when exposed to various hydraulic loadings. This section describes methods for assessing the effect of currents and waves on the levee surface, and provides limited guidance for the design of measures to protect against those effects.

8.4.1.1 Currents

The importance of considering currents during levee design derives from the potential that exists for moving water to mobilise material on the levee surface or in locations that would impact levee stability. This section describes currents that should be considered during analysis or design of levees.

Currents in the main channel

Flow in the main channel interacts with and shapes the channel boundary. The continual change in channel boundary identified in the morphologic assessment described in Chapter 7 may indicate that protective measures are needed to prevent damage to the levee. Such protective measures may involve armouring the channel bank or installing features that redirect the current direction. Levee planning and design has to account for future changes in the channel to ensure acceptable system performance. Sediment transport studies done in site characterisation (Chapter 7) provide indication of long-term trends in channel erosion and deposition. So, it is necessary to expand those estimates to locations where there is potential threat to the levee. Specifically, local velocity at the exterior bank of bends and resulting scour depth has to be determined so that protection schemes can be designed. As described in Section 7.3, velocity distributions vary with cross-section shape and alignment. So, it is necessary to apply correction factors to mean channel velocity or to develop multi-dimensional numeric models to determine the near bank velocity in bends.

Currents on the levee surface

As flow moves across and along the surface of a levee it imposes not only static and dynamic forces that the levee has to resist, but also a drag on levee surface materials as it moves across the levee. The drag, caused by boundary shear stress (Section 7.3), can mobilise materials leading to erosion and eventual failure of the levee embankment. Currents induced by the stream flow during various levels of flood, including the maximum anticipated event, impose boundary shear stresses at different magnitudes. So, it is necessary to evaluate the shear stresses at various flow levels.

In steady flow, the current-induced shear stress acting on the bed may be calculated using Equation 7.45.

The first parameter to estimate is the cross-sectional averaged velocity for the portion of cross-section near the levee. This velocity is often available from numerical models developed during site characterisation (Section 7.3), or can be calculated from the model results. The Manning-Strickler or Chézy equations (Section 7.3.6.1) provide a simplified method for calculating the average cross-section velocity. The cross-section velocity gives some indication of velocities that may exist near the levee surface. The shear stress computed with average velocity acts at the stream bed. Evaluation of shear stress at locations other than at the bed requires adjustment in the values of average channel velocity. There are correction factors that can be used to adjust the mean velocity to better reflect local flow conditions (Section 8.4.1.2). An alternative approach is to use multi-dimensional models in local areas to calculate velocity magnitude and direction where excessive velocities are anticipated.

8.4.1.2 Basis of critical concepts for erosion

Analysis of the hydraulic stability of armourstone and sediments generally concerns individual stones and particles. By comparison, geotechnical stability analysis discussed elsewhere in Section 8.6 always concerns material in bulk. Movements of stones and sediment due to current and/or wave action are observed as 'displacements' of individual particles or as 'scour' holes when the bed consists of sand, small stones or gravel. This shows that the relative magnitudes of the movements of coarse and fine particles are of different order. Displacements of individual stones are of the order of several times the stone diameter, while scour depths/lengths in sediments are at least several orders of magnitude of the grain size.

Conventional design methods aim to prevent the initial movement of coarse and fine particles by defining 'threshold' conditions. These conditions are expressed in terms of critical values for shear stress, velocity, wave height, or discharge.

There is usually considerable experimental scatter around the point of initial movement, eg the critical shear stress parameter, ψ_{cr}, or the critical velocity, V_{cr}. The designer can take advantage of a probabilistic approach as described in CIRIA; CUR; CETMEF (2007) to account for uncertainties. In addition to the uncertainty in resistance or strength, certain damage may be accepted. This implies that some movement is allowed, but only up to predefined levels of displacement or scour. These threshold levels may be defined, for example, as the:

- maximum amount of displaced stones or concrete units (per unit time and area)
- critical scour depth
- maximum transport of material.

The concept of allowing some damage below a certain limit is the most common concept for the design of protective measures consisting of armourstone or structures armoured with concrete armour units.

The exceedance of the threshold conditions previously highlighted, leads to instability of loose materials. Waves, current velocities and differences in water levels, all acting through shear stresses, can be regarded as the principal hydraulic loadings. The principal stabilising or resistance forces are gravity and cohesion. Cohesion is only relevant to sediments in the clay and silt range ($D < 5\ \mu m$ and $D < 50\ \mu m$, respectively) or fine sand ($D < 250\ \mu m$) with appreciable silt content. In this regard it is convenient to classify material of erodible layers or subsoil as either:

- cohesive sediments — **silt**, $D < 50$ mm and **clay**, $D < 5\ \mu m$
- non-cohesive, fine sediment — **sand**, $50\ \mu m < D < 2$ mm
- non-cohesive, coarse sediment — **gravel**, $D > 2$ mm and **stone**, $D > 50$ mm

Box 8.11 contains information relating sediment material classification and material classification used in geotechnical engineering.

Box 8.11 ***Sediment classification***

> Sediment material characteristics relative to erosion and sediment mobility are defined differently than are soil material properties used in geotechnical soil classifications.
>
> Sediment particle sizes for sediment mobility as stated in this manual refer to European designations. Sediment size classes in the US differ and can be found in Vanoni (1975), (Lane *et al*, 1947).

The structural response of particles can be practically described with one or more of the following hydraulic loading variables and parameters:

- specific discharge, q, across a structure ($m^3/s/m$)
- shear stress, τ (N/m^2), or non-dimensional, Shields parameter, ψ (-), or shear velocity, u^* (m/s)
- velocity, either depth-averaged, V, or local, u (m/s)
- water level, h, or head H or H–h (m).

The most prominent strength or resistance variables with regard to stability are:

- particle size, D (m) or nominal diameter, D_n (m) or mass, M (kg)
- relative buoyant density, $\Delta = (\rho_s - \rho_w)/\rho_w$, where ρ_s is the apparent mass density of the solid particle (kg/m^3) and ρ_w is the mass density of water (kg/m^3)
- mitigating factors that may bind individual particles together include inter particle cohesion or density of any grass root mass (kg/m^3).

Two basic concepts or methods exist to evaluate the hydraulic stability of a rock structure:

- the critical shear concept
- the critical velocity concept.

In practice, from these two methods other criteria can be derived in terms of mobility or stability numbers, Table 8.9.

Table 8.9 ***Stability concepts and the relation with structure types and stability formulae for design***

Stability concept	Stability parameter	Structure type
Shear stress	ψ_{cr} (Shields parameter)	Bed and bank protection Spillways and outlets
Velocity	$U^2/(2g\Delta D)$ (Izbash number)	Bed and bank protection Near-bed structures Toe and scour protection
Discharge	$q/(g(\Delta D)^3)^{1/2}$	Sills Weirs (eg levee embankment)
Wave height	$H/(\Delta D)$ (stability number)	Rock armour layers Concrete armour layers Toe and scour protection
Hydraulic head	$H/(\Delta D)$	Sills Weirs (eg levee embankment)

The use of a velocity stability concept, although it is the simplest and most straightforward, may become difficult when a representative velocity has to be determined. It is often a local value that is required and not the depth-averaged value.

The bed shear stress concept incorporates the basic grain mechanics and so is most generally applicable. However, the vertical velocity profile has to be known first, and subsequently a reliable transfer should be performed from this velocity profile into shear stress. Some approaches are not purely based on grain mechanics, but rather on model tests and dimensional analysis.

In the cases of movement and erosion resistance of sediments under current attack, the method of critical shear stress and the method of permissible or critical velocity are most frequently used.

Critical velocity concept

According to the permissible velocity method, initiation of motion of material occurs when the critical or permissible velocity is exceeded. Selection of the proper velocity is essential to guarantee reliable application of these criteria. Usually, the depth-averaged flow velocity, V (m/s), is used and various corrective factors are added to adjust for local velocity conditions. Table 8.10 presents typical critical velocities, V (m/s) for non-cohesive materials where water depth, h, is 1.0 m. Critical velocities for water depths ranging from h = 0.3 to 3.0 m can be obtained by multiplying the values in Table 8.10 by the factors, K_1, given in Table 8.11.

To prevent damage due to erosion, calculated flow velocities have to be less than those given by this method. In addition to the ultimate hydraulic loading case, velocities for multiple flow conditions should be checked to verify that critical thresholds are not exceeded.

Table 8.10 Critical depth-averaged velocities, V, for loose granular material in water depth of 1 m

Material	Sieve size, *D* (mm)	Critical velocity *V* (m/s) for *h* = 1 m
Very coarse gravel	200–150	3.9–3.3
	150–100	3.3–2.7
Coarse gravel	100–75	2.7–2.4
	75–50	2.4–1.9
	50–25	1.9–1.4
	25–15	1.4–1.2
	15–10	1.2–1.0
	10–5	1.0–0.8
Gravel	5–2	0.8–0.6
Coarse sand	2–0.5	0.6–0.4
Fine sand	0.5–0.1	0.4–0.25
Very fine sand	0.1–0.02	0.25–0.20
Silt	0.02–0.002	0.20–0.15

Table 8.11 Velocity correction factors, K_I, for water depths ($h \neq 1.0$ m) in the range of 0.3 m < h < 3 m

Depth, *h* (m)	0.3	0.6	1.0	1.5	2.0	2.5	3.0
Ki (-)	0.8	0.9	1.0	1.1	1.15	1.2	1.25

Critical shear concept

The critical shear concept for unidirectional flow is based on the Shields criterion (Shields, 1936). The criterion expresses the critical value of the ratio of the de-stabilising fluid forces to the stabilising forces that act on a particle. The forces that tend to move the particle are related to the maximum shear stress exerted on the bed by the moving fluid, so the stabilising forces are related to the submerged weight of the particle. When the ratio of the two forces, represented by the Shields parameter, ψ, exceeds a critical value, ψ_{cr}, movement initiates. The Shields criterion for steady uniform flow is expressed in Equations 8.86 and 8.87. The Shields curve is given in Figure 8.46.

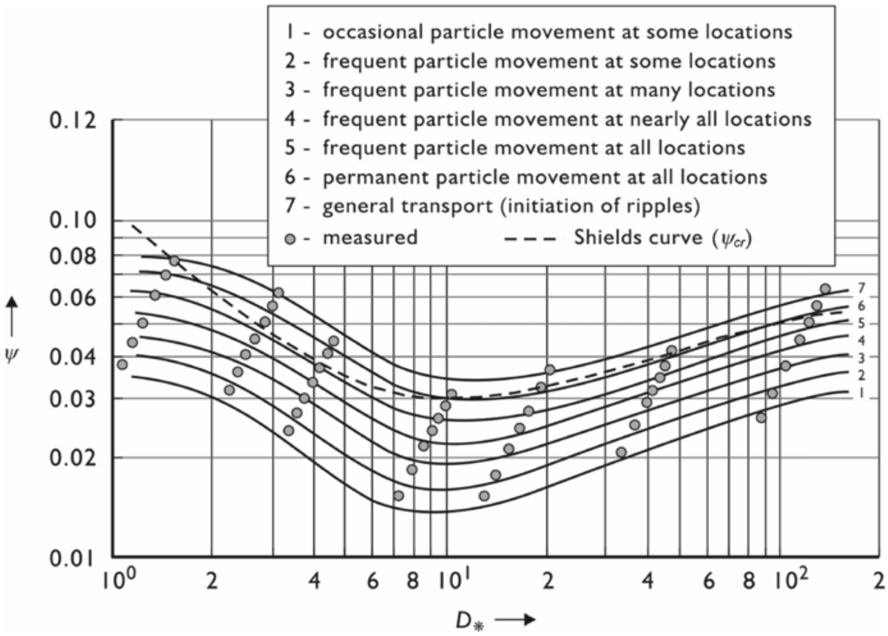


***Figure 8.46** The modified Shields diagram for steady flow (CIRIA; CUR; CETMEF, 2007)*

Equation 8.86 gives the Shields parameter, ψ_{Cr}, as a function of the critical value of the shear velocity, $u*_{cr}$ (m/s):

$$\psi_{cr} = \frac{\tau_{cr}}{(\rho_s - \rho_w)\; g\, D} = \frac{u_{*cr}^2}{\triangle g D} = f\,(Re) \tag{8.86}$$

Equation 8.87 gives the Shields parameter as a function of the depth-averaged critical velocity, V_{cr} (m/s):

$$\psi_{cr} = \frac{1}{C^2}\frac{V_{cr}}{\triangle D} \tag{8.87}$$

where:

τ_{cr}	=	$\rho_w g V_{Cr}/C^2$, critical value of bed shear stress induced by the fluid at which particles first begin to move (N/m^2)
ρ_s	=	apparent mass density of the particles (kg/m^3)
ρ_w	=	mass density of water (kg/m^3)
D	=	sieve size of material (m). The median size, D_{50}, is often as a characteristic value
$D*$	=	$D_{50}(g\Delta/\nu^2)^{1/3}$, non-dimensional grain size (-)
u^*_{cr}	=	$(\tau/\rho_w)^{1/2}$, critical value of the shear velocity (m/s)
ν	=	kinematic fluid viscosity (m^2/s)
C	=	Chézy friction coefficient (m$^{1/2}$/s)
Re^*	=	u^*D/ν, Reynolds number, based on shear velocity (-)
Δ	=	relative buoyant density of the particles (-)

Following are approximate values of ψ , associated with state of particle mobility as indicated:

- ψ = 0.03 for initiation of movement
- ψ = 0.05 for limited movement
- ψ = 0.10 for general movement/transport.

For fluvial conditions, the average shear stress on the channel boundary across the entire cross-section of the river is calculated with Equation 8.88:

$$\tau = \gamma\, K_b\, R\, S_f \tag{8.88}$$

where:

K_b = bend coefficient (-)

γ = unit weight of water (kN/m³)

R = hydraulic radius of river (area divided by wetted perimeter) (m)

S_f = slope of energy grade line (m/m)

Figure 8.47 presents a plot of critical shear stress as a function of mean grain size of particles. This diagram shows that the most erodible material is fine sands with a mean grain size range of 0.1 to 0.5 mm. It also shows that for fine grain size material with cohesion (silt, clay) erosion threshold does not correlate with mean particle size.

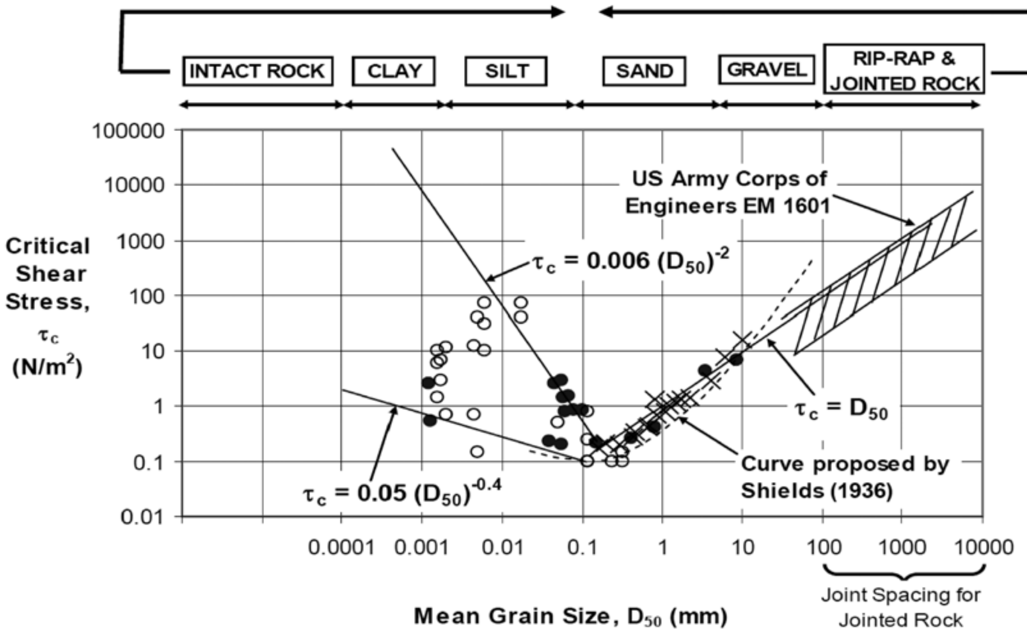


Figure 8.47 ***Critical shear stress vs. particle grain size (Briaud et al, 2001)***

Both critical velocity and critical shear methods may use the depth-average velocity. This approach can be expanded to adjust for localised conditions if appropriate factors are included. These factors may be in the form of load amplification factors or strength reduction factors as shown in Table 8.12. A summary of equations used to calculate these factors is provided in Table 8.13. Further details of adjustment factors can be found in various literature eg CIRIA; CUR; CETMEF (2007).

Table 8.12 ***Amplification and reduction factors to adjust depth-averaged velocity***

Loading	**Factor type**	**Factor**	**Multiply with:**
Additional waves	Amplification	$k_w (\geq 1$, limited to $\tau_w < 2.5\ \tau_c)$	u^2, ψ, τ, q^2, H
Excessive turbulence	Amplification	$k_t (\geq 1)$	$u, \psi^{1/2}, \sqrt{\psi}, \tau^{1/2} q, H^{1/2}$
Depth or velocity profile (logarithmic distribution)	Amplification	Λ_{hf}	V
Slope	Reduction	$k_{sl} (\geq 1)$	V

Table 8.13 Amplification and reduction factor formulae

Factor	Equation	Parameters
Wave amplification	$k_w = 1 + \frac{1}{2} f_w \frac{C^2}{2\,g} \left(\frac{u_0}{V}\right)^2$	f_w is the rough bed friction factor • $f_w = 0.3$ for $a_0/z_0 \leq 19.1$ • $f_w = 1.39(a_0/z_0)^{-0.52}$ for $a_0 > 19.1z_0$. C is the Chézy coefficient u_0 is peak orbital velocity near the bed (m/s²)
Turbulence	$k_t = \frac{1+3r}{1.3}$	r is the turbulence factor as described in Section 7.3.7.5
Depth or velocity profile	$\Lambda_h = \frac{1}{f_c} = \frac{C^2}{2\,g} = \frac{18^2}{2\,g} \log^2 \left(\frac{12\,h}{k_s}\right)$	k_s is the bed roughness: • $k_s = 2(D_{90})$ or $\approx 4(D_{50})$ for sediments and gravel • k_s for armourstone depends on the situation • f_c is friction factor for currents.
Slope	$k_{sl} = \frac{\cos\psi\,\sin\beta + \sqrt{\cos^2\beta\,\tan^2\phi - \sin^2\psi\,\sin^2\beta}}{\tan\phi}$	ψ = angle made by flow to upslope direction (deg) β = angle of the sloping embankment with the horizontal (deg) ϕ = angle of repose of material

Combining the adjustment factors with the Shields parameter yields:

$$\frac{V^2}{2\,g} = k_{sl}\,k_t^{-2}\,k_w^{-1}\,\Lambda_h\,\psi_{cr}\,\triangle\,D \tag{8.89}$$

In Equation 8.89 ψ_{cr} can be used as a damage parameter with:

- $0.03 < \psi_{cr} < 0.035$ representing no damage or movement
- $0.05 < \psi_{cr} < 0.055$ representing some movement.

A variety of stability formulae can be derived from these concepts for special applications such as riverbanks. An example of stability criterion for stones is given in Box 8.12.

Box 8.12 Velocity-type stability criterion for stones on a sill

The well-known example of a velocity-type stability criterion was presented by Izbash and Khaldre (1970). Their empirically-derived formulae for exposed and embedded stones *on a sill* are given by:

Exposed stones:

$$\boxed{\frac{v_b^2}{2\,g} = 0.7\,\triangle\,D_{50}} \tag{8.90}$$

Embedded stones:

$$\boxed{\frac{v_b^2}{2\,g} = 1.4\,\triangle\,D_{50}} \tag{8.91}$$

where D_{50} is the median sieve size (m).

Range of validity: these equations, as developed by Izbash and Khaldre (1970), are valid for relative water depths, *h/D*, in the range of *h/D* = 5 to 10.

Note that Izbash and Khaldre (1970) defined v_b as the critical velocity for stone movement (m/s), which can be interpreted as the velocity near the stones and not as the depth-averaged flow velocity, *V* (m/s).

Cohesive soils

In the hydraulic resistance (erodibility) of cohesive sediments, the physical-chemical interaction between the particles plays a significant role. So, the determination of critical velocities relies heavily on empirical data based on various experiments and *in situ* observations. The existing knowledge of the correlation of the Shields factor and/or the critical flow velocity U_{cr} with mechanical properties of the soil (silt content, plasticity index, shear stress etc) is still not sufficient to allow for a general approach. Cohesive materials such as clay generally have higher resistance to erosion than non-cohesive material. As an indication, the following values of critical velocities may be used:

- fairly compacted clay (e = 0.50) U_{cr} = 0.8 m/s
- stiff clay (e = 0.25) U_{cr} = 1.5 m/s.

While it is accepted that there is uncertainty in predicting the erosion of a soil because of the range of factors that can affect the state and the erodibility of a soil, as well as uncertainty in the performance of protection layers such as grass cover, methods are available to estimate performance as follows.

More detailed discussion of soil erodibility can be found in Section 8.10 as part of the discussion of breach processes.

8.4.2 Resistance of grass systems to external erosion

The potential for slopes to erode and scour can be determined by calculating current velocities and boundary shear stresses as outlined in Sections 8.4.1.1 and 8.4.1.2 and comparing these values to allowable limits for the materials. Soil movement (erosion) can be expected if calculated values exceed allowable limits for the embankment material. Once it has been determined that erosion and/or scour is a concern for levee safety, it is necessary to consider measures that can reduce or mitigate their resulting effects. Of these protection using grass systems is always worth considering as an option.

While there has been a lot of research into the effects of vegetation and grass on flow within channels, the degree of guidance available on the performance of grass cover for levees during overflow or wave overtopping conditions is more limited. Guidance divides into grass performance under overflow conditions (often misquoted as overtopping) and performance under wave overtopping conditions. A review of current research and guidance for both can be found in Morris *et al* (2012a).

Research and guidance often originates back to three sources:

- in the USA research by USDA at Stillwater, Oklahoma
- in the UK publications from CIRIA
- in the Netherlands ongoing research into grass performance on dikes during wave overtopping.

There are notable differences in approaches from each of these sources (Temple *et al*, 1987, Temple, 1997, Temple and Hanson, 1994, and Hanson and Temple, 2002). US guidance looks at the combination grass type and soil resistance to erosion, while UK guidance looks only at grass condition. Dutch guidance focuses upon wave overtopping, but applied to the performance of Dutch dikes, which are normally constructed from a grass covered clay layer sitting over a sand core. Performance analysis for the outer layer should be generically applicable though.

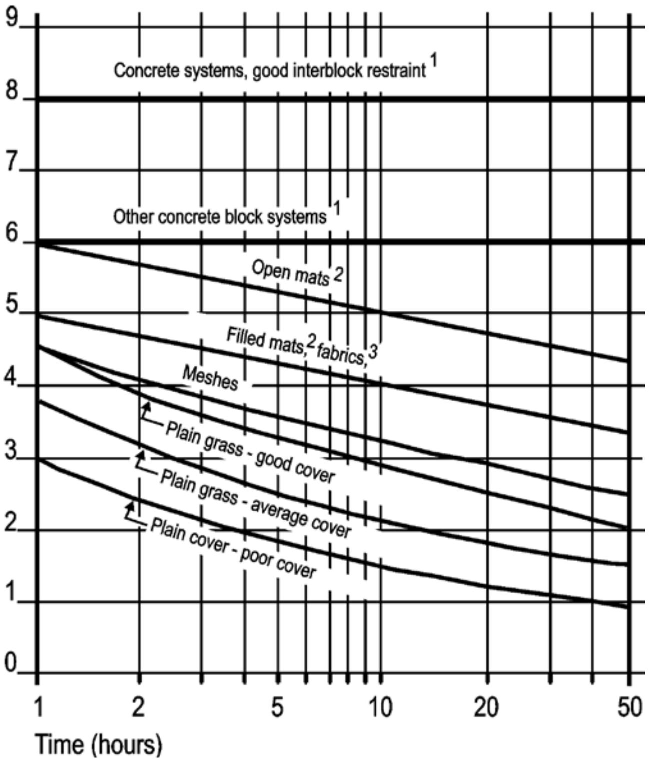
8.4.2.1 Grass resistance under overflow conditions

Existing guidance relates back to two sources:

1. In Europe, guidance often relates or refers to work by CIRIA during the 1970/1980s, drawing from Whitehead (1976) or Hewlett *et al* (1987).
2. In the US, guidance typically builds from Temple *et al* (1987).

CIRIA guidance provides design curves, which suggest acceptable limits for combinations of flow velocity and duration. The US approach estimates shear stress at the soil surface (as a function of vegetation type and impact) followed by acceptability in relation to the soil erodibility.

Figure 8.48 may be used as a first guide in designing the appropriate measure to protect embankments. Detailed calculations should follow use of Figure 8.49 to confirm adequate performance of the selected measure under site specific conditions. If Figure 8.48 indicates that more substantial revetment systems (such as mats and concrete blocks) are required, the more detailed guidance in Sections 8.4.4 should be followed. Where proprietary measures are to be used, manufacturer guidelines should be followed. Due to the inexact nature of defining erosion and scour and significant variation in various design formulae, it is advisable to use several methods to calculate a range of possible requirements.



Notes

1 Minimum superficial mass 135 kg/m^3.

2 Minimum nominal thickness 20mm.

3 Installed within 20 mm of soil surface, or in conjunction with a surface mesh.

4 These graphs should only be used for erosion resistance to unidirectional flow. Values are based on available experience and information contained in Hewlett *et al* (1987).

5 All reinforced grass values assume well-established, good grass cover.

6 Other criteria (such as short-term protection, ease of installation and management, and susceptibility to vandalism) have to be considered in choice of reinforcement.

Figure 8.48 ***Recommended limiting design values for erosion resistance of select erosion counter measures (Hewlett et al, 1987)***

The design curves in Figure 8.48 appear to contain a factor of safety as compared to the performance curves presented in Whitehead *et al* (1976) (Figure 8.49). So, while these may be appropriate for use in design, the earlier curves (shown as dashed lines in Figure 8.49) should be used when undertaking a levee performance assessment.

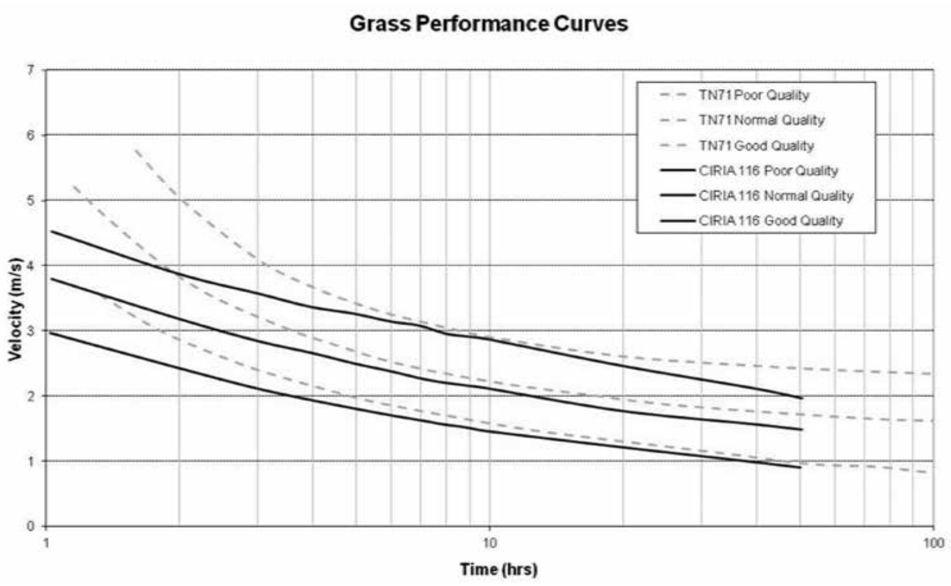


Figure 8.49 ***Comparison between R116 grass performance curves (Hewlett et al, 1987) and the original field test data (Whitehead et al, 1976)***

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8.4.2.2 Grass resistance under wave overtopping conditions

Early approaches to assessing wave overtopping resistance by grass cover used the same CIRIA performance curves but with an averaged rate of overflow arising from the periodic wave overtopping. This ignored the surges in flow that arise from wave action and might be assumed to under-predict the impact of wave action.

Recent (ie post 2000) and ongoing Dutch studies using a wave overtopping simulator (Figure 8.50) are allowing guidance on performance under wave overtopping conditions to be developed. Dutch dikes typically comprise a grass covered clay soil layer, covering an inner sand core (Figure 8.51). The analysis of grass performance relates to the grass cover, turf and top layer only. The top layer may be up to 0.2 m thick, including the turf that may be 0.05 m thick.

Figure 8.50 Recent Dutch studies into grass performance under wave overtopping, using the wave overtopping simulator (Morris et al, 2012a)

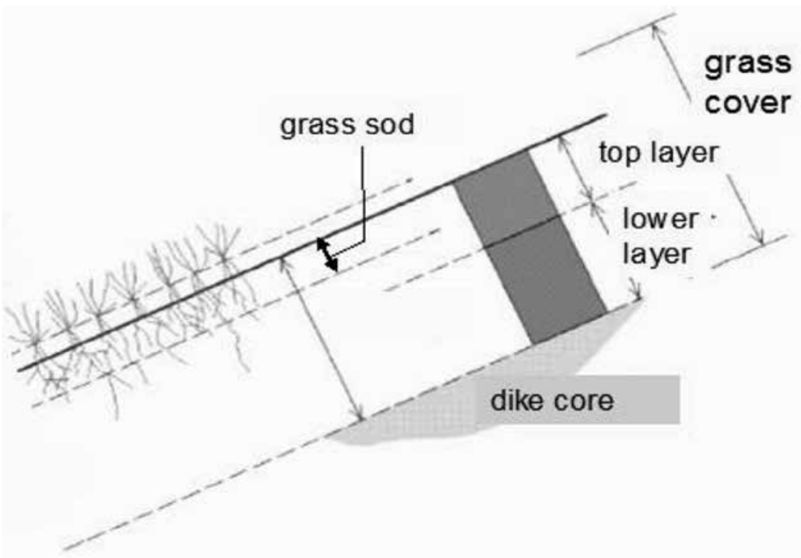
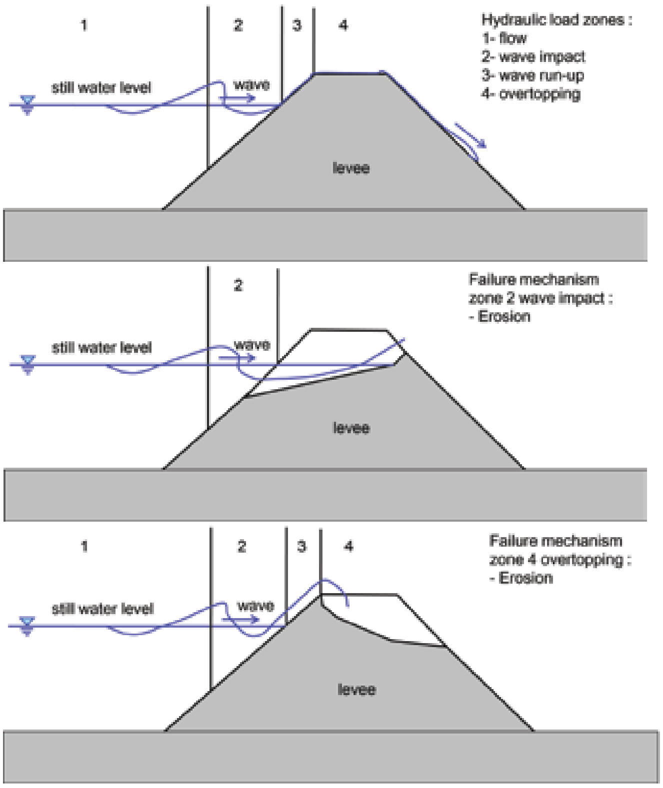


Figure 8.51 Recent Dutch studies into grass performance under wave overtopping, using the wave overtopping simulator (from TAW, 1997 and Rijkswaterstaat)

Three conditions of grass strength are described, being closed turf, open turf and fragmented turf. Damaged patches of less than 0.15 m square are not considered to significantly affect the performance of closed or open turf under wave overtopping. Fragmented turf is considered to offer little protection against erosion.

Four hydraulic load zones are identified (Figure 8.52) and failure mechanisms considered for zones 2 and 4.



Note
The SBW is the overall research project of the Rijkswaterstaat (Morris *et al*, 2012a). Within the project, destructive testing has been undertaken using the Wave Overtopping Simulator on real dikes to give preliminary conclusions on strength of grassed inner slopes of dikes against wave overtopping.

Figure 8.52 Hydraulic load zones (1 to 4) and failure mechanisms addressed in the SBW research program (Morris et al, 2012a)

The failure model suggested for erosion in the wave impact zone (2) compares the wave impact load time t_i (hour) with the wave impact resisting time t_r (hour) for different wave height H_s (m) as given in Figure 8.53. The turf is sufficiently strong if $t_r > t_i$. The model does allow some minor damage to occur to the turf.

Model limitations concerning the slope angle are 1H:2.5V (or less steep) for $H_s \geq 0.5$ m and 1V:1.5H (or less steep) for $H_s < 0.5$ m. For a slope angle gentler than 1V:4H the resisting time t_r will increase, however, the model has no prediction capability on how much t_r will increase.

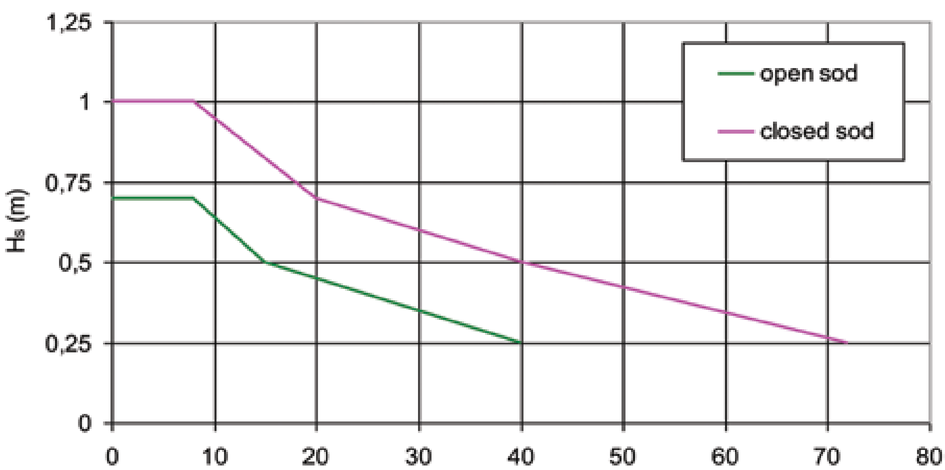


Figure 8.53 Wave impact resisting time t_r (hour) for different wave height H_s (m) and turf quality (open or closed) (Morris et al, 2012a)

No research within the SBW framework (Morris *et al*, 2012a) was aimed at erosion in the wave run-up zone (zone 3 in Figure 8.52), however, if the turf present in the wave impact zone (zone 2 in Figure 8.52) is sufficient, the turf in the run-up zone will also be sufficient. Pressure gradients in the turf and subsoil, causing erosion, are significantly larger in the wave impact zone than in the wave run-up zone. Grass cover will fail in the wave impact zone before it fails in the wave run-up zone if the grass cover is of equal quality in both zones.

The hydraulic load for erosion of the grass cover in the wave overtopping zone (crest and landward slope of the dike, zone 4 in Figure 8.52) consists of the overtopping wave volumes. Each of the overtopping volumes can be characterised by the maximum depth averaged flow velocity and maximum water layer thickness. As shown in Figure 8.54, each overtopping wave volume will result in a triangular shaped flow velocity development against time.

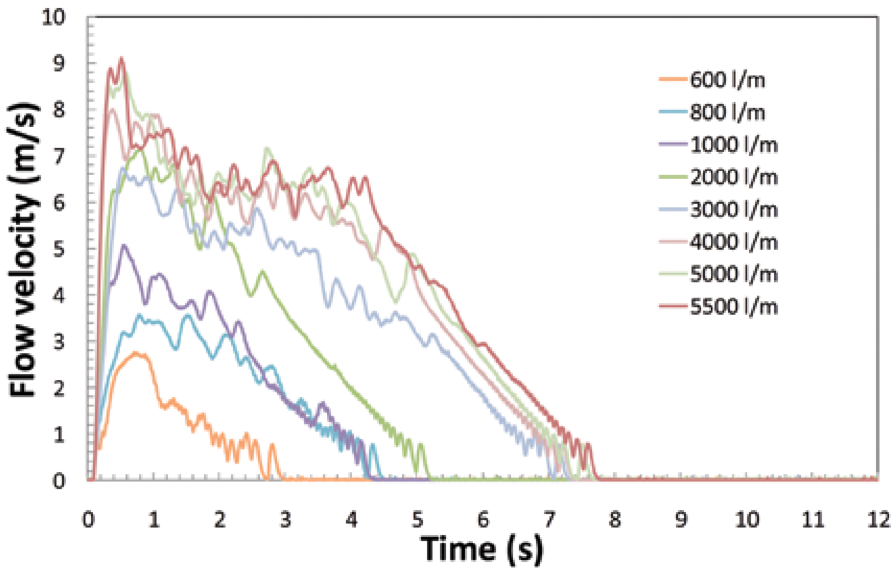


Figure 8.54 ***Velocity (m/s) against time (s) for different wave overtopping volumes (600–5500 l/m) measured at one wave-overtopping test sloped 1V:4.5H (van der Meer et al, 2010)***

The maximum depth averaged velocity U (m/s) in a wave overtopping event with a volume, V (m^3/m), can be estimated by the empirical formula $U = 5V^{0.34}$ (van der Meer *et al*, 2010). For example, a wave overtopping volume of 1000 l/m results in a maximum depth averaged velocity of 5 m/s and an overtopping volume of 5500 l/m in 8.9 m/s. Measurements at a relatively steep (1V:2.3H) and long slope showed an increase in velocity as the volume progressed down slope. Measurements at a relatively mild slope (1V:5H) showed a decreasing velocity. However, until further research gives conclusive insight in the development of the velocity depending on slope angle and slope length, the above estimate of the correlation between V and U is used for slopes of 1V:2.3H and more gentle. For steeper slopes the model presented in the following paragraphs is advised. Research to determine the velocity as a function of slope angle and slope length is in progress.

For steeper slopes, the distribution of wave overtopping volumes during a storm can be calculated using the formulae in Pullen *et al* (2007). Parameters involved are the storm duration and the average wave period, which determine the number of waves reaching the dike. The water level, slope geometry and roughness, wave height and period, determine the number of waves that reach the crest, and overtop, and the average overtopping discharge q (l/s per m).

In engineering practice the wave overtopping load is often described by the average wave overtopping discharge only. However, it is important to include consideration of the wave height as part of the erosion load. It is not enough to just use the average overtopping discharge when describing wave overtopping.

A fragmented turf does not have any strength that can be relied upon. If there is any significant wave overtopping to be expected (ie more than 0.1 l/s per m) a fragmented turf is not recommended. So, if the risk of undermining can be excluded, and if a closed turf is present, the cumulative overload model is suggested (van der Meer *et al*, 2010):

$$\sum_{n=1}^{N_{ov}} \left(U_n^2 - U_c^2\right) < C \tag{8.92}$$

With $U_n \geq U_c$, where:

N_{ov} = number of overtopping waves

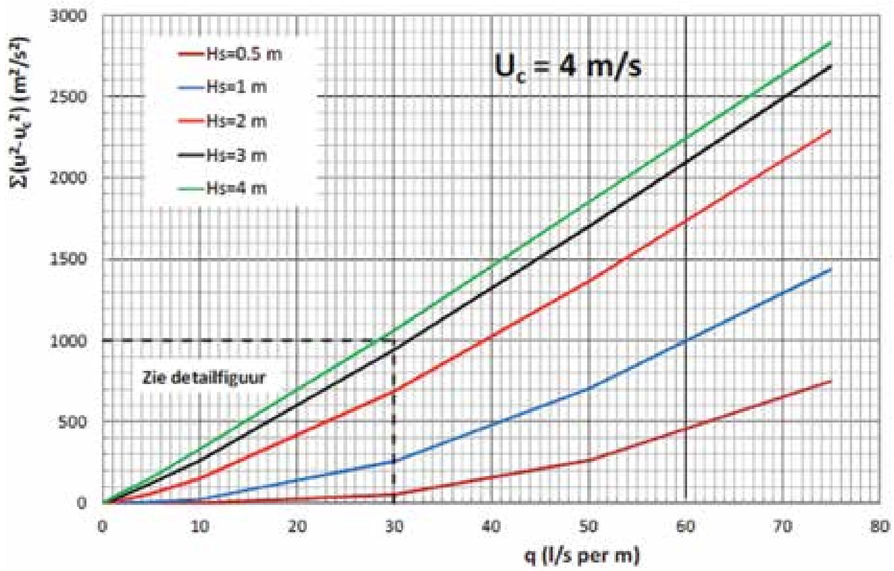
U = maximum depth averaged flow velocity from an overtopping wave (m/s), for cases where $U > UC$

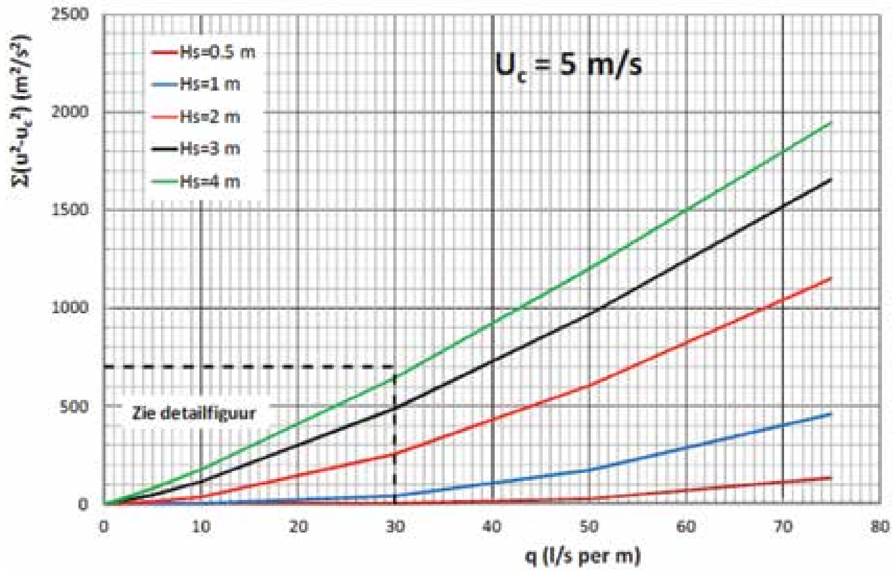
U_c = critical maximum depth averaged flow velocity depending on the top layer strength (m/s)

C = critical value (m²/s²) where:

- C = 500 (m²/s²) resembles a situation where initial damage occurs. A large scatter in the initial damage value is however observed
- C = 1000 (m²/s²) multiple spots with initial damage (not yet failure of the top layer)
- C = 3500 (m²/s²) failure of the top layer.

The cumulative overload depends mainly on U_c, the storm duration, and the combination of the average overtopping discharge and the wave height, H_s. From the wave overtopping tests, critical velocities were back-calculated and showed a range from U_c = 4 m/s (critical volume 500 l/m) up to 6.3 m/s (critical volume 2000 l/m), excluding tests with fragmented turf. The cumulative overload can be compressed in the graphs in Figure 8.55. The graph gives (on the vertical axis) the cumulative overload for a one hour storm condition (Figure 8.55).





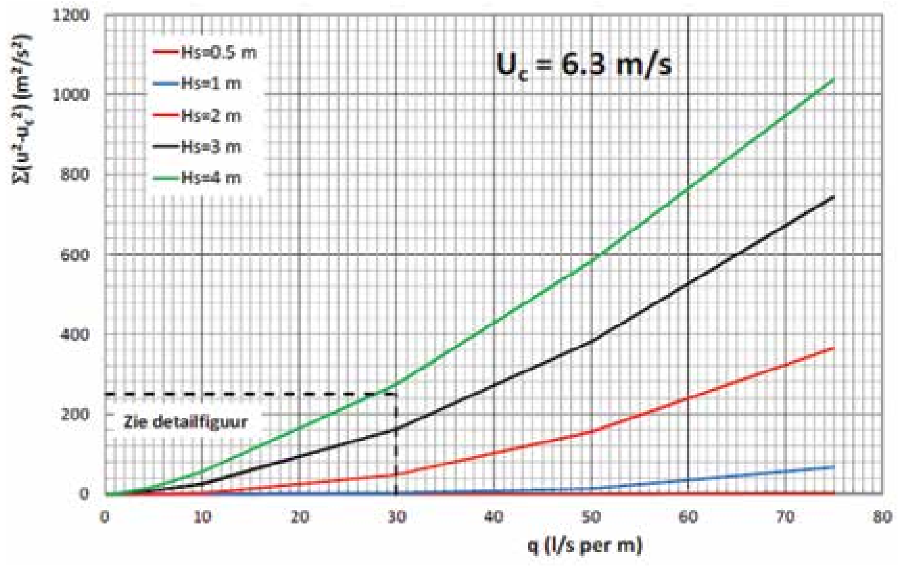


Figure 8.55 *Cumulative overload (m^2/s^2) as a function of U_c (m/s), H_s (m) and q (l/s per m) for a one hour storm condition (Morris et al, 2012a)*

The research within the SBW framework (Morris *et al*, 2012a) has not yet led to a reliable relation between U_c and field parameters. Based on the test results a value of U_c = 4 m/s and C = 1000 m^2/s^2 is advised for closed turf, and excluding cases too far beyond the range of wave overtopping tests, the most important being the slope angle of 1, 2, 3. For a closed turf it is likely that the critical velocity will be larger than 4 m/s, however, the research to predict U_c is still work in progress.

Example

For example, consider wave overtopping that lasts six hours, with Hs = 2 m. There are two hours of q = 10 l/s per m (water level rise and fall) and four hours of q = 25 l/s per m at the peak water level. Using the graph with U_c= 4 m/s for a closed turf, shows a cumulative overload of 150 m^2/s^2 per hour for 10 l/s per m and Hs = 2 m and 550 m^2/s^2 per hour for 25 l/s per m and Hs = 2 m. The total cumulative overload during the storm event will be 2 × 150 + 4 × 550 = 2500 m^2/s^2. This is larger than 1000 m^2/s^2, so the suggested criterion is not met.

If, in the same case U_c = 5 m/s than the graph shows a cumulative overload of 25 m^2/s^2 per hour for 10 l/s per m and Hs = 2 m and 200 m^2/s^2 per hour for 25 l/s per m and Hs = 2 m. The total cumulative overload during the storm event will be 2 × 25 + 4 × 200 = 950 m^2/s^2. This is smaller than 1000 m^2/s^2, so the suggested criterion is met.

8.4.3 Resistance of other protection systems to erosion due to currents

A large number of stability formulae for armourstone under current attack have been suggested by various authors, which tend to give quite different results in terms of the required stone size. CIRIA; CUR; CETMEF (2007) presents three methods selected from the range of formulae available in the literature. The three formulae addressed in CIRIA; CUR; CETMEF (2007) have been used extensively for current attack. Synopses of the methods follow.

Pilarczyk (1995) combined various design formulae to present a unified relationship between required armourstone size for stability and the hydraulic and structural parameters. Special factors and coefficients were added to the Shields (1936) formulations to derive Equation 8.93. Guidance related to parameters in the equation is presented in Table 8.14.

$$D = \frac{\phi_{sc}}{\triangle} \frac{0.035}{\psi_{cr}} k_h \, k_{sl}^{-1} \, k_t^2 \, \frac{U^2}{2\,g} \qquad (8.93)$$

where:

D = characteristic size of the protection element (m), $D = D_{n50}$ for armourstone
ϕ_{sc} = stability correction factor (-)
Δ = relative buoyant density of the protection element (-)
ψ_{Cr} = critical mobility parameter of the protection element (-)
k_t = turbulence factor (-)
k_h = velocity profile factor (-)
k_{s1} = side slope factor (-)
U = depth averaged flow velocity (m/s)

Table 8.14 Design guidance for parameters in the Pilarczyk design formula

Characteristic size, *D*	Armourstone and rip-rap: Box gabions and gabion mattresses: Note that the armourstone size is also determined by the need to have at least two layers of armourstone inside the gabion.	$D = D_{n50} \cong 0.84\ D_{50}$ (m) D = thickness of element (m)
Relative buoyant density, Δ	Rip-rap and armourstone: Box gabions and gabion mattresses: where n_v = layer porosity Δ 0.4 (-), ρ_r = apparent mass density of rock (kg/m^3) and ρ_w = mass density of water (kg/m^3)	$\Delta = \rho_r/\rho_w - 1$ $\Delta = (1-n_v)(\rho_r/\rho_w - 1)$
Mobility parameter, ψ_{cr}	Rip-rap and armourstone: Box gabions and gabion mattresses: Rock fill in gabions:	$\psi_{cr} = 0.035$ $\psi_{cr} = 0.070$ $\psi_{cr} < 0.100$
Stability factor, φ_{sc}	Exposed edges of gabions/stone mattresses: Exposed edges of rip-rap and armourstone: Continuous rock protection: Interlocked blocks and cabled blockmats:	$\varphi_{sc} = 1.0$ $\varphi_{sc} = 1.5$ $\varphi_{sc} = 0.75$ $\varphi_{sc} = 0.5$
Turbulence factor, k_t	Normal turbulence level: Non-uniform flow, increased turbulence in outer bends: Non-uniform flow, sharp outer bends: Non-uniform flow, special cases:	$k_t^2 = 1.0$ $k_t^2 = 1.5$ $k_t^2 = 2.0$ $k_t^2 > 2.0$
Velocity profile factor, k_H	*Fully developed logarithmic velocity profile:* where h = water depth (m) and k_s = roughness height (m), k_s = 1 to $3D_n$ for rip-rap and armourstone, for shallow rough flow ($h/D_n < 5$), $k_h \approx 1$ can be applied *Not fully developed velocity profile:*	 $k_h = (1 + h/D_n)^{-0.2}$
Side slope factor, k_{sl}	The side slope factor is defined as the product of two terms, a side slope term, k_d, and a longitudinal slope term, k_l: where $k_d = (1-(\sin^2\alpha/\sin^2\varphi))^{0.5}$ and $k_l = \sin(\varphi-\beta)$, α is the side slope angle (°), φ is the angle of repose of the armourstone (°) and β is the slope angle in the longitudinal direction (°)	$k_{sl} = k_{dkl}$

Escarameia and May (1992) provide an equation that is a form of the Izbash equation. The Escarameia and May formulation (Equation 8.94) includes effects of turbulence and can be particularly useful in situations where turbulence levels are higher than normal (near river training structures, at bridge piers, downstream of hydraulic structures such as gates, weirs, spillways and culverts). Guidance for parameters used in Equation 8.94 is presented in Table 8.15 and Table 8.16.

$$D_{n50} = c_T \frac{u_b^2}{2g\Delta} \tag{8.94}$$

where C_T is the turbulence coefficient (-) and u_b is the near-bed velocity, defined at 10 per cent of the water depth above the bed (m/s).

Table 8.15 Design guidance for parameters in Escarameia and May formula

Median nominal diameter, D_{n50}	Armourstone: Gabion mattresses: Note that equations were developed from results of tests on gabion mattresses with a thickness of 300 mm	$D_{n50} = (M_{50}/\rho_r)^{1/3}$ (m) D_{n50} = stone size within gabion
Turbulence coefficient, c_r	Armourstone: (valid for r ≥ 0.05): Gabion mattresses: (valid for r ≥ 0.15): where r = turbulence intensity defined at 10% of the water depth above the bed (-), $r = u'_{rms}/u$	$c_r = 12.3\,r - 0.20$ $c_r = 12.3\,r - 1.65$
Near bed velocity, u_b	If data are not available an estimation can be made based on the depth-averaged velocity, *U* (m/s), as:	u_b = 0.74 to 0.90 U

Table 8.16 Typical turbulence levels for use in Escarameia and May formula

Situation	Turbulence level	
	Qualitative	*Turbulence intensity, r*
Straight river or channel reaches	Normal (low)	0.12
Edges of revetments in straight reaches	Normal (low)	0.20
Bridge piers, caissons and spur-dikes, and transitions	Medium to high	0.35–0.50
Downstream of hydraulic structures	Very high	0.60

Maynord (1993) developed the USACE design procedure based on an assumption that stability for rip-rap and armourstone should not be based on the threshold of movement criterion. Maynord instead based his formula on not allowing the underlying material to be exposed. As a result the layer thickness is included. Equation 8.95 gives the relationship between the characteristic stone sieve size, D_{50} (m) required to achieve stability subject to the imposed hydraulic and structural parameters. Guidance for parameters used in Equation 8.95 is given in Table 8.17.

$$D_{50} = f_g^{0.32} S_f\, C_{st}\, C_v\, C_T\, h \left(\frac{1}{\sqrt{\Delta}} \frac{V}{\sqrt{g\, h\, k_{sl}}} \right)^{2.5} \tag{8.95}$$

where:

f_g = gradation (factor = D_{85}/D_{15} (-))
S_f = safety factor (-)
C_{st} = stability coefficient (-)
C_v = velocity distribution coefficient (-)
C_T = blanket thickness coefficient (-)
h = local water depth (m)
Δ = relative buoyant density of stone (-)
V = depth-averaged flow velocity (m/s)
k_{sl} = side slope factor (-)

Maynord's blanket thickness coefficient, C_T, takes account of the increase in stability that occurs when stone is placed thicker than the minimum thickness ($1D_{100}$ or $1.5D_{50}$) for which $C_T = 1.0$.

Table 8.17 Design guidance for parameters in Maynord (1993)

Safety factor, S_f	**Minimum value**	**S_f = 1.1**
Stability coefficient, C_{st}	Angular armourstone: Rounded armourstone:	C_{st} = 0.3 C_{st} = 0.375
Velocity distribution coefficient, C_v	Straight channels, inner bends: Outer bends: where r_r = centre radius of bend (m) and B = water surface width just upstream of the bend (m) Downstream of concrete structures or at the end of dikes	C_v = 1.0 $C_v = 1.283 - 0.2 \log(r_b/B)$ C_v = 1.25
Blanket thickness coefficient, C_T	Standard design: Otherwise see Maynord (1993)	C_T = 1.0
Side slope factor, k_{sl}	$k_{sl} = {}^{-}0.67 + 1.49\cot\alpha + 0.045\cot\alpha$	

Note

The methods presented in this section are indicative methods. Other design methods can be found in CIRIA; CUR; CETMEF (2007). In view of the differing results, it is advisable in most instances to try more than one design formula for the evaluation of the required armourstone size and to use engineering judgement for the final selection.

These methods can be used to calculate the nominal size of rock required based on site hydraulic data, namely velocity. It is recommended that at least these three methods be used in selecting the size material to use. In order to achieve adequate protection armour, void spaces within the layer thickness must not be excessive and there should be good interlocking between the individual armourstones. This requires that a variety of rock sizes be included in the final placement. Once nominal stone size is determined the full gradation of the armour layer has to be specified to achieve this. Typically this involves defining a D_{15}, D_{85}, and/or D_{100} sizes. The approach to defining the required gradation varies by nation. In Europe, the gradation approach set out in BS EN13383-1:2002 should be followed. Further details can be found in CIRIA; CUR; CETMEF (2007).

These methods can also be used to size individual concrete armour blocks. Where blocks have interlocking features or external anchoring, the appropriate size should be based on manufacturer recommendations. Current recommendations for design of concrete block armour units on levee embankments is to determine sizing based on individual units without the benefit of anchoring.

8.4.4 Resistance of other protection systems to erosion due to waves

The principal requirement of an armouring system is dissipation of wave energy, and protection of the finer materials in the core. The armour has to remain stable under wave attack, and should dissipate energy over and within the voids in the armour and under layer(s), thus limiting wave run-up and overtopping, and reflections. In resisting severe wave action, armoured structures may suffer damage or failure in many different ways. The main failure modes for which functional relationships have been established may be defined as:

1. **Armour movement on the front face:** deemed to include rocking, displacement, and breakage of armour units.
2. **Armour movement on the rear face:** caused by wave overtopping.
3. **Crown wall movement:** principally sliding backwards or tilting under wave forces, horizontal and uplift.
4. **Toe erosion:** localised erosion of the foundation material at the toe of the breakwater.

Only mechanism (1) is discussed here as the others are discussed in detail in CIRIA; CUR; CETMEF (2007). Also, note that the advice of a coastal engineering specialist should be sought. The front face armour has to limit wave run-up and/or overtopping, and restrict reflections from the structure. Both of these are assisted by breaking the waves on the sloping face of the structure, and by dissipating wave energy in flow over/within rough and permeable armour layers. The seaward slope angle and crest freeboard generally have the most significant influence on the hydraulic performance. Armour porosity and permeability are particularly important in determining the potential for wave energy dissipation, with both influencing the armour stability.

The main parameters used to describe wave attack, and to calculate the principal hydraulic responses may be summarised as:

Significant wave height (inshore or offshore):	H_s, H_{si}, H_{so}
Mean or peak wave periods:	T_m, T_p
Mean or peak offshore wave length:	$L_m = gT_m^2/2\pi$, $L_p = gT_p^2/2\pi$
Mean or peak wave steepness:	$s_m = H_s/L_m$, $s_p = H_s/L_p$

The main parameters describing the structure geometry are summarised in CIRIA; CUR; CETMEF (2007). Some frequently used terms are listed here:

Nominal unit dimension:	$D_n = (M/\rho_r)^{1/3}$, generally D_{n50}
Unit mass:	M, (eg M_{50})
Material density:	ρ_p or ρ_c (usually in kg/m^3)
Depth of water:	h, or h_s at the structure
Armour crest or structure freeboard:	A_c, R_c
Front armour slope:	α or α_f

8.4.4.1 Armourstone design formulae

Simple approaches to the design of rock armour to such structures have often concentrated on extraction of individual armour units, generally termed 'damage'. The armour size required was derived from formulae using a regular wave height and value of a stability coefficient derived from model tests at a 'no damage' limit (often zero to five per cent extractions). The influences of many other parameters were ignored. Most design methods for rock or concrete armour calculate the median unit mass, M_{50}, or the nominal median stone diameter, D_{n50}, defined as: $D_{n50} = (M_{50}/\rho_r)^{1/3}$.

The two most commonly used methods are:

1 The Hudson formula, as used in USACE (2006a).

2 Van der Meer's equations, as used in CIRIA; CUR; CETMEF (2007).

In each instance, the design method is used to determine the limiting value of the armour size for given wave conditions, and structure geometry.

Hudson's formulae

Hudson developed a simple expression for the minimum armour weight required to resist a (regular) wave height, H, which may be re-written:

$$M_{50} = \rho_r\, H^3/K_D\, \cot\alpha\, \triangle \qquad (8.96)$$

where:

ρ_r, ρ_w	=	density of armour/water (kg/m^3)
Δ	=	buoyant density of rock = $(\rho_r/\rho_w)-1$
α	=	slope angle of the structure face
K_D	=	is a stability coefficient to take account of the other variables.

For wide graded rock armour, or rip-rap, values of a coefficient K_{RR} are substituted for K_D. Values of K_D were initially derived from model tests using regular waves with permeable cross-sections subject

1 2 3 4 5 6 7 8 9 10

to no overtopping. A range of wave heights and periods were studied. In each case the value of K_D corresponded to the wave condition giving the worst stability condition. Some rearrangement of the armour was expected, and values of K_D suggested for design correspond to a 'no damage' condition where up to five per cent of the armour units may be displaced.

The Hudson equation has many limitations, which include:

- potential scale effects from the tests used to generate the data
- the use of regular waves only
- no account taken of wave period or storm duration
- insufficient definition of the damage level
- the use of non-overtopped and permeable core structures only.

Before turning to other methods, however, it is convenient to consider another way of looking at Equation 8.96. The use of ($K_D$cotα) does not always best describe the effect of the slope angle, and it is often convenient to substitute a single stability number for ($K_D$cotα), and to work in terms of the nominal armour unit diameter $D_{n50} = (M_{50}/\rho_r)^{1/3}$. The Hudson equation may be re-arranged in terms of the stability number N_s:

$$N_s = \frac{H_s}{\triangle D_{n50}} = (K_D \cot\alpha)^{1/3} \tag{8.97}$$

The Hudson formula does not itself give any information on the level of damage. However, information is available in USACE (2006a) that allows the derivation of a similar equation relating a damage parameter,$N_{d\%}$, to the relative wave height. Taking $S_d = 0.8N_{d\%}$, a damage formula based on Equation 8.97 may be written:

$$\frac{H_s}{\triangle D_{n50}} = a\ (K_D \cot\alpha)^{1/3}\, S_d^b \tag{8.98}$$

where for rock armour = 0.67, b = 0.16, for Tetrapods or cubes = 0.69, b = 0.14 and S_d, design damage number = $A_e/D_{n50}{}^2$ (below for definitions and critical values).

Van der Meer's formulae

Van der Meer (1988) derived formulae to include the effects on armour size:

- of random waves
- of a wide range of core/underlayer permeabilities
- of the chosen level of damage
- and to distinguish between plunging and surging wave conditions.

For plunging waves:

$$\frac{H_s}{\triangle D_{n50}} = 6.2\, P^{0.18} \left(\frac{S_d}{\sqrt{N_z}}\right)^{0.2} \frac{1}{\sqrt{\xi_m}} \tag{8.99}$$

For surging waves:

$$\frac{H_s}{\triangle D_{n50}} = 1.0\, P^{-0.13} \left(\frac{S_d}{\sqrt{N_z}}\right)^{0.2} \sqrt{\cot\alpha}\, \xi_m^P \tag{8.100}$$

where parameters not previously defined are:

P	=	notional permeability factor, see Figure 8.56a
S_d	=	design damage number = A_e/D_{n50}, see Table 8.18
A_e	=	erosion area from profile
N_z	=	number of zero-crossing waves
ζ_m	=	Iribarren number = tan $\alpha/s_m{}^{1/2}$

s_m = wave steepness for mean period = $2\pi H_s/gT_m^2$

T_m = mean wave period

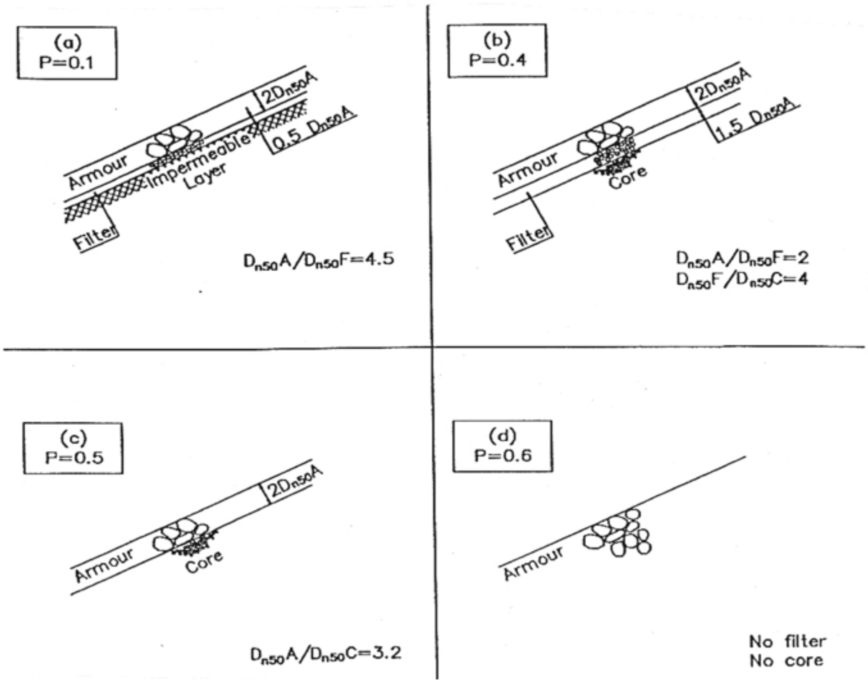


Figure 8.56 Permeability factors used in van der Meer's stability formulae for rock armour

The transition from plunging to surging waves is calculated using a critical value of $\xi_m = \xi_{mcr}$:

$$\xi_{mcr} = \left[6.2\, P^{0.31} \sqrt{\tan\alpha}\right]^{1/(P+0.5)} \quad (8.101)$$

The recommended values of the design damage number, S_d, equivalent to the number of D_{n50} sized stones extracted from a D_{n50} wide strip of the structure, are given in Table 8.18, for initial damage, intermediate damage, and failure. Failure is assumed when the filter layer is first exposed.

A range of core/underlayer configurations were used in the test programme, each with an armour layer thickness, $t_a = 2.2D_{n50}$. To each of these a value of the permeability factor, P, was assigned. In most cases for levee design the conservative value of P = 0.1 should be assumed, comparable to the value given by van der Meer (1988) for armour on an underlayer over an impermeable embankment. Other values for P are given by van der Meer (1988) for more permeable situations, but these should only be adopted after referring to detailed guidance available, for example CIRIA; CUR; CETMEF (2007).

Table 8.18 Suggested levels of damage, S_d, of armourstone protection systems

Slope	Damage, S_d		
	Initial	*Intermediate*	*Failure*
1:1.5	2	–	8
1:2	2	5	8
1:3	2	8	12
1:4–6	3	8	17

8.4.4.2 Design formulae for other revetment systems, slabs and blocks

Alternative forms of armouring for slopes shallower than 1:2 use concrete slabs, concrete blocks, pitched stone grouted by bitumen or concrete, or asphaltic materials. The stability of this type of armouring requires that the net uplift pressures acting across the concrete are balanced by the net weight force.

Simple stability formulae have been suggested for the preliminary analysis or design of blockwork or stone pitching on revetment slopes. The formula may be used to determine the block thickness, t_a:

$$\frac{H_s}{\Delta t_a} = \frac{S_b}{\xi_{op}^{0.67}} \tag{8.102}$$

where:

ζ_{op} = $\tan\alpha / s_{op}^{1/2}$
s_{op} = H_s/L_{op}

Ranges of values of the stability parameter S_b for different block types and underlayer materials are given in Table 8.19.

Table 8.19 ***Values of block stability parameter, S_b for different block and underlayer materials***

Block type	Underlayer	$S_{b,min}$	$S_{b,max}$
Loose	Granular	2.6	5.6
Loose	Geotex. + sand	3.7	8.0
Loose	Clay	5.1	11.0
Linked	Granular	3.7	8.0
Linked	Geotex. + sand	5.1	11.0

Using the highest value of S_b will give the slab thickness beyond which the structure will be unstable. Using the lowest value of S_b will give the slab thickness that will be stable under the design conditions. In practice, as little guidance is available on performance of the structure between these two limits, the designer will be likely to use the more conservative value of the two.

Yarde *et al* (1996) gave particular consideration to the case of reservoir dams, and to wave conditions generated over limited fetch lengths such as those occurring on inland bodies of water, where wave periods are short and wave steepness is large. They extended the general method of Klein Breteler and Bezuijen (1991) for short wave periods and for larger slabs, and suggested the following modified equation:

$$\frac{H_s}{\Delta t_a} = \frac{S_c}{\xi_{op}} \tag{8.103}$$

Yarde *et al* (1996) quantified the stability coefficient, S_c, as a function of the dimensions and permeabilities of the cover layer and underlayer:

$$S_c = 3.3 \ln\left[\frac{\sqrt{A_s}}{t_f}\left(\frac{w}{D_{f15}}\right)^{0.1}\right] + 4.0 \tag{8.104}$$

where:

A_s = slab area (m²)
t_f = thickness of the filter layer (m)
w = gap between slabs representing drainage area or cover layer permeability (m)
D_{f15} = 15 per cent non-exceedance diameter of the filter layer material, obtained from the grading curve (m), and is taken as indicating the relative permeability of the filter layer

A comparison between the outputs of the alternative design formulae is given in Figure 8.57.

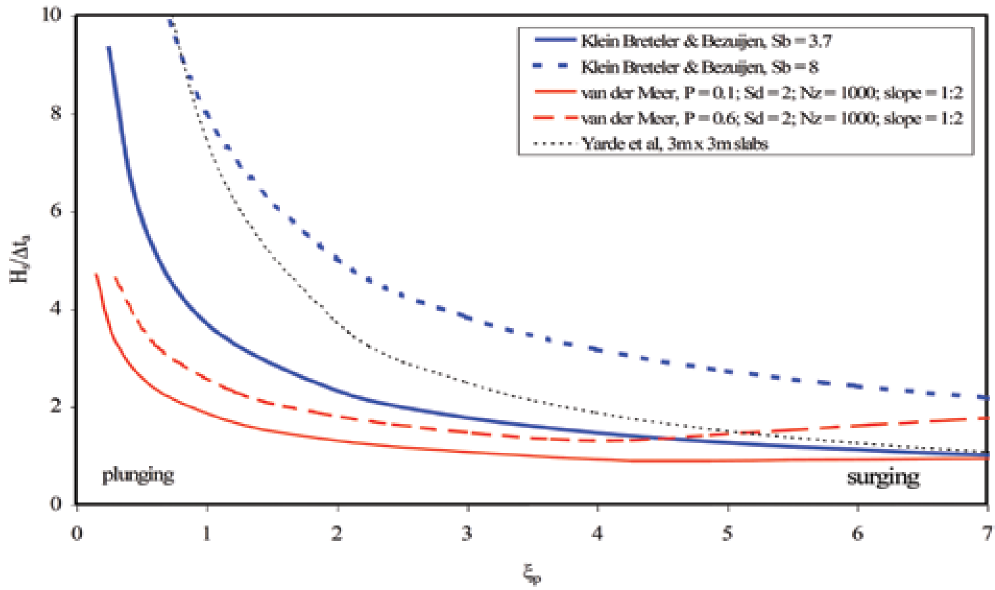


Figure 8.57 *Comparisons between stable revetment thickness predictions for rock armour, blockwork and slabbing (from McConnell and Allsop, 1999)*

8.4.5 Resistance of armourstone to ice

Brown and Clyde (1989) identified that ice (Section 7.3.13) can affect surface protection systems in a number of ways:

- moving surface ice can cause crushing and bending forces and large impact loadings
- the tangential flow of ice along a protected levee can cause high lateral shearing forces
- the thawing of upstream ice jams can cause a rapid release of water and blocks of ice leading to flooding and possible overtopping of water and ice.

Ice forces should be evaluated on a case-by-case basis using past experience and local codes of practice. In most instances, ice flows will not be of sufficient magnitude to warrant detailed analysis. For example, historic observations of ice flows in rivers in New England, USA indicate that rip-rap sized to resist design fluvial flow events will also resist ice forces (Brown and Clyde, 1989, and Colorado Department of Transportation, 2004).

Where ice flows have historically caused problems, the diameter of armourstone calculated using procedures such as those set out in Sections 8.4.4, should be multiplied by an additional stability factor based on local experience. Table 8.20 provides an initial guide to the magnitude of this stability factor developed by Brown and Clyde (1989).

Table 8.20 *Guidelines for the selection of stability factors for rip-rap design (Brown and Clyde, 1989)*

Condition	Stability factor* range
Uniform flow, straight or mildly curving reach (curve radius/channel width > 30), impact from wave action and floating debris is minimal, little or no uncertainty in design parameters	1.0–1.2
Gradually varying flow, moderate bend curvature (30 > curve radius/channel width > 10), impact from waves or floating debris moderate	1.3–1.6
Approaching rapidly varying flow, sharp bend curvature (10 > curve radius/channel width), *significant potential impact from floating debris and/or ice*, significant wind and/or boat generated waves (0.30 m to 0.61 m), high flow turbulence, turbulently mixing flow at bridge abutments, significant uncertainty in design parameters	1.6–2.0

Note

* Testability factor is the number by which the design rock diameter for hydraulic design should be multiplied to take account of ice effects.

Vaughan *et al* (2002) carried out independent assessments and calculations to investigate the appropriateness of the Brown and Clyde (1989) recommendations in five relatively severe ice related scenarios:

1. Anchor ice rafting and rip-rap specific gravity reduction.
2. Raft ice impact damage.
3. Raft ice push-up onto shore.
4. Ice jams causing velocity increase.
5. Increased longitudinal effective tractive force imposed by stream ice cover.

They concluded that, for the scenarios investigated, the higher stability factors in Table 8.20 (ie in the range 1.6 to 2.0) were still relevant.

Note

Consideration of the stability formulae in Sections 8.4.3 and 8.4.4 shows that as an alternative to increasing the size of the armourstone, flatter levee slopes may be adopted to deliver the same increase in size of the stability factor. This is reflected by the practice described in Box 8.13.

Box 8.13 USA practice for levee slopes prone to ice action

General practice in the Midwestern USA is to keep slopes at 1V:4H or flatter. If use of a 1V:4H slope is not an option, the size of the armourstone is increased.

Extension of rip-rap protection up to the 10 per cent event ice water surface profile should also be considered (a practice adopted by the Omaha District Corps of Engineers). If required, numerical modelling may be used to estimate the 10 per cent event ice water surface profile.

When considering the design methodologies available for blockwork, it became apparent the methodologies are mainly based on loose or interlocking blocks of low permeability. Many proprietary cellular block systems are available that have much higher permeabilities. Consideration of model test data from Lindenberg (1983) suggested that the method of Klein Breteler and Bezuijen (1991) could be applied with careful choice of the stability coefficient S_b.

Often concrete blockwork may be cable-tied with nylon or steel cables being used to create blockwork mats which facilitate placement of the blocks. While it is generally agreed that the cables should not be considered to provide additional strength in the structure allowing thinner blocks to be used, they may help to provide a restraining force in the event that sliding failure of the revetment occurs.

Model studies by Lindenberg (1983) and practical experience suggest that gravel blinding of blockwork may help provide an increase in the stability of concrete blocks. This enhancement would, however, only work if both the concrete blocks and the binding material were sufficiently robust/durable to resist crushing over the life of the revetment. There is much debate as to whether this stability increase can be relied upon and McConnell (1998) recommends that this improvement be ignored in performing ultimate stability calculations.

8.4.5.1 Design formulae for asphaltic revetments

Many coastal levees are protected from erosion of their core by an asphalt revetment, typically between 15 cm and 30 cm in thickness. The thickness is larger in the lower part of the revetment in order to avoid uplift when the sea level drops. The discussion in this section is an introduction, focusing mainly on impermeable asphaltic revetments. For such revetments, three failure mechanisms are normally considered detailed as follows:

Uplift

The failure of an asphalt revetment layer by uplift forces can be described by a simplified analytical solution, in which the maximum water head difference is related to the thickness of the revetment. This solution can be applied to an impermeable asphalt revetment on a sand bed with an open toe construction (Figure 8.58).

The driving load is expressed in terms of a head difference H_{max}.

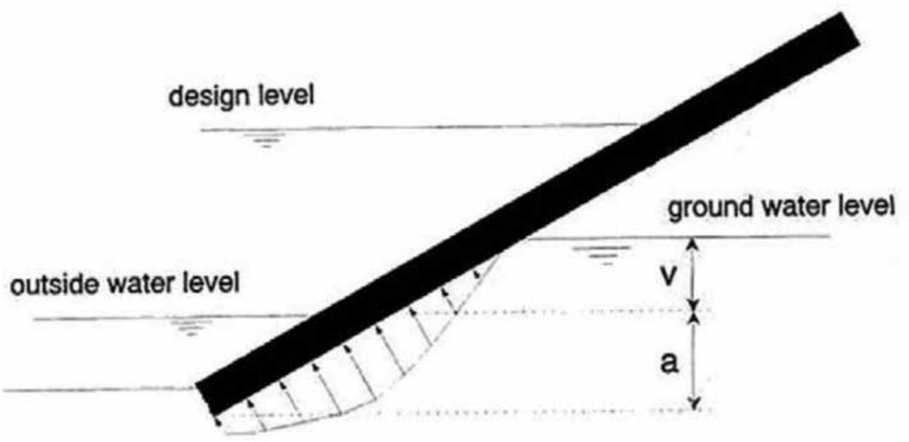


Figure 8.58 Uplift sketch for asphaltic revetment (from FLOODsite, 2007)

The layer thickness h can be derived from the equilibrium equation for uplift:

$$h = H_{max} \triangle / \cos \alpha \tag{8.105}$$

where:

Δ = relative buoyant density of the asphalt = $(\gamma_{\rho} - \gamma_w)/\gamma_w$, where γ_r for asphalt = 23 kN/m^3

h = thickness of asphalt layer (m)

α = slope angle (°)

Two situations need to be considered:

1 Where the outside water level at which the maximum uplift pressure occurs is higher than the average outside water level, Equation 8.106 should be used (Van Herpen, 1998):

$$H_{max} = \frac{v}{\pi} \arccos \left(2 \left[\frac{v + h \cos \alpha}{a + v} \right]^{\pi/\theta} - 1 \right) \tag{8.106}$$

with $\theta = \arctan(n) + \pi/2$, and a and v as shown in Figure 8.58.

For a given groundwater level and a variable outside water level Equation 8.106 can be maximised to $v/(a + v)$. For slopes between 1:1 and 1:8 this gives $v/(v + a) = 0.53$. This can be inserted in Equation 8.106 in order to obtain an equation for $H_{max\ critical}$ depending on h and α at the critical outside water level. This equation for $H_{max\ critical}$ can be inserted into Equation 8.105. The resulting equation can be solved numerically to find $h/(a + v)$ and where the groundwater level $(a + v)$ is known this calculation results in a value for the layer thickness h.

In the case of a slope angle 1:4, the numerical results have been fitted by means of the function Q_n, which results in the following formula for the layer thickness h:

$$h = 0.21\, Q_n\, (a + v) / \triangle \tag{8.107}$$

where:

Q_n = $0.96/(\cos \alpha)^{1.4}$

2 Where the critical outside water level at which the maximum uplift pressure occurs is lower than the average outside water level, $a/(a + v)$ is defined with reference to the average outside water level. This means that a correction factor R_w is needed in Equation 8.107. R_w varies with $v/(a + v)$ as shown in Figure 8.59.

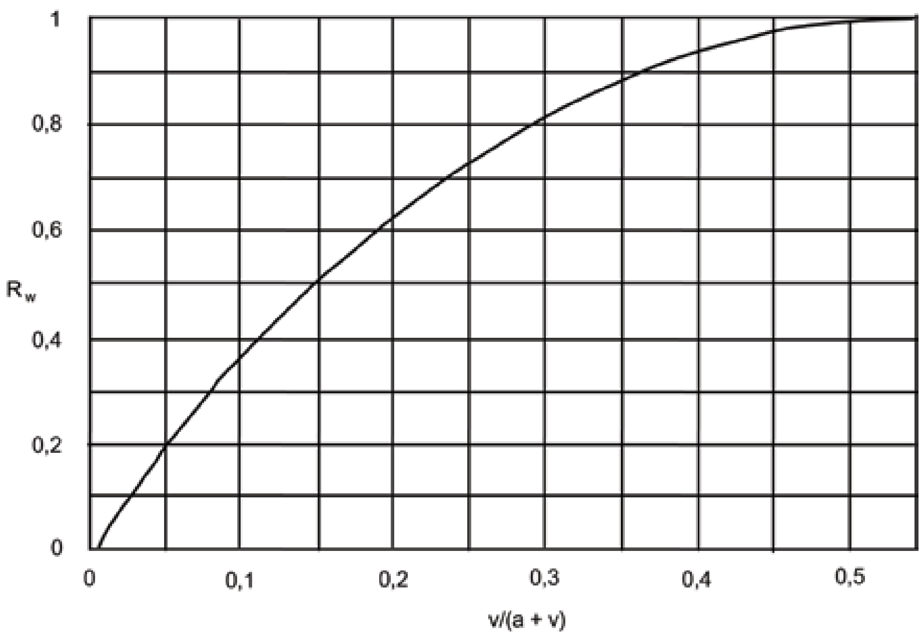


Figure 8.59 ***Reduction factor R_w for h where the outside water level at which the maximum uplift pressure occurs is lower than the average outside water level (Morris et al, 2012a)***

Resistance against wave loading

Asphaltic revetments may be able to resist repeated waves with significant heights of up to 4.5 m. However, the asphalt layer can fail as a result of fatigue due to repeated loading under storm conditions. Indeed, in the event of very high wave loads, the asphalt can fail after just a few large waves. So, in conditions of severe wave attack, calculations should be carried out to ensure that the asphalt has sufficient fatigue strength to resist the impact forces of wave loading, which cause bending in the asphalt due to limited support from the underlying materials. Appropriate fatigue calculations can be facilitated (de Looff *et al*, 2006) by suitable software such as 'Golfklap' (wave attack in English).

Sliding

Sliding is avoided when:

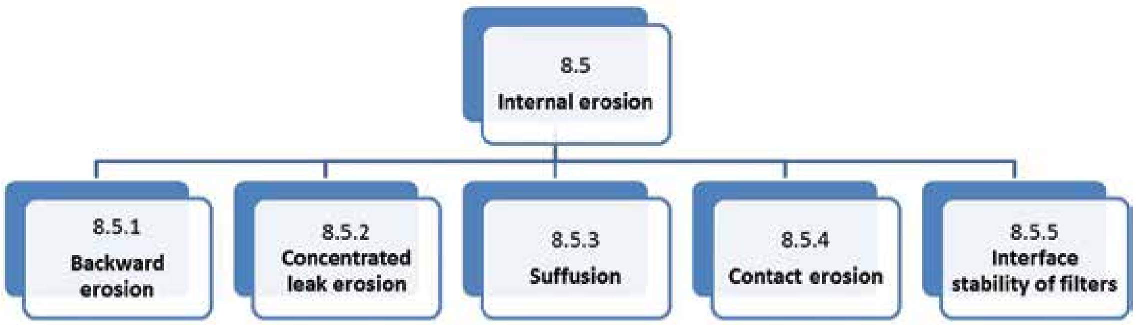
$$H_{max} = \leq h \cos\alpha \left[\frac{\gamma_w}{\gamma_r}\left(1 - \frac{\tan\alpha}{f}\right) - 1\right] \tag{8.108}$$

where:

h = thickness of revetment (m)

f = coefficient for friction (-), for $\theta < \varphi$: $f = \tan\theta$, for $\theta \geq \varphi$: $f = \tan\varphi$

8.5 INTERNAL EROSION



Internal erosion is related to all processes that involve soil particles detachment and transport by seepage flow within the dam or levee, or its foundation. Such processes can ultimately lead to the instability of the levee. Failure by internal erosion is categorised into three general modes:

- internal erosion through the embankment
- internal erosion through the foundation
- internal erosion at the levee foundation contact.

Basic mechanisms of internal erosion

Four different mechanisms may be identified ICOLD (2012). These mechanisms form the basis for information presented in this section and shown in the section flow chart.

1. **Backward erosion:** detachment of soil particles when the seepage exits to an unfiltered surface, leading to worm-holes and sand boils.
2. **Concentrated leak erosion:** detachment of soil particles through a pre-existing path in the embankment or foundation.
3. **Suffusion:** selective erosion of the fine particles from the matrix of coarse particles.
4. **Contact erosion:** selective erosion of the fine particles from the contact with a coarser layer.

General conditions for occurrence of internal erosion

Two conditions should be fulfilled for internal erosion to occur described as follows, and shown in Figure 8.60:

1. The first condition is that particles can be detached, ie that hydraulic shear stresses are larger than resistant contact forces. To reach this hydro-mechanical criterion, water seeping through the flood defence should have sufficient velocity to provide the energy needed to detach particles from the soil structure.
2. The second condition is that detached particles can be transported through the soil. Two criteria should be fulfilled:
   a. A hydro-mechanical criterion, where flow is sufficient to carry the eroded particles.
   b. A geometric criterion (which is specific to internal erosion), where voids exist in the soils within the flood defence that are large enough for detached particles to pass through. This void is either a pipe inside the soil, as in backward erosion or concentrated leak erosion, or pore space within the grains of a coarse layer, as observed in suffusion and contact erosion.

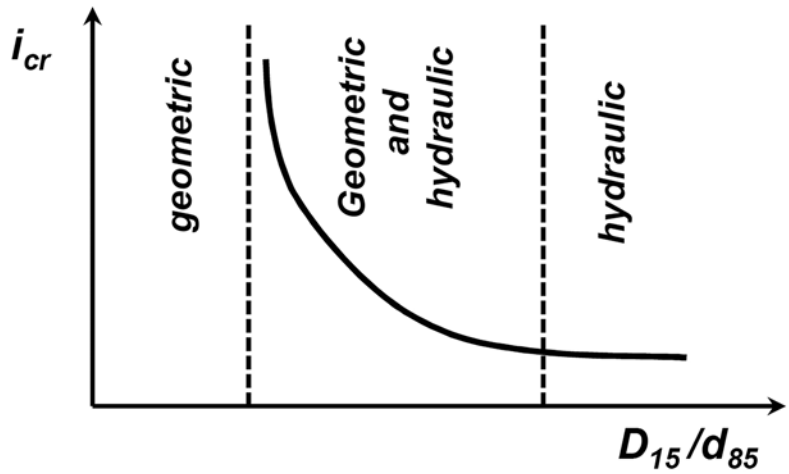


Figure 8.60 *Interaction of geometric and hydraulic influence on internal erosion mechanisms*

The nature of the soil in the embankment determines its vulnerability to erosion. Two main classes have to be distinguished.

1 **Granular non-cohesive soils:** erosion resistance is related to particle buoyant weight and friction. Hydro-mechanical transport criterion is linked to rolling and sliding resistance of the grains.
2 **Cohesive soils:** erosion resistance is mainly related to attractive contact forces in between the soil particles. The main transport mode is suspension flow.

Successive phases of internal erosion

From ICOLD (2012), the process of internal erosion of embankment dams or levees and their foundations can be represented by four phases.

1 **Initiation:** first phase of internal erosion, when one of the phenomenon of detachment of particles occurs.
2 **Continuation:** phase where the relationship of the particle size distribution between the base (core) material and the filter controls whether or not erosion will continue.
3 **Progression:** phase of internal erosion, where hydraulic shear stresses within the eroding soil may or may not lead to the erosion process being ongoing and, in the case of backward and concentrated leak erosion, to formation of a pipe. The main issues are whether the pipe will collapse, or whether upstream zones may control the erosion process by flow limitation.
4 **Breach:** final phase of internal erosion (Section 8.10).

8.5.1 Backward erosion

Backward erosion involves the detachment of soil particles when the seepage exits to a free unfiltered surface. The seepage flow erodes particles upwards and backwards below the embankment through erosion pipes, sometimes called worm-holes, and sand boils form on the surface. In critical circumstances, such as floods, the head difference increases, these pipes may grow progressively from the area with a lower hydraulic head towards the higher head.

The erosion shortens the seepage path and increases the gradient leading to higher flow velocities causing further backward erosion, increasing the length of the worm-hole, and causing failure when the worm-hole extends backwards to greater than half the width of the embankment base. Two configurations are identified:

- backward erosion in a sandy layer below an impermeable roof (clay layer, horizontal structure). This configuration involves the development of retrogressively growing pipes in the sand layer below the levee due to groundwater flow

- backward erosion in a cohesive soil. In this configuration, erosion is initiated by a leakage at the exit of a cohesive core to the foundation. The formation of a hole in the core increases erosion rate and hence leads to progressive backward extension of a pipe.

Criteria for initiation and progression of backward erosion may be related to local hydraulic conditions (exit gradients) or global hydraulic conditions averaged along the flow path.

8.5.1.1 Local criteria

Since erosion primarily involves removal of granular material, backward erosion is only possible if there is a prior destabilisation of the surface of the soil in the exit flow zone. When the exit zone is constituted by a pervious top soil layer, the destabilisation process of the near surface may exist depending on the flow direction and hydraulic gradient. In the special case of horizontal layer with vertical upward flow, this mechanism is called heave (or fluidisation). When the exit zone is composed of an impervious soil layer, the destabilisation process develops at the layer scale (development of cracks within the top layer) and is called uplift.

Heave (fluidisation)

In pervious (granular) soils, movement of soil at the downstream seepage exit may not occur as flotation followed by particle-by-particle movement. A mass of soil may be lifted, followed by piping. This phenomenon is called heave (or fluidisation) and occurs when the upward seepage force due to differential head equals the overlying buoyant weight of soil. Slope stability condition for purely frictional soils (Box 8.14) may be expressed in terms of slope angle:

$$\beta \leq \varphi' - \sin^{-1}\left\{\frac{i}{i_{cr}}\sin\left(\varphi' + \lambda\right)\right\} \tag{8.109}$$

where:

i_{cr} = γ'/γ_w is the Terzaghi critical gradient

As mentioned by Philippe and Richard (2008), the ratio i/i_{Cr} can be interpreted as a Shields' number, which enables making the link with sand erosion framework. The stability condition may also be expressed in terms of gradient:

$$i \leq i_{cr}\frac{\sin\left(\varphi' - \beta\right)}{\sin\left(\varphi' + \lambda\right)} \tag{8.110}$$

In the special case of a horizontal pervious (granular) soil layer ($\beta = 0$) and vertical upward flow ($\lambda = 0$), the stability condition may be written in terms of hydraulic gradient, $i_v \leq i_{Cr}$. The critical gradient may also be written in terms of intrinsic parameters of the soil, $i_{Cr} = (\rho_s - 1)/(1 + e)$, where ρ_s is the specific density of soil grains ($\rho_s \approx 2.7$) and e the void ratio (-).

Uplift

When seepage occurs beneath an impervious soil layer, the layer at its base is subject to a hydraulic force, which tends to lift the soil upward. The stability of soil against heave may be checked by verifying vertical equilibrium of a soil column. This condition may be expressed by:

$$\sigma_v > u \tag{8.111}$$

where σ_v is the stabilising vertical stress (kPa), and u the destabilising pore water pressure (kPa) beneath the imperious layer.

Box 8.14 Slope stability under water flow

Shallow sliding of slopes under-seepage conditions depends on the flow direction and hydraulic gradient, particularly near the ground surface. In the case of homogeneous slopes, analytical solutions based on the infinite slope model may be used. In the infinite slope stability model, the slip surface is assumed to be a plane parallel to the ground surface and the end effects are neglected. This analysis is valid if the ratio of depth to length of the sliding mass is small (a ratio of 1: 20 is commonly used). The slope element is subjected to both seepage and gravitational forces, in a block stability approach.

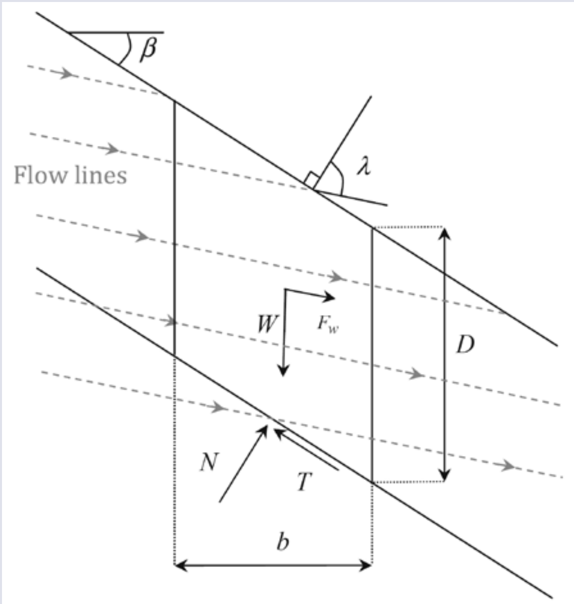


Figure 8.61 Infinite slope model with parallel flow lines

From geometrical considerations, the gradient can be derived as a function of seepage direction (λ) and slope angle (β). This exit gradient, corresponding to a locally uniform seepage, may be expressed as:

$$i = \frac{\sin\beta}{\sin\lambda} \qquad (8.112)$$

It can be shown (Delinger and Iverson, 1990, and Ghiassian and Ghareh, 2008) that equilibrium condition of the sliding mass may be expressed in terms of slope geometry parameters and effective shear strength parameters of the soil:

$$\sin\beta + i\,\frac{\gamma_w}{\gamma'}\sin\lambda \le \frac{c'}{\gamma'\,D\,\cos\beta} + \left(\cos\beta - i\,\frac{\gamma_w}{\gamma'}\cos\lambda\right)\tan\varphi' \qquad (8.113)$$

where:

D = vertical soil depth (m)

β = inclination of the slope from the horizontal (°)

C' = soil effective cohesion (kPa)

φ' = soil effective internal friction angle (°)

γ' = unit weight of submerged soil (kN/m^3)

γ_w = unit weight of water (kN/m^3)

8.5.1.2 Global criteria

Global criteria models apply only in the configuration of backward erosion in a sandy layer below an impermeable roof, which is considered as perfectly rigid (not erodible). They introduce the concept of the length of the path travelled by seeping water and lead to the development of creep ratios or creep coefficients. Figure 8.62 shows the basic parameters required for the analysis.

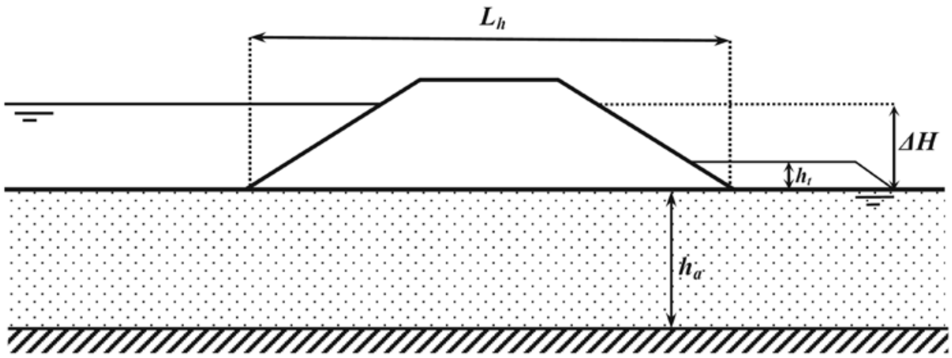


Figure 8.62 Definition of geometrical parameters

Bligh model

The rule of Bligh (1927) states that failure from backward erosion occurs if:

$$\triangle H - 0.3\, h_t > \frac{L_h}{C_{Bligh}} \tag{8.114}$$

where:

ΔH = hydraulic head over the levee (m)
h_t = thickness of the top layer (m)
L_H = horizontal seepage length (m)
C_{Bligh} = creep factor of Bligh (-)

Lane model

The presence of a structure, such as a cut-off wall, causes an extra barrier for the seepage path. Lane (1935) introduced a vertical seepage length L_v so that the rule of Lane states that backward erosion occurs if:

$$\triangle H - 0.3\, h_t > \frac{L_h/3 + L_v}{C_{Lane}} \tag{8.115}$$

where:

L_v = vertical seepage length (m)
C_{Lane} = creep factor of Lane (m)

The creep factors depending on the type of soil are given in Table 8.21.

Table 8.21 Value of creep coefficient

Type of soil	C_k (Lane)	C_k (Bligh)
Very fine sand or silt	8.5	18
Fine sand	7	15
Medium size sand	6	–
Coarse sand	5	12
Fine gravel or sand and gravel	–	9
Medium size gravel	3.5	–
Coarse gravel	3	–
Boulders, gravel and sand	–	4 to 6
Clay	2 to 3	–

Sellmeijer model

Large research programs were performed in the 1970s and 1980s to create a better understanding of the piping mechanism. More recently, scale effects have been studied with small, medium and full-scale experiments. Recent advances (van Beek *et al*, 2011, and Sellmeijer *et al*, 2011) in understanding the process has led to the improvement of a theoretical model, in which the equilibrium of grains in the bed of the pipe is used as criterion for development of the pipe. The critical gradient can be calculated by combining groundwater flow with the flow conditions in the pipe. Curve-fitting resulted in a formula relating sand characteristics to the geometric properties of the sand bed. This model takes into account scale effect (ratio between grain size and gradient $\Delta H/L$). According to the model of Sellmeijer (for horizontal retrogressive erosion in a sand layer below a clay dike) backward erosion is prevented if (Sellmeijer and Koenders, 1991):

$$\Delta H - 0.3\, h_t < F_{resistance}\, F_{scale}\, F_{geometry}\, L_h \tag{8.116}$$

with:

$$F_{resistance} = \frac{\gamma_p'}{\gamma_w}\,\eta \tan\theta \left(\frac{D_R}{D_{Rm}}\right)^{0.35} \left(\frac{C_U}{C_{Um}}\right)^{0.13} \left(\frac{R}{R_m}\right)^{-0.02} \tag{8.117}$$

$$F_{scale} = \frac{d_{70}}{\sqrt[3]{\kappa L_h}} \left(\frac{d_{70m}}{d_{70}}\right)^{0.6} \tag{8.118}$$

$$F_{geometry} = 0.91 \left(\frac{h_a}{L_h}\right)^{\frac{0.28}{(h_a/L_h)^{2.8}-1}+0.04} \tag{8.119}$$

where the *m* index refers to the characteristics of the small scale tests and:

ΔH = actual hydraulic head over the flood defence (m)
L_h = horizontal seepage length (m)
h_a = thickness of the aquifer (uppermost sand layer sensitive for retrogressive erosion) (m)
h_t = thickness of top layer (m)
γ_s' = unit weight of sand grains under water (16.5 kN/m^3)
γ_w = unit weight of water (10 kN/m^3)
θ = bedding angle of sand grains (°)
η = White's constant (0.25)
κ = intrinsic permeability of aquifer (m^2)
d_{70} = d_{70} of aquifer (m), (d_{70} = 208μm)
D_R = relative density (%)
R = roundness of the particles (%)

The parameters κ and d_{70} may be determined from grain size distribution analyses. The bedding angle determines how a grain is disposed on the other grains. It is only related to the weight and geometry since the model assumes that the grain rolls over the others without friction. Experimental data show that θ = 37° is a good estimation from current cases.

It should be noted that the above set of equation does not include a margin of safety and that for design purposes, a factor of safety may be necessary.

Hoffman's method

Another approach has been developed by Hoffmans (2012) to determine the critical gradient. Considering that progression of backward erosion needs transport of the detached particles through the piping channel, the critical gradient is decomposed in a critical Shields gradient and a critical Darcy's gradient. The most important variables of this model are hydraulic conductivity, particle sizes d_{50} and d_{15} and some coefficients determined experimentally. This approach enables capturing the influence of permeability on the magnitude of the critical gradient. This model does not account for all physical processes but secondary effects are included by calibration of some parameters of the model.

Schmertmann's method

Based on several laboratory tests on fairly uniform soils ($1.0 < C_u < 6$) ranging from fine to medium sands, Schmertmann (2000) proposed the following linear expression of the critical gradient:

$$i_{cr} = 0.05 + 0.183\,(C_U - 1) \tag{8.120}$$

This approach has the advantage of simplicity however this correlation was not confirmed for different types of soils.

An example of the use of a simplified method for under-seepage analysis is given in Box 8.15.

Box 8.15 ***Simplified method for under-seepage analysis***

In fluvial environments, levees are often placed on alluvial floodplains covered with silty or clayey soils that form impervious foundations. These impervious layers are frequently founded on a sandy soil stratum (aquifer), generally anisotropic, with permeability that is much greater, enabling horizontal flows. So, the simplified model (USACE, 1993) based on the following basic assumptions may be used:

- flow through the blanket is vertical
- flow through the pervious foundation is horizontal
- all flows are laminar and steady state
- the levee material (or its core) is impervious
- aquifer has a constant thickness and is horizontal.

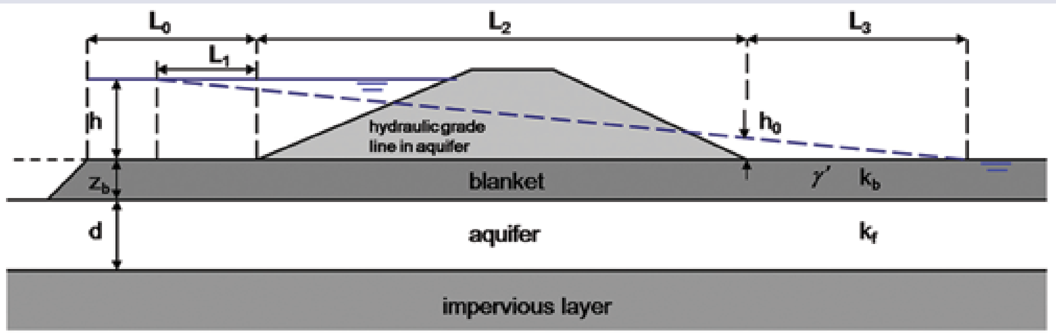


Figure 8.63 Geometric notations for under-seepage analysis (after USACE, 1993)

When the upstream impervious blanket is not continuous, its upstream effective length, L_1 (m), has to be defined as

$$L_1 = \sqrt{\frac{k_f}{k_{bu}} Z_{bu}\, d} \tag{8.121}$$

Box 8.15 *Simplified method for under-seepage analysis*

where:

k_f = horizontal permeability of the pervious foundation (m/s)
k_{bu} = vertical permeability of the upstream blanket (m/s)
Z_{bu} = thickness of the upstream blanket (m)
d = thickness of the pervious aquifer (m)

The effective length of the downstream blanket, L_3 (m), is:

$$L_3 = \sqrt{\frac{k_f}{k_{bd}} Z_{bd}\, d} \tag{8.122}$$

where:

k_{bd} = vertical permeability of the downstream blanket (m/s)
Z_{bd} = thickness of the downstream blanket (m)

The pressure head under the blanket at the downstream toe of the levee is estimated as follows:

$$h_0 = \frac{h\, L_3}{L_1 + L_2 + L_3} \tag{8.123}$$

where:

L_2 = length of impervious core or levee base (m)

This pressure head is then compared to the critical pressure head $h_b = Z_{bd}\, \gamma'/\gamma_w$, so that the factor of safety against uplift at the downstream toe of the levee is:

$$F = \frac{h_0}{h_c} = \frac{Z_{bd}}{h} \frac{\gamma'}{\gamma_w} \frac{L_1 + L_2 + L_3}{L_3} \tag{8.124}$$

8.5.2 Concentrated leak erosion

Concentrated leak erosion appears in a preferential path such as crack openings or pre-existing holes. Along this path, water flow is sufficient to initiate soil particle detachment from lateral surfaces and transport away inducing enlargement of the path. In the presence of cohesive materials able to 'hold a roof', theses openings result in the formation of a continuous tunnel called a 'pipe' between the upstream and the downstream side of the embankment or its foundation.

8.5.2.1 Model for concentrated leak erosion

The first model to interpret concentrated leak erosion was proposed by Wan and Fell (2002, and 2004a and b) for a specific type of tests, called the hole erosion test (HET) (Box 8.16 or Section 7.8.3). This test reproduces concentrated leak erosion in a pre-existing cylindrical pipe. More recently, a model combining hydrodynamic equations for a turbulent pipe flow and tangential erosion law was able to interpret more accurately experimental HET results (Bonelli *et al*, 2006, Bonelli and Brivois, 2008, Bonelli, 2012, and Benahmed and Bonelli, 2012). These models use a local erosion law, which is often written in the form of a threshold law:

$$\varepsilon = C_e \left(\tau - \tau_c\right) \tag{8.125}$$

where:

ε = eroded mass rate per unit surface (kg/m²s)
τ = hydraulic shear stress applied to the surface of the hole (Pa)
τ_c = critical shear stress (Pa)
C_e = coefficient of erosion (s/m)

Critical shear stress and coefficient of erosion characterise the 'erodibility' of the soil. The critical shear stress is the minimum hydraulic shear stress required to initiate the detachment of soil particles. Below this value, no erosion is observed. The coefficient of erosion reflects the rate of the detachment of soil particles when the stress is maintained constant above the critical shear-stress. Piping occurs if $P_0 > \tau_c$ where P_0 is the driving pressure, equal to the tangential shear stress exerted by the piping flow on the soil, and τ_c is the critical stress. The evolution of pipe radius during erosion with constant pressure drop obeys an exponential scaling law:

$$R(t) = R_0 \left[\frac{\tau_c}{P_0} + \left(1 - \frac{\tau_c}{P_0}\right) e^{+\frac{t}{t_{er}}} \right] \qquad (8.126)$$

with

$$P_0 = R_0 \frac{\triangle p}{2\,L} \qquad (8.127)$$

and

$$t_{er} = \frac{2\,\rho_{dry}\,L}{C_e\,\triangle p} \qquad (8.128)$$

where:

P_0 = driving pressure (Pa)
τ_{er} = characteristic piping erosion time (s)
R_0 = initial radius (m)
Δp = pressure drop in the hole (Pa)
L = hole length (m)
ρ_{dry} = dry soil density (-)
C_e = Fell coefficient of soil erosion (s/m)

The Fell coefficient of soil erosion is related to the Fell erosion index by $I_e = -\log(C_e/C_{ref})$ with $C_{ref} = 1$ s/m.

Box 8.16 Hole erosion test (HET)

Concentrated leak erosion resistance of soils can be tested in laboratory using a HET apparatus (Figure 8.64).

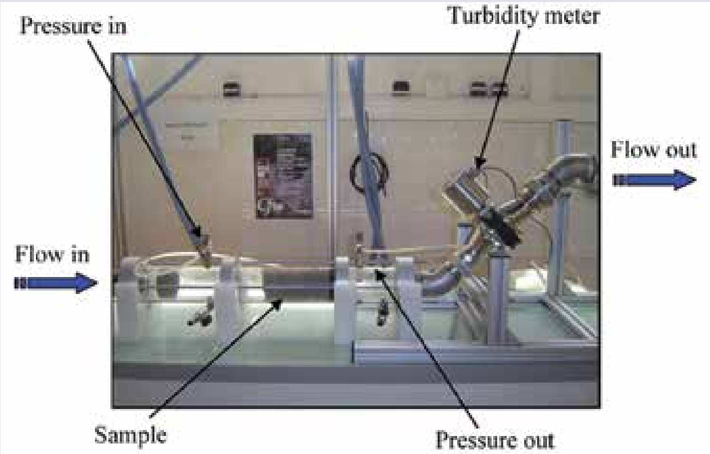


Figure 8.64 Hole erosion test (HET) apparatus at Irstea

A typical experimental result of a HET is shown in Figure 8.65. The experimental data are expressed in terms of pressure gradient and turbidity versus time.

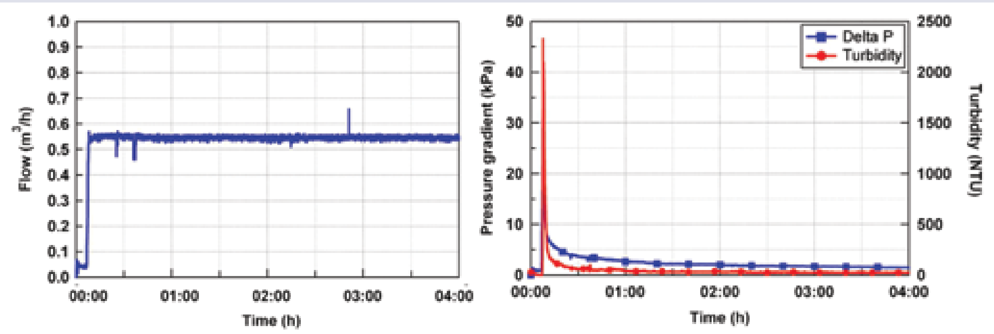


Figure 8.65 Example of evolution of turbidity and pressure gradient during a hole erosion test (Benahmed and Bonelli, 2012)

Box 8.16 *Hole erosion test (HET)*

An example of an eroded sample is shown in Figure 8.66. The longitudinal section of the sample cut at the end of the test clearly shows the enlargement of the initial hole after the erosion process. It can also be seen that the shape of the enlargement is fairly uniform.

Figure 8.66 ***Example of enlargement of initial hole by erosion on white kaolinite sample, sample before the test (a), sample after the test (b), and longitudinal section of the sample after the test (c) (Benahmed and Bonelli, 2012)***

The same model as presented in Section 8.5.2.1 is used to interpret a HET and provides the values of the erodibility parameters of the soil sample, namely the critical shear stress, τ_c, and the coefficient of erosion, C_e.

8.5.2.2 Factors affecting time to failure

Consider the case of a straight and circular pipe of current radius R(t), in an embankment of height and base width (Figure 8.67) (Bonelli and Benahmed, 2011, and Bonelli *et al*, 2012).

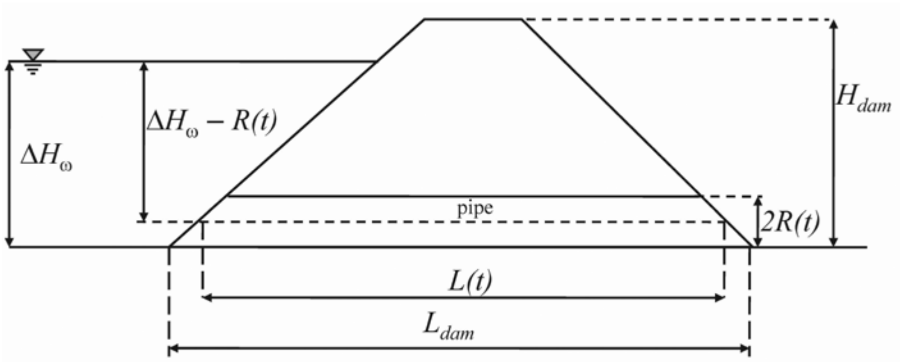


Figure 8.67 ***Sketch of a pipe flow with erosion***

The rate of pipe enlargement is highly dependent on the erodibility of the soil as measured by the erosion coefficient and the critical shear stress. The enlargement of the pipe ultimately causes roof collapse and creates a breach. The scaling law of the piping erosion process with a constant hydraulic gradient is given in Equation 8.129. An expression for the time remaining to breaching can then be proposed. The piping process begins at time, t_0, with the initial radius R_0, both of which are unknown.

Visual inspection defines the initial time $t_d > t_0$ for detection and can provide an estimation of the output flow rate, and so an estimation of the radius $R_d > R_0$. R_u and t_u can be used to denote the maximum radius of the pipe before roof collapse and the collapse time, respectively. For $t > t_u$, piping failure continues to cause erosion in a way similar to that of an overtopping failure (Section 8.4.2). So, the remaining time before breaching may be estimated as follows:

$$\Delta t_u \approx t_{er} \ln\left(\frac{R_u}{R_d}\right) \propto \frac{1}{C_e} \qquad (8.129)$$

This significant result means that erosion coefficient C_e can serve as an indicator of the time remaining to breaching unlike the critical shear stress τ_c. The peak flow is assumed to correspond to the maximum radius of the pipe. Consequently, the time before breaching is also the time from detection (eg eyewitness observations) to peak discharge, and visual detection of the piping event as well as reporting are required. The following orders of magnitude (Bonelli and Benahmed, 2011) are found: if the erosion index I_e is of the order of magnitude of 2 ($C_e \approx 10^{-2}$ s/m), an d the levee failure will take place very quickly, within a few minutes. If the erosion index I_e is of the order of magnitude of 3 ($C_e \approx 10^{-3}$ s/m), the levee failure will take place within several hours. If it is greater than 4 ($C_e < 10^{-4}$ s/m), then the levee failure will not occur until several days, allowing time to take appropriate action. This coefficient can be obtained with the HET. However, the change of scale (from the laboratory to the structure) could affect the coefficient of erosion, which remains to be addressed.

8.5.3 Suffusion

Both geometric and hydraulic conditions must be fulfilled for suffusion to occur. Many granulometric criteria exist in various literature. However, one of the most commonly used is the standard proposed by Kenney and Lau (1985), which combines grain size distribution and filtration rules. More recently, Fannin and Li (2006) have compared this criterion with another proposed by Kezdi (1979) while Wan and Fell (2008) have shown that the previous commonly used methods are conservative for silt-sand-gravel or clay-silt-sand-gravel soils.

8.5.3.1 Kenney and Lau model

This model considers that grains smaller than a given diameter *d* can be detached if there are not enough grains in the interval (*d* to *4d*) to keep them trapped (Figure 8.68). They proposed the following criterion:

$$\min_{F_d < X} \left(\frac{F_{4d}}{F_d} - 1 \right) \geq 1 \quad (8.130)$$

where:

d = diameter of grains (m)

F_d = cumulative mass percentage of grains smaller than the diameter d (-)

For coefficient of uniformity of the soil (C_u, defined as the ratio d_{60}/d_{10}) smaller than 3, the parameter X is taken equal to 0.3. For $C_u \geq 3$, it may be taken equal to $X = 0.2$.

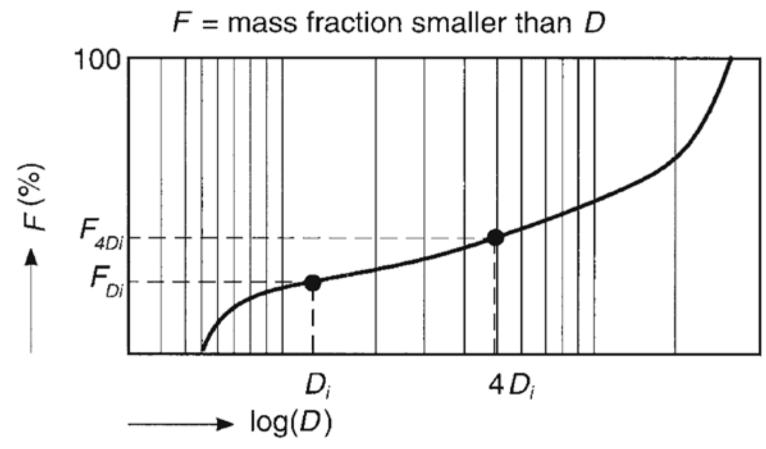


Figure 8.68 Definition of cumulative mass percentage criterion (from CIRIA; CUR; CETMEF, 2007)

8.5.3.2 Model of Kezdi

This model proposes a simpler criterion:

$$\min_{d} (F_{4d} - F_d) \leq 0.15 \quad (8.131)$$

This criterion is more conservative than the Kenney and Lau model for $F_d < 0.15$.

8.5.3.3 Li and Fannin approach

Using some new experiments and those existing in various literature, Li and Fannin (2008) have recently proposed to use Kezdi criterion for gap-graded size distribution whereas Kenney and Lau criterion is suited for widely-graded soils as shown in Figure 8.69.

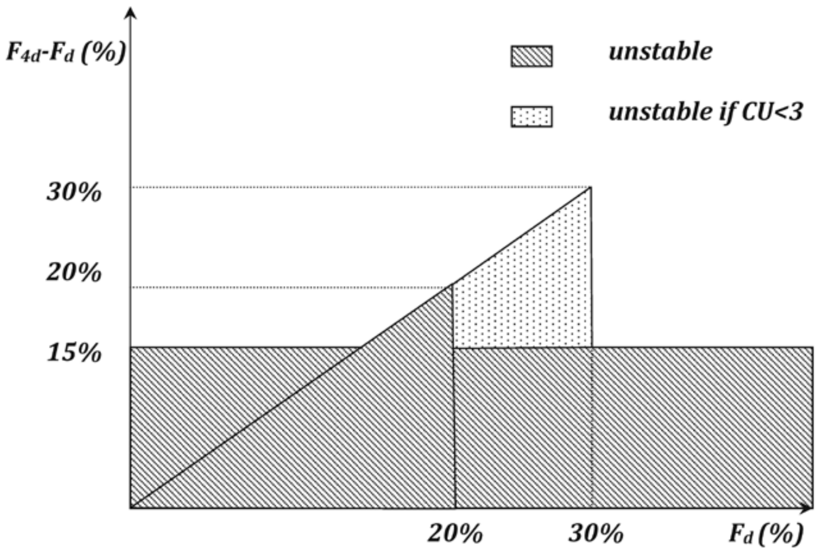


Figure 8.69 Graphical synthesis of Kenney and Lau and Kezdi approaches

Li (2008) proposed also a hydro-mechanical criterion in terms of threshold hydraulic gradient, validated for experiments on unstable soils, which is simply a fraction of the critical gradient i_{Cr} first introduced by Terzaghi:

$$i_{suf} = \chi\, i_{cr} = \chi \frac{\gamma'}{\gamma_w} \tag{8.132}$$

with:

$$\chi = 3.85 \frac{d'_{85}}{O_{50}} - 0.616 \tag{8.133}$$

where:

i_{suf} = threshold hydraulic gradient initiating suffusion (-)
i_{Cr} = critical hydraulic gradient initiating heave (-)
d'_{85} = d_{85} of the fine fraction of soil (m)
O_{50} = effective constriction size of the coarse fraction (m)

The χ parameter generally falls between one-fifth and one-third. Other methods may be used in the case of well-graded soils (Burenkova, 1993, and Lubockov, 1965).

8.5.4 Contact erosion

As in suffusion, both geometric and hydraulic conditions must be fulfilled. But unlike suffusion, which concerns a unique material with a broad graded grain size distribution, contact erosion appears at the interface between two different materials having distinct grain size distributions. Consequently, the geometric condition for contact erosion to occur is simply fulfilled when the classical filter rules are not satisfied and the studies related to contact erosion have mainly focused on hydraulic threshold.

Most of the models proposed for contact erosion are dedicated to the first configuration, ie underlying fine material layer with non-cohesive soils (sand). They result from an adaptation of Shields criterion (Shields, 1936) with an empirical coefficient that accounts for the specific geometry of the coarse layer (Brauns, 1985, and Bezuijen *et al*, 1987). Darcy velocity has been chosen by the majority of the models' authors as a good indicator of the hydraulic loading. This threshold reads:

$$U_{cr} = \alpha\, n_D \sqrt{\frac{\gamma'}{\gamma_w} g\, d_{50}} \tag{8.134}$$

where:

α	=	empirical coefficient (-)
n_D	=	porosity of the coarse layer (-)
γ'	=	buoyant specific weight (kN/m^3)
γ_w	=	unit weight of the water (10 kN/m^3)
d_{50}	=	median diameter of sand grading curve (m)

The empirical coefficient α is equal to 0.65 as proposed by Brauns (1985), or depends on the type of fine soil and flow characteristics (Bezuijen *et al*, 1987). More precisely, Béguin *et al* (2013) showed that α may be explained by the existence of a hydrodynamic transition zone just above the layer of the fine soil.

The inverse configuration as well as cohesive soils have been studied recently (Schmitz, 2007, Guidoux *et al*, 2010, and Beguin, 2011). Based on experimental results of contact erosion tests with silts and clays, Guidoux *et al* (2010) adapted empirically Brauns' expression to take into account the adhesive forces. Beguin (2011) proposed to use the same threshold erosion law as for concentrated leak erosion (Section 8.4.3). This requires a relation between shear stress and hydraulic gradient (or equivalently Darcy velocity) as the ones proposed by Reddi *et al* (2000) or Wörman and Olafsdottir (1992). Note that for cohesive soils, Béguin (2011) also successfully used the excess shear stress erosion law proposed for concentrated leak erosion. Information from all of these sources is summarised in Figure 8.70.

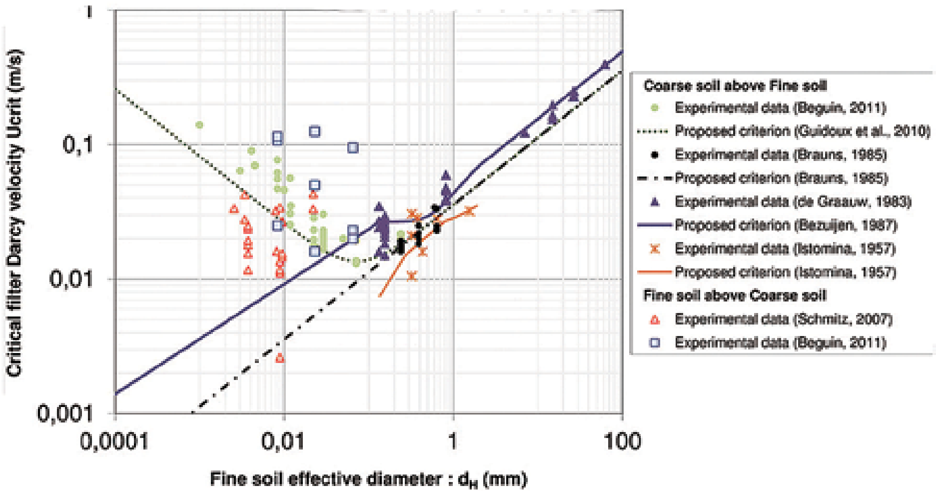


Figure 8.70 ***Summary of experimental data and models for the critical Darcy velocity at the initiation of contact in the configuration with a coarse material over a layer of fine soil (Béguin, 2011)***

8.5.5 Interface stability of filters

8.5.5.1 Granular filters

Granular filters have to fulfil five requirements:

- soil retention
- drainage
- self-filtering
- not crushable
- not cohesive.

The crushability of the filter depends on the nature of soil particles. For silicate soil particles, it may be assumed that this criterion is intrinsically met as long as no shear failure develops within the drain. The non-cohesiveness is generally met when the fine content FC < 5 per cent, and self-filtering is guaranteed when the filter is not subjected to suffusion (Section 8.5.3). The following paragraphs will focus on the first two requirements.

The filter stability at the interface of two different granular materials is called interface stability. The finer of the two materials is called the 'base' (index *b*) and the coarser the 'filter' (index *f*).

Terzaghi retention criterion

For a matrix consisting of grains of diameter *D*, a particle of diameter *d* is prevented from being transported through the matrix, based on geometrical considerations, using criterion developed by Terzaghi (1940):

$$\frac{D_{15}}{d_{85}} < 4 \text{ to } 5 \tag{8.135}$$

where:

D_{15} = particle size diameter for 15 per cent passing of the filter (mm)
d_{85} = particle size diameter for 85 per cent passing of the base soil (mm)

This purely geometric retention criterion has been shown to be generally quite conservative and applicable for truncated d < 4.75 mm fraction of the soil.

When the soil has relatively few particles in a certain size range, the soil may be considered as 'gap-graded' and the grading curve may be characterised by a concave shape with a relatively flat curve in the intermediate range. The criteria mentioned here may still be relevant provided that it is considered as a mixture of two subgradings with quite different particle size ranges. When the base is gap-graded, the d_{85base} value may be replaced by the sieve size d'_{85base} (mm) of the smaller of the two subgradings. Mlynarek *et al* (1993) suggest that this size may more or less correspond to the D_{30base} of the overall base material. So, the Terzaghi criterion would become:

$$\frac{d_{15f}}{d_{30b}} < 5 \tag{8.136}$$

Criteria for design purposes

Based on extensive laboratory research, Sherard and Dunnigan (1985 and 1989) proposed filter design criteria for drains based on the distinction of four soil classes. These criteria, presented in Table 8.22, are generally well accepted by practitioners for design purposes of new structures.

Table 8.22 Filtering criteria of Sherard and Dunnigan (1989)

FC	Soil class	Filter condition
< 15 %	Sand, gravel	$d_{15f} < 4d_{85b}$
15–40%	Silty and clayey sands	$d_{15f} < 0.7 + (40–FC)\ (4d_{85b}–0.7)/25$
40–85 %	Sands, silts, clays	$d_{15f} < 0.7$ mm
> 85 %	Fine silts, clays	$d_{15f} < 9d_{85b}$
FC: percentage of fines passing 75µm (%)		

Criteria for assessment purposes

Based on an extensive investigation of existing dams, Foster and Fell (2001) showed that the criteria previously defined include some built-in factors of safety. They proposed less conservative criteria adapted to the assessment of filter performance of existing hydraulic structures. Although it has been shown that other factors such as clay content influence the erosion behaviour of the filter, the D_{15}/d_{85} ratio is so dominant that these new criteria only use this ratio (Table 8.23).

Table 8.23 Filtering criteria of Foster and Fell (2001)

Base soil	Filter condition	
d_{95b} < 0.3 mm		$d_{15f} < 9\ d_{95b}$
0.3 mm < d_{95b} < 2 mm		$d_{15f} < 9\ d_{95b}$
d_{95b} > 2 mm	FC < 15%	$d_{15f} < 7\ d_{85b}$
	15% < FC < 35	$d_{15f} < 1.6\ (0.7 + (35–FC)\ (4\ d_{85b}–0.7)/20)$
	35% < FC < 85	d_{15f} < 0.7 mm
	FC > 85	$d_{15f} < 9\ d_{85b}$
FC = percentage of fines passing 75µm (%). Criteria are applicable if d_{95b} < 4.75 mm. Otherwise, passing has to be determined on the 0 mm to 4.75 mm fraction.		

Giroud's approach

Giroud (2003) suggested that the approach used for geotextile filters could also be used for granular filters. This approach leads to the graph shown in Figure 8.71 for the proposed retention criterion for granular filters in the case of a dense soil. The retention criteria proposed by Terzaghi, is represented by the horizontal dashed line in the graph. For large coefficients of uniformity, greater than five, Terzaghi's retention criteria may be unconservative. It is for this reason that truncation of the particle size distribution curve is traditionally employed in the design of granular filters. Truncation artificially decreases the coefficient of uniformity of the soil to compensate for this potential unconservatism in the case of high coefficients of uniformity. The graph shown in Figure 8.71, as proposed by Giroud (2003), is applicable regardless of the maximum particle size and is not limited to particles smaller than 4.75 mm.

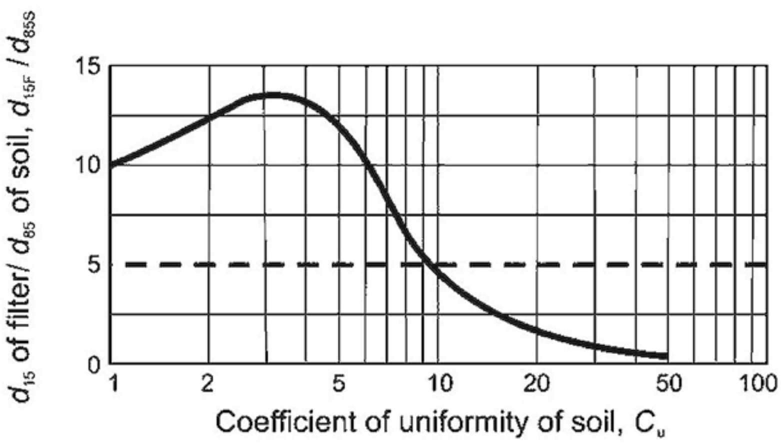


Figure 8.71 Retention criterion for granular filters for the case of dense soils (after Giroud, 2003)

Permeability requirements

The general requirements relative to permeability are $k_f > 3.5\ 10^{-5}$ m/s and $k_f/k_b > 25$. Considering the Vaughan and Soares' formula, this condition is equivalent to the following geometrical criteria:

$$d_{15f} > 0.1\,\mathrm{mm} \tag{8.137}$$

and

$$\frac{d_{15f}}{d_{15b}} > 5 \tag{8.138}$$

8.5.5.2 Geotextile filters

Geotextile filters have to be designed and installed carefully as defined in Chapter 9.

Soil retention requirements

The criterion for interface stability of a geotextile filter is generally formulated according to a geometrically tight principle. The filtration opening size of the geotextile filter O_{95} (Figure 8.72) should meet the following:

$$D_{min} \leq O_{95} \leq D_I \tag{8.139}$$

where:

D_{min} = largest fine particle being transported in suspension (mm)

D_I = indicative diameter of the soil particle to be filtered (mm)

Giroud *et al* (1998) estimated the minimum value $D_m$in $\approx 50\ \mu$m. The diameter of the particles to be filtered may be estimated (AFNOR, 1993):

$$D_I = C\, d_{85b} \tag{8.140}$$

where C is a coefficient depending on the state of the soil. For example, for a uniform soil ($C_u < 5$), the coefficient may be taken as $C = 0.4$ if the soil is in a loose state and $C = 0.6$ for a soil in a dense state. In the case of non-cohesive soils with uniformity coefficient $C_u > 5$, criteria proposed by Giroud (1988) may be used. If the soil is dense ($I_D > 50$ per cent) then:

$$D_1 = 18{C_u}^{-1.7} d_{85b} \tag{8.141}$$

If the soil is loose ($I_D < 50$ per cent), then:

$$D_1 = 9{C_u}^{-1.7} d_{85b} \tag{8.142}$$

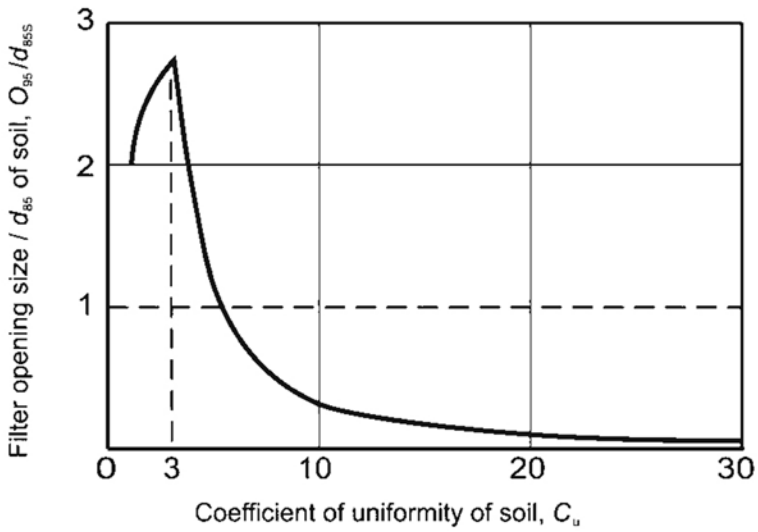


Figure 8.72 Retention criterion for geotextile filters for the case of dense soils (from Giroud, 1982)

Permeability requirements

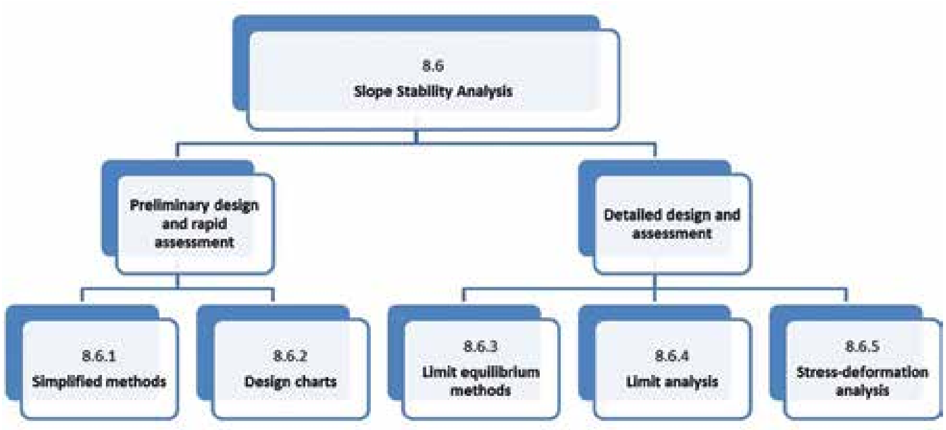
Because the geotextile permeability may be reduced considerably during its lifetime (blocking, clogging etc) the target values of permeability are generally much higher than those required for granular filters. The permeability requirements are defined in terms of a permeability ratio as shown in Table 8.24.

Table 8.24 ***Permeability requirements for geotextile filters***

Type of structure	k_f/k_b
Coastal protection structures	≥ 100
Hydraulic structures	≥ 100
Standard dewatering trench	≥ 10

It is important to mention that the minimum values correspond to long-term reduced values.

8.6 SLOPE STABILITY



Slope sliding is one of the prevalent forms of instability encountered in levees and will be detailed in this section according to the flowchart. It is a 3D phenomenon in which a certain volume of soil moves down the slope under the influence of gravity and/or external actions. The sliding mass is bounded above by the surface of the slope and bellow by a surface of sliding (Figure 8.73) characterised by a discontinuity in strain and velocity field (it is in fact a transitional zone generally sufficiently thin to be considered as a surface as regards to the sling soil volume).

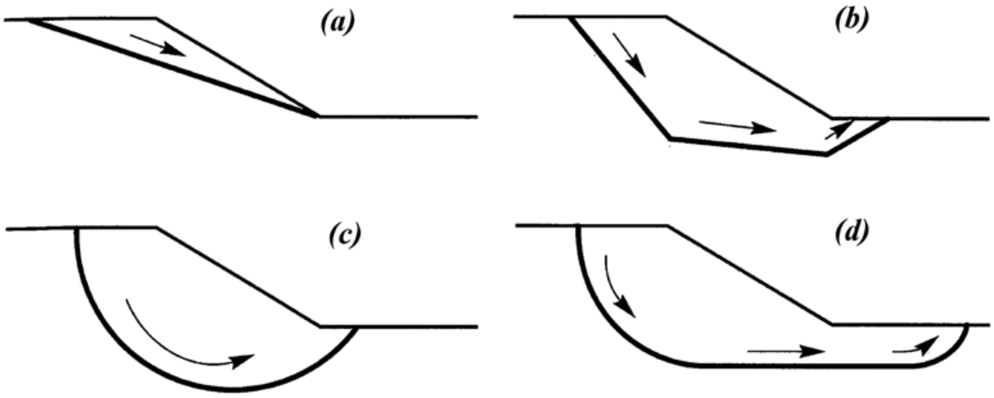


Figure 8.73 ***Common failure surface geometries: planar (a), multiplanar (b), circular (c), and noncircular (d)***

Theoretical analysis of homogeneous slope stability (Baker and Garber, 1978) leads to the determination of two families of possible critical slip surfaces. The first is a straight line, the second a logarithmic spiral. In reality, the homogeneous case is marginal and the failure surfaces often have different shapes, which mostly depend on the geometrical model, the geological context and the hydro-geological condition.

Slope stability analysis methods

The procedures for analysis of slope stability under static conditions are well-established. Currently, the most used methods of static slope stability analysis are:

- limit equilibrium analyses
- stress-deformation analyses.

Theoretical and practical comparison of some approaches has been provided in the literature (Jiang and Magnan, 1997, Yu *et al*, 1998, and Duncan, 1996). These approaches share some common features, and different theoretical backgrounds, which should be understood by engineers applying these methods.

2D versus 3D analysis

Most of the slope stability methods have been developed in the 2D plane strain context, but have also been extended to 3D. 3D analysis may be a more accurate representation of the critical failure surface, however, there are valid arguments to continue the use of 2D models in practice.

From a theoretical point of view, it has been proven that for a given slope the 3D factor of safety always exceeds the 2D factor of safety (Cavounidis, 1987). From a practical point of view, Duncan (1996) showed that this theoretical result was generally verified on actual cases and that in publications showing otherwise, significant inaccuracies and simplifying assumptions led to neglect of important aspects of the problem. In particular, it is noted that a 2D factor of safety is calculated for the most critical 2D section. Here, the use of any rule of thumb, such as a 10 per cent increase to compensate for the neglect of 3D effects, is not advisable in all cases because the ratio between the two may vary within a range of 1.0 to as high as 1.4 (Morgenstern, 1992, and Hungr *et al*, 1989).

Moreover, the validity of 2D slope stability methods has been demonstrated by back analysis of actual cases and models, as well as by extensive practical applications. From a numerical point of view, this generalisation to 3D models are still quite consuming in terms of resources and implies complementary assumptions (except for numerical methods), which may be difficult to calibrate and pose additional problems of numerical convergence.

Therefore, for all these reasons, the slope stability problem is generally simplified in a 2D problem in plane strain state. In this handbook, guidance and technical references have been developed for the 2D plane strain formulation. However, 2D analysis may need attention when estimating the strength of certain materials through back analysis (for example, in the diagnosis of an existing levee). Neglecting a strong 3D effect in the back analysis may result in a serious over-estimation of the back-calculated strength.

Effective versus total stress analysis

For given loading and drainage conditions the response of the soil may be considered as drained or undrained. In the undrained case, the analysis has to be performed in total stress, considering undrained shear strength parameters, whereas in the drained case, an effective stress analysis considering effective shear strength parameters is relevant. Given that the slope stability analysis methods do not presume the type of analysis performed, the shear strength parameters involved in their description may be either effective or undrained shear strengths (Duncan 1996).

Accuracy of the methods

The accuracy of a slope stability method depends on:

- governing parameters estimation, ie the accuracy with which the geological model, strength properties, pore water pressure and geometric conditions can be defined
- the inherent accuracy of the method of analysis
- the degree of understanding of the program by the engineer and ability to evaluate the results to avoid mistakes and misuse.

In most cases, the uncertainties related to definition of geometry, pore water pressures and soil properties are greater than those that arise from the approximations involved in the analytical technique. In this section, it is considered that the most accurate evaluation of the geometrical and geological model has been made and that the characteristic values of each soil layer have been determined (Chapter 7). The stability analysis conditions and the choice of the shear strength parameters to be used are also assumed to have been appropriately determined (Chapter 9). The tools concerning the determination of pore water pressure issues are addressed in different sections: pore water pressure build-up related to consolidation processes is treated in Section 8.7 concerning settlement analysis, and wave-induced pore water pressure is discussed in Section 8.3.2.

In the following sections, focus is given to the presentation of the methods of analysis in terms of their inherent accuracy to provide guidance for choosing an appropriate slope stability analysis method according to the need and the tools the engineer can mobilise. The presentation follows a tiered approach presenting the different alternatives from the simplest (stability charts and simplified methods) to the most complex (numerical analysis).

8.6.1 Simplified methods

The simplified methods may be used as preliminary verification in the case of levees resting on soft soils. They should be completed by limit equilibrium or stress-deformation methods according to the relevant geotechnical standards.

8.6.1.1 At-rest pressure approach

The at-rest earth pressure method is used to estimate the potential for lateral spreading and horizontal sliding of an embankment, as shown in Figure 8.74.

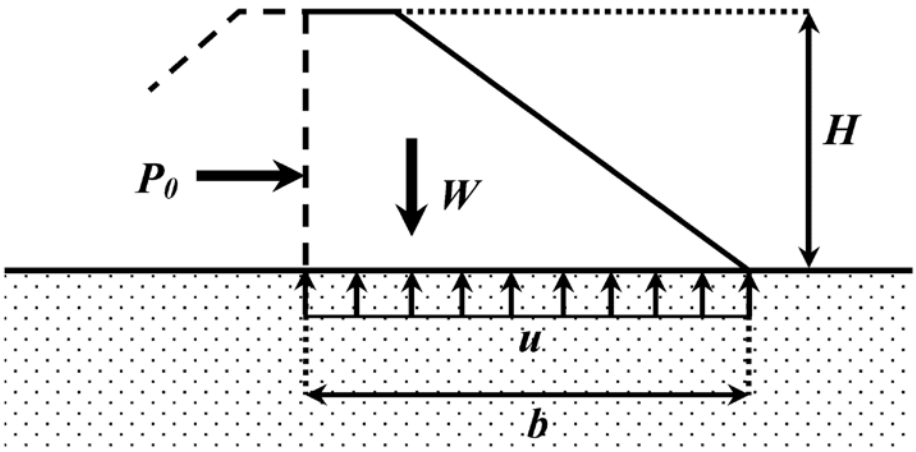


Figure 8.74 At rest pressure approach for stability analysis

The method compares the at-rest earth pressure, P_0, on a vertical plane through the embankment to the shear resistance along the base of the embankment. The method is only partly a limit equilibrium method, because the at-rest earth pressures are calculated independently of any equilibrium conditions and then compared to the limiting shear resistance. The safety factor is expressed as:

$$F_s = 2\frac{cb + (W - ub)\tan\varphi}{K_0\gamma H^2} \qquad (8.143)$$

where:

c	=	cohesion along the embankment-foundation contact (kPa)
φ	=	friction angle along the embankment-foundation contact (°)
u	=	average pore water pressure along embankment-foundation contact (kPa)
K_0	=	at-rest earth pressure coefficient (-)
γ	=	unit weight of embankment (kN/m^3)
d	=	half width of the levee (m)
H	=	height of the levee above foundation (m)

Ensuring that an embankment has an adequate factor of safety by this analysis will assist in limiting deformation where two or more materials with significantly different stress-strain behaviour are present. A common example application is a zoned levee with a clay core.

8.6.1.2 Bearing capacity approach

The concept of bearing capacity of the foundation refers to a criterion of shear failure of the foundation for punching failures. These are failures of the foundation soil characterised by the fact that the embankment collapses while undergoing traction. The failure of the foundation is general because it concerns the entire width of the embankment. The failure pattern of the foundation soil is similar to that which occurs under a shallow foundation, and can be studied as such.

The bearing capacity methods are limited to homogeneous foundations where simple bearing capacity equations are applicable. These methods are also used primarily for evaluating short-term, undrained stability of embankments resting on soft, saturated clay foundations. These methods are intended only for preliminary analyses and for use as an approximate check of more rigorous and thorough analyses.

This simple bearing capacity approach ignores the shear strength of the embankment fill and is conservative in this respect. Because the shear strength of the embankment material is ignored, questions about incompatibility between the stress-strain behaviour of the embankment and the foundation do not arise. Although more sophisticated approximations can be made, bearing capacity analyses should not be considered to be a substitute for detailed slope stability analyses.

When new levees or projects of heightening of existing levees are concerned, the worst case is generally the end of construction (short-term situation). So, the stability check should assume the embankment being built instantly, without dissipation of pore pressures in the foundation: the short-term undrained characteristics have to be considered.

The bearing capacity limit state is defined by the same methodology as the one concerning rigid footings stability, considering the design vertical stress $q = \gamma H$. The limit pressure on a soil with undrained cohesion C_u can be written:

$$q_f = N_c c_u \tag{8.144}$$

where:

N_c = a factor function of b/t

b = is the half-width of the embankment

t = the thickness of soft cohesive foundation

Several authors have established bearing capacity factors (Prandtl, Terzaghi, Meyerhof, Hansen, Vesic, Mandel and Salençon etc) in function of unit weight, cohesion and friction angle of a uniform semi-infinite soil layer, but generally in the case of a rigid footing. Michalowski (1993) proposed a solution taking into account a finite thickness of soft cohesive foundation t. The boundary condition at the contact with the soft soil embankment is characterised by the parameter χ defined as:

$$\chi = \frac{\tau_m}{c_u} \tag{8.145}$$

where τ_m is the mean shear stress at the base of the embankment, and the roughness of the contact soft soil-substratum is characterised by the parameter κ defined as:

$$\kappa = \frac{c_0}{c_u} \tag{8.146}$$

where c_0 is the shear resistance at the base interface.

The bearing capacity factor N_c may then be determined from Figure 8.75. This figure shows that for a perfect contact interface between the embankment and its foundation ($\chi = 0$) and a semi-infinite foundation layer (b/t = 0), the N_c factor takes the classical value of $\pi + 2$.

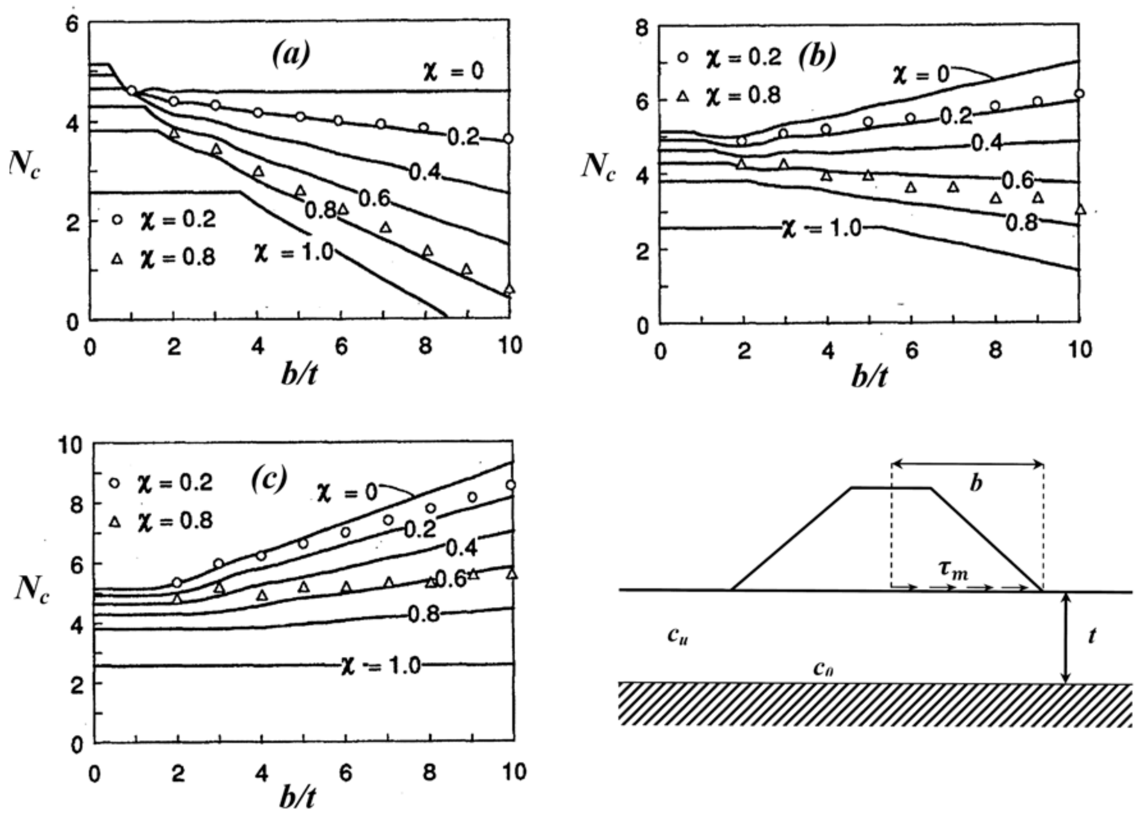


Figure 8.75 ***Dimensionless limit load q/c for outward horizontal loads on the foundation layer, homogeneous soil: smooth base (a), base interface strength equal to half of the shear strength of the soil (b), perfectly rough base (c). Solid lines indicate the numerical solution and bullets mark the closed-form solution (from Michalowski, 1993)***

8.6.2 Design charts

Slope stability charts provide a means for rapid analysis of slope stability. They can be used for preliminary analyses, for checking detailed analyses, or for complete analyses. They are especially useful for making comparisons between design alternatives, because they provide answers so quickly. The accuracy of slope stability charts is usually as good as the accuracy with which shear strengths can be evaluated.

In this section, chart solutions are presented for four types of slopes:

- slopes in soils with $\varphi = 0$ and uniform strength throughout the depth of the soil layer
- slopes in soils with $\varphi > 0$ and $c > 0$ and uniform strength throughout the depth of the soil layer
- infinite slopes in soils with $\varphi > 0$ and $c = 0$ and soils with and $\varphi > 0$ and $c > 0$
- slopes in soils with $\varphi = 0$ and strength increasing linearly with depth.

Using approximations in slope geometry and carefully selected soil properties, these chart solutions can be applied to a wide range of nonhomogeneous slopes.

Averaging procedure

For simplicity, charts are developed for homogenous soil conditions with simplified slopes. To apply these to complex conditions, it is necessary to approximate the real conditions with an equivalent simplified slope. The most effective method of developing a simple slope profile for chart analysis is to begin with a cross-section of the slope drawn to scale. On this cross-section, using judgment, draw a geometrically simple slope that approximates the real slope as closely as possible.

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Shear strength averaging

To average the shear strengths for chart analysis, it is useful to know the location of the critical slip surface. The charts contained in the following parts of this section provide a means of estimating the position of the critical circle. Average strength values are calculated by drawing the critical circle, determined from the charts, on the slope. Then the central angle of arc subtended within each layer or zone of soil is measured with a protractor. The central angles are used as weighting factors to calculate weighted average strength parameters, c* and φ^* as follows:

$$c^* = \frac{\sum \theta_i c_i}{\sum \theta_i} \qquad (8.147)$$

$$\varphi^* = \frac{\sum \theta_i \varphi_i}{\sum \theta_i} \qquad (8.148)$$

where:

c^* = average cohesion (kPa)
φ^* = average angle of internal friction (°)
θ_i = central angle of arc, measured around the centre of the estimated critical circle, within zone *i* (°)
c_i = cohesion in zone *i* (kPa)
φ_i = angle of internal friction in zone *i* (°)

To average the unit weights for use in chart analyses, it is usually sufficient to use layer thickness as a weighting factor, as indicated by the following expression:

$$\gamma^* = \frac{\sum h_i \gamma_i}{\sum h_i} \qquad (8.149)$$

where:

γ^* = average unit weight (kN/m³)
γ_i = unit weight of layer *i* (kN/m³)
h_i = thickness of layer *i* (m)

Unit weights should be averaged only to the depth of the bottom of the critical circle. If the material below the toe of the slope is a $\varphi = 0$ material, the unit weight should be averaged only down to the toe of the slope, since the unit weight of the material below the toe has no effect on stability in this case.

Use of the charts

The slope stability charts were developed by Janbu (1973)as described following:

- for purely cohesive soils refer to Figure 8.76
- for $\varphi > 0$ soils refer to Figure 8.77
- for frictional soils refer to charts providing adjustment factors for surcharge loading at the top of the slope as shown in Figure 8.78
- charts providing adjustment factors for submergence and seepage are shown in Figure 8.79
- charts providing adjustment factors to account for tension cracks are shown in Figure 8.80.

First, the engineer has to decide which cases should be investigated. For uniform soil conditions, the critical circle passes through the toe of the slope if the slope is steeper than about 1H/1V. For flatter slopes, the critical circle usually extends below the toe, and is tangent to some deep firm layer. The chart in Figure 8.76 can be used to compute factors of safety for circles extending to any depth. Multiple possibilities should be analysed, to be sure that the overall critical circle and overall minimum factor of safety have been found. The following criteria can be used to determine which possibilities should be examined:

- if a soil layer is weaker than the one above it, the critical circle may be tangent to the base of the lower (weaker) layer. This applies to layers both above and below the toe
- if a soil layer is stronger than the one above it, the critical circle may be tangent to the base of either layer, and both possibilities should be examined. This applies to layers both above and below the toe.

The following steps are performed for each circle:

- calculate the depth factor $d = D/H$ where D is the depth from the toe of the slope to the lowest point on the slip circle and H the slope height above the toe of the slope. The value of d is 0 if the circle does not pass below the toe of the slope. If the circle being analysed is entirely above the toe, its point of interaction with the slope should be taken as an 'adjusted toe', and all dimensions like *D*, *H*, and *Hw* has to be adjusted accordingly in the calculations
- find the centre of the critical circle using the charts at the bottom of Figure 8.76
- determine the average value of the strength for the circle considered, using the previously developed averaging procedure
- calculate the quantity P_d using the formula:

$$P_d = \frac{\gamma H + q + \gamma_w H_w}{\mu_q \mu_w \mu_t} \tag{8.150}$$

where:

γ = average unit weight of homogenous soil (kN/m³)
H = slope height above toe (m)
q = surcharge (kPa)
γ_w = unit weight of water (kN/m³)
H_w = height of external water level above toe (m)
μ_q = surcharge adjustment factor (-), see Figure 8.78
μ_w = submergence adjustment factor (-), see Figure 8.79
μ_t = tension crack adjustment factor (-), see Figure 8.80

- use the chart at the top of Figure 8.76, determine the value of the stability number, N_0, which depends on the slope angle, β, and the value of d. The factor of safety can be estimated following the formula:

$$F_s = N_0 \frac{c}{P_d} \tag{8.151}$$

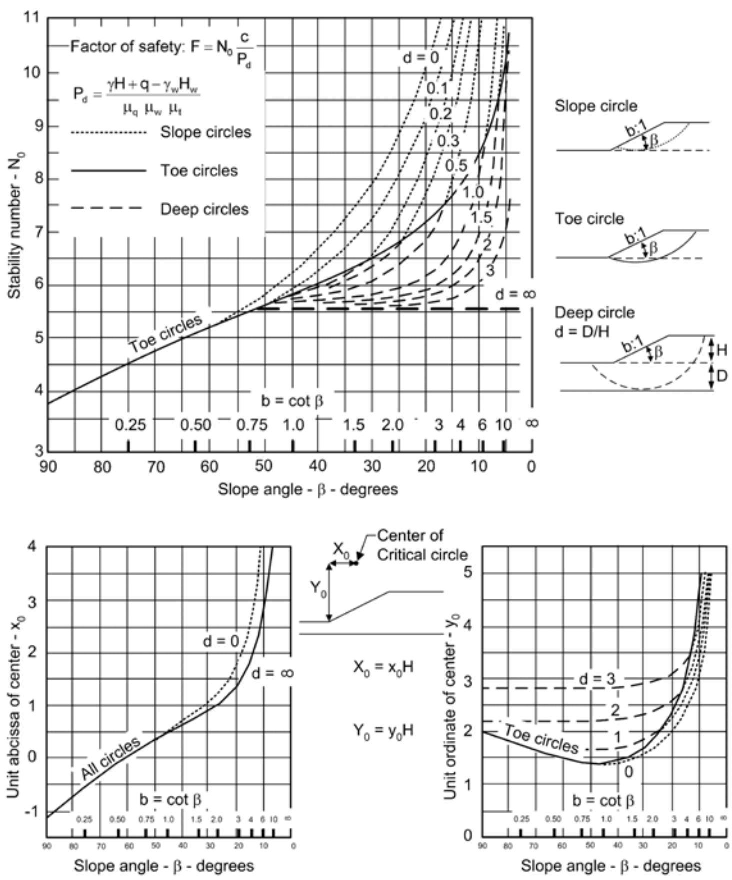
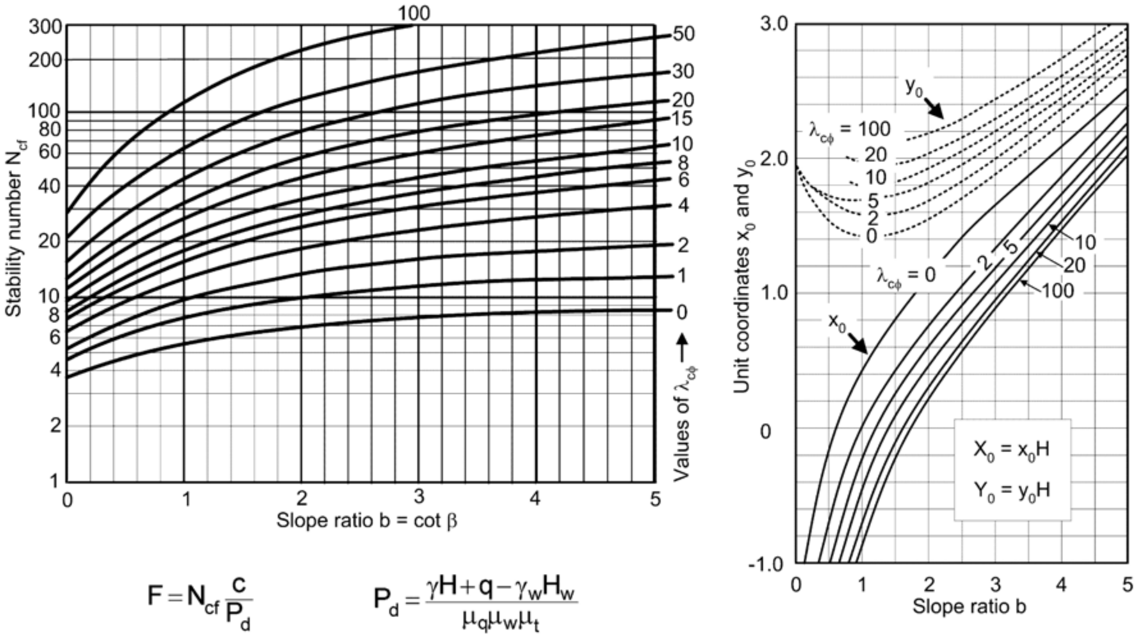


Figure 8.76 Slope stability chart for purely cohesive soils (from USACE, 2003)



$$F = N_{cf}\frac{c}{P_d} \qquad P_d = \frac{\gamma H + q - \gamma_w H_w}{\mu_q \mu_w \mu_t}$$

$$\lambda_{c\phi} = \frac{P_e \tan\phi}{c} \qquad P_e = \frac{\gamma H + q - \gamma_w H_w'}{\mu_q \mu_w'}$$

Stability numbers and center coordinates for circles passing through the toe of the slope.

Figure 8.77 Slope stability chart for φ > 0 soils (from USACE, 2003)

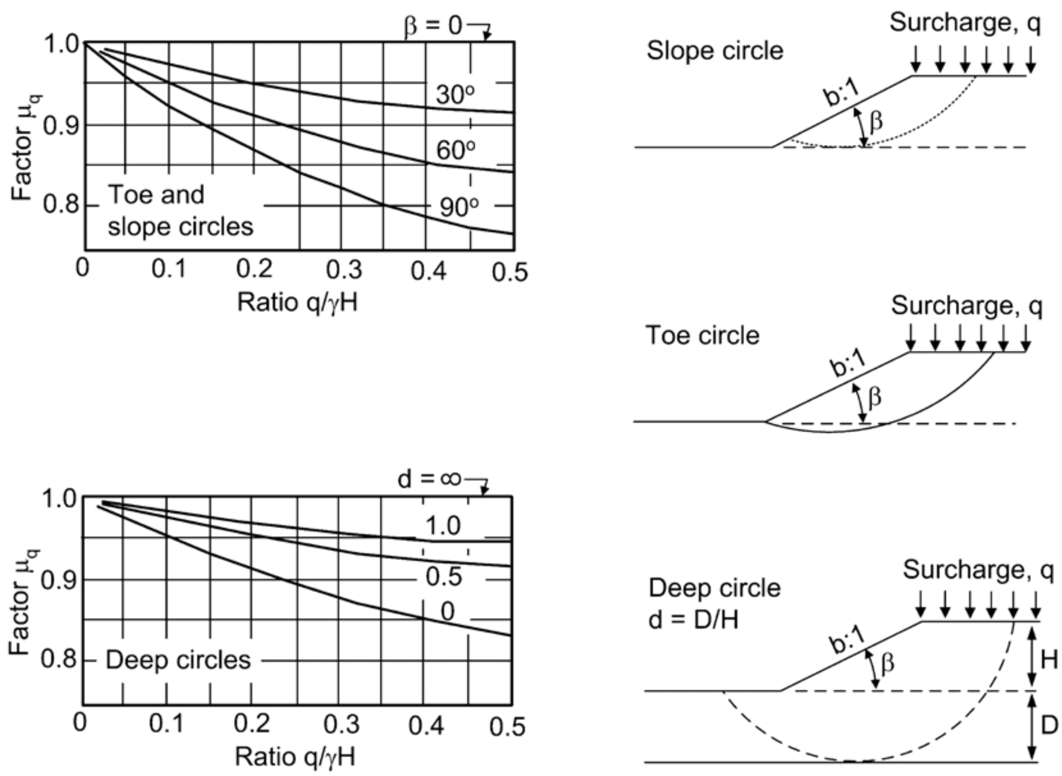


Figure 8.78 Surcharge adjustment factors (from USACE, 2003)

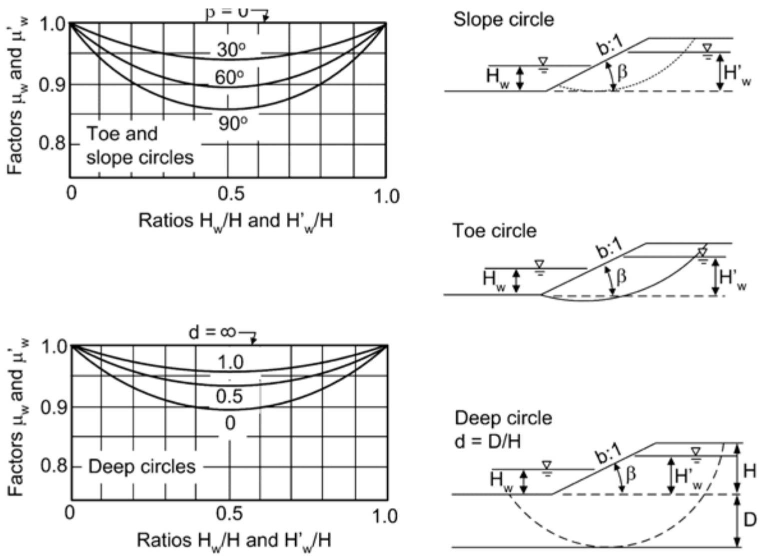


Figure 8.79 Submergence and seepage adjustment factors (from USACE, 2003)

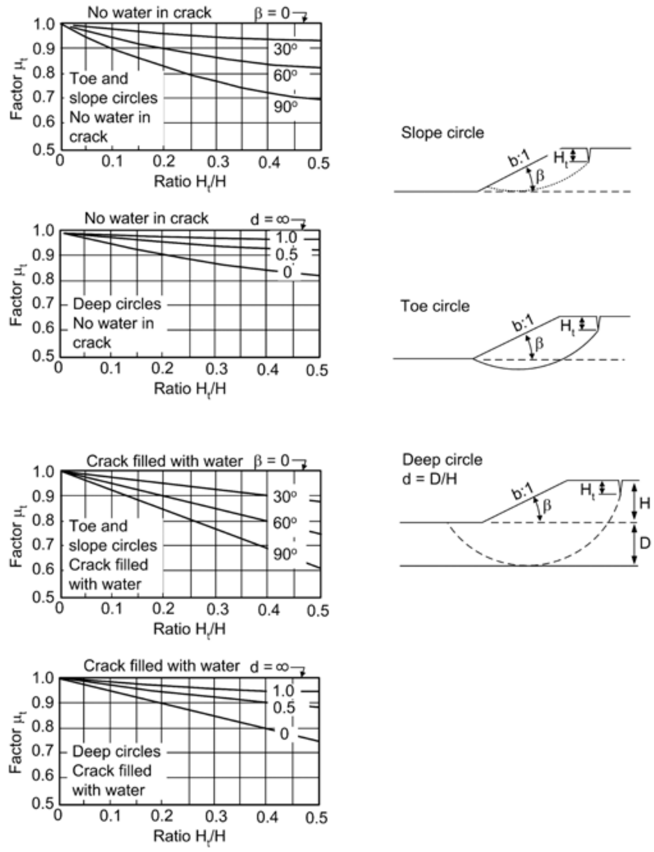


Figure 8.80 Tension crack adjustment factors (from USACE, 2003)

For frictional soils, one has to calculate the P_d parameter and then calculate the parameter P_e using the formula:

$$P_e = \frac{\gamma H + q + \gamma_w H'_w}{\mu_q \mu'_w} \tag{8.152}$$

where:

H_w' = height of water within slope (m)

μ_w' = seepage correction factor (-)

H_w' = the average level of the piezometric surface within the slope. For steady seepage conditions this is related to the position of the phreatic surface beneath the crest of the slope as shown in Figure 8.81. If the circle being studied passes above the toe of the slope, H_w' is measured relative to the adjusted toe.

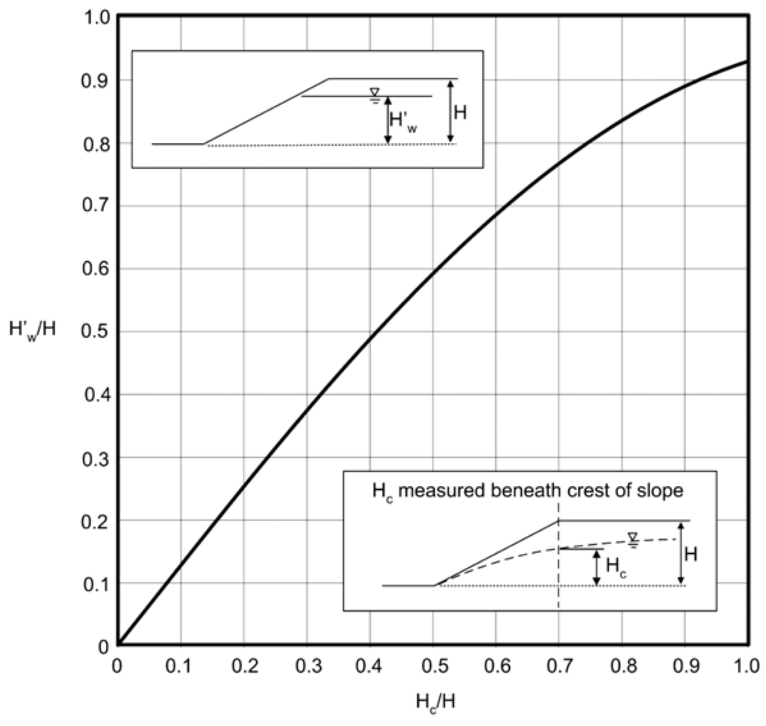


Figure 8.81 Steady seepage adjustment factor for φ > 0 soils (after Duncan et al, 1987)

The default values of adjustment factors are $\mu_w' = 1$ if there is no seepage and $\mu_q = 1$ if there is no surcharge. In a total stress analysis, internal pore water pressure is not considered, so $H_w' = 0$ and $\mu_w' = 1$ in the formula for P_e. Calculate the dimensionless parameter P_c using the formula:

$$\lambda_{c\varphi} = P_e \frac{\tan\varphi}{c} \tag{8.153}$$

where:

φ = average value of φ (°)

c = average value of c (kPa)

Then, it is possible to estimate the factor of safety, F_s, using the formula:

$$F_s = N_{cf} \frac{c}{P_d} \tag{8.154}$$

8.6.3 Limit equilibrium methods

Limit equilibrium analysis method has been the most popular method for slope stability calculations. A major advantage of this approach is that complex soil profiles, seepage and a variety of loading conditions can be easily handled. Using a global equilibrium condition, the limit equilibrium approach is purely static and neglects the plastic flow rule of the soil. In the limit equilibrium approach, it is postulated that the slope might fail by mass of soil sliding on a failure surface. These methods have been widely used for assessing the stability of natural or man-made slopes. These methods were successively developed in order to deal with circular or arbitrary shaped slip surfaces. The common features of limit equilibrium methods are as follows:

- the problem is considered as 2D in plane strain formulation
- the Mohr-Coulomb failure criterion is assumed
- the factor of safety is defined in reference to a given slip surface as a ratio between the shear strength of soil and the shear stress required for equilibrium of the sliding body

- the strength of the slip surface is mobilised to the same degree to bring the sliding body into a limiting state. The overall slope and each part of it are in static equilibrium
- the factor of safety estimation is based on force and/or moment equilibrium equations.

If the soil at failure is assumed to be a rigid, perfectly plastic material obeying an associated flow rule, then collapse mechanisms selected by the limit equilibrium method are usually kinematically inadmissible. In addition, the static admissibility of the stress field is not satisfied because some arbitrary assumptions are made to remove the static indeterminacy and, in some methods, only a global equilibrium condition (rather than equilibrium conditions at every point in the soil) is satisfied. The different limit equilibrium methods may be merged into three groups:

1. **Analytical and graphical methods:** explicitly solved (even manually) methods based on the hypothesis of a simple shape of slip surface. These are the simplest methods and are useful for first approximation calculations.
2. **Slices and blocks methods:** iteratively solved methods based on the decomposition of the sliding mass into slices or blocks and requiring assumptions regarding interslice forces to solve the non-linear implicit problem.
3. **Perturbations methods:** explicitly solved methods based on assumptions regarding the normal stress distribution along the slip surface.

Although some of the methods presented (Table 8.25) in this section are not widely used in engineering practice, they are given to cover most of the methods implemented in commercial software, in order to provide a wide range of users useful tools for slope stability analyses.

Table 8.25 Characteristics of limit equilibrium procedures

Procedures		Equilibrium conditions satisfied			Shape of slip surface
		V	*H*	*M*	
Analytical and graphical methods	Infinite slope	x	x		planar
	Culmann	x	x		planar
	Swedish			x	circular
	Wedge method	x	x		three segments
Slice methods	Fellenius	x		x	circular
	Bishop simplified	x		x	circular
	Van's method	x	x	x	one segment and two arcs of circle
	Carter	x	x		any
	Janbu simplified	x	x		any
	USACE	x	x		any
	Lowe-Karafiath	x	x		any
	Spencer	x	x	x	any
	Morgenstern-Price	x	x	x	any
	Janbu rigorous	x	x	x*	any
Multi-block method	Sarma	x	x	x	any
Perturbation methods	Bell, Faure, Zhu	x	x	x	any non-planar

Notes

V = vertical equilibrium, *H* = horizontal equilibrium, *M* = global moment equilibrium

* moment equilibrium satisfied for each individual slice

Some slope stability analysis methods were developed based on variational calculus. However, in view of the fact that both practical results and theoretical basis are questionable (Duncan, 1996) it appears that these types of approaches have not resulted in significant advancement to the practical state of art for slope stability analysis. Also, this technique is mathematically complex and very few calculation tools exist using this type of approach. So, this handbook does not explain the theoretical details of this approach.

Because of the approximate and somewhat arbitrary nature of limit equilibrium analysis, there is often concern about how accurate these types of solutions are. However, limit equilibrium methods have shown great accuracy in geotechnical engineering with justification procedures and guidelines largely making reference to required factors of safety calibrated as regards to this approach (USACE, 2003).

8.6.3.1 Analytical and graphical methods

- **Infinite slope model**

  This method assumes that the slope is of infinite lateral extent and that sliding occurs along a plane surface parallel to the surface of the slope. Solving the problem requires vertical and horizontal equilibrium of the vertical block as shown in Figure 8.82. The factor of safety may be expressed as:

$$F_s(z) = \frac{c + \gamma z\,(1 - k\tan\beta)\cos^2\beta\tan\varphi}{\gamma z\cos\beta\,(\sin\beta + k\cos\beta)} \tag{8.155}$$

where:

z = vertical soil depth (m)

β = inclination of the slope from the horizontal (°)

c = soil cohesion (kPa)

φ = soil internal friction angle (°)

γ = unit weight of soil (kN/m³)

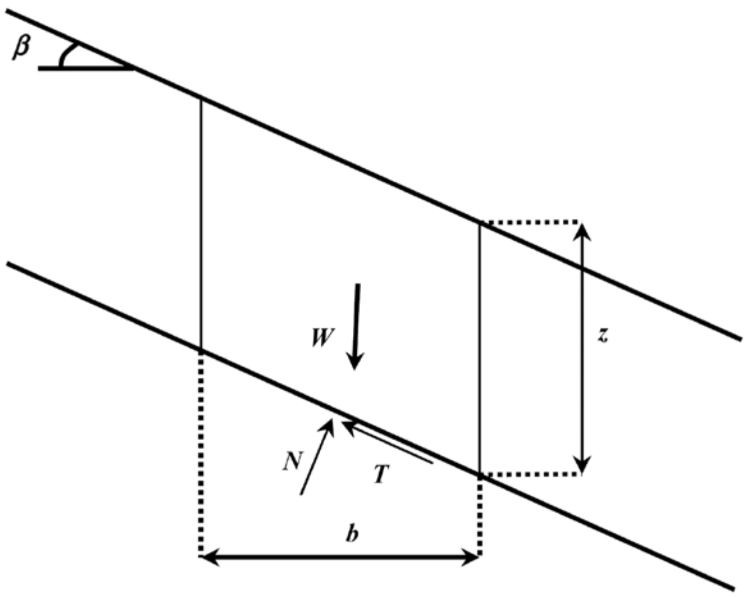


Figure 8.82 Infinite slope model

Real slopes are not infinite up and down. When the thickness of the sliding mass is not negligible as regards to its length, active and passive wedges may be introduced.

Culmann method

Culmann analysis is based on the assumption that the failure of a slope occurs along a plane when the average shearing stress tending to cause the slip is more than the shear strength of the soil, Figure 8.83. Consider a failure surface defined by an angle θ with the horizontal plane, the plane of length l elimiting a sliding mass and the associated linear weight W are dependent on the θ angle and the factor of safety may be expressed as:

$$F_s(\theta) = \frac{cl(\theta) + (W(\theta)\cos\theta - U(\theta))\tan\varphi}{W(\theta)\sin\theta} \tag{8.156}$$

where U is the water pressure applied on the failure surface.

The minimum factor of safety is obtained when the derivative of the safety factor function becomes null. This null criterion defines the optimum angle θ^* giving the minimum factor of safety $F_s(\theta^*)$.

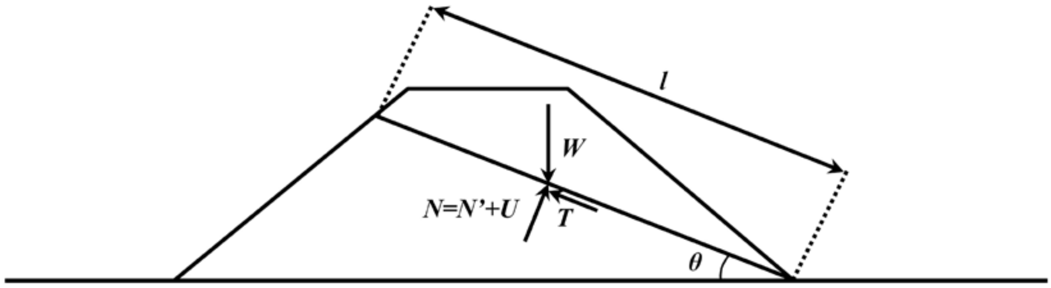


Figure 8.83 Culmann model for planar sliding surface

Swedish method

This method is the simplest circular analysis used to analyse the short-term stability for both homogeneous and non-homogeneous slopes. It assumes that a rigid cylindrical block fails by rotation about its centre (x_0, y_0) and the soil is assumed to be purely cohesive ($\varphi = 0$) (Figure 8.84).

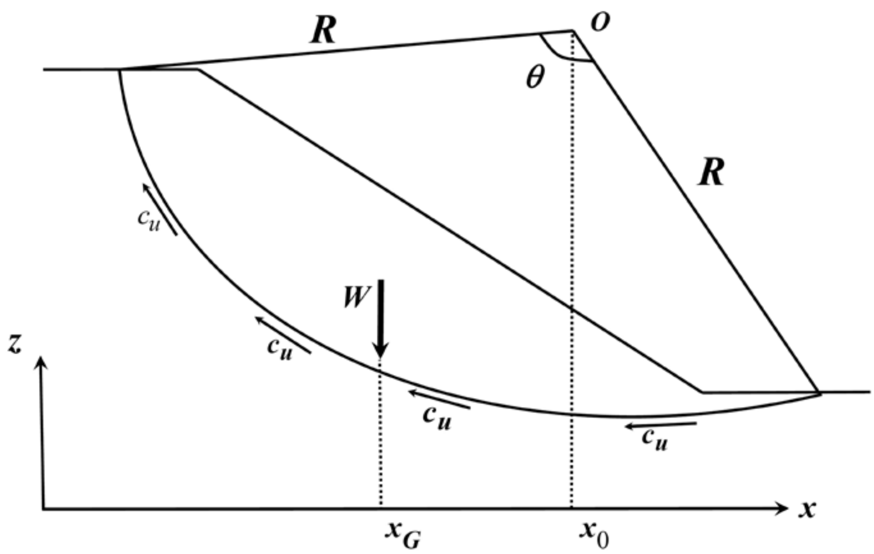


Figure 8.84 Swedish circle method model (φ = 0)

The factor of safety is defined in terms of moment equilibrium:

$$F_s = \frac{c_u R^2 \theta}{W(x_G - x_0)} \tag{8.157}$$

where:

c_u = undrained shear strength (kPa)
R = radius if circular slip surface (m)
θ = angle between entry and exit of slip surface (°)
W = weight of soil mass above sliding surface (kN)
x_G = abscissa of centre of gravity of soil mass (m)

Wedge method

The Wedge method assumes that the sliding mass is composed of three regions, the active wedge, the central block, and the passive wedge (Figure 8.85). The inclination angles of the forces on the vertical boundaries between the zones are assumed. The Wedge method is actually a special case of the force equilibrium procedure. The Wedge method fully satisfies equilibrium of forces in the vertical and horizontal directions and ignores moment equilibrium.

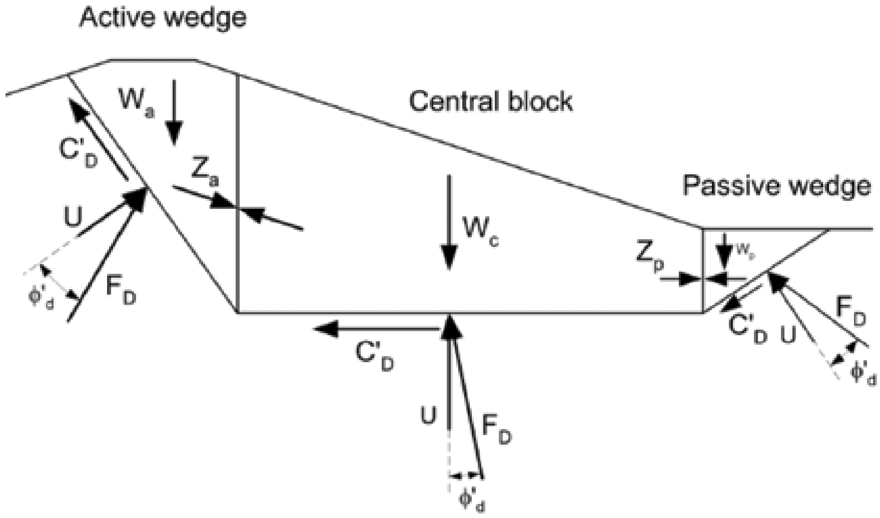


Figure 8.85 ***Block decomposition of the Wedge method***

The Wedge method has the same limitations as other force equilibrium procedures. In addition, the specific 'wedge' shape of the slip surface restricts use of the procedure to slopes where slip surfaces of this shape are likely to be critical. Factors of safety calculated using the Wedge method are sensitive to the assumed inclinations of the side forces. The Wedge method may be used to check Spencer's solutions for three-part noncircular shear surfaces. In this case, the side force inclination is taken as the same side force inclination found in Spencer's approach.

8.6.3.2 Slice methods

The conventional methods of slices involve division of the sliding body into n vertical slices. Figure 8.86 shows the different notations used in the methodology.

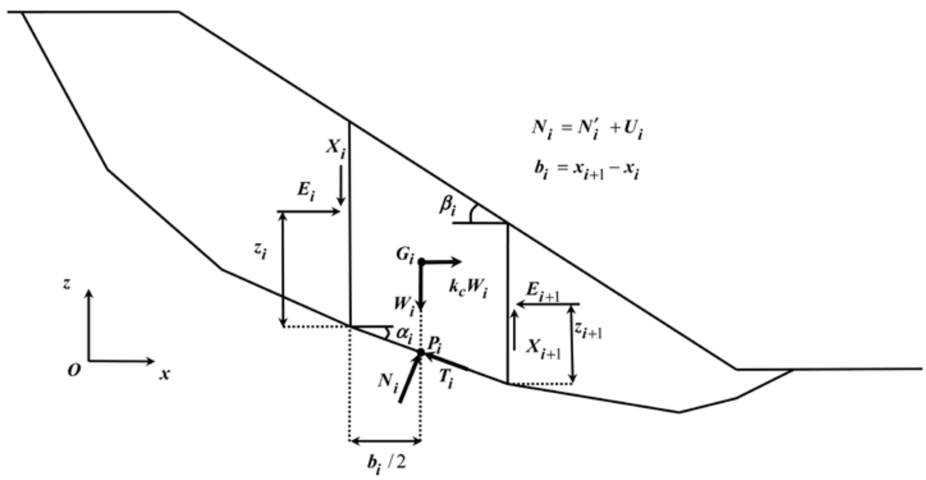


Figure 8.86 ***General slice method model***

Problem determination

The verification of vertical, horizontal and moment equilibrium for all slices gives $3n$ equations. The unknowns are:

- n values of the normal reaction on the slice base N_i
- n values of the location of l_i
- n-1 values of vertical interslice forces X_i
- n-1 values of horizontal interslice forces E_i
- n-1 locations of interslice forces z_i
- *1* value of safety factor F_s.

However, following the limit equilibrium principle, the values of tangential reaction T_i must also be accounted for in the slice at limit equilibrium. In total, there are 5n–2 unknowns. So, for more than one slice or block, the number of unknowns exceeds the number of equations by 2n–2. In conventional slice methods, the number of unknowns is reduced by considering that the normal reaction on the slice base acts at the centre of the base ($l_i = b_i/2$), assuming that it introduces very little uncertainty, which is often the case when the slices are narrow. In the same manner, the horizontal gravity centre is often assumed to be vertical to the centre of the slice. These assumptions leave n–2 non trivial assumptions required to make the number of equations balance the number of unknowns.

Solving the problem consists of making as many assumptions as the equilibrium conditions chosen to verify. The slice methods differ in:

- the static equations employed in deriving the factor of safety equation
- the assumptions used to render the problem determinate.

The methods of slices have become the most common methods due to their ability to accommodate complex geometries and variable soil and water pressure conditions. Also, their implementation in commercial codes contributed greatly to their popularity among the geotechnical community.

General framework

From the general slice model represented on the Figure 8.86, the following equations concerning vertical and horizontal equilibrium of each slice can be written:

$$N_i \cos\alpha_i + T_i \sin\alpha_i = W_i - (X_{i+1} - X_i) \tag{8.158}$$

$$N_i \sin\alpha_i - T_i \cos\alpha_i = k_c W_i - (E_{i+1} - E_i) \tag{8.159}$$

The relation between the normal and tangential reaction forces is given by the limit equilibrium condition so that according to the definition of the factor of safety:

$$T_i = \frac{1}{F_s}\left\{c_i b_i + (N_i - U_i)\tan\varphi_i\right\} \tag{8.160}$$

The normal reaction equation may be expressed as:

$$N_i = \frac{W_i + \dfrac{1}{F_s}(U_i \sin\alpha_i \tan\varphi_i - c_i b_i) + (X_{i+1} - X_i)}{\cos\alpha_i + \dfrac{\sin\alpha_i \tan\varphi_i}{F_s}} \tag{8.161}$$

The limit equilibrium condition may be written from a global horizontal force point of view:

$$F_s = \frac{\sum_{i=1}^{n} \cos\alpha_i \left\{c_i b_i + (N_i - U_i)\tan\varphi_i\right\}}{\sum_{i=1}^{n}\left\{k_c W_i + N_i \sin\alpha_i\right\}} \tag{8.162}$$

or from a global moment point of view:

$$F_s = \frac{\sum_{i=1}^{n} (y_{Pi}\cos\alpha_i + x_{Pi}\sin\alpha_i)\left\{c_i b_i + (N_i - U_i)\tan\varphi_i\right\}}{\sum_{i=1}^{n}\left\{W_i(x_{Gi} + k_c y_{Gi}) + N_i(x_{Pi}\cos\alpha_i - y_{Pi}\sin\alpha_i)\right\}} \tag{8.163}$$

Note

The interslice assumptions do not appear explicitly in the global equilibrium conditions. However, determination of the normal reaction, N_i, depends on the assumptions made on the interslice forces. The limit equilibrium methods differ on the assumptions made concerning interslice forces.

Given that these assumptions have n-1 interslice force relationships, the problem becomes over-determinate. Some methods render the problem determinate by only verifying one of the global equilibrium conditions (force or moment). But more rigorous methods introduce one more degree of freedom into the relationship between the vertical and horizontal components of the interslice forces by assuming the general pattern:

$$X_i = (\lambda f_i + g_i)\, E_i \tag{8.164}$$

where $f_i = f(x_i)$and $g_i = g(x_i)$ are assumed functions of x and λ a coefficient to be determined (Chen and Morgenstern, 1983). The prescribed functions that were proposed in the literature are constant (Spencer, 1967), half-sine (Morgenstern and Price, 1965), extended half-sine (Zhu *et al*, 2006), clipped sine, trapezoid, data-point specified, and others.

Some authors have developed methods using other relationships defining directly a shear interslice force function $X_i = \lambda f_i$ (Pan, 1980, Madej, 1984, and Correia, 1988). These methods will not be detailed because theoretically there is no guarantee that the failure criterion is not violated along the interslice boundary, and these methods are not implemented in commercial codes. Finally, there is need to mention Sarma's method (Sarma, 1973), which considers all equilibrium equations and assumes that every interslice boundary is also at limiting equilibrium. The resolution procedure may be analogous to the Morgenstern and Price method. The resolution of the problem consists in determining the λ value for which the factors of safety given by the force equilibrium and the moment equilibrium are equal. The search of the λ parameter can be a trial and error one (Fredlund and Krahn, 1977) or guided by moment equilibrium of each individual slice (Zhu *et al*, 2005).

Assumptions that are made for each of the slice methods are presented in Table 8.26. Examples of selected methods are presented in Boxes 8.17 to 8.20.

Table 8.26 Assumptions referring to the different slice methods

Slice method procedures	Assumptions
Fellenius	Interslice forces neglected
Bishop simplified	Resultant of interslice forces horizontal
Carter	Resultant of interslice forces horizontal
Janbu simplified	Resultant of interslice forces horizontal and correction factor to account interslice shear force
USACE	Direction of resultant interslice forces parallel to the ground surface
Lowe-Karafiath	Direction of resultant interslice forces equal to the average of the ground surface and slope of the base of the slip surface
Spencer	Resultant interslice forces are of constant slope throughout the sliding mass
Morgenstern-Price	Direction of the interslice forces defined using an arbitrary function
Janbu rigorous	Location of the horizontal interslice force is defined by an assumed line of thrust
General multi-block	The shear strength is mobilised on the sides of all inclined slices

Box 8.17 *Ordinary slice method*

This method, first developed by Fellenius (1936) is applicable to circular slip surfaces. The method assumes that the interslice forces can be neglected because they are thought to be parallel to the base of each slice. So, the normal reaction at the base of each slice may be written as:

$$N_i = W_i\left(\cos\alpha_i - k_c\sin\alpha_i\right) \tag{8.165}$$

The factor of safety is then simply derived from summation of moments about a common point (either a fictitious or real centre of rotation of the sliding mass). Given that the slip surface is circular, the moment produced by the normal force equals zero. So, the explicit expression of the factor of safety is obtained:

$$F_s = \frac{\sum_{i=1}^{n}\left\{c_i b_i + (N_i - U_i)\tan\varphi_i\right\}}{\sum_{i=1}^{n} W_i\left(\sin\alpha_i + k_c\cos\alpha_i\right)} \tag{8.166}$$

It is important to see that under these assumptions, Newton's principle of 'action equals reaction' is not satisfied between the slices. The indiscriminate change in direction from one slice to the next result in factor of safety errors, which may be as much as 60 per cent (Whitman and Bailey, 1967).

Also, from the tangential reaction expression, note that there are situations where the shear reaction may be negative (when r_u is close to 1). This implies that there is a negative shear stress on the base of the slice, which is physically impossible. In order to complete the calculation, one may set the shear stress to zero for all such slices, but this may result in substantial errors. As shown by Bishop (1955), the computed factor of safety is generally too small and errors may be as much as 20 per cent.

Box 8.18 *Bishop's simplified method*

In this method (Bishop, 1955), the slip surface is also assumed to be an arc of circle and the normal force is assumed to be at the centre of the base of each slice. So, *n–2* additional assumptions are required to make the problem determinate. Bishop proposed to neglect the shear interslice terms (X_{i+1}–$X_i = 0$), considering that the discrepancy introduced by this assumption is usually much less than the probable error in measured values of shear strength characteristics. Then, the vertical force equilibrium equation leads directly to the normal force by:

$$N_i = \frac{W_i + \dfrac{1}{F_s}\left(U_i\sin\alpha_i\tan\varphi_i - c_i b_i\right)}{\cos\alpha_i + \dfrac{\sin\alpha_i\tan\varphi_i}{F_s}} \tag{8.167}$$

Considering a circular slip failure, the factor of safety can be calculated using equation 8.166 with the normal force, N_i, as determined in equation 8.167. However, this relation is no longer explicit (F_s appears on both sides of the equation) so that the calculation has to be performed iteratively.

Carter method

The Carter method has been developed to generalise the Bishop's method to a general form of slip surfaces. Carter (1971) noticed that the global momentum equilibrium tends to equate to the global horizontal equilibrium when the centre of rotation is taken high enough. In this method, the factor of safety is determined from the global horizontal equilibrium of the sliding mass and applied to any slip surface.

Box 8.19 *Van's method (from Van, 2001)*

For many embankments built on soft deposits with a relatively rigid, permeable sand layer underneath, failure may be induced by the uplift mechanism. A high water level in the river or estuary in front of the embankment may generate high pore pressures in the sand layer under and behind the embankment. Consequently, the shear stresses at the interface between the sand layer and the soft deposits are reduced, eventually to zero, in case of actual uplift of the soft deposits, and failure along a relatively deep sliding plane may occur as indicated in Figure 8.87.

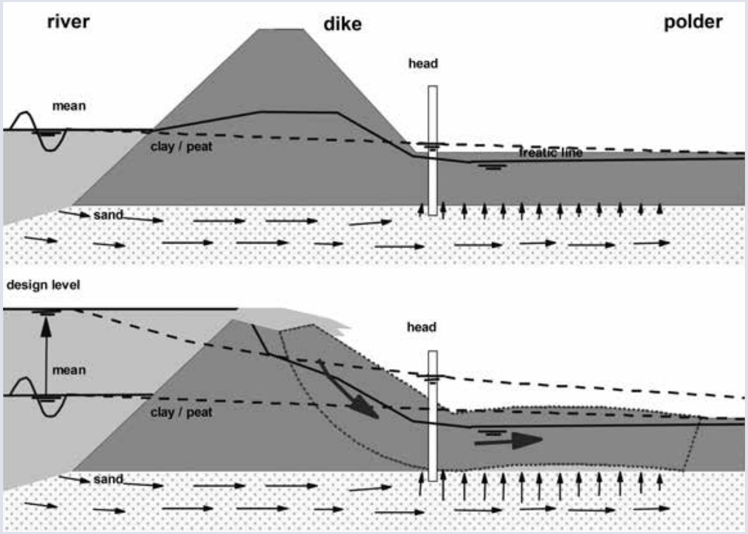


Figure 8.87 ***Uplift induced embankment failure***

In the lower parts of the Netherlands, the uplift phenomenon turns out to be the dominant failure mechanism for the majority of the embankments if the rather high design water levels are applied. The standard approach in the Netherlands for checking stability is a circular slip surface (Bishop method). But in the case of uplift, the zone in which the shear stresses are reduced most significantly is hardly included in a circular analysis.

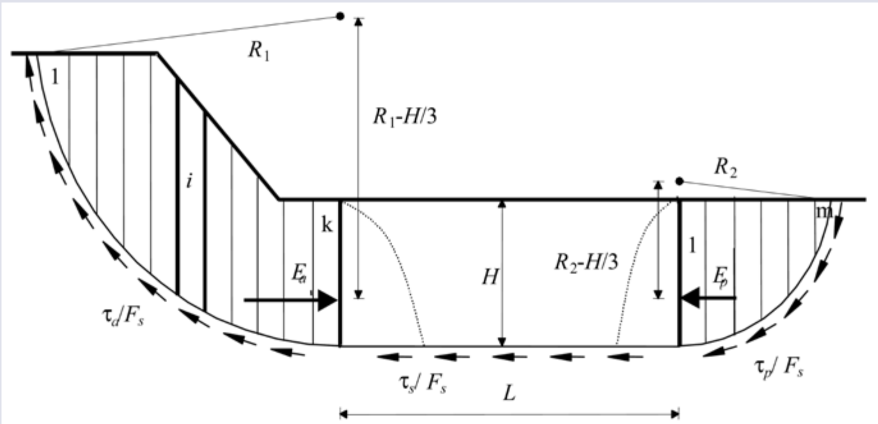


Figure 8.88 ***Van's slip surface model***

In the method presented by Van (2001) the principles of Bishop's simplified method are applied to determine the stability factor of the slip plane shown in Figure 8.88. In accordance with Bishop's method, the safety criterion applies to the stability factor being the lowest denominator of the shear stress τ along the sliding plane, which results in equilibrium. The inter-slice horizontal forces E_a and E_p are supposed to act at one-third of the beam segment height above the sliding plane, which is a safe assumption. The horizontal and momentum equilibrium conditions lead to the following expression of the factor of safety (Van *et al*, 2005):

$$F_s = \frac{\dfrac{\sum_{i=1}^{k}\left\{\tau_i \dfrac{b_i}{\cos\alpha_i}\right\}}{1-\dfrac{H}{3R_1}} + \dfrac{\sum_{j=1}^{m}\left\{\tau_j \dfrac{b_j}{\cos\alpha_j}\right\}}{1-\dfrac{H}{3R_2}} + \tau_s L}{\sum_{i=1}^{k}\gamma_i h_i b_i \sin\alpha_i + \sum_{j=1}^{m}\gamma_j h_j b_j \sin\alpha_j} \qquad (8.168)$$

For $R_1 = R_2$ and $L = 0$ the method is equal to Bishop's method. In the more general case, some of the geometrical limitations of Bishop's method are relaxed, as required for an accurate description of the uplift mechanism, while the approach is consistent with a model that has turned out to be accurate in practice in cases where the slip surface is indeed more or less circular. In both the Bishop and Van methods the stability factor needs to be calculated by iteration. Fortunately, in both methods, convergence proceeds without any complications.

Box 8.20 Janbu rigorous procedure (from Janbu, 1973)

This method considers all the force and moment equilibrium conditions by assuming the location of the thrust line $z(x)$ (generally about one-third of the slice height). In order to solve the factor of safety, the interslice forces have to be evaluated. For the first iteration, the shear forces are set to zero ($X_i = 0$). For subsequent iterations the interslice forces are computed from the moment equilibrium of individual slice about the centre of the slice base, which is assumed to be the point of application of the normal force N_i. As the width of the slice is assumed to be infinitesimal, some terms are becoming negligible and a recurrence relationship can be exhibited on the X_i values. The horizontal interslice forces, E_i, are obtained by combining horizontal equilibrium equation and vertical equilibrium and the moment equilibrium of each slice being satisfied, so the force equilibrium given by Equation 8.162 should be considered.

Generalisation to multi-block failure analysis

Sarma (1973) was the first to generalise the approach of slices to inclined slice interfaces. The inclinations of slices are chosen so that a kinematic slip mechanism can develop. Since these inclinations are not known in advance, one may start with assumed inclined planes where sliding can take place inside the mass and later iterate to find a critical set. This approach may be seen as a generalisation of the Wedge method presented earlier.

Even if the mass contained within the slip surface is in a state of limiting equilibrium, the mass will not be able to move unless shear surfaces are formed within the body (Figure 8.89). To fulfil the kinematic compatibility condition, the inclinations of slices may be chosen so that a reasonable kinematic slip mechanism can develop.

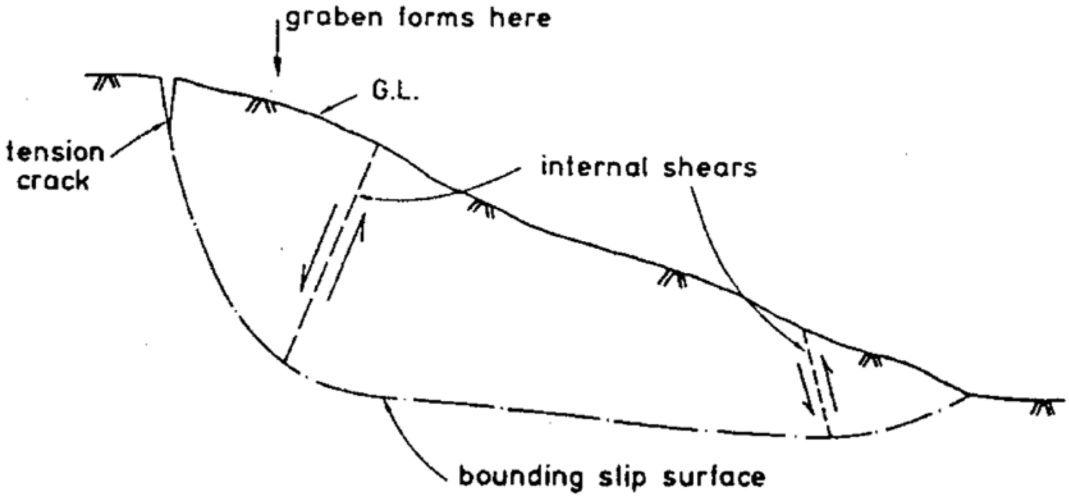


Figure 8.89 Typical internal shear required to permit movement in a non-circular slide (from Hutchinson, 1987)

Since inclinations of sliding interfaces within the sliding mass are not known in advance, one may start with assumed inclined planes and later iterate to find a critical set. Once a decomposition of the sliding mass is done, assume that the body forces X and E on the slice boundaries are such that they are in a state of limit equilibrium. It is then possible to write, for each i^{th} interslice boundary:

$$X_i = \frac{1}{F_s}\{(E_i - P_{w,i})\tan\widetilde{\varphi}_i + \widetilde{c}_i b_i\} \tag{8.169}$$

where d is the length of the inclined interslice boundary and P_w the force due to water pressure on the plane and the 'tilde' shear strength parameters are those averaged along the blocks interfaces. In this analysis, there are n–1 assumptions regarding the X_i and E_i relationship. In order to make the problem completely determinate, additional assumptions need to be made about the point of applications of all but one N_i normal force. Or, alternatively, points of applications of N_i can be determined by assuming the line of thrust of interslice forces. A suitable assumption may be to consider the point of application at the middle of the block base or vertical to the gravity centre.

As in the methods of slices, the solution obtained should satisfy the criterion of acceptability, ie all the N_i and T_i values should be positive. The values of z_i should lie within the slice, preferably in the middle third. Since the moment equilibrium equation does not play any part in the determination of the k_c, the slices can be as large as possible and in fact should be controlled by the change of inclination of the slip surface. The solution k_c depends on the assumed block decomposition of the sliding mass. The technique for finding the optimal sliding mass decomposition is a trial and error procedure.

Numerical difficulties

Computational difficulties may occasionally be encountered in solving the factor of safety equations. Three of the most common problems, which have been discussed in the literature (Ching and Fredlund, 1983) are:

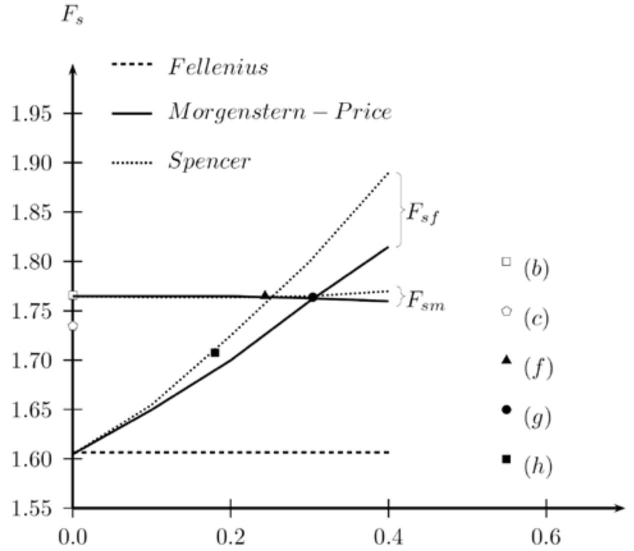
- unreasonably large and/or negative magnitude of the normal force on the base of the slice calculated as a result of the denominator term of N_i (Equations 8.162 and 8.163)approaching zero and/or going negative
- computation of a negative normal force on the base of a slice if the soil slope is highly cohesive
- convergence difficulties encountered when unreasonable side force function is assumed. For example, when the inter-slice force assumptions depend directly on the geometry of the problem (eg USACE, Lowe and Karafiath, 1960, and rigorous Janbu methods, 1973), some numerical difficulties may arise when the ground surface, the slip surface or the thrust line are not smooth, resulting in unreasonable discontinuity of interslice force distribution. To overcome this numerical difficulty smoothing techniques may be used (eg Zhu *et al*, 2003).

Several authors presented suggestions to resolve these difficulties (eg Ching and Fredlund, 1983). It is beyond the scope of this handbook to treat these questions. In cases of complex geometry, it is recommended to use two different methods to detect the potential numerical difficulties.

Accuracy comparison of slice methods

It should be noted that the limit equilibrium solutions are neither upper nor lower bound for the actual solutions. However, the solutions calculated within a rigorous context provide a rather narrow range for possible solutions. It has been pointed out in different studies (Fredlund and Krahn, 1997, Duncan, 1996, and Zhu *et al*, 2003) that the differences between factors of safety calculated by rigorous methods generally do not exceed ± six per cent. This limit is represented by the dotted line in Figure 8.90. This is certainly close enough for practical purposes, because slope geometry, water pressures, unit weights and shear strength can seldom be defined with accuracy as good as ± six per cent.

Thus, if an engineer performs slope stability analyses using methods satisfying all conditions of equilibrium of the sliding mass, it is justified in virtually every case to conclude that the accuracy of the analyses is as good as, or better than, the accuracy with which the analysis conditions are defined. The engineer can then devote his or her attention to the most important and most difficult issues involved in analyses of slope stability: those of defining geometry, shear strengths, unit weights and water pressures, and of determining the possible uncertainties in these quantities.



Notes

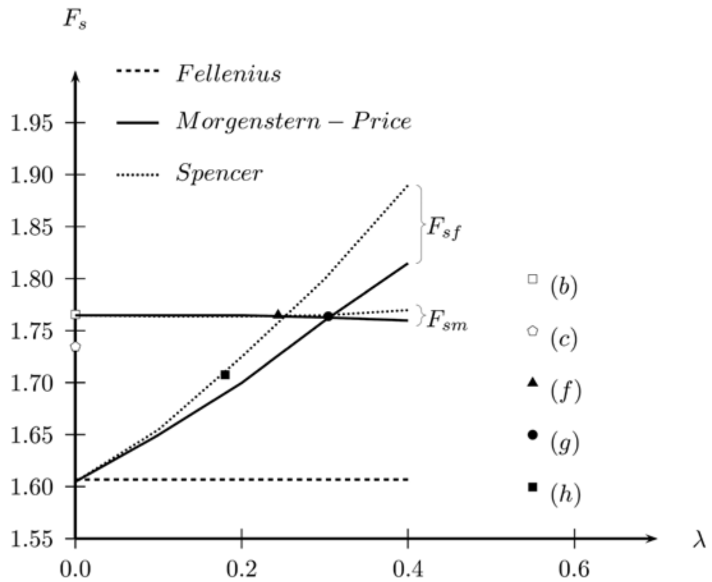
Fs* (solid line is Fs = Fs*)
(a) Ordinary method of slice
(b) Bishop simplified
(c) Janbu simplified
(d) Lowe-Karafiath
(e) USACE
(f) Spencer
(g) Morgenstern-Price
(h) Janbu Rigorous

Figure 8.90 ***Comparison of factors of safety obtained by different methods as regards to the mean factor of safety for rigorous methods (from Fredlund and Krahn, 1977, Zhu and Jiang, 2003, Yuang and Yamasaki, 1993, and Sinha, 2008)***

To understand the differences between the factors of safety determined from various methods, some authors have drawn plots of F_{sf} and F_{sm} as functions of λ values. Figure 8.91 shows the influence of interslice force assumptions on the computed factor of safety on the base of one example taken from Fredlund and Krahn (1977).

It is important to note that the factor of safety calculated using the ordinary method of slices (ie Fellenius method, 1936) is almost equal to the one calculated using the uncorrected simplified Janbu method (without the f_0 correction factor). The Janbu generalised method does not use an explicit λ factor, but given that this method is based on the force equilibrium equations, the Janbu rigorous method has been placed along the force equilibrium line to give an indication on the equivalent λ value.

The main observation to make is that the factor of safety obtained by the Spencer, Morgenstern-Price and Bishop methods are generally similar, ie the factor of safety based on the moment equilibrium has a small influence on the interslice forces assumptions. However, the factors of safety based on overall force equilibrium are far more sensitive to the side force assumptions (Figure 8.91).



Notes

(b) Bishop simplified
(c) Janbu simplified (corrected)
(h) Spencer
(g) Morgenstern-Price
(h) Janbu rigorous

Figure 8.91 ***Example of influence of interslice forces assumptions on the factor of safety***

Limitations and recommendations

The general remarks formulated on the different approaches are as follows:

- ordinary method of slices may be highly inaccurate for effective stress analyses of slopes with high pore pressures – the computed factor of safety is too low. The method is more accurate for purely cohesive soils in total stress analyses using circular slip surfaces. The method does not have numerical problems
- Bishop's simplified method is accurate for all conditions. Its limitations are that it is applicable only to circular slip surfaces and that numerical problems can be encountered under some conditions. If a factor of safety calculated using Bishop's method is smaller than the factor of safety for the same circle calculated using the ordinary method of slices, then it can be concluded that there are numerical problems with the Bishop's modified method analysis
- methods considering only force equilibrium conditions (eg Janbu simplified, USACE, Lowe-Karafiath) are sensitive to the assumed inclinations of side forces between slices. A poor assumption regarding side force inclination can result in a serious error in the computed factor of safety. These methods are inclined to have numerical problems
- methods satisfying all conditions of equilibrium of the sliding mass (eg Janbu rigorous, Spencer, Morgenstren-Price, Sarma) are generally accurate for any condition and slip surface forms. However, all of these methods have numerical problems under some conditions.

8.6.3.3 Perturbation methods

Other approaches consist of trying to estimate directly the normal stress distribution along the slip surface. That is the aim of the perturbation methods. A typical slope profile with a general-shaped slip surface is presented in Figure 8.92. In this 2D, the cross-section of the slope is visualised as having a unit length. The sliding body is bounded by the ground surface $y = g(x)$ and the slip surface $y = s(x)$.

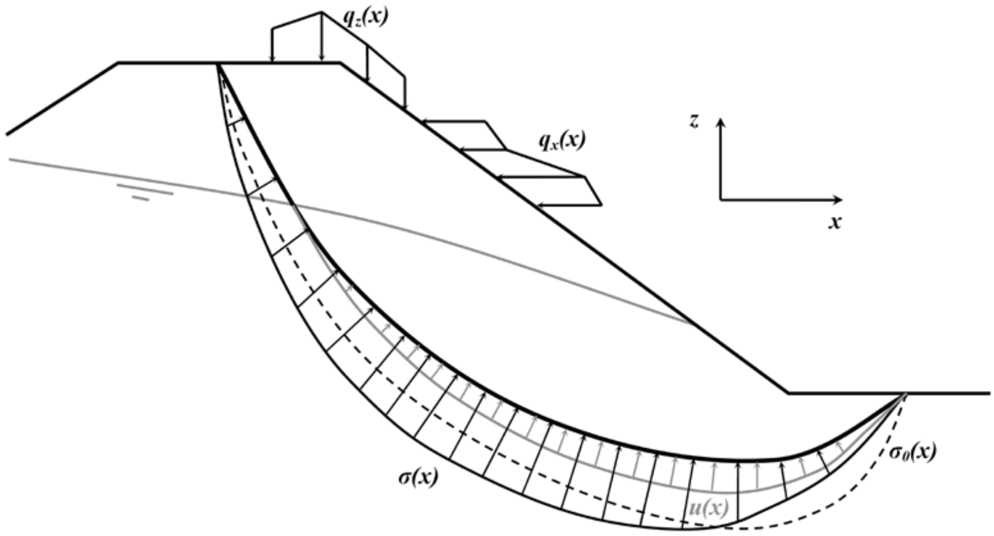


Figure 8.92 Geometry of slope stability model – sliding mass

By assigning a constant factor of safety, F_s, to the whole failure surface the sliding body is brought into a limiting state under the combined action of:

- $w(x)$: unit weight of the soil
- $k_c\, w(x)$: horizontal seismic force
- $u(x)$: pore water pressure along the slip surface
- $q_x(x)$, $q_z(x)$: horizontal and vertical surcharges or reinforced pressures.

The normal and tangential stresses on the slip surface are σ *(x) and* σ *(x)* respectively. In terms of effective stresses, the Mohr-Coulomb failure criterion is given by:

$$\tau = \frac{1}{F_s}\{c + (\sigma - u)\tan\varphi\} \tag{8.170}$$

where σ and c are friction angle and cohesion respectively, u is the pore water pressure.

When the sliding body is in an equilibrium state, three equations have to be verified: horizontal force equilibrium, vertical force equilibrium and overall moment equilibrium about a selected centre (x_c, y_c).

Solving the problem consists of determining the couple ($s(x)$,$\sigma(x)$) that minimise the safety factor F_s. In limit equilibrium methods, assume the shape of the slip surface $s(x)$ and look for stress distribution $\sigma(x)$ and F_s parameter consistent with the equilibrium equations. The minimum value of the factor of safety is obtained by exploration of kinematically admissible slip surfaces. In the perturbation method, statically admissible normal stress distribution is determined by assuming an *a priori* normal stress distribution $\sigma_0(x)$ and 'perturb' it to satisfy the required equations of equilibrium. It can be shown (Baker and Garber, 1978) that every $\sigma(x)$ function that has at least two degrees of freedom can satisfy the three equilibrium equations. The modifying function should then involve two auxiliary unknowns (λ, μ) to make the problem determinate. The general normal stress function is put in the form:

$$\sigma = \lambda\sigma_\lambda + \mu\sigma_\mu + \sigma_\delta \tag{8.171}$$

where σ_λ, σ_μ and σ_Δ are determinate functions depending on x. The verification of equilibrium equations leads to a 3D linear system in terms of λ, μ and F_s. The condition of existence of the solution leads to a polynomial cubic equation in terms of F_s, which can be analytically solved in the simplest cases. In the more complex cases the resolution is made based on the discretisation of the slip surface and linear interpolation of integrals. The accuracy of the result depends on the assumed functions σ_λ, σ_μ and σ_δ. Different functions have been proposed in the literature. The most commonly used methods consider the reference functions:

$$\sigma_\lambda = \frac{w\left(1 - k_c s'\right) + q_z - s' q_x}{1 + s'^2} \tag{8.172}$$

and

$$\sigma_\delta = 0 \tag{8.173}$$

In Bell's method (Bell, 1966), the function σ_μ is defined as a sin function:

$$\sigma_\mu = \sin\left(2\pi\frac{x-a}{b-a}\right) \tag{8.174}$$

Whereas in the method proposed by Faure (1985) the function σ_μ depends on the reference function.

$$\sigma_\mu = s'\sigma_\lambda \tag{8.175}$$

There is also a method proposed by Zhu and Lee (2002), for which the components are expressed as cubic polynomial functions.

Equivalence with slice methods

The direct approach presented here is related to the slice procedures developed earlier in the case of infinitesimal width slices. To check the reasonableness of the normal stress distribution, some verification may be useful. When the factor of safety has been obtained, the λ and μ parameters and the normal stress distribution are known. So, the horizontal and vertical forces, $E(x)$ and $T(x)$ respectively, may be obtained by considering horizontal and vertical force equilibrium conditions of the sliding slice from a to x, while the moment of forces acting on the same part of the sliding mass about a centre of rotation (x_0, y_0) gives the position of the point of action of the internal force. If the point of action lies within the interslice boundary the solution is statically reasonable.

8.6.3.4 Shape of the slip surface

All of the limit equilibrium methods require that a potential slip surface be assumed in order to calculate the factor of safety. In practice, calculations are repeated for a sufficient number of trial slip surfaces to ensure that the minimum factor of safety has been calculated. For computational simplicity the candidate slip surface is often assumed to be circular or composed of a few straight lines. However, the slip surface will need to have a more complicated shape in complex stratigraphy. The assumed shape is dependent on the problem geometry and stratigraphy, material characteristics (especially anisotropy), and the capabilities of the analysis procedure used. Commonly assumed shapes are as follows:

Planar: failures occurring along a planar surface are generally relevant for very steep slopes or specific geological contexts (thin weak layers).

Circular: observed failures in relatively homogeneous materials often occur along curved failure surfaces. A circular slip surface is often used because it is convenient to sum moments about the centre of the circle, and because using a circle simplifies the calculations. A circular slip surface should be used in the ordinary method of slices and Bishop simplified method. Circular slip surfaces are almost always useful for starting an analysis. Also, circular slip surfaces are generally sufficient for analysing relatively homogeneous embankments, or slopes and embankments on foundations with relatively thick soil layers.

Wedge: 'wedge' failure mechanisms are defined by three straight line segments defining an active wedge, a central block, and a passive wedge. This type of slip surface may be appropriate for slopes where the critical potential slip surface includes a relatively long linear segment through a weak material bounded by stronger material. A common example is a relatively strong levee embankment founded on weaker, stratified alluvial soils. Wedge methods, including methods for defining or calculating the inclination of the base of the wedges, are discussed in the following section.

General, non-circular shape: slope failure may occur by sliding along surfaces that do not correspond to either the wedge or circular shapes. The term general slip surface refers to a slip surface composed of a number of linear segments that may each be of any length and inclined at any angle. The term noncircular is also used to describe such general-shaped slip surfaces. Recently improved search techniques and computer software have increased the capability to analyse such slip surfaces. Stability analyses based on general slip surfaces are now much more common and are useful as a design check of critical slip surfaces of traditional shapes (circular, wedge) and where complicated geometry and material conditions exist. It is especially important to investigate stability with noncircular slip surfaces when soil shear strengths are anisotropic.

8.6.3.5 Location of the critical slip surface

A full slope stability analysis generally comprises evaluation of the critical slip surface for which the factor of safety is minimal. Because different analysis procedures employ different assumptions, the location of the critical slip surface may vary among different methods of analysis. The critical slip surface for a given problem analysed by a given method is found by a systematic procedure of generating trial slip surfaces until the one with the minimum factor of safety is found. Searching schemes vary with the assumed shape of the slip surface and the computer program used. Common schemes are discussed as follows.

Circular slip surfaces: a circular surface, Figure 8.93, is defined by three parameters that may be:

- centre co-ordinates (X_c, Y_c) and radius (R). The trial is generated on a grid of centre points and eventually on radii
- centre co-ordinates (X_c, Y_c) and a point through which the circle must pass (X_p, Y_p). The trial relies on the definition of a grid of centre points, the radius being given by the definition of the anchor point
- centre co-ordinates (X_c, Y_c) and a plane to which the circle should be tangent. The trial also relies on the definition of a proper grid of centre points, the radius being given by the tangent line.

In the case of homogeneous slopes and circular slip surfaces, Jiang *et al* (2003) provided a chart for critical slip surface location. Depending on the shear strength parameters, he defined ranges of values for which shallow toe circles (which exit directly through the toe of the slope), deep toe circles (which pass below the toe of the slope before exit at the toe of the slope) or deep base circles (DB), are the most critical (which pass below the toe of the slope and exit down the slope). This kind of analysis could be useful for a qualitative verification of the credibility of the critical slip surface location.

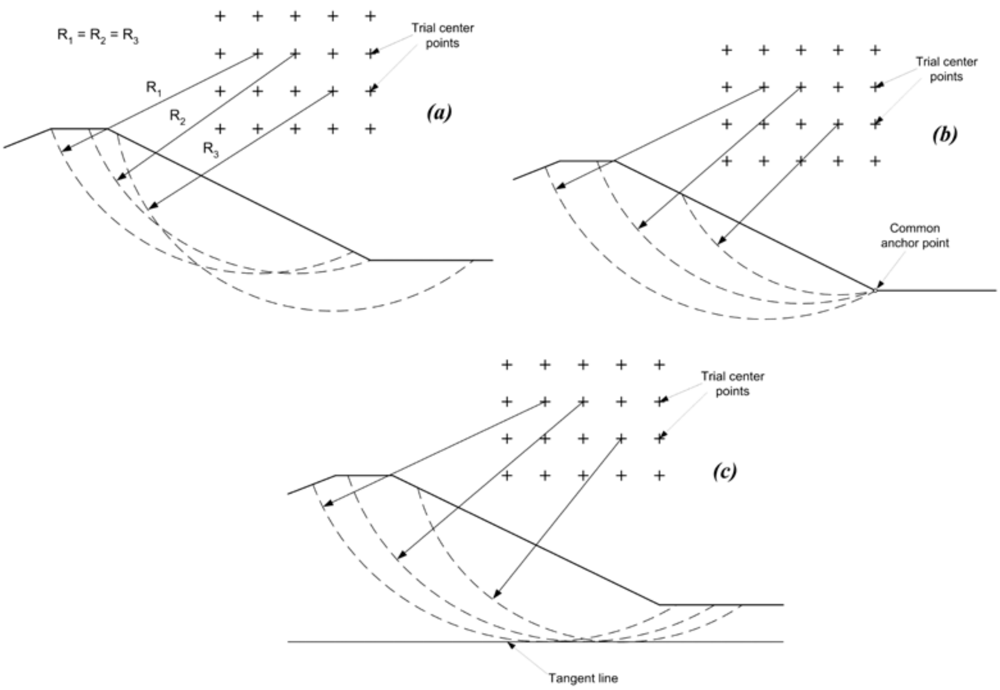


Figure 8.93 Different types of searching patterns for circular slip surfaces

Wedge-shaped slip surfaces: wedge-shaped slip surfaces require searching for the critical location of the central block and for the critical inclination of the bases of the active and passive wedges. Searching for the critical location of the central block involves varying systematically horizontal and vertical co-ordinates of the two ends of the base of the central block, until the central block corresponding to the minimum factor of safety is found. For each trial position of the central block, the bases of inclinations of the active and passive wedge segments should be set based on searching critical inclinations (Figure 8.94).

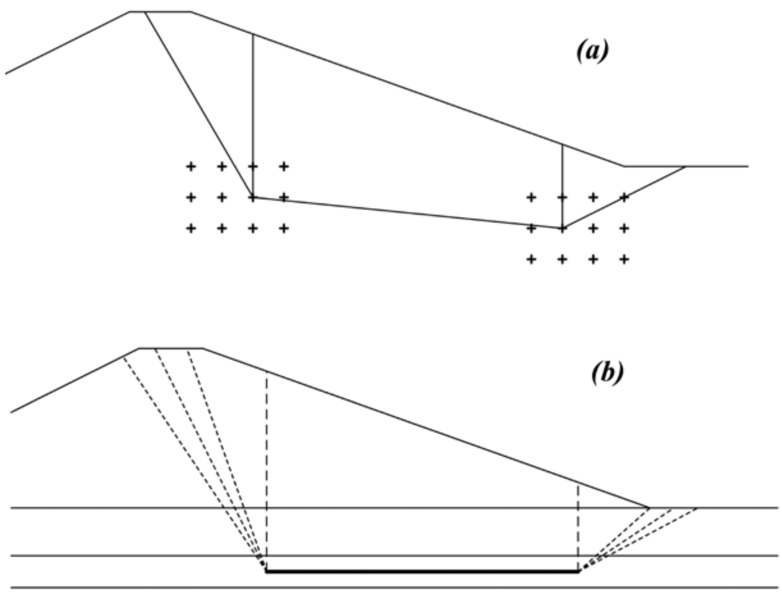


Figure 8.94 Different types of searching patterns for wedge slip surfaces: search scheme for critical central block (a), and search scheme for wedge inclinations (b) (from USACE, 2003)

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General shapes: a number of techniques have been proposed and used to locate the most critical general-shaped slip surface. One of the most robust is the one developed by Celestino and Duncan (1981), shown in Figure 8.95. In this iterative method, an initial slip surface is assumed and represented by a series of points that are connected by straight lines. The factor of safety is first calculated for the assumed slip surface. Next, all points except one are held fixed, and the 'floating' point is shifted a small distance in two directions. The directions might be vertically up and down, horizontally left and right, or above and below the slip surface in some assumed direction. The factor of safety is calculated for the slip surface with each point shifted as described. This process is repeated for each point on the slip surface.. Once all points have been shifted in both directions and the factor of safety has been computed for each shift, a new location is estimated for the slip surface based on the computed factors of safety. The slip surface is then moved to the estimated location and the process of shifting points is repeated. This process is continued until no further reduction in factor of safety is noted and the distance that the shear surface is moved on successive approximations becomes minimal.

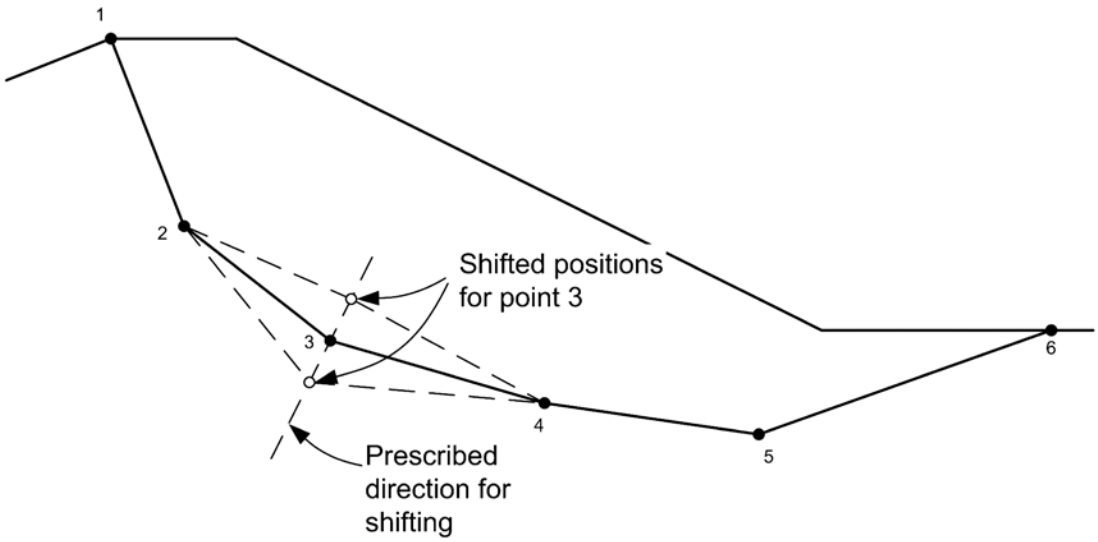


Figure 8.95 Search scheme for non-circular slip surfaces (Duncan and Celestino, 1981)

Genetic algorithms: in cases of complex geotechnical conditions, the minimisation solution may have several local minimum. Some authors proposed to use genetic algorithms to locate the global critical slip surface under general conditions with general constraints (eg Zolfaghari *et al*, 2005, Sun *et al*, 2008, Sengupta and Upadhayay, 2009, van de Meij, 2010, and Li *et al*, 2010). The advantage of this approach is that convergence to any prescribed degree of precision can be achieved and the algorithm has been demonstrated to be computationally superior to most of the optimisation routines, like the Monte-Carlo method and grid-points approaches. The disadvantage is that this kind of approach is rarely implemented in commercial codes.

Limitations considering slip surface assessment

Any search scheme employed in computer programs is restricted to investigating a finite number of slip surfaces. In addition, most of these schemes are designed to locate one slip surface with a minimum factor of safety. The schemes may not be able to locate more than one local minimum. The results of automatic searches are dependent on the starting location for the search and any constraints that are imposed on how the slip surface is moved. Automatic searches are controlled largely by the data that the user inputs into the software. The first thing to ensure is that the critical surface found is located inside the exploration domain and not on its borders. Regardless of the software used, a number of separate searches should be conducted to confirm that the lowest factor of safety has been calculated.

These limitations come from the fact that the problem of locating the critical slip surface can be viewed as a form of nonlinear, non-smooth, global optimisation and the objective function to be minimised is the factor of safety function. Some of the difficulties in the location of the critical slip surfaces are:

- the objective function of the factor of safety is non-smooth and can be non-convex in nature. The constraints, which include kinematically acceptable shapes of failure surfaces, rock and soil profile etc, may also be non-smooth, non-convex functions
- the existence of multiple minima is a fundamental feature of a slope stability problem
- a good trial for general ground conditions with arbitrary loadings can be difficult to develop for optimisation analysis.

Despite the fact that some modern heuristic optimisation methods (genetic algorithms, artificial networks etc) have been employed with success in the research field, most engineers still rely on their experience at present.

8.6.3.6 Cracking assessment

When soils at the crest of the slope have cohesion, the calculated values for the normal forces and side forces in this area are often negative. Negative forces are consistent with what would be calculated by classical earth pressure theories for the active condition. The negative stresses result from the tensile strength that is implicit for any soil having a Mohr-Coulomb failure envelope with a cohesion intercept. This type of shear strength envelope implies that the soil has tensile strength. Because few soils have tensile strength that can be relied on for slope stability, tensile stresses should be eliminated before an analysis is considered acceptable. Tensile stresses can be eliminated from an analysis by introducing a vertical tension crack near the upper end of the slip surface. The slip surface is terminated at the point where it reaches the bottom of crack elevation (Figure 8.96). The appropriate crack depth can be determined in either of the following ways:

- a range of crack depths can be assumed and the factor of safety calculated for each depth. The crack depth producing the minimum factor of safety is used for final analyses. The depth yielding the minimum factor of safety will correspond closely to the depth where tensile stresses are eliminated, but positive (driving) stresses are not
- the crack depth can be estimated as the depth over which the active Rankine earth pressures are negative. For total stresses and homogeneous soil the depth is given by:

$$d_{crack} = \frac{2c}{\gamma \tan\left(\pi/4 - \varphi/2\right)} \tag{8.176}$$

where C and φ represent the developed cohesion (kPa) and friction angle (°) respectively, γ the soil unit weight (kN/m^3). Similar expressions can be developed for the depth of tension for effective stresses and/or non-homogeneous soil profiles.

In some cases the depth of crack computed using Equation 8.176 will be greater than the height of the slope. This is likely to be the case for low embankments of well-compacted clay. For embankments on weak foundations, where the crack depth computed using Equation 8.176 is greater than the height of the embankment, the crack depth used in the stability analyses should be equal to the height of the embankment, so the crack should not extend into the weak foundation. In this case, the engineer must take great care concerning the validity of the limit equilibrium assumptions and the definition of the slip surface. Stress-deformation may be necessary.

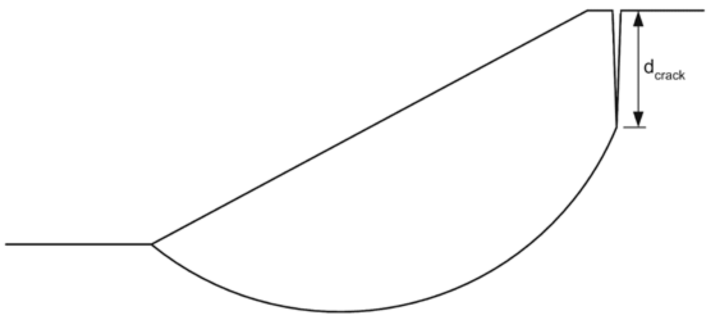


Figure 8.96 Vertical crack modelling

8.6.4 Limit analysis approaches

Limit analysis (Chen, 2007) approaches consist in modelling the soil as a perfectly plastic material obeying an associated flow rule. Two approaches were developed in this theoretical framework, static and kinematic. The general procedure is to assume a statically admissible stress field or a kinematically admissible failure mechanism and then optimise the objective function with respect to a very limited number of variable parameters. In this way, an upper or lower bound value of the limit load can be respectively found and the theoretically true collapse load is bracketed from above and below. This feature is particularly valuable in cases for which an exact solution cannot be determined, because it provides a built-in error check on the accuracy of the approximate collapse load.

The limit analysis framework can be used considering the upper bound solutions based on kinematically admissible rigid-block velocity fields (associated with the Mohr-Coulomb criterion) with the same practical advantages of limit equilibrium methods. In this case, it was shown (Michalowski, 1989) that the global force equilibrium was satisfied so that an upper bound limit analysis solution may be regarded as a special limit equilibrium solution, but not vice versa. Limit analysis approaches may also be implemented in finite element codes as a lower (Yu *et al*, 1998) or upper bound (Jiang and Magnan, 1997) formulation.

Limit analysis applied to rigid block assumption offers the advantage of simplicity. Just as for limit equilibrium methods, it requires the definition of the shear parameters (cohesion and friction angle) and of the slip surface. The rigorous elasto-plasticity formulation need no other assumptions related to interslice forces and give an upper bound of the factor of safety.

The disadvantage of this approach is that it requires that the materials modelled obey the associative flow rule. In effect this requires that all shearing resistance is modelled as dilation rather than a combination of dilation and friction as occurs with real soils. This is accurate for undrained problems where the angle of shearing resistance is zero, however for drained problems it typically leads to a small overestimate of load capacity. In extreme cases it can lead to volumetric locking and no collapse. Experience has shown that for moderately unconstrained problems, the increase in load estimate is minor. Manzari and Nour (2000), indicate, for example, that non-associative results for cohesive-frictional slope stability problems typically give values three to 10 per cent lower than for the associated flow rule case. To put this into context, this corresponds to using an angle of shearing resistance in an associated flow model approximately three per cent lower than the actual angle, which is of the order of 1°. However, it is not possible to give guidance on its effect on all problems.

8.6.5 Stress-deformation analysis

Stress-deformation may be performed by finite element or finite difference codes. This approach enables the estimation of stresses within the soil and the magnitude of the induced displacements. It is possible to model irregular geometries, complex soil behaviour, complex boundary conditions and a variety of construction phases.

For static slope stability analysis, stress-deformation approaches offer the advantages of being able to identify the most likely failure mode by determining the slope deformation, locating the most critically stressed zones within a slope and predicting the effect of slope failure on the adjacent or supported structures. These advantages come at the cost of increased engineering time for problem formulation, characterisation of material properties, interpretation of results and increased computational efforts.

8.6.5.1 Sources of inaccuracy

Duncan (1996) provided a very comprehensive review of the experience of using finite element methods to estimate stresses and deformations in slopes and embankments. Most of those conclusions are still valid and are summarised as follows. The sources of uncertainty in the results of the stress-deformation analyses are related primarily to the difficulties in predicting the actual densities and water contents of soils in the field, and with being able to anticipate the sequence of operations that will be followed during construction.

According to Kramer (1996), the accuracy of stress-deformation analyses is strongly influenced by the accuracy of the stress-strain model of the soil. Many behaviour laws have been developed in the past 30 years, each of them having advantages and limitations. It is beyond the scope of this handbook to discuss stress-deformation analysis tools in detail. It is important to emphasise that the accuracy of simple models is usually limited to certain ranges of strain and/or certain stress paths. Models that can be applied to more general stress and strain conditions are often quite complex and may require a large number of input parameters that may be difficult to determine experimentally.

There are generally three types of behaviour laws used for slope stability analyses:

1. **Linear elastic laws:** they have the advantage of simplicity and the limitation that they only model the behaviour of real soils at low stress levels and small strains, which is not the domain pertinent for slope stability analyses.
2. **Hyperbolic laws:** they have the advantage of simplicity, they model nonlinear behaviour, the parameters involved have physical significance and that they can easily be determined by conventional triaxial tests. They have the limitation that they are inherently elastic and do not model plastic deformations in a fully logical way.
3. **Elasto-plastic laws:** they have the advantage that they can model more realistically the behaviour of soils close to failure, at failure and after failure. They have the limitation that they are more complex to calibrate and some parameters have no real physical significance.

Comparisons of the results of FEM with field measurements have shown that the calculated deformations have a tendency to be larger than the measured deformations. According to Duncan (1996), the reasons for differences may be significantly influenced by the approximation of field parameters from laboratory testing procedures on intact or reconstituted samples (Chapter 7).

8.6.5.2 Factor of safety evaluation

The concept of factor of safety is not pertinent in the context of deformation analysis. But, given that most of the standards and recommendations express the requirement in terms of factors of safety, it may be a necessary output of the analysis.

Strength reserving approach

The definition of the factor of safety given by Duncan (1996) is particularly efficient in the framework of limit equilibrium methods. However, in a FEM, there are some difficulties related to the determination of the critical slip surface. In this approach, the factor of safety is generally obtained through the strength reduction technique as the value for which division of the shear strength parameters by F_s would onset a slope failure. So far, there has been no generally accepted failure criterion. The definition of the critical equilibrium state as the moment at which the plastic zones that enclose the critical sliding surface are linked together and pass through the slope from the toe to the top is much more preferable than the 'non-convergence of resolution algorithm' criterion.

Also, determination of the critical sliding surface requires some technical measures to visualise the shear bands, for example, the adaptive mesh refinement procedure (Zienkiewicz and Taylor, 1991), the technique of enhanced visualising failure mechanism (Griffiths and Kidger, 1995) etc. When this definition is used in finite element or finite difference analysis, some precautions should be taken.

Overloading approach

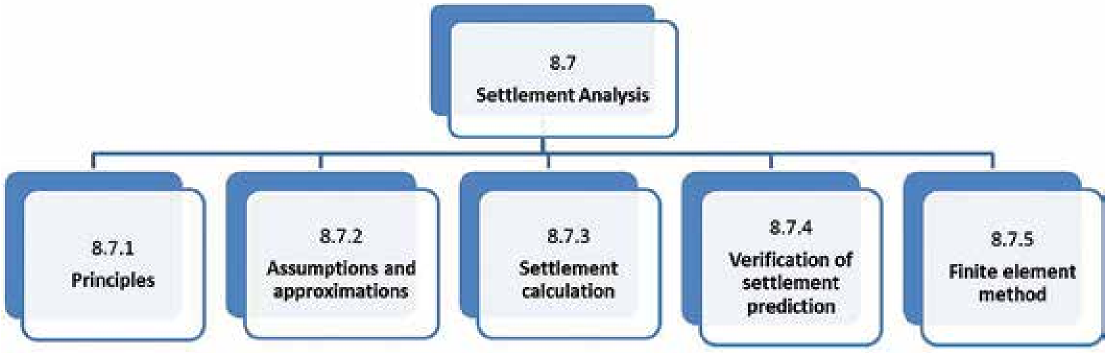
For all the reasons previously discussed, some authors have proposed another definition. The factor of safety is then defined as the ratio of total resisting forces to total driving forces along a certain slip line. The critical sliding surface is then regarded as the passage along which the ratio is a minimum. As compared with the previous definition, there are some advantages using this definition:

- only one model is needed in finding factor of safety associated with the loading definition
- this approach considers the effect of different stress paths on the degree of safety of the slope.

Choice of approach

When the FEM is used for slope stability analysis, the results from the overloading definition might be significantly different from both results from the strength reduction technique and the results from the limit equilibrium methods, particularly in terms of the position of the critical slip surface. Usually, the critical slip line associated with the overloading definition of the factor of safety is for the most part shallower than that associated with the strength reserving definition. Some authors (Zheng *et al*, 2006) recommend that the results from the strength reduction technique be taken as the standards in design and safety assessment of slopes, but when considering man-made slopes, the factor of safety associated with the overloading definition could be used to compare the effect of different construction procedures.

8.7 SETTLEMENT



8.7.1 Principles

When a structure is built on soil, the stress state in soil is modified not only in the loaded area but also widely in an influence zone. The vertical displacement of soil due to this modification is called settlement. For sandy foundations, settlements appear in the short-term (during construction phases). For clay soils and specifically for soft soils or peats, this modification leads to consolidation of soils and then to displacements (horizontal and vertical). For a new levee construction or enlargement of an existing levee, the prediction of total settlements and differential settlements is an important issue for the project because it is directly linked to the capacity of reaching the design level of protection of the levee. The design process has to take into account a sufficient overbuild of the levee to accommodate predicting settlements, and to find building solutions to limit settlement or to accelerate them.

For linear structures such as levees, the problem can be considered as 2D with quite good accuracy. The methods presented rely on a pertinent definition of the geotechnical model consistent with the models used for other analyses (eg slope stability analysis). When the 2D assumption is no longer valid, ie when drainage, deformations, applied loads and geometry cannot be considered as 2D, a 3D model may be necessary.

Simple to complex methods are commonly used to estimate settlements but it is assumed that a 1D method or oedometer method is often sufficient to predict settlements. This section will detail conventional methods and introduce control methods and numerical methods as shown in the flow chart.

8.7.2 Assumptions and approximations

This section presents the most commonly used method to evaluate the settlement of a compressible foundation layer.

Unidimensional consolidation

As it is quite difficult to determine the load-induced stress field within the foundation layer (other than with a linear elastic model), the stress is first estimated vertical to the axis of the levee. In this central zone, provided that the compressible foundation layer is thin in regards to the levee width ($b/H > 1$), assume that the drainage path is vertical and that horizontal deformations are negligible. Under these assumptions, consider that the conditions for application of unidimensional consolidation theory are met. These concepts are shown in Figure 8.97.

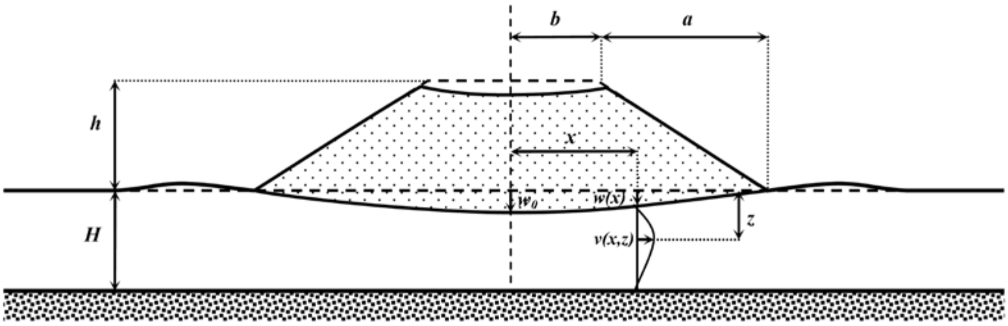


Figure 8.97 Definition of the geometrical parameters

Lateral deformations

At the toe of the slope, drainage conditions and deformations are much more complex (Figure 8.98). On one hand, there is an additional horizontal drainage component, while on the other hand, lateral deformations may occur as well as shear deformations along potential sliding surfaces. The smaller the factor of safety, the larger the shear deformations will be. Regarding the pore water pressure dissipation process, two concurrent phenomena may occur:

- primary consolidation processes due to the embankment loading, which tend to dissipate pore water pressure with time
- pore water pressure build-up due to contractive behaviour of the soil under shear stress.

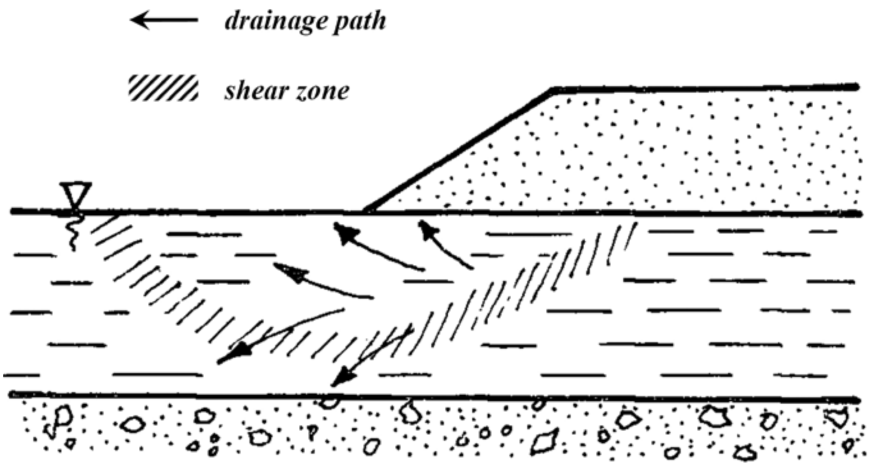


Figure 8.98 Drainage and deformation under the toe of the embankment

So, the determination of settlement under the embankment becomes a function of several parameters expressed as:

$$w\left(x\right) = w_0\,\mathcal{F}\left(\frac{x}{H}, F_s, \frac{x}{b}\right) \tag{8.177}$$

where w_0 is the settlement at the centre of the embankment, F_s is the factor of safety deduced from slope analysis, H the height of the embankment, b the half width of the crest of the embankment and x, the distance from central axis.

8.7.3 Settlement calculation

According to consolidation theory, total settlement w_t is the sum of the following components:

w_i = instantaneous settlement occurring under undrained condition
w_c = consolidation settlement (or primary settlement)
w_s = secondary settlement (or creep settlement)
w_l = settlement due to irreversible lateral movement (deduced from w_i).

Then total settlement w_t is given by:

$$w_t = w_i + \mu w_c + w_s + w_l \tag{8.178}$$

where μ is a correction factor, introduced by Skempton and Bjerrum (1957), which takes into account the 2D aspect of the consolidation process. The different components of settlement, w_t, are shown in Figure 8.99.

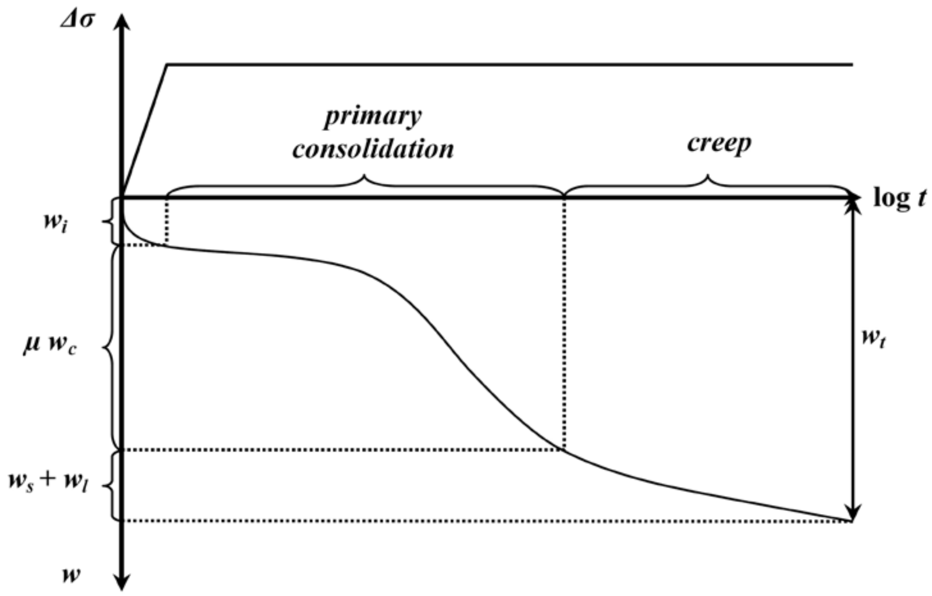


Figure 8.99 ***Different components of the settlement w_t (negative ordinate) as function of load $\Delta\sigma$ (positive ordinate), and time (t)***

Determination of vertical stress

When an embankment is constructed, applying a uniform pressure to the soil surface, the increase of total vertical stress depends on height and geometry of the embankment. Since maximum vertical stress is situated at the centreline of the levee, some authors have developed practical graphs to obtain the vertical stress increase in foundation due to surface loads.

Considering the geometrical definition of the embankment (Figure 8.97), the vertical stress increment at depth $\Delta\,\sigma_v(z)$ along the axis of the embankment may be estimated from the chart proposed by Osterberg (1957) to obtain vertical stress increment from the following equation:

$$\Delta\sigma_v\,(z) = 2\,I\,(z)\;\Delta\sigma \tag{8.179}$$

where I (-) is the coefficient of influence and $\Delta\sigma$ (kPa) the vertical stress increment at the surface of the soil foundation. In Figure 8.100, a (m) is the width of slope's base, b (m) is the half width of the levee crest and z (m) represents the depth.

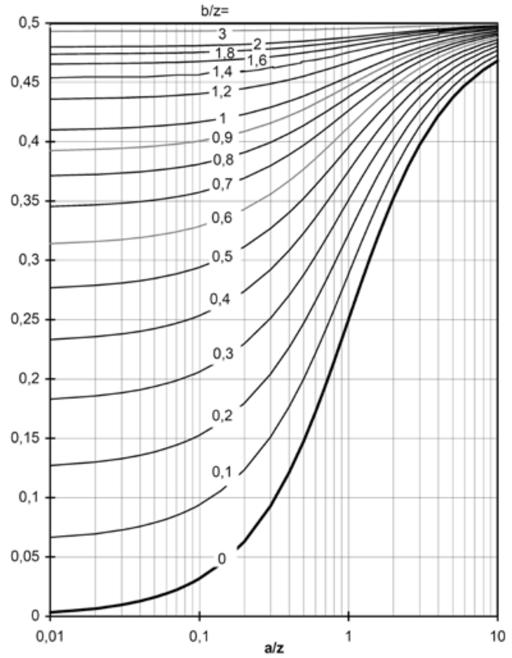


Figure 8.100 Graph giving vertical stress under half an embankment (after Osterberg, 1957)

8.7.3.1 Instantaneous settlement

Soil strains contributing to instantaneous settlement are caused during initial loading for undrained conditions. Loading for levee construction is not applied instantaneously and the soil is generally in a partly saturated state. So, the strict separation between w_i and w_c is not exact. However, some simple methods and charts are available to determine instantaneous settlements, w_i, according to elasticity theory by the following equation:

$$w_i = \frac{\Delta\sigma}{E_u} I \quad (8.180)$$

where:

$\Delta\sigma_v$ = incremental load (kPa)
E_u = elastic modulus of compressible soil for undrained condition (kPa)
I = influence factor (see Figure 8.101)

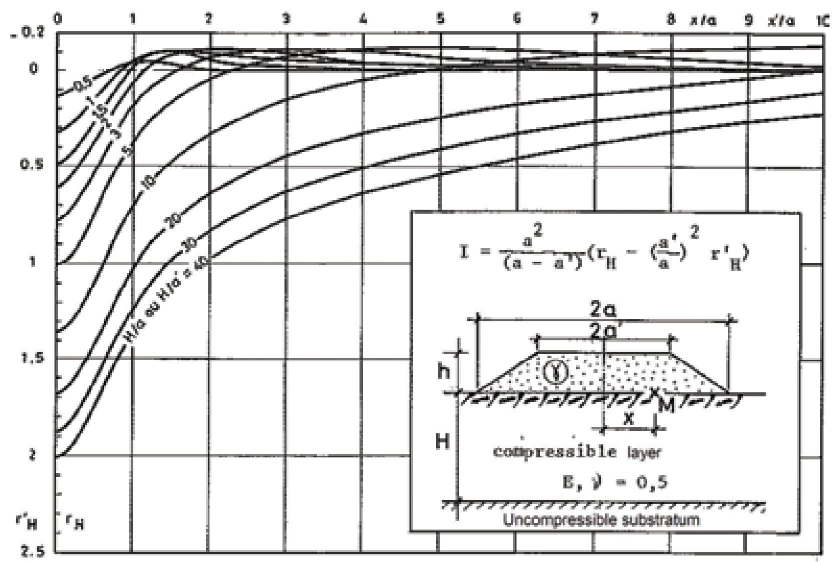


Figure 8.101 Elastic instantaneous settlement charts (Giroud, 1973)

8.7.3.2 Primary consolidation

This method was developed by Terzaghi and proposes to individualise soil into homogeneous layers with thickness H_0, characterised by oedometer test. This test yields the consolidation state of the soil and the vertical preconsolidation stress σ'_p that will govern the soil behaviour, based on an increase of vertical load $\Delta\sigma_v$, such as due to an embankment. The value, σ'_p, indicates if the soil has been previously loaded to a stress exceeding the actual stress σ_{v0}. From this test (Section 7.8.3), oedometric settlements can be defined by Equations 8.181 or 8.182 depending on the soil consolidation state and position of final stress $\sigma'_{v0} + \Delta\sigma$. In the common case of normally consolidated soils, the final primary settlement may be calculated as:

$$w_{c\infty} = \frac{H_0}{1+e_0} C_c \log \frac{\sigma'_{v0} + \triangle\sigma}{\sigma'_{v0}} \qquad (8.181)$$

where:

e_0 = initial void ratio of soil (-)
C_c = consolidation coefficient (-)
H_0 = initial height of the compressible soil layer (-).

This relationship may be extended at any time during primary consolidation considering a consolidation ratio U(t) (Figure 8.102) defined as:

$$w_c\,(t) = U\,(t)\; w_{c\infty} \qquad (8.182)$$

with:

$$t = \frac{H^2}{C_v} T_v\,(U) \qquad (8.183)$$

where:

C_v = consolidation coefficient (m²/s)
T_v = non-dimensional time parameter (-)
H_d = drainage path length (m)

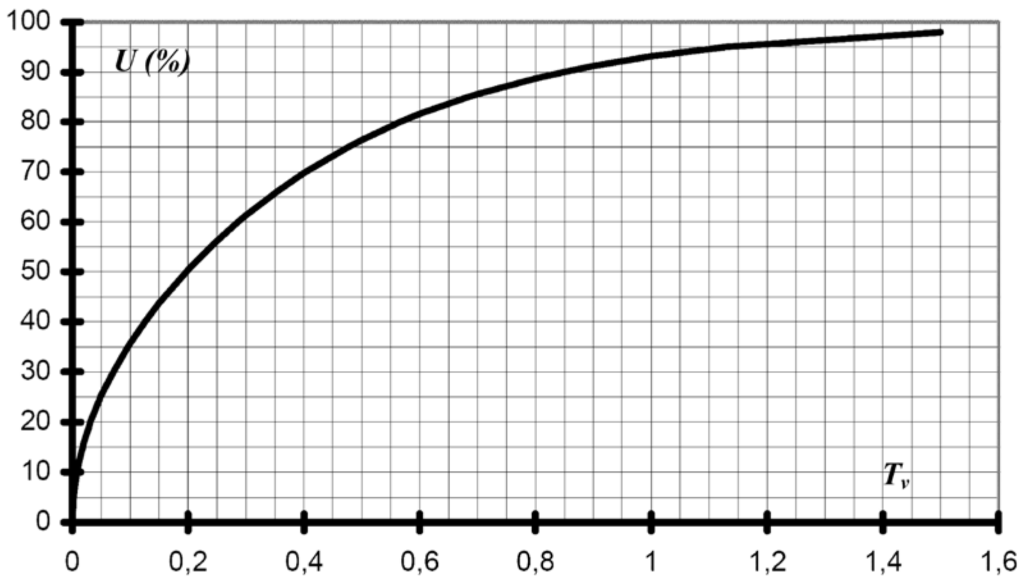


Figure 8.102 Consolidation ratio (U) as a function of the non-dimensional time (T_v)

Because compressible soils have relatively poor geomechanical characteristics (defined by undrained cohesion C_u), their bearing capacities are often limited and embankment works need to be phased into several steps. For each phase, settlement values and time of consolidation are designed but the accuracy of the prediction is insufficient and necessitates controls during construction. Construction techniques to anticipate and measure settlements during construction phases are given in Section 10.5.4. Controls are very important for construction works on compressible soils because predictions are highly dependent on the soils character and drainage potential, while geotechnical investigations during design phases are often insufficient to properly capture the soils heterogeneity. Settlement control programs also contain soil pore pressure measurement devices to track drainage of the soil and anticipate possible soil failure (loss of bearing capacity of soil due to excessive pore pressure) and embankment failures.

Methods to estimate total settlements during construction regarding settlement evolution are given at the end of this section. Section 7.7.5 details some observational methods useful for determination of base settlement.

8.7.3.3 Secondary compression

After primary compression, for clayey and particularly organic soils such as peats, a second compression phase takes place (secondary or creep consolidation) corresponding to soil grain reorganisation without lateral displacement. For very soft soils and peats, the secondary settlement phase could be important regarding the life time of the structure and should be taken into account earlier in the project.

Different methods (field or laboratory devices) exist to determine consolidation characteristics of soils. Classically, consolidation behaviour of soils can be appreciated by laboratory oedometer compression tests (developed in Section 7.8.3). The secondary settlement is given by:

$$w_s(t) = \frac{H_{ref}}{1+e_{ref}} C_\alpha \log \frac{t}{t_{ref}} = H_{ref}\, C_{\alpha e} \log \frac{t}{t_{ref}} \tag{8.184}$$

where:

C_{ae} = creep index (determined with long-term oedometer test)
t_{ref} = reference time from which the creep settlement is calculated (eg at 90 per cent of consolidation)
H_{ref} = corresponding thickness of compressible layer
e_{ref} = corresponding reference void index

Settlement due to lateral creep deformations

For compressible soils, the displacement of soil during earthen construction is not only vertical (settlement) but also horizontal. Note that this phenomenon can cause deteriorations to existing structures placed in the influence zone and has to be take into account during design (eg for choice of levee location) and controlled during construction (Section 10.5.4).

Settlement due to lateral displacement is difficult to obtain. The order of magnitude may be appreciated from the empirical relationship (**to use with extreme caution as a rule of thumb**):

$$w_l = 0.11 \frac{H}{a+b} w_s \tag{8.185}$$

where H is the thickness of compressible layer, and $a + b$ the half equivalent width of the embankment.

Other 1D methods are available (such as stress path method initiated by Skempton and Bjerrum (1957) and developed by Lambe, 1964). As lateral displacements can be expressed as a function of settlement along the embankment axis, different methods to avoid failure (eg observational methods) and to verify predicted total settlements (eg construction monitoring controls) should be employed.

8.7.4 Verification of settlement prediction

Asaoka method based on settlement measurement

To verify final settlement predictions during construction and react if necessary, Asaoka (1978) proposed a simple method based on measurement of soil settlements at regular time intervals. The method consists of measuring settlements, (S_{i-1}, S_i), at constant time intervals,Δt, and plotting them such as shown in Figure 8.103. In this method, S_i is the settlement measurement at t_{i-1} and S_i is the settlement measurement at $t_i = t_{i-1} + \Delta t$.

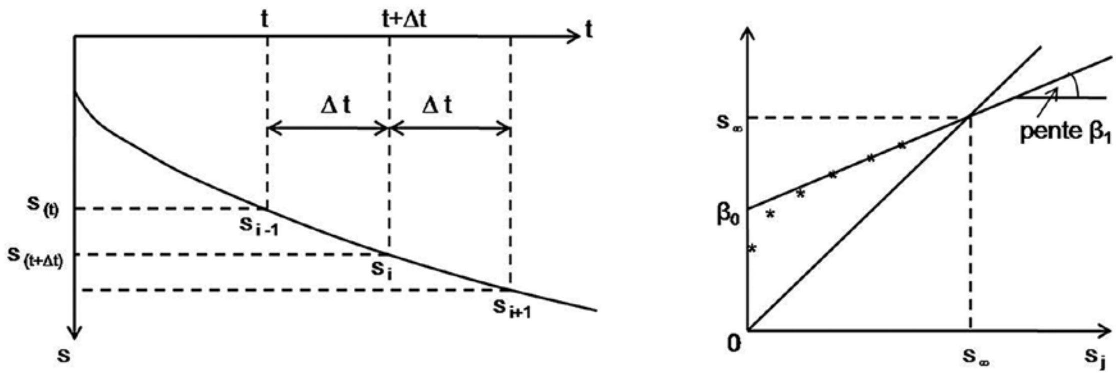


Figure 8.103 Total settlement curve of compressible layer (a), and Asaoka's construction curve (b) (Asaoka, 1978)

Steps for using the Asaoka method are:

- construct a time-settlement curve (as shown in Figure 8.103a) and select a series of settlement values at increasing time intervals
- plot the settlement values (S_{i-1} versus S_i) as shown in Figure 8.103b
- the plotted values will form a straight line as shown in Figure 8.103b as the β_0 line
- the estimated total settlement is where the β_0 line crosses the 45° line ($S_i = S_{i-1}$).

This method also enables adjustment to the time consolidation prediction by knowing the average vertical coefficient of consolidation, *C*v, of the compressible layer given by equation of the β_0 line:

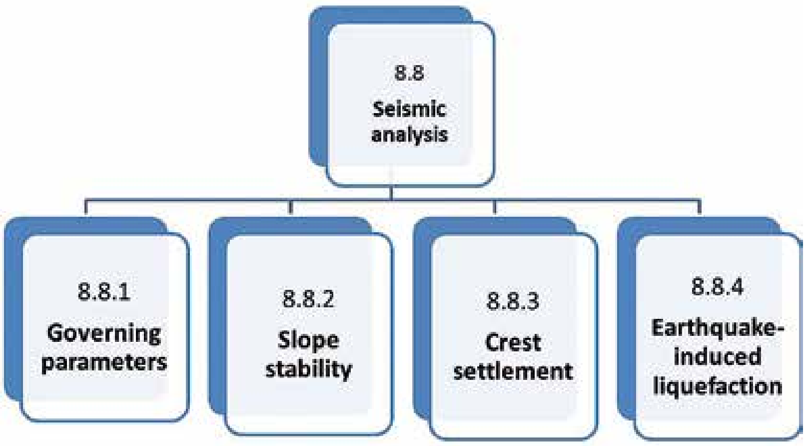
$$C_v = -\frac{5H^2}{12\Delta t}\ln \beta \tag{8.186}$$

Other methods are based on lateral displacement measurements or interstitial pressure measurements in clay soils. The reader is referred to any soil mechanics text book for further information.

8.7.5 Finite element method (FEM)

Numerical calculations are available for settlement predictions and some software proposes models for nonhomogeneous soil, anisotropic soil etc. Such models require complex input data, which are not easy to obtain from classical laboratory tests. Generally it is often more accurate for a levee project to predict settlements with 1D methods such as oedometer testing than with complex numerical methods.

8.8 SEISMIC ANALYSIS



Two critical design issues must be addressed when evaluating the seismic performance of a levee:

- loss of significant strength of the material within or below the levee as a result of cyclic loading (eg soil liquefaction, water pressure build up in sands and silts, or post-peak reduction in sensitive clays)
- significant deformations that may jeopardise satisfactory performance.

Methods for stability analysis and evaluation of seismically induced permanent displacements attributed to deviatoric shear deformation are developed below as shown graphically above.

In addition to deformation of the embankment from slipping in response to earthquake shaking, the levee may settle in response to the stresses developed in each soil element. This generalised settlement can be estimated by using soil mechanics consolidation, empirical, and/or finite element procedures. Post seismic settlement in the foundation due to pore pressure dissipation is also a key issue under the scope of this section (Tokimatsu and Seed, 1987, Ishihara and Yoshimine, 1992, and Tsukamoto and Ishihara, 2010).

Other factors that may play a role in determining the acceptability of the performance of the levee following an earthquake are:

- the occurrence of flooding
- the ability or lack thereof to quickly repair a damaged structure.

8.8.1 Governing parameters

8.8.1.1 Seismic action

The seismic action to be considered for design purposes should be based on the estimation of the ground motion expected at each location in the future, ie it should be based on the hazard assessment (deterministic seismic hazard analysis or probabilistic seismic hazard analysis).

Probabilistic seismic hazard analysis gives hazard curves that depict the exceedance probability of a certain seismologic parameter (eg the peak ground acceleration, velocity or displacement) for a given period of exposure, at a certain location (normally assuming a rock ground condition).

For most countries, the seismic hazard is described by a zonation map defined by the national authorities.

Elastic response spectra represent maximum responses of a series of single-degree-of-freedom systems of different natural periods to a given ground-motion excitation. The response spectrum amplifications vary with the value of damping.

1 2 3 4 5 6 7 8 9 10

The standard response spectra are commonly used. The spectra is developed using the peak or effective ground motion parameters in conjunction with a standard spectral shape. It incorporates soil property effects, but ignores the influence of earthquake magnitude and distance on the shape of the spectra (Figure 8.104).

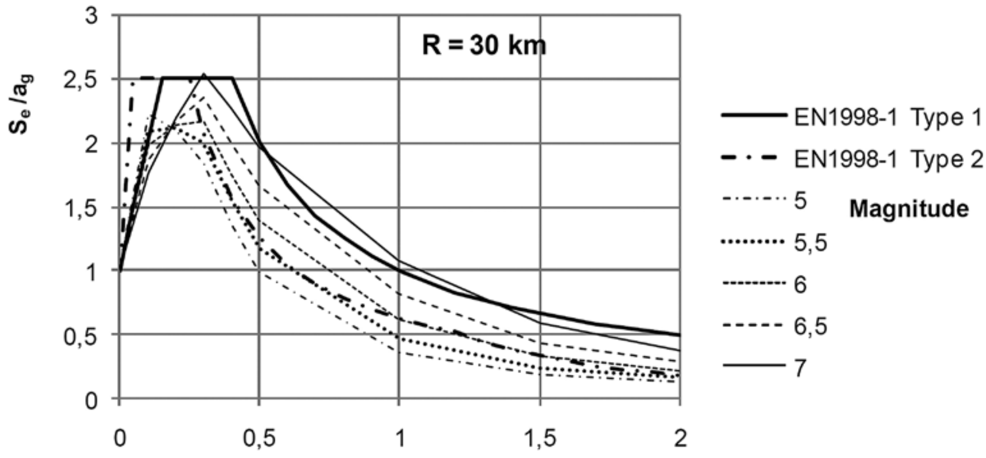


Figure 8.104 Recommended spectral shapes for Type 1 and Type 2 seismic action in EN1998-1 and illustration of the effect of magnitude (from Bisch et al, 2011)

Site-specific procedures are used to produce response spectra that correspond closely with those expected, based on the seismological and geological conditions at the site. These procedures use either the deterministic or probabilistic method to develop site-specific spectra.

The earthquake vibration at the surface is strongly influenced by the underlying ground conditions and correspondingly the ground characteristics very much influence the seismic response of structures. EN 1998-1 provides for example five ground profiles (A to E) and corresponding site coefficient of amplification (S).

Guidance in the choice of selecting seismic parameters can be found in ICOLD (2010).

8.8.1.2 Soil properties

Strength properties

For cohesive soils the relevant strength characteristic is the undrained shear strength (C_u). For most materials this value can be taken equal to the conventional 'static' shear strength. Some plastic clays may be subject to cyclic degradation with a loss of strength. Consequently most investigators recommend that the static undrained strength of soft clay be reduced by 20 per cent to account for strength loss during strong earthquake shaking. However, some clays may exhibit a shear strength increase with the rate of loading. These phenomena should ideally be given due consideration in the choice of the relevant undrained shear strength.

For pulverulent (powdery) soils the relevant properties are the drained friction angle ϕ' and the drained cohesion c'. These parameters are directly usable for dry or partially saturated soil. For saturated soils they would require the knowledge of the pore water pressure variation, u, during cyclic loading, which directly governs the shear strength (Section 7.8.3.3). EN1998-5 suggest an alternative approach, which consists of using the undrained shear strength under cyclic loading, $\tau_{cy,u}$. This undrained shear strength may be determined from laboratory tests or experimental relationships with, for example, the soil relative density or any other index parameter like blow counts, N, measured in standard penetration tests (SPT).

Note

Those considerations serve for assessing the characteristic value of the relevant strength characteristic in the sense of Eurocode 7 Part 1 and also its design value (for example, by applying the partial factor γ M 'material factor' in approach 3, see Section 7.5.4).

Deformation characteristic

The soil stiffness is defined by the soil shear modulus G:

$$G = \frac{\Delta\tau}{\Delta\gamma} \tag{8.187}$$

where $\Delta\tau$ and $\Delta\gamma$ are respectively the shear stress and shear strain variations. The small strain value may be estimated equal to $G_{max} = \rho V_s^2$ where ρ is the unit mass and V_s is the shear wave propagation velocity of the ground (Section 7.9.5). The relevant values to use in most of the calculation models are not the elastic ones but secant values compatible with the average shear strain induced by the earthquake. EN1998-5 proposes a set of reduction factors correlated to the peak ground surface. Soil internal damping $\eta(\gamma)$, generally taken between five and 20 per cent, shall be considered in some analysis.

8.8.2 Slope stability

The seismic stability of slopes has been a topic of considerable interest in geotechnical engineering practice for the past 40 years. During that period, the state of practice has moved from simple pseudostatic analyses to more complicated permanent displacement analyses. A variety of analytical tools ranging from sliding block analyses to multidimensional nonlinear dynamic response analyses are now available for prediction of permanent displacements. These tools represent the mechanics of the seismic slope stability problem with different levels of rigor, and require different levels of information on material behaviour. The most useful are those that can represent the important physical mechanisms of a particular seismic stability problem using material information that can be obtained practically and economically (Kramer and Smith, 1997).

8.8.2.1 Pseudostatic approach

Among the methods of analysis of the seismic stability of slopes, the pseudostatic methods are the oldest and the most widely used in engineering practice. Pseudostatic analysis represent the transient effects of an actual earthquake motion by applying constant unidirectional accelerations (horizontal and vertical) to a mass of potentially unstable material. The resulting inertial forces are taken to act in directions that destabilise the slope. The magnitudes of the horizontal and vertical pseudostatic loads are usually expressed in terms of seismic coefficients, k_h and k_v, numerically equal to the ratios of the inertial forces to the weight of the potentially unstable material. By solving force and/or moment equilibrium of the potentially unstable soil, a pseudostatic factor of safety can be computed. The pseudostatic factor of safety provides an index of stability under seismic conditions in a form familiar to geotechnical engineers. Selection of an appropriate seismic coefficient, however, is a crucial and complicated matter (Kramer and Smith, 1997).

The seismic inertia forces F_H and F_V acting on the soil sliding mass (Figure 8.105), for the horizontal and vertical directions respectively, in pseudostatic analyses shall be taken as:

$$F_H = k_H\, W \tag{8.188}$$

$$F_V = k_V\, W \tag{8.189}$$

where:

k_H = pseudostatic horizontal seismic coefficient (-)
k_V = pseudostatic vertical seismic coefficient (-)
W = total weight of the sliding mass (kN)

Vertical seismic coefficient is usually not taken into account. Simplifications made in using the pseudostatic approach to evaluate seismic slope stability include:

- replacing the transient earthquake motion by a constant horizontal acceleration equal to $k_H\, g$ (where g is acceleration of gravity)

- simplify amplification in the embankments using peak horizontal average acceleration of the failure mass.

Slope stability methods

Most of the slope stability methods developed in Section 8.6 may be used by adjusting the weight, *W*, of each slice to accommodate the seismic inertia forces F_H and F_V.

Soils properties

Static undrained strength should be used in the analysis. Most investigators recommend that the static undrained strength of soft clay be reduced by 20 per cent to account for strength loss during strong earthquake shaking.

Selection of the seismic coefficient

Recommendations for selecting an appropriate pseudostatic seismic coefficient were provided by different authors. The first recommendations were developed for embankment dams and were based on a level of acceptable deformation that would not compromise the integrity of the embankment. Using a limit of 1 m as a criterion, for acceptable performance, Seed (1979) recommended using seismic coefficients of 0.1 and 0.15 (together with a factor of safety of 1.15) for earthquake magnitude 6.5 and 8.25 respectively (crest acceleration less than 0.75 g).

The general expressions for seismic coefficients are given by the following equations:

$$k_H = \pm \alpha S \frac{a_g}{g} \tag{8.190}$$

$$k_V = \pm \beta S \frac{a_{vg}}{g} \tag{8.191}$$

where:

a_g = horizontal peak ground acceleration at bedrock (m/s²)
a_{vg} = vertical peak ground acceleration at bedrock (m/s²)
S = site amplification factor (-)
g = acceleration of gravity (m/s²)

The parameters α and β define the average peak horizontal acceleration of the potential failure mass (including amplification in the embankment) from the ground acceleration. It needs to be emphasised that choosing α < 1.0 implies that if there are sliding surfaces for which the condition F_s < 1 is met, permanent displacements will occur during the earthquake.

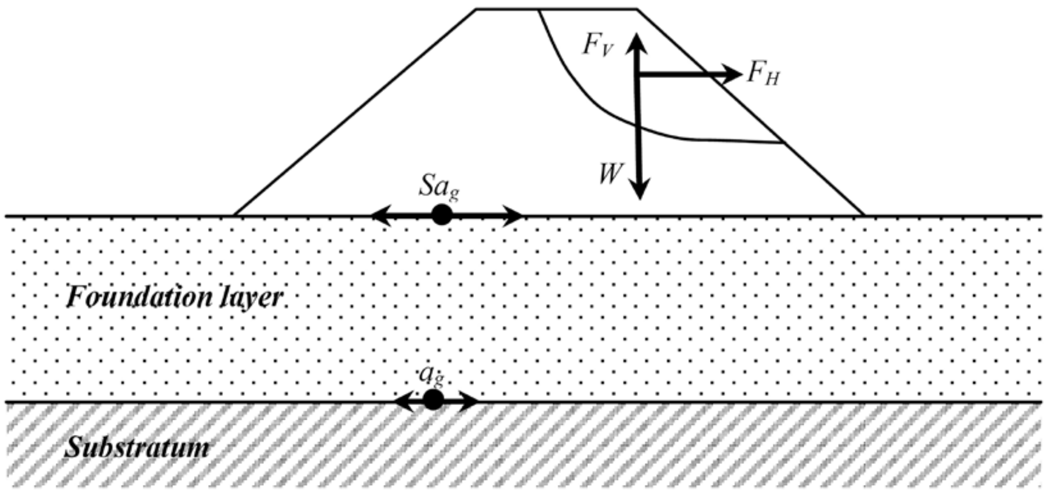


Figure 8.105 Definition of pseudo-static approach terms

The values generally accepted in engineering practice are α = 2/3 and β= 0. However, for slope design, BS EN 1998-5: 2004 proposes using horizontal seismic coefficient α = 0.5 and β = 0.17–0.25 depending on a_{vg}/a_g ratio. This value of the α coefficient has been selected based on empirical analysis, observed performance of slopes and embankments during earthquakes, and back-calculations. In the case of the design of a sensitive structure, implying the decision of limiting the induced permanent displacements, higher values of seismic coefficient may be chosen, possibly equal to or greater than the peak ground acceleration at the base of the levee ($\alpha \geq 1$) if amplification is expected in the levee. For example, the practice of dam engineering in Switzerland is to consider amplification α = 1.5 (OFEG, 2003).

Pseudostatic slope analysis conservatively evaluates the potential for occurrence of a slope failure due to earthquake loading. If the results of the pseudostatic analysis indicate a factor of safety less than one, then the potential for slope movement exists (not necessary failure) and a deformation analysis may be appropriate to quantify the permanent seismic deformations.

8.8.2.2 Pseudo-dynamic approaches

Sliding block analysis

This approach is based on the analogy of a rigid block resting on an inclined plane representing a potential sliding mass of soil (Figure 8.106). A simple procedure for estimating displacement of slopes during earthquake shaking is based on the concept of critical (or yield) acceleration (a_c) originally proposed by Newmark (1965). The yield acceleration is the minimum pseudostatic acceleration required to produce a displacement of the block (factor of safety F_s = 1). When equivalent acceleration applied to the block, corresponding to the inertial forces due to the earthquake, which exceeds the critical acceleration, a displacement of the block occurs.

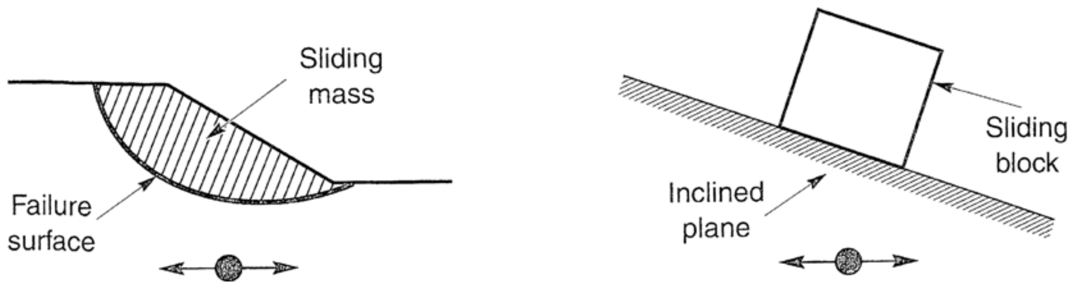


Figure 8.106 Analogy between potential sliding mass and rigid block resting on an inclined plane (from Kramer, 1996)

With the soil mass being rigid, the permanent displacement is obtained by a simple double integration of the excess acceleration (Figure 8.107). Given that an earthquake motion can exceed the yield acceleration many times, it may produce a number of increments of displacement. So, the total displacement is influenced by strong-motion duration as well as amplitude and frequency content of the earthquake spectra.

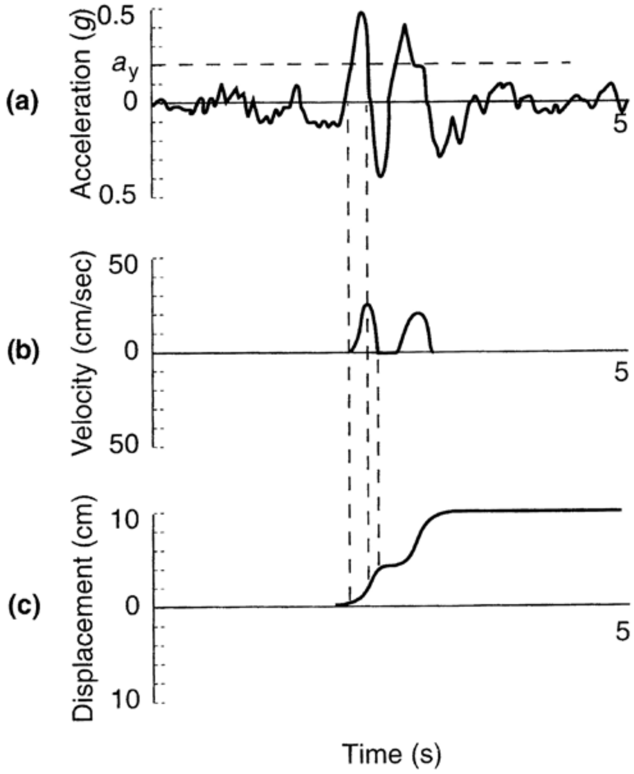


Figure 8.107 Newmark integration scheme (from Wilson and Keefer, 1985)

Different approaches were developed to refine the Newmark method by using a variety of acceleration pulses as well as large collections of actual strong motion records.

Ambraseys and Menu formula (1988)

Ambraseys and Menu (1988) proposed various regression equations to estimate Newmark displacement as a function of the critical acceleration ratio based on analysis of 50 strong-motion records from 11 earthquakes. They concluded that the following equation best characterised the results of their study:

$$\ln(D) = 0.90 + \ln\left[\left(1 - \frac{k_c}{k_{max}}\right)^{2.53}\left(\frac{k_c}{k_{max}}\right)^{-1.09}\right] \pm 0.30 \qquad (8.192)$$

where D is the expected displacement of the sliding mass of soil (cm), $k_c = a_c/g$ the critical seismic factor (-) and $k_{max} = a_{max}/g$ the maximum averaged seismic factor (-).

Jibson formula (2007)

Jibson (1993) suggested using Arias intensity (I_a) rather than peak ground acceleration to characterise the strong shaking. Arias (1970) defined this measure of the shaking content of a strong-motion record as:

$$I_a = \frac{\pi}{2}\int_0^T [k(t)]^2\, dt \qquad (8.193)$$

where:

g = the acceleration of gravity (m/s^{-2})
T = the duration of the significant shaking (s)
$k(t)$ = $a(t)/g$, the seismic coefficient history

Because Arias intensity measures the total acceleration content of the record rather than just the peak value, it provides a more complete characterisation of the shaking content of a strong-motion record than the peak ground acceleration. Jibson (2007) proposed an equation based on a rigorous analysis on hundreds of strong-motion records to generate the following regression equation:

$$\ln(D) = 0.561 \ln I_a - 3.833 \ln\left(\frac{k_c}{k_{max}}\right) - 1.474 \pm 0.616 \quad (8.194)$$

Validity domain

The accuracy of a sliding block analysis depends on the accuracy of the input motion applied to the inclined plane (Figure 8.108). The sliding block method assumes the potential sliding mass of soil to be rigid, in which case the appropriate input motion would be the ground motion at the level of the failure surface. However, actual levee slopes deform during the earthquake shaking. Their dynamic response depends on their geometry, stiffness and spectral content of the underlying ground motion. For levees composed of stiff soils and/or subjected to low frequency motion, lateral displacement throughout the potential sliding mass may be nearly in phase and the rigid block assumption is a good approximation. In the case of softer soils and/or higher frequency motion, the displacement field throughout the potential sliding mass may not be in phase. When this occurs, the inertial forces acting on the sliding mass have different directions and the resultant inertial force may be significantly smaller than that by the rigid block assumption.

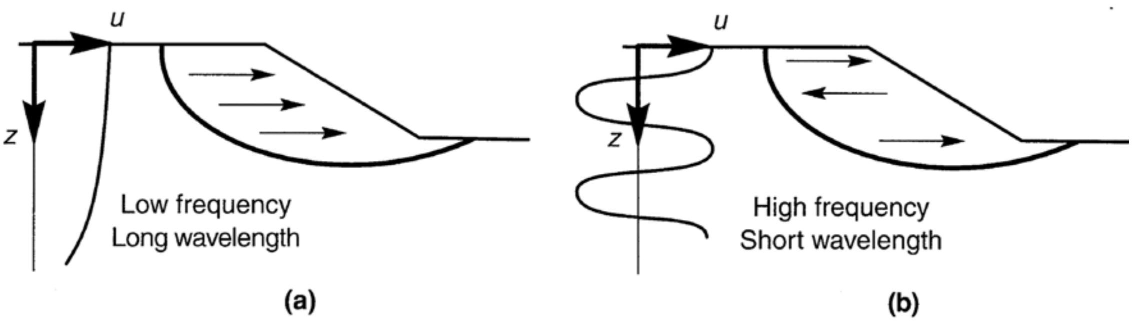


Figure 8.108 Influence of frequency on earthquake-induced displacements, soil motion in phase (a), and soil motion in opposite directions (b) (from Kramer, 1996)

Makdisi and Seed approach

Makdisi and Seed (1978) used dynamic finite element analysis to determine the horizontal component of the dynamic stresses acting on a potential failure surface. The resultant inertial force was divided by the mass of soil above the failure surface to produce the average acceleration of the potential sliding mass. Then they performed a sliding block analysis to estimate the permanent earthquake-induced displacement of earth dams and embankments. By making simplifying assumptions about the results of the numerical analyses, a simplified procedure was developed.

Critical seismic coefficient, k_c

In the simplified procedure, the critical acceleration ($k_c g$) for a particular potential failure surface is computed using dynamic yield strength of the soil (assuming a 20 per cent reduction of the undrained strength). The critical yield acceleration, k_c, may be determined using slope stability analysis and limiting equilibrium methods. To this purpose, it has to be noted that the Sarma method explicitly produces a critical (yield) seismic coefficient.

Seismic coefficient at the crest, $k_{0,max}$

The next step of the method consists in determining the maximum seismic coefficient at the crest of the levee ($k_{0,max}$). It may be done by the following equation:

$$k_{0,max} = \frac{1}{g}\sqrt{(1.6\, S_{a,1})^2 + (1.06\, S_{a,2})^2 + (0.86\, S_{a,3})^2} \quad (8.195)$$

where S_a, n is the spectral acceleration for the nth mode corresponding to the T_n period. The first three natural periods may be determined by:

$$T_1 = 2.62 \frac{h}{V_s} \quad , \quad T_2 = 1.14 \frac{h}{V_s} \quad , \quad T_3 = 0.73 \frac{h}{V_s} \tag{8.196}$$

where h is the height of the levee and V_s the average strain-dependent shear wave velocity of soil.

Averaged seismic coefficient of the sliding mass, k_{max}

The dynamic response of the embankment is accounted for by an acceleration ratio that varies with the depth of the potential sliding surface (z) relative to the height of the embankment (H). The relation between these parameters is represented on Figure 8.109.

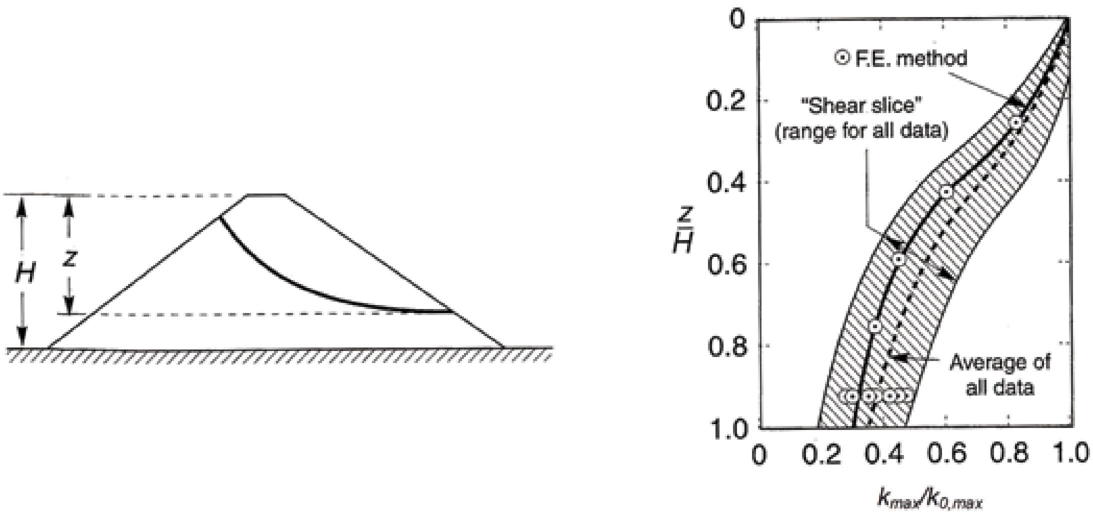


Figure 8.109 Influence of the depth of the failure surface on the average maximum acceleration of the potential sliding mass (from Makdisi and Seed, 1978)

Validity domain

This is one of the greatest limitations of this method. As shown in Figure 8.109, which presents results based on linear equivalent method analyses of columns of waste placed on top of a firm foundation for a number of ground motions, the maximum seismic coefficient at the top of the levee $k_{0,max}$ varies significantly. There is great uncertainty regarding what value of $k_{0,max}$ to use. So, the uncertainty in the estimate of k_{max} is high, because the uncertainty in estimating the crest maximum acceleration is high. Even with advanced analyses, estimating the maximum seismic coefficient at the crest is difficult, and the need to perform any level of dynamic analysis to estimate the crest acceleration conflicts with the intent of a simplified method that should not require more advanced analysis.

Also, the bounds shown on the Makdisi and Seed (1978) plot of $k_{max}/k_{0,max}$ vs z/H (Figure 8.109) are not true upper or lower bounds. Stiff earth structures undergoing ground motions with mean periods near the degraded period of the earth structure can have k_{max} values exceeding 50 per cent of the crest maximum acceleration for the base sliding case (ie, $z/h = 1.0$), and flexible earth structures undergoing ground motions with low mean periods can have k_{max} values less than 20 per cent of the maximum acceleration of the crest for base sliding (Kramer, 1996).

The variation of normalised permanent displacement with the critical seismic coefficient for different magnitude earthquakes is shown in Figure 8.110.

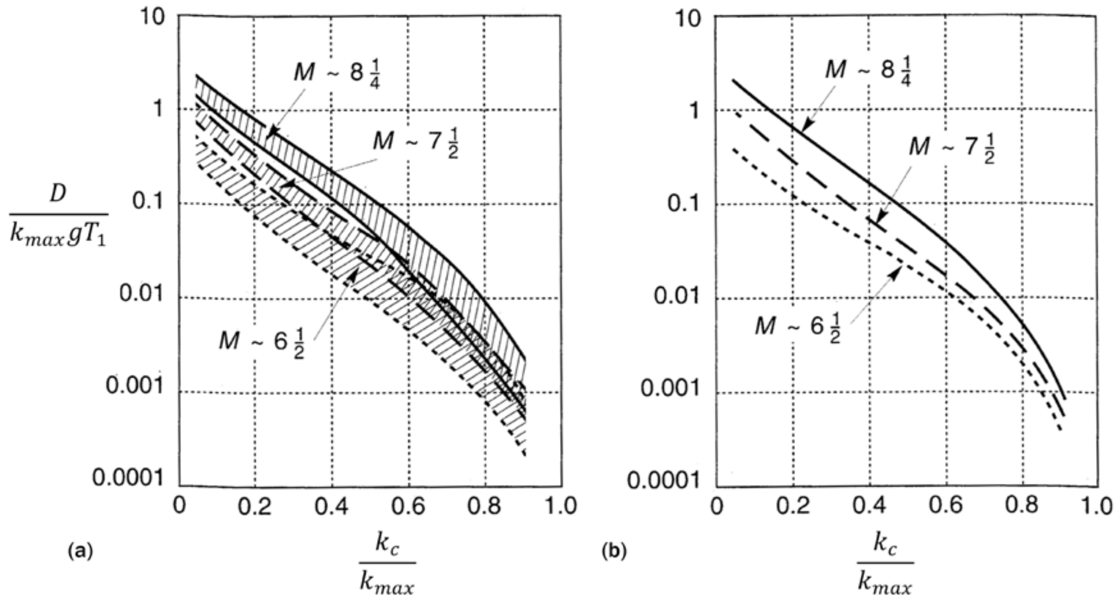


Figure 8.110 Variation of the normalised permanent displacement (D) with the critical seismic coefficient for different magnitudes, summary of several earthquakes (a), and average values (b) (from Makdisi and Seed, 1978)

Bray and Travasarou method (2007)

The earthquake ground motion is one of the most important components of a seismic displacement analysis in terms of its contribution to the calculation of the amount of seismic displacement. Yet, currently available simplified slope displacement estimation procedures are largely based on a relatively limited number of earthquake records or simulations. Bray and Travasarou (2007) tried to overcome this limitation working on a large database of case studies.

Spectral acceleration has been commonly employed in earthquake engineering to characterise an equivalent seismic loading on a structure from the earthquake ground motion. Similarly, Bray and Travasarou (2007) found that the five per cent damped elastic spectral acceleration at the degraded fundamental period of the potential sliding mass, equal to 1.5 times the initial fundamental period, was the optimal ground motion intensity measurement in terms of efficiency and sufficiency.

Contrary to the previously developed methods, Bray and Travasarou (2007) use a fully coupled model taking into account the vibratory behaviour of the structure, and deviatoric and volumetric behaviour of the soil constitutive of the embankments. As uncoupled models are not always conservative, the approach eliminates this. The first step is to determine critical acceleration, k_c, using slope stability analysis (limiting equilibrium methods). Then the model for estimating seismic displacement, D, consists of two discrete computations of:

- the probability of negligible ('zero') displacement (typically $D \leq \delta = 1$ cm)
- the likely amount of 'non-zero' displacement.

The probability of negligible displacement is calculated by the following equation:

$$P(D \leq \delta) = 1 - \Phi(-1.76 - [3.22 + 0.484T_s] \ln(k_c) + 3.52 \ln(S_a(1.5T_s))) \tag{8.197}$$

where:

D = seismic displacement (cm)

Φ = the standard normal cumulative distribution function

k_c = yield coefficient (-)

T_s = fundamental period of the sliding mass (s) (Figure 8.111)

S_a = spectral acceleration of the input ground motion at a period of $1.5T_s$(g)

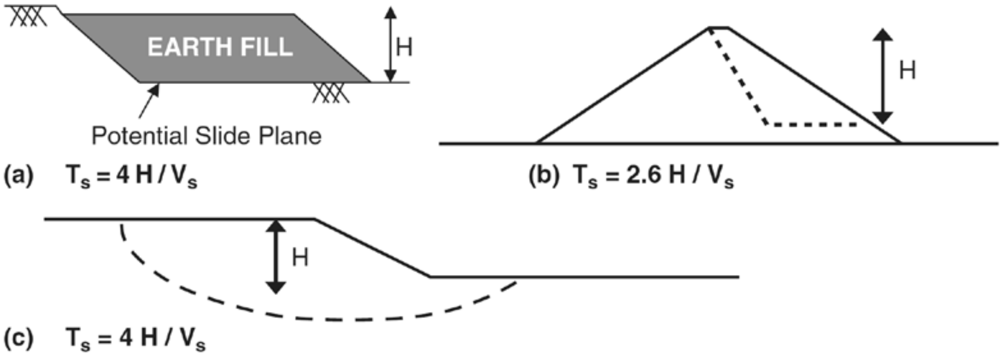


Figure 8.111 Initial fundamental period of potential sliding mass (from Bray, 2007)

If there is not a high probability of negligible displacement, the amount of 'non-zero' displacement, D, can be estimated by the following equation:

$$\begin{aligned}\ln(D) = {} & -1.10 - 2.83\ln(k_c) - 0.333\,[\ln(k_c)]^2 + 0.566\ln(k_c)\ln(S_a(1.5\,T_s)) \\ & +3.04\ln(S_a(1.5\,T_s)) - 0.244\,[\ln(S_a(1.5\,T_s))]^2 + 1.50\,T_s + 0.278\,(M-7) \pm \varepsilon\end{aligned} \quad (8.198)$$

where:

M = magnitude of the earthquake (-)

ε = normally distributed random variable with zero mean and standard deviation $\sigma = 0.66$

To eliminate the bias in the model when $T_s \approx 0$ s, the first term of equation should be replaced with −0.22 when $T_s < 0.05$ s. The median minus one standard deviation to median plus one standard deviation range of seismic displacement can be approximated as half the median estimate to twice the median estimate of seismic displacement. So, the median seismic displacement calculated using equation with $\varepsilon = 0$ can be halved and doubled to develop approximately the 16 to 84 per cent exceedance seismic displacement range estimate.

Validity domain

The Bray and Travasarou method was originally developed for the analysis of embankments (dams and waste dumps) and natural slopes (soil and rock). It was developed to study the maximum deviatoric component of the movement of the embankments on their bases. This approach does not intend to deal with soils susceptible to pore water pressure increase during an earthquake and the related soil softening regime. The statistical model was constructed from 688 recorded accelerograms from 41 earthquakes with magnitudes between 5.5 and 7.6 at an epicentral distance less than 100 km on broad soil foundation types. Calculations concern embankments of height between 12 m and 100 m, shear wave velocities ranging from 200 m/s to 425 m/s, critical seismic coefficient ranging between 0.02 and 0.4, and fundamental periods varying from 0 to 2 seconds. The method takes into account that gravity is constant and equal to 17.6 kN/m^3, with a single curve of shear modulus and damping. They justify this choice by a low incidence of these curves on the results of a sensitivity analysis.

Validation of the method over 16 dams showed good predictions for structures having undergone the lowest seismic displacements (< 5 cm). The model gives the order of magnitude for the largest seismic displacements (5 cm to 50 cm), and generally offers a better prediction than previous methods. The interpretation of this validation test suggests that the estimated displacement would be zero when the method predicts a probability of negligible displacement greater than or equal to 50 per cent. However, a probability of zero per cent of negligible displacements corresponds to measured displacements higher than 15 cm.

8.8.3 Crest settlement

Swaisgood (2003) has carried out an extensive study of case histories of embankment dam behaviour during earthquakes, particularly those that are not susceptible to liquefaction problems. The objectives of the study were to determine if there is a 'normal' trend of seismic deformation that can be predicted and if there are certain factors that consistently have an effect on the amount of damage and deformation incurred during earthquakes. Nearly 70 case histories have been reviewed, compared and statistically analysed in this effort. The results of this empirical study have shown that the most important factors that appear to affect embankment crest settlement during earthquake include the:

- peak ground acceleration at the site
- earthquake magnitude.

The relationship between the magnitude of measured settlement and the peak ground accelerations during earthquake are plotted in Figure 8.112.

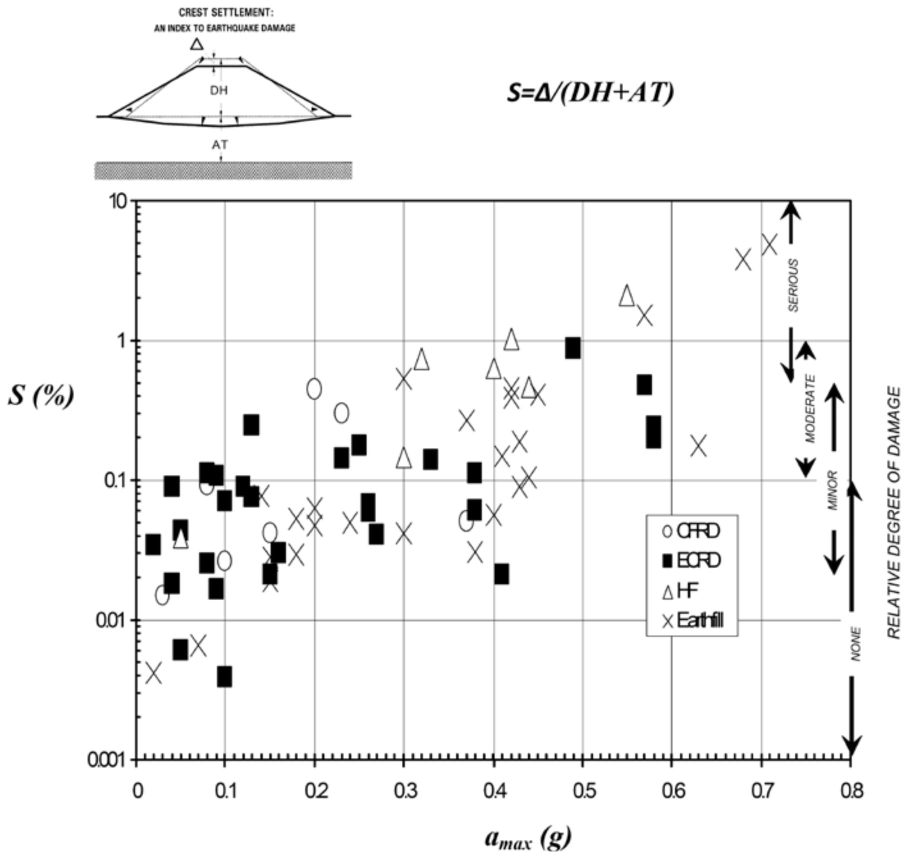


Figure 8.112 Empirical relationship between the peak ground acceleration and crest settlement (Swaisgood, 2003)

An empirical equation was formulated as an aid in estimating the amount of embankment crest settlement as follows:

$$S = \exp\left(6.07a_{max} + 0.57M - 8.00\right) \tag{8.199}$$

where:

S = crest settlement in per cent (%)
a_{max} = peak ground acceleration (g) at the foundation rock
M = earthquake magnitude (-)

Validity domain

As reliability of this kind of method depends on the database where it has been established, this approach only gives an order of the magnitude of the crest settlement. Differences between calculated and measured settlements ranging from one to six are possible. Due to its exponential trend, this formula may be limited to moderately seismic zones. This method may only be used as a rule of thumb in early phases of the project or rapid assessment.

8.8.4 Earthquake-induced liquefaction

Liquefaction designates the generic term for the loss of strength of cohesionless soils due to excess pore water pressure caused by cyclic loading. In many strong earthquakes, liquefaction was observed and caused significant damage to infrastructure and buildings. The mechanism of liquefaction has been studied in detail since the 1960s, starting with the Niigata and Alaska earthquakes in 1964. The knowledge of this mechanism has been gradually improved, allowing a better prediction of its occurrence during earthquakes.

8.8.4.1 Physical processes

The term liquefaction is used to describe phenomena in which the generation of excess pore pressure leads to reduction of effective stress and to softening and/or a significant weakening of effective soil strength (Kramer and Elgamal, 2001). The increase in pore pressure in the soil mass (related to the contractive behaviour of the soil under undrained loading) may be the result of applying quasi-static or dynamic loading, monotonic or cyclic stresses, shock or water transfers between layers. The term liquefaction covers several different physical phenomena such as flow liquefaction and cyclic mobility, which are defined as follows:

- **flow liquefaction** is a phenomenon that occurs when the liquefaction is initiated in a soil whose residual strength is smaller than the resistance necessary for static equilibrium of the environment. This type of failure occurs only in loose soils of low residual strength. It is the consequence of instability, which when triggered causes this movement. It can lead to extremely large deformations (slip-type flow). However, these strains are actually caused by the static shear stresses still present. The cases of flow liquefaction are relatively rare in practice, but they can cause immense damage.
- **cyclic mobility** is a phenomenon in which the shear stresses produce cyclic pore pressures in soil with residual strength greater than that which is necessary for static equilibrium of the medium. This mechanism is often manifested *in situ* in the form of lateral movement (lateral spreading), the process of accumulation of permanent displacements under the effect of static stresses during an earthquake. These deformations can occur in both relatively dense and loose soil with amplitudes more or less pronounced.

The contrasting views on the definition of soil liquefaction are due to the different approaches considered, depending on whether one prefers sites (and structures) or actions, or behavioural aspects of sandy soils in terms of description by laboratory tests or in terms of modelling. However, these definitions separate the effects of movement within the soil mass from the mechanism that drives the movement. Often, it is difficult or impossible to differentiate in the field in many cases. Note that the term residual strength seems a misnomer to sand if referring to the ultimate shear strength, but it is has become customary.

Finally, it is necessary to distinguish the concept of susceptibility to liquefaction and the liquefaction potential. According to Youd and Perkins (1978), the susceptibility to liquefaction of the soil corresponds to the fact that the soil is unable to withstand cyclic shear stresses. It depends on the particle properties, soil structure (texture), void ratio, and initial conditions. The liquefaction potential of a mass of soil concerns the risk of liquefaction of the soil for given seismic conditions. The potential for liquefaction depends on the seismic excitation and susceptibility of the soil to liquefaction as a behaviour.

8.8.4.2 Governing parameters

In the field, liquefaction generally concerns cohesionless soils and particularly fine to coarse sands, especially when such materials have a uniform size. But this view, oriented towards the behaviour of an elementary volume of soil (laboratory test), is not sufficient to characterise liquefaction phenomenon at the scale of a soil layer, because many additional factors are involved in this process.

The soil liquefaction susceptibility is the inability of soil to resist shear stress and monotonic or cyclic loading. It does not only depend on the physical and mechanical properties of soil. Liquefaction potential of a soil mass concerns the risk of liquefaction of the mass in the considered loading conditions, monotonic or cyclic. Liquefaction potential of a soil layer depends on several factors, which are not always easy to distinguish in the field (Prakash, 1981). These factors may be listed as follows:

1 2 3 4 5 6 7 8 9 10

- parameters related to the site:
  - thickness and depth of the layer
  - morphology of the site
  - profile of the underlying soils, the depth of substratum, surface layer (and all their physical, mechanical and hydraulic properties)
  - saturation and drainage conditions (hydraulic boundary conditions)
  - degrees of freedom of ground motion in the kinematic conditions of the site
  - structures built on the site, including soil reinforcement
- parameters related to the load:
  - type of load applied to the soil from its original state, static (monotonic) or dynamic (cyclic)
  - in static mode, the load and speed
  - dynamic mode, intensity, frequency and duration of loading and, in terms of earthquakes, the intensity and duration of seismic motion, the distance to the source etc
- parameters related to the soil:
  - history and age of the deposit
  - the soil physical properties (particle size, specific gravity etc), Its structure, its homogeneity, cementing
  - mechanical properties (undrained strength, deformability), hydraulic (permeability), and its anisotropy
  - the initial state mechanical, with the depth and variable depending on the history of the soil.

Density

The mechanical behaviour of sandy soils depends on two main factors, their physical nature and state. The physical nature of the soil corresponds to the mineralogical composition of grains, their size distribution, shape and angularity, fines content etc. Soil state refers to conditions under which the soil occurs. This state is described by variables such as relative density (represented by the density index, I_D. See Section 7.8.3), soil texture, and effective stress in the field.

In general, the cyclic strength of sands depends strongly on density. Loose sands are collapsible under drained conditions and have a higher susceptibility to liquefaction under undrained conditions.

Age effect

The resistance of sandy soils to liquefaction is not only influenced by the relative density and grain size distribution, including the presence of fines, but also by the age of the formation, which affects the soil structure. The precise mechanism of aging of soils is still not well understood. However, these changes are related to mechanical processes such as sedimentation, over-consolidation or rearrangement of particles in configurations more stable and/or physicochemical processes of cementation by creating links to soil particle contacts.

Youd and Perkins (1978) noted that the most recent soils, ie younger than 500 years, have a susceptibility to liquefaction of high to very high. The oldest soils of Holocene age (500 to 10000 years) have moderate to high susceptibility and soils of Pleistocene age (10000 to 1.8 million years) a very low to low susceptibility.

Initial stress conditions

The stress states encountered *in situ* are not isotropic and there are many situations in practice where the soil foundation supports a non-zero initial shear stress on a horizontal plane (on a slope or at the foot of a foundation or earth structure). In the simplest situations, the initial stress states are defined by a coefficient of earth pressure at rest K_0 different from unity, which is the ratio of effective horizontal stress σ'_{h0} and effective vertical stress σ'_{v0} to the depth z ($K_0 = \sigma'_{h0}/\sigma'_{v0}$). During the earthquake, the soil element is subjected to additional shear stresses (cyclic) due to wave propagation in the soil mass. The presence of initial static stress plays an important role on the cyclic response of the soil.

Work dedicated to evaluate the effect of initial stresses on cyclic resistance lead to contradictory conclusions. It appears that the cyclic strength of sands can both increase or decrease in the presence of anisotropic initial stress, according to the relative density of sand (as it is expanding or contracting), the level of static shear stresses, and amplitude of cyclic shear, Δq_{SA} (alternating cycles or not).

Loading mode

In the laboratory, the cyclic strength of sands depends on the mode of loading (triaxial, simple shear, torsion or other). Correction factors have been proposed to standardise the measurements of resistance from different sources. More generally, the resistance also depends on the nature of the unidirectional or multidirectional seismic signals.

Soil structure

The cyclic resistance of reconstituted sands in the laboratory depend strongly on the method of specimen preparation (pluviation, moist tamping, compaction). This shows the importance of structural effects.

Many other factors have been studied experimentally concerning the nature of the sands, their structure (given by the method of preparation for the soil reconstituted), shape and grain size, grain crushability etc. These factors often appear to have significant influence, at least in small deformations until the outbreak of a possible instability of the sand, and seem to have less influence during the regime of steady flow.

8.8.4.3 Liquefaction criteria for sands

From a phenomenological point of view, the definition of liquefaction of sands based on monotonic and cyclic undrained laboratory tests expresses:

- two successive stages in the process, a step of triggering (or not) of the instability of sand, followed by a step of flowing (or not)
- trigger thresholds, which can be defined by different criteria formulated in terms of deformations, stresses, or pore pressures or another combination of the previous parameters.

All definitions given in the literature are not equivalent and all thresholds are not interchangeable because they often depend on the conditions under which they were defined (including loading modes, the initial state, and the structure of sand etc). So, it is important to distinguish the triggering criteria of liquefaction of sands and its potential effects.

In terms of cyclic loading, failure is defined by a threshold axial strain reached for a given number of cycles of uniform shear stress. This definition of liquefaction corresponds to the point where a sudden loss of resistance is observed followed by unlimited deformation (steady-state deformation). Other definitions are based on the ratio of pore pressure $r_u = \sigma_v/u$ and liquefaction initiation defined as the moment when r_u = 100 per cent.

In practice, the definition of criteria for liquefaction is quite arbitrary. Indeed, the thresholds are defined in the range of small and medium deformations. One great difficulty is that the state conditions strongly affect the behaviour of sands in this deformation range and that the criteria are highly dependent on, among others, the loading modes. So, it is difficult to define criteria for liquefaction triggering in general and applicable to *in situ* (unknown) conditions prevailing at the sites.

In situ state of sands

Studies of the behaviour of natural sand cannot be undertaken without an effective means of collecting these materials *in situ*. Various techniques are used including large diameter sampling or freezing before sampling and transportation to the laboratory. Testing has shown that the cyclic resistance of natural sands is generally larger than their equivalent reconstituted samples at the same density. These additional resistances were probably acquired at the time of their deposition and subsequent aging.

8.8.4.4 Clayey soils liquefaction potential

It is difficult to determine the susceptibility to liquefaction of silty and clayey soils and, where they are susceptible, how to characterise this. In other words, "are the test methods and criteria for sands transferable and applicable to clayey and/or silty soils?" These questions arise primarily for recent unconsolidated deposits, in which these materials are generally soft, not very resistant and very deformable.

Boulanger and Idriss (2006) propose new criteria to qualify the "liquefaction susceptibility" of saturated silts and clays. The term liquefaction is used incorrectly in this case, even if these materials can exhibit high levels of strain during monotonic or cyclic loads. Also, considering the fine soils as a whole, the authors advocate using the term liquefaction to describe the large deformation or loss of strength that appear in sandy soils (sand-like) and the term of cyclic softening to describe similar mechanisms that appear in clayey soils (clay-like).

Recent poorly consolidated clayey soils are soft and deformable. They have a very low resistance, undrained shear strength, c_u, in terms of total stress, with low deformation moduli. In natural homogeneous normally consolidated deposits, c_u increases approximately linearly with depth. The relationship between c_u and effective vertical stress σ'_{v0} is of the order of 0.2 ($c_u/\sigma'_{v0} \approx 0.2$). But the undrained cohesion also depends on the degree of over-consolidation of the clay. Undrained cohesion is used to normalise the mechanical properties of clays, as the ratio τ_{cyc}/C_u for example, where τ_{cyc} is the cyclic shear strength. Undrained strength of clays also increases with the speed of loading, five to 15 per cent per log cycle of the shear rate.

Undrained cyclic tests generally show a progressive amplification of deformations, associated with a gradual increase in pore pressure, showing no signs of instability, unless a particular case of sensitive clays. The state of zero effective stress is not reached during cycles (r_u < 100 per cent).

8.8.4.5 Silty soils liquefaction potential

Silts can be considered intermediate between sands and clays in terms of undrained behaviour. Many authors have emphasised their liquefiable character in support of *in situ* observations from different earthquakes. As for the sandy soils, many experimental studies focus on laboratory tests on reconstituted soils by mixing sand with silt particles, or even clay particles. As for reconstituted sands, the representativeness of these mixtures is often discussed.

Intact silt samples under undrained conditions show a behaviour under cyclic leading intermediate between natural unconsolidated clays and sands. By filiation with the sands, their dilatancy helps limit deformations. But dilatancy cannot be exacerbated as well as in clean sands because the voids are filled by fine particles. However, according to several studies, it appears that silts or silty clayed sands with low to medium plasticity behave differently from the sands during cyclic loading under undrained conditions, in terms of progression of the deformations and pore pressure generation in particular. The data available to date does not reveal any instability mechanism in intact silts.

There is a degree of confusion in the engineering profession about the liquefaction susceptibility of silty soils (Andrews and Martin, 2000). Because the grain size of silt falls between that of sand and clay, it is often assumed that the liquefaction susceptibility of silts must also fall somewhere between the high susceptibility of sands and the non-susceptibility of clays. Confusion about the liquefaction susceptibility of silty soils is further exacerbated whenever silts and clays are coupled under the one heading 'fines'.

8.8.4.6 Physical properties of soils criteria

Procedures to identify potentially liquefiable soils have been developed around the consistency limits, particle size distribution and the water content, or combinations of these properties. These procedures are based on a proposal by Wang (1979), later developed by Seed and Idriss (1982) as the Chinese criteria (Figure 8.113).

These criteria are used to identify suspicious soils with respect to the risk of liquefaction and non-susceptible soils, considering the site conditions and the seismic level. However, authors such as Boulanger and Idriss (2006) and others believe that these criteria are often misinterpreted as evidence of liquefaction risk exclusion, and should be abandoned in practice.

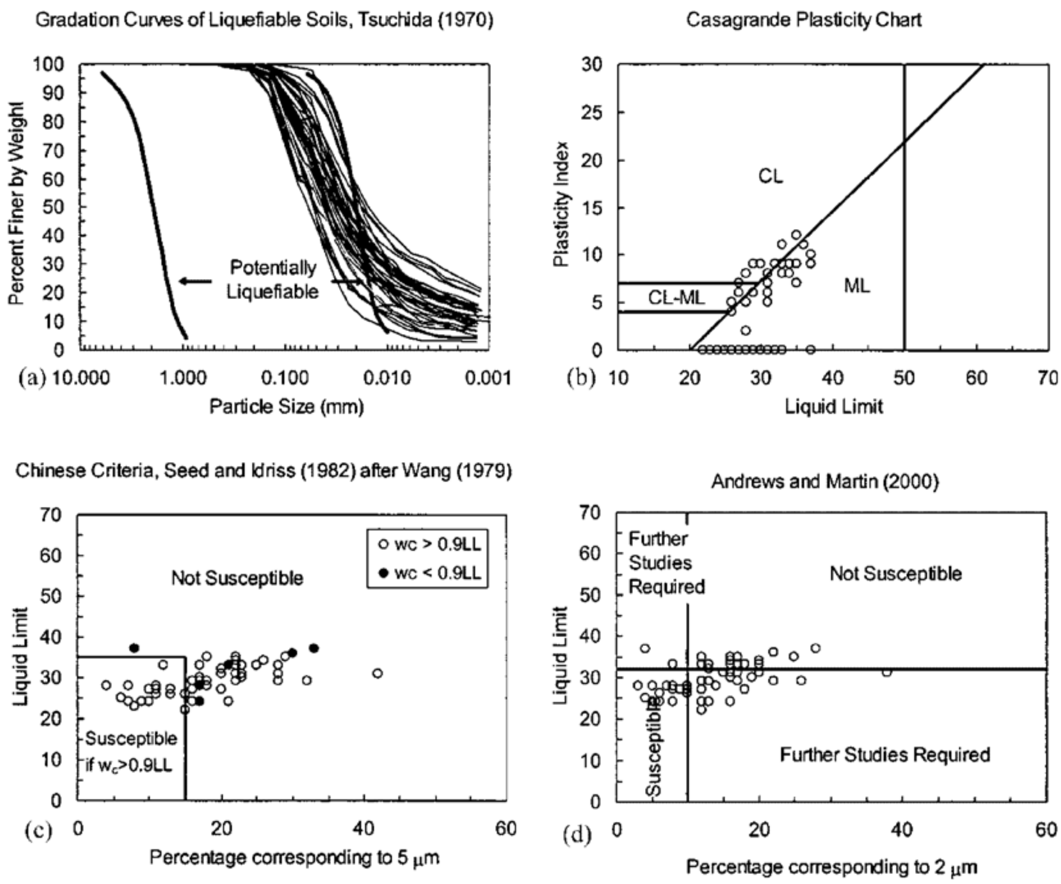


Figure 8.113 Different criteria for determining liquefaction susceptibility of fine-grained soils (from Seed and Idriss, 2004)

However, the physical properties of fine soils may still constitute a guide to a soils' susceptibility to liquefaction. These properties can provide useful clues to distinguish soils with sand-like behaviour from those with clay-like behaviour, as advocated by Boulanger and Idriss (2004). Based on a large number of references to undrained monotonic and cyclic tests compiled in various literature, these authors propose to classify soils into these two families of behaviour. These classifications are associated with consistency limits, which allow display of an intermediate class between sandy type soils and clay type soils. The transition is on a very narrow range of the plasticity index ranging from three to seven (Figure 8.114).

The authors then propose rules for practical applications. Soils with plasticity index greater than 7 (PI ≥ 7) can be considered as clay-like. This includes clay of low plasticity (CL). For soils classified as silt and clay of low plasticity (ML-CL), the criterion may be reduced to PI ≥ 5. Soils that do not meet this criterion should be considered sand-like and eventually liquefiable, unless specific laboratory or *in situ* tests show otherwise. These soils are those for which the correlations with field tests are most appropriate to assess their cyclic strength (CPT, SPT). For soils whose behaviour is intermediate and PI whose indices range from three to six, it is recommended to perform laboratory tests in conjunction with field tests, which are not considered totally reliable in this case. In the absence of laboratory tests, the threshold remains at PI = 7.

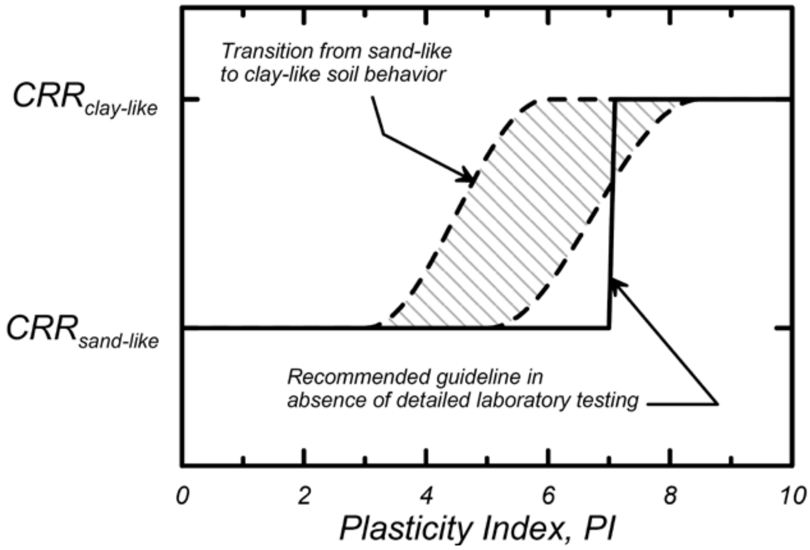


***Figure 8.114** Schematic illustration of the transition from sand-like to clay-like behaviour for fine-grained soils with increasing PI, CRR = cyclic resistance ratio (from Seed and Idriss, 2004)*

8.8.4.7 Simplified methods

A simplified procedure for the evaluation of soil resistance to liquefaction was proposed by Seed and Idriss (1971). The liquefaction resistance is expressed by means of the cyclic resistance ratio (CRR), while the cyclic loading imposed on the ground by the earthquake is expressed by the cyclic stress ratio (CSR). Soil liquefaction is possible if CSR ≥ CRR.

Two hypotheses form the basis of the simplified methods. The first hypothesis assumes that the shear stresses act on a horizontal plane in the soil mass. This assumption is based on an approximation that the shear waves propagate vertically from the bottom to the top of the soil column. The second hypothesis is to assume that irregular seismic signals can be converted into equivalent signals whose amplitude is uniform and in relation with the peak acceleration surface.

Determination of cyclic stress ratio

Under these hypotheses, seismic induced stresses into the soil mass reduce to a shear stress where the maximum value at depth, z, is given by:

$$\tau_{max} = \sigma_{v0} \frac{a_{max}}{g} r_d \tag{8.200}$$

where a_{max} is the maximum surface acceleration (m/s^2), g the gravitational constant (9.81 m/s^2), σ_{v0} the total vertical stress and r_d a stress reduction coefficient that accounts for the flexibility of the soil column (ie r_d = 1 corresponds to rigid body behaviour), which decrease from one at the surface to approximately 0.9 at 12 m depth. Equivalent uniform cyclic stress produced by the seismic loading, τ_{sis}, at the considered depth may be expressed as:

$$\tau_{sis} = 0.65\, \tau_{max} \tag{8.201}$$

The coefficient 0.65 defines a value of cycles more representative of loading experienced by the soil mass during the earthquake. Other close values have been proposed in the literature (0.67 or 0.66). In this approach, the cyclic stress ratio is defined by:

$$CSR = \frac{\tau_{sis}}{\sigma'_{v0}} \tag{8.202}$$

where σ'_v is the effective vertical stress at the considered depth. The method also introduces a magnitude scaling factor (MSF) to provide an approximate representation of the effects of shaking duration or the equivalent number of stress cycles. The cyclic stress ratio is defined as a normalising factor to enable the comparison of different magnitude earthquakes:

$$MSF = \frac{CSR_M}{CSR_{7.5}} \tag{8.203}$$

Finally, normalised cyclic stress ratio is calculated by:

$$CSR_{7.5} = 0.65\, a_{max} \frac{\sigma_{v0}}{\sigma'_{v0}} \frac{r_d}{MSF} \tag{8.204}$$

Determination of MSF

The relation proposed by Idriss (1999) is shown in Figure 8.115 and expressed as:

$$MSF = 6.9\, \mathrm{e}^{-M/4} - 0.058 \leq 1.8 \tag{8.205}$$

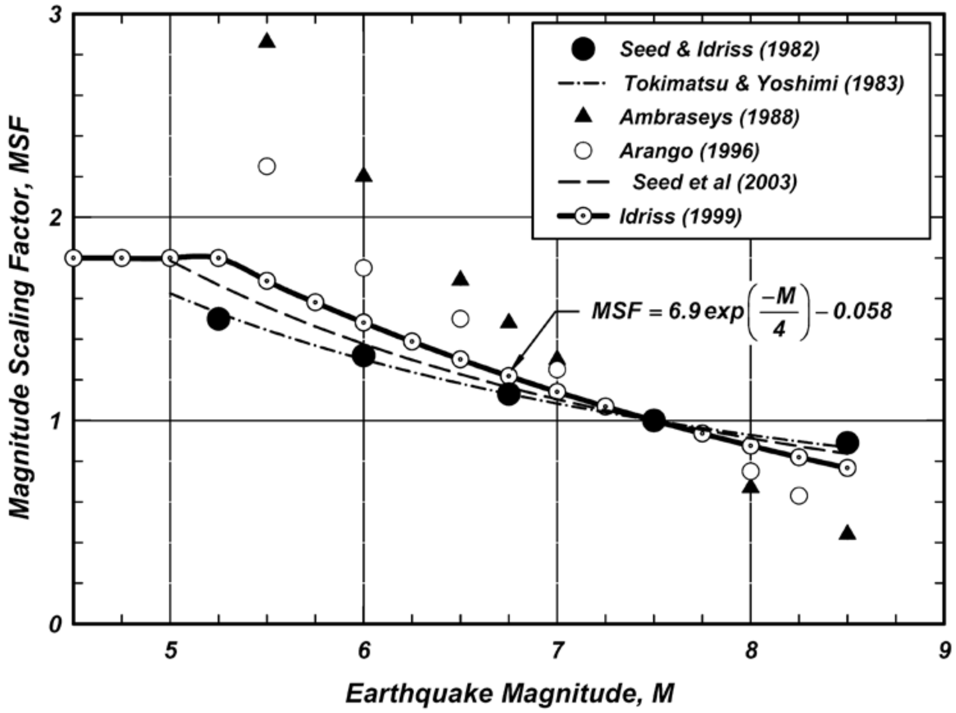


Figure 8.115 Magnitude scaling factor proposed by various investigators (Boulanger and Idriss, 2004)

Determination of r_d factor

The r_d parameter should be expressed in terms of depth and earthquake magnitude (Figure 8.116). The following empirical relation was derived by Idriss (1999):

$$r_d = -1.012 - 1.126 \sin\left(\frac{z}{11.73} + 5.133\right) + M\left[0.106 + 0.118 \sin\left(\frac{z}{11.28} + 5.142\right)\right] \tag{8.206}$$

where:

z = depth (m)

M = magnitude of the earthquake (-)

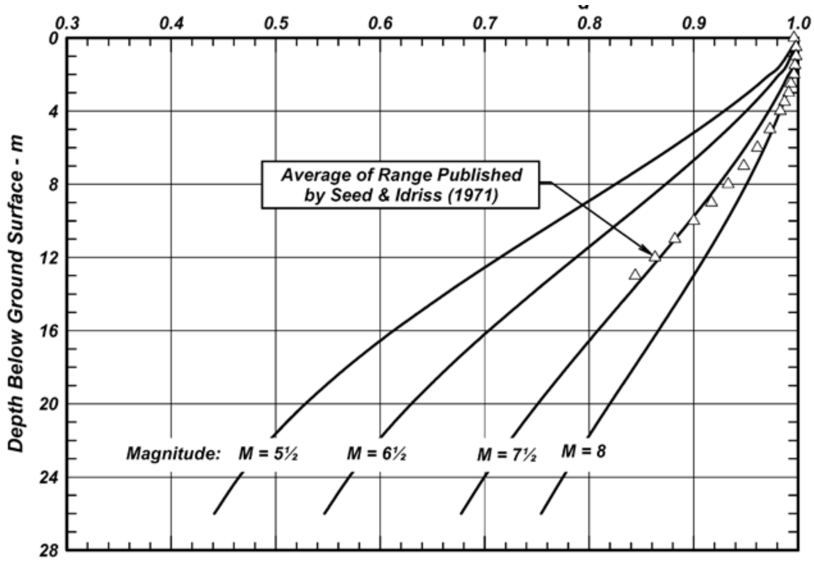


Figure 8.116 Variation of strength reduction factor with depth and earthquake magnitude (Boulanger and Idriss, 2004)

Normalised SPT and CPT resistances

According to Idriss and Boulanger (2004), the effective use of SPT blow count and CPT tip resistance as indices for soil liquefaction characteristics require that the effect of soil density and effective confining stress on penetration resistance be separated. Consequently, Seed *et al* (1975) included the normalisation of penetration resistance in sand to an equivalent σ'_{v0} = 1 atm (101 kPa) as part of the empirical procedure. The purpose of the overburden normalisation is to obtain quantities that are independent of σ'_{v0} and so are more likely to relate to the sands relative density (Boulanger and Idriss, 2004).

SPT test

For SPT tests, this normalisation currently takes the form:

$$N_{1,60} = C_N \frac{E}{E_{60}} N \tag{8.207}$$

where:

C_N = correction factors (-)
E = transmitted SPT energy blow (J)
E_{60} = 60 per cent energy blow efficiency (J)
N = SPR blow count (-)

Boulanger and Idriss (2004) proposed the following expressions for determining correction factor from an iteration process:

$$C_N = \left(\frac{P_a}{\sigma'_{v0}}\right)^{0.784-0.0768\sqrt{N_{1,60}}} \leq 1.7 \tag{8.208}$$

CPT test

The normalised cone tip resistance is given by:

$$q_{c,1N} = C_q \, q_c \tag{8.209}$$

where:

C_q = correction factors (-)
q_c = cone tip resistance (kPa)

The correction factor is also estimated iteratively from the empirical formula:

$$C_q = \left(\frac{P_a}{\sigma'_{v0}}\right)^{1.338-0.249q_{c1N}} \leq 1.7 \tag{8.210}$$

Shear wave velocity test

As for SPT and CPT resistances, the shear wave velocity is normalised as follows:

$$V_{s1} = C_V\, V_s \tag{8.211}$$

with

$$C_V = \left(\frac{P_a}{\sigma'_{v0}}\right)^{0.25} \leq 1.4 \tag{8.212}$$

Determination of cyclic resistance ratio (CRR)

Once the resistances have been normalised, the value of cyclic resistance ratio for a magnitude M = 7.5 and an effective vertical stress σ'_{v0} = 1 atm may be estimated based on the field test data (SPT, CPT, Shear wave velocity) and Equations 8.213 to 8.217 as detailed following.

SPT test

To estimate the cyclic resistance ratio, the SPT penetration resistance is adjusted to an equivalent clean sand value as:

$$N_{1,60cs} = N_{1,60} + \exp\left(1.63 + \frac{9.7}{\mathrm{FC}} - \left(\frac{15.7}{\mathrm{FC}}\right)^2\right) \tag{8.213}$$

where FC is the soil fine content (%) defined as the proportion of fines retained by a no 200 sieve (D < 0.075 mm). The variation of SPT blow count with correction factor, C_N, is shown in Figure 8.117.

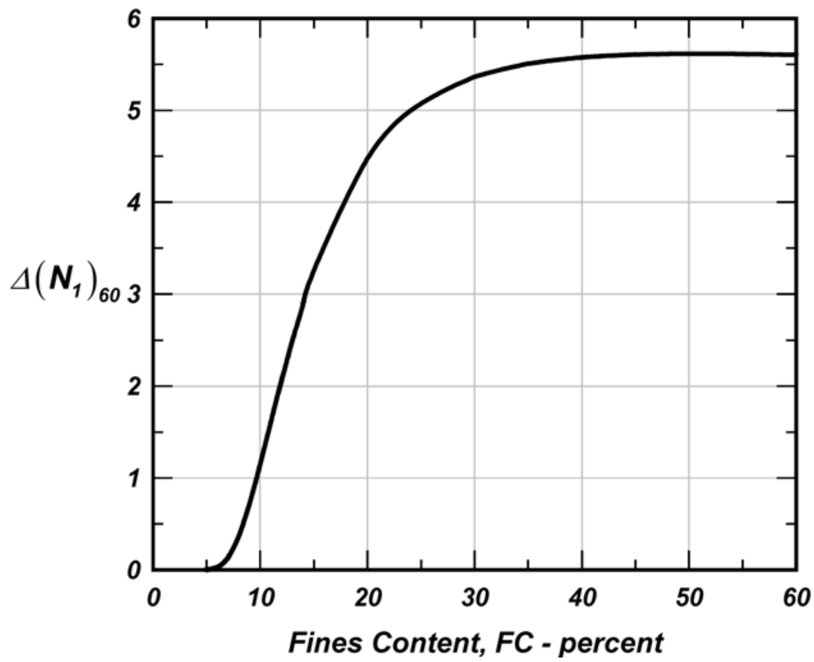


Figure 8.117 Variation of SPT blow count correction factor with fine content (Boulanger and Idriss, 2004)

So, following Boulanger and Idriss (2004), the cyclic resistance ratio is shown in Figure 8.118 and expressed as:

$$CRR_{7.5} = \exp\left(\frac{N_{1,60cs}}{14.1} + \left(\frac{N_{1,60cs}}{126}\right)^2 - \left(\frac{N_{1,60cs}}{23.6}\right)^3 + \left(\frac{N_{1,60cs}}{25.4}\right)^4 - 2.8\right) \tag{8.214}$$

1 2 3 4 5 6 7 8 9 10

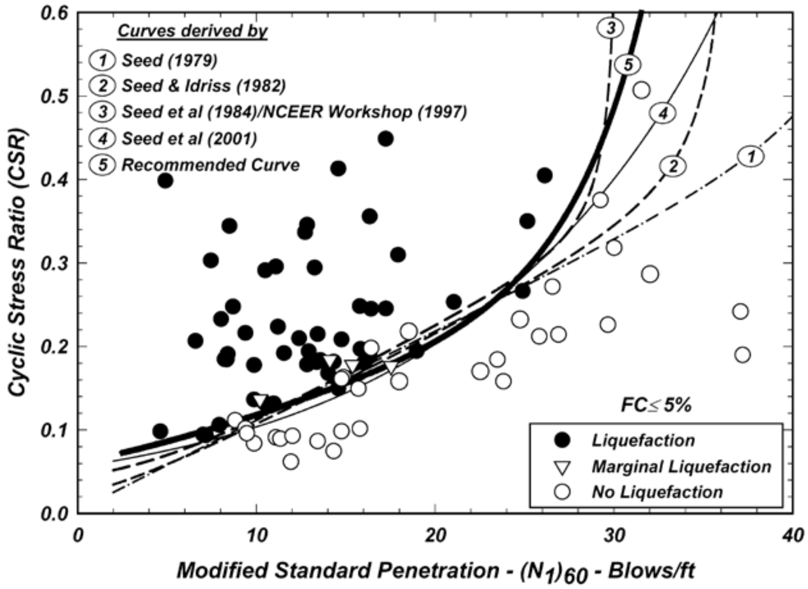


Figure 8.118 Curves relating CRR to $N_{1,60}$ for clean sands and the curves recommended by Boulanger and Idriss (2004) for M = 7.5 and σ'$_0$ = 1 atm

Figure 8.118 has plotted the corrected SPT blow counts versus the corresponding cyclic stress ratio for numerous field sites where liquefaction was or was not observed following an earthquake. The different lines correspond to different curves proposed to fit the limit between liquefaction and no liquefaction zones.

CPT test

In the case of CPT tests, and for FC ≤ five per cent soils, the expression of cyclic resistance ratio is shown in Figure 8.119 and may be expressed as:

$$CRR_{7.5} = \exp\left(\frac{q_{c1N}}{540} + \left(\frac{q_{c1N}}{67}\right)^2 - \left(\frac{q_{c1N}}{80}\right)^3 + \left(\frac{q_{c1N}}{114}\right)^4 - 3.0\right) \tag{8.215}$$

For fine contents ≥ five per cent, specific procedures may be used to determine CPT resistance, such as Robertson and Wride (1997), which introduce a soil behaviour type index as a function of cone tip resistance and sleeve friction ratio, or Moss (2003), which use CSR and R_f values to estimate the fine content adjustment.

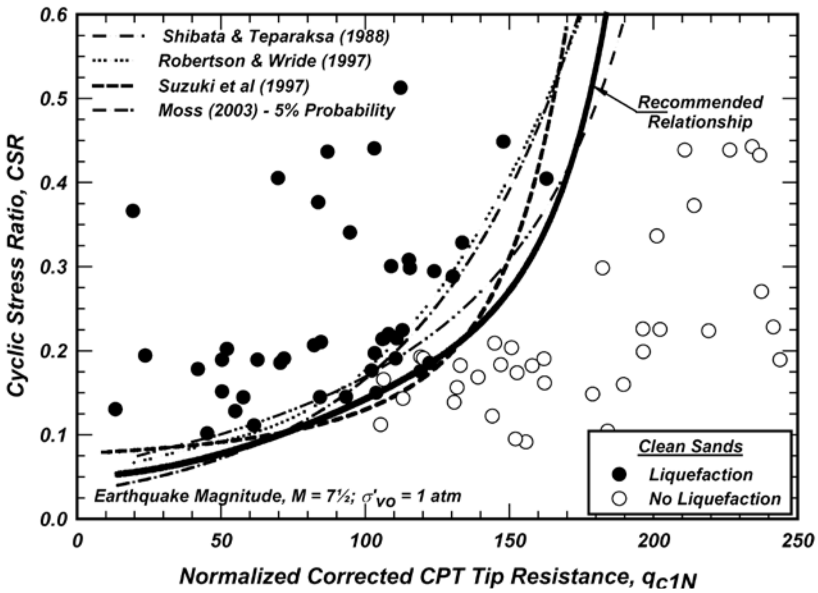


Figure 8.119 Curves relating CRR to q_{c1N} for clean sands and the curves recommended by Boulanger and Idriss (2004) for M = 7.5 and σ'$_{v0}$ = 1 atm

Shear wave velocity test

The cyclic resistance ratio based on shear wave velocity is shown in Figure 8.120 and expressed as:

$$CRR_{7.5} = \left\{ 0.22 \left(\frac{V_{s1}}{100} \right)^2 + 2.8 \left(\frac{1}{V_{s1}^* - V_{s1}} - \frac{1}{V_{s1}^*} \right) \right\} \tag{8.216}$$

with

$$200 \leq V_{s1}^* = 215 - 0.5\,(FC - 5) \leq 215 \tag{8.217}$$

where FC is the soil fine content and V_{s1}^* the limiting upper value of V_{s1} for liquefaction occurrence. The curves recommended by Andrus and Stokoe (2000) are shown in Figure 8.120. On this figure, the dashed line indicates that field performance data are limited. They do not extend much below 100 m/s, because no field data were available to support extending them to the origin. It is important to note that these boundary curves are for extreme behaviour, where boils and ground cracks occur.

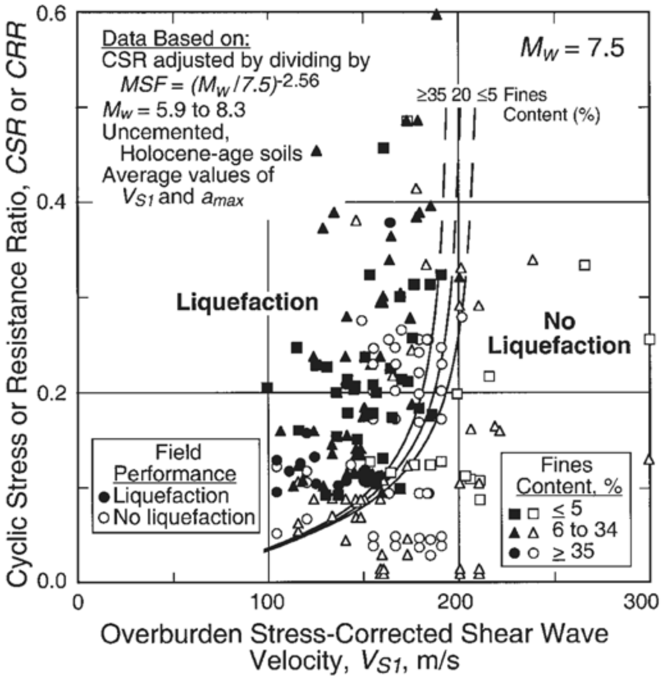


Figure 8.120 Curves relating CRR to V_{s1} for clean sands and the curves recommended by Andus and Stokoe (2000) for M = 7.5 and σ'_{v0} = 1 atm

8.8.4.8 Modelling soil liquefaction

Most commonly used methods of ground response analysis are based on the equivalent linear model (Seed, 1973). This model is a total stress approach and so does not take into account the effect of pore water pressures on soil properties and dynamic response during earthquake or cyclic loading. This was a major factor leading to the development of effective stress analysis models that are able to generate pore water pressures during earthquake or cyclic loading.

Semi coupled models

- **Martin-Finn-Seed model**

The first model of this kind was developed by Martin *et al* (1975) and Seed *et al* (1976). They proposed a relationship between pore water pressure and the number of uniform shear stress cycles that cause liquefaction (determined from cyclic triaxial tests).

It has to be noted that some commercial codes propose successive use of equivalent linear models (eg QUAKE/W) to determine a pore pressure ratio function based on equivalent number of uniform cycles.

The pore pressure ratio is then implemented in slope stability analysis tools (SLOPE/W) to determine the effective shear strength of the soils along a given slip surface. This simplified approach may be useful in the context of a tiered approach.

- **Pecker model**

The basic idea of this model (Pecker *et al*, 2001) consists of splitting the time into two separate scales associated with (a) the cyclic loading (fast time) and (b) with the steady pore pressure increase and dissipation (slow time). The fastest phenomena (pore pressure build-up or pore pressure dissipation) will govern the residual pore pressure at any time. Under constant mean pressure, the pore pressure increment depends on the bulk modulus of soil skeleton and the volumetric strain increment.

Fully coupled constitutive models

Various constitutive models have been developed that can capture the liquefaction behaviour of sands (Jefferies 1993, Drescher *et al*, 1995, Byrne *et al*, 1995, Gudehus, 1996, Wolffersdorff, 1996, Drescher and Mróz, 1997, Puebla *et al*, 1997, Niemunis and Herle, 1997, Beaty and Byrne, 1998, Yu, 1998, Boukpeti and Drescher, 2000, Boukpeti *et al*, 2002, Jefferies and Shuttle, 2002, Mróz *et al*, 2003, and Imam *et al*, 2005). The practical application of a constitutive model for a geotechnical problem is only possible when the model is implemented in a finite element/finite difference program. Some of the constitutive models that are implemented (as user defined soil models) in commercial finite element/difference codes are described here:

- **UBCSAND Model**

This model was developed at the University of British Columbia (Byrne *et al*, 1995, Puebla *et al*, 1997, and Beaty and Byrne, 1998). It is an elastic–plastic model developed specifically for liquefaction behaviour of sand. The model is implemented in the commercial computer code FLAC (Fast Lagrangian analysis of Continua, ITASCA 2005). The UBCSAND model has also been implemented (Tsegaye, 2010) in the finite element program PLAXIS (Brinkgreve *et al*, 2010).

- **Hypoplastic model for sand**

Hypoplasticity is a newly developed framework for constitutive modelling of granular materials. Unlike elasto-plasticity, hypoplasticity does not make use of the concepts such as yield surface and plastic potential (Kolymbas, 2000). There are several versions of hypoplasticity available in literature. The Hypoplastic model (Wolffersdorff, 1996) has been implemented (Masin, 2010) in the finite element program PLAXIS.

- **NorSand model**

NorSand is a critical state elastic-plastic constitutive model (Jefferies, 1993, and Jefferies and Shuttle, 2002). NorSand has been used for modelling a range of soils from clayey silt to sand (Shuttle and Jefferies, 2010). This model is capable of capturing the liquefaction behaviour of sands.

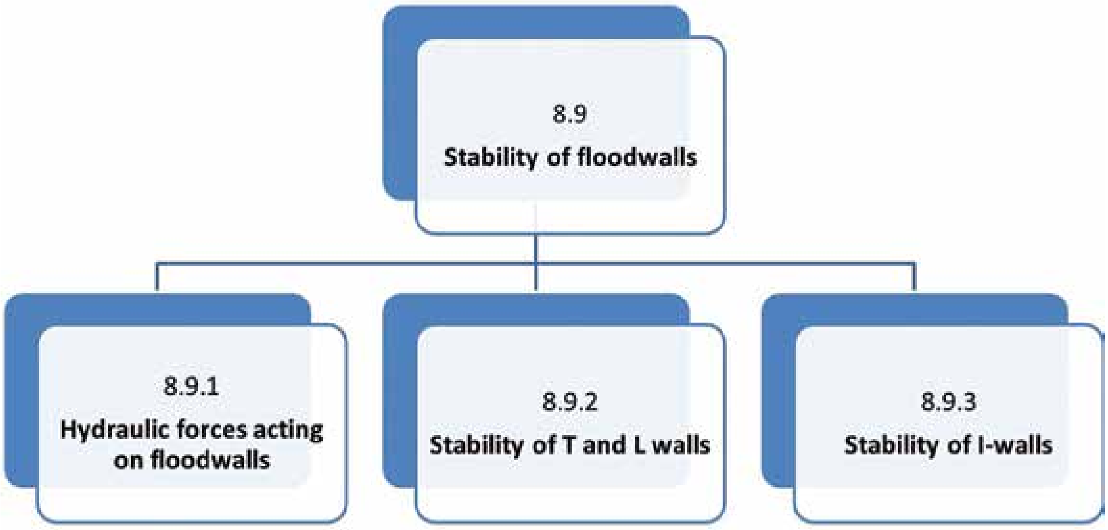
- **CASM – a unified state parameter model for clay and sand**

CASM (clay and sand model) is a critical state elastic-plastic model developed by Yu (1998) and further extended by Yu *et al* (2006). CASM is capable of predicting the behaviour of clay and sand under both drained and undrained loading conditions. CASM has been implemented (Khong, 2004) into the finite element program CRISP (CRItical State soil mechanics Program).

Limitations of phenomenological models

It has been shown (Finn, 1999) that comparison between observed cyclic response and model predictions for general loading paths were largely disappointing. In fact, despite the theoretical generality of these models, the predictions of elastic-plastic methods can be strongly path dependant. The predictions are good for loading paths close to those used to calibrate the models, but for paths far from these the predictions are often poor.

8.9 STABILITY OF FLOOD WALLS



Flood walls are often an integral part of a flood protection system and are of particular concern when they are embedded into the levee section or form transitions with a levee segment. This section will detail the stability of flood walls as presented in the flow chart.

8.9.1 Hydraulic forces acting on flood walls

Analysis and design of flood wall components of levee systems requires consideration of both static and dynamic forces. Static forces result when the structure contains a level of water on one side that is stationary so pressures over the face of the wall are hydrostatic.

Where the water is moving, additional dynamic forces come into play. These wave forces are primarily estimated using impulse-momentum methods, often using empirical methods developed specifically for estimating wave forces on vertical walls.

Wave action on the wall is the primary dynamic loading to be considered for flood walls (USACE, 1989). In the case of waves, a distinction is made between the action of nonbreaking, breaking, and broken waves, where the methods are recommended for calculation of wave forces on vertical walls. Wave forces on other types of walls (ie sloping, stepped, and curved) are less well understood, so general analytical design methods may need further extension. For these instances, a coastal engineer should be involved in establishing wave forces for the design of important structures where wave forces can be expected.

8.9.1.1 Hydrostatic forces

The horizontal force produced by water acts perpendicular to the surface of the object containing it (in this case the flood wall). The pressure that water exerts on a vertical surface can be calculated by multiplying the density of water, γ, with the depth of water at the point of interest, y, as indicated in Equation 8.218. The pressure varies linearly with depth increase as indicated in Figure 8.121. The water density may be assumed constant for depths associated with flood walls, but will be determined by whether the water body is composed of fresh, brackish, or seawater in the case of rivers, estuaries, and oceans, respectively.

$$p(y) = \gamma\, y \tag{8.218}$$

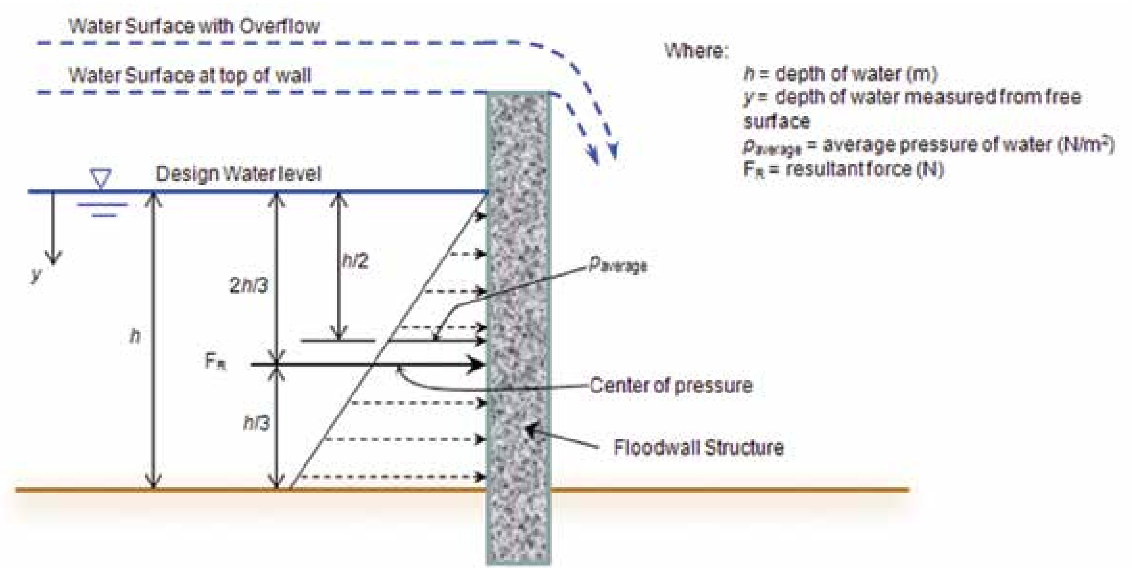


Figure 8.121 ***Hydrostatic pressure acting on vertical flood wall***

The horizontal force acts at the centroid of the pressure distribution, which is 2/3×h below the water surface (Figure 8.121) for a vertical wall. In general, the force at any point on a vertical wall is a function of the depth of water to the point of interest. Where a flood wall has a sloped face on the waterside, both horizontal and vertical forces must be considered. The same methodology can be applied where curved or other irregularly shaped flood wall sections may exist. The reader is referred to classical hydraulics texts for formula needed for calculating centroids and areas for these shapes.

Due to the uncertainty associated with estimating the magnitude of flow on river levee systems, it is recommended that forces acting upon flood walls are calculated for:

- the design water level
- the water levels equal to the top of the flood wall crest and
- the maximum possible water level that results in overflow, if applicable (Figure 8.121).

The critical loading case to be considered for design should be where h equals the full height of the wall or the highest anticipated water level if greater than the wall height.

8.9.1.2 Dynamic forces

It has been appreciated for many years that apparently similar wave conditions may give rise to dramatically different wave pressures or forces depending on the form of wave breaking at, onto, or close to the wall. Under wind waves, there will inevitably be a wide range of wave breaking, but it is generally convenient to use categories of wave load/breaking conditions from the following:

1. Nonbreaking or pulsating.
2. Impulsive breaking or impact.
3. Broken waves.
4. Post breaking or bore waves.

Wave pressures on a vertical wall for two of these breaking types are illustrated in Figure 8.122 – nonbreaking versus impulsive. The simplest case, (type a), is generally when the wave is nonbreaking, also termed reflecting or pulsating. For this condition, the wave motion is relatively smooth, and the main processes can be predicted by simple wave theories. Simple prediction methods for pulsating wave loads by Goda or Ito generally predict average pressures up to about $p_{av} = 2\rho g H_s$ where H_s is the incident (local) significant wave height.

Much more intense wave forces/pressures arise if the wave can break directly against the wall, termed plunging, breaking, impulsive or impact (type b). Research studies in Europe have measured local wave impact pressures up to or greater than $p_{impact} = 40\rho g H_s$, much higher than would be predicted by simple design methods (Allsop and Vicinanza, 1996 and Allsop *et al*, 1996a). In extremis, tests by Kirkgoz (1995) suggest impact pressures up to $p_{impact} = 100\rho g H$, although these are highly unlikely in practice.

Impulsive breaking is strongly influenced by any mound, berm, or steep bed slope in front of the wall with conditions difficult to predict, and producing significant variability/uncertainty. In the past, these variations have led to significant lack of clarity in advice on wave forces.

Rather lower forces arise if waves have already broken before reaching the wall (type c). The wave motion is turbulent, but often highly aerated. Predictions of broken wave loads are uncertain, with relatively few laboratory or field data. The last class is the post-breaking or bore wave (type d) usually applied to a wall where the toe is above the static water level, but where the run-up bore can still reach the wall.

Broken waves occur when the local water depth is insufficient to support unbroken waves. For simple vertical walls with no significant mound, waves may start to break when the local wave height to depth exceeds, for example, $H_{si}/d > 0.35$. As local wave conditions approach the breaking limit, so the proportion of broken waves increases, and the probability of a large but un-broken wave reduces.

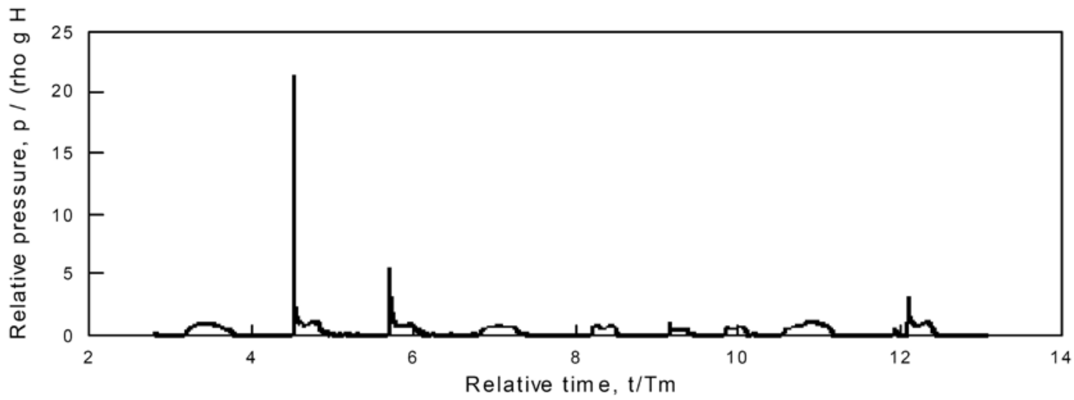


Figure 8.122 Example wave pressure traces on a vertical wall with toe berm: model test results (after Allsop et al, 1996a)

Predicting types of wave load

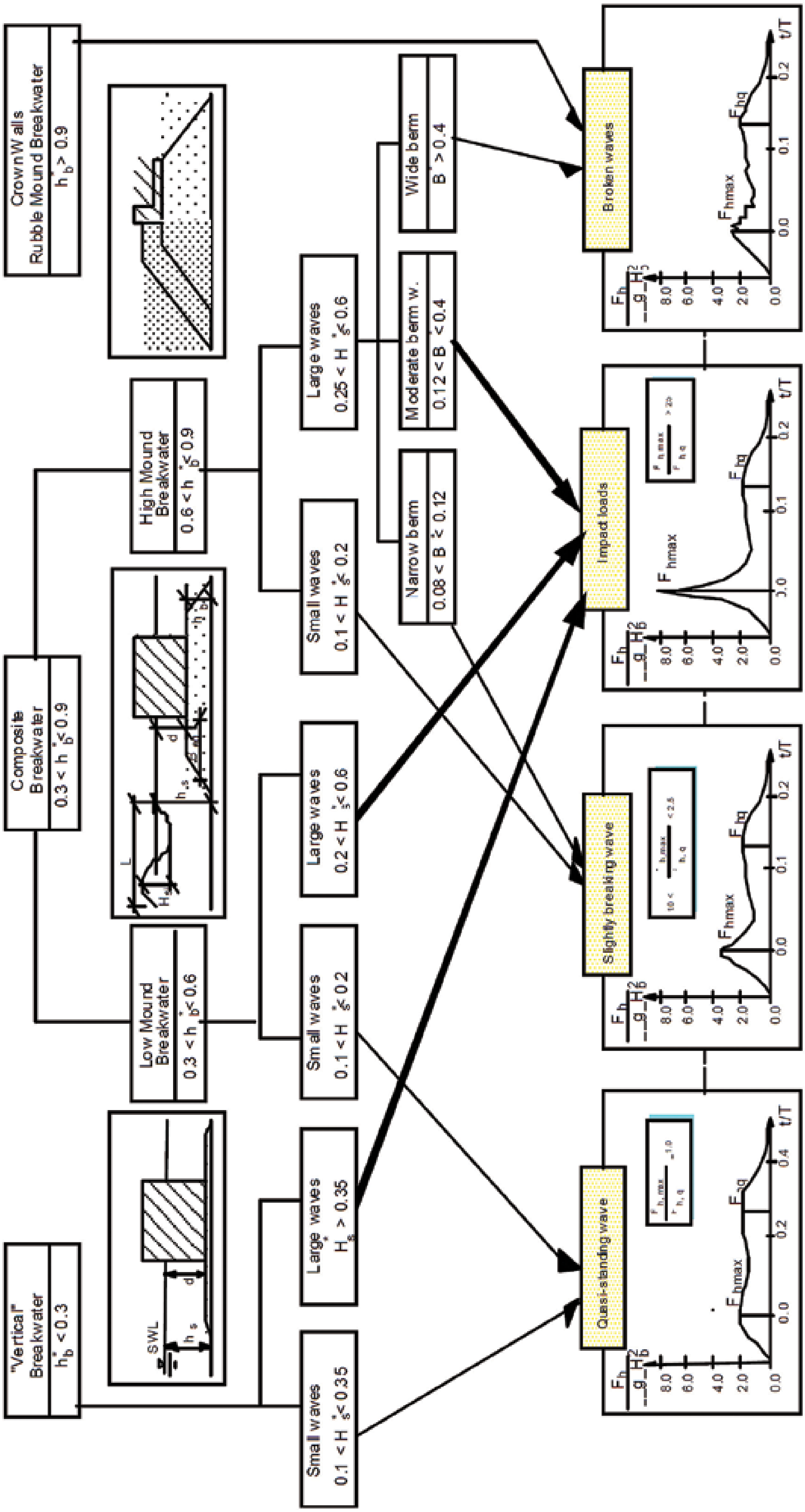
A method to identify the occurrence of some types of breaking and wave load, was developed in the PROVERBS project (Allsop *et al*, 1999), and is shown in Figure 8.123.

The version shown in Figure 8.123 was derived for approach slopes no steeper than 1:50. The parameter map indicates that wave impacts are most likely to occur for three categories of conditions:

- vertical walls with large waves ($H_{si}/d > 0.35$)
- walls on low mounds with large waves ($0.65 < H_{si}/d < 1.3$)
- high mounds with moderate berm widths ($0.14 < B_{eq}/L_{pi} < 0.4$) and large waves ($0.65 < H_{si}/d < 1.3$).

Using this general approach, methods to predict wave forces on vertical wall and, where applicable, are described in the following guidance:

- Goda (1985)– use for nonbreaking waves
- Takahashi modification to Goda (Takahashi *et al*, 1994) – use when a berm may cause impulsive breaking of waves
- Allsop and Vicinanza (1996) – estimate impulsive force of breaking waves
- Cuomo *et al* (2010a and b, and 2011) – estimate impulsive force of breaking waves
- Blackmore and Hewson (1984) – estimate force when wave action is broken before reaching the wall
- Camfield (1991) – estimate force when a breaking/broken bore travels over a slope or beach.



where $h_b^* = h_b/h_s$, $H_s^* = H_{si}/h_s$, $B^* = B_{eq}/L_{pi}$

Figure 8.123 Parameter map to predict occurrence of wave load types (after Allsop et al, 1999)

Pulsating (or non-impulsive) wave loads

The main default method to calculate quasi-static wave loads should be Goda's, or Takahashi's modified version of Goda's method.

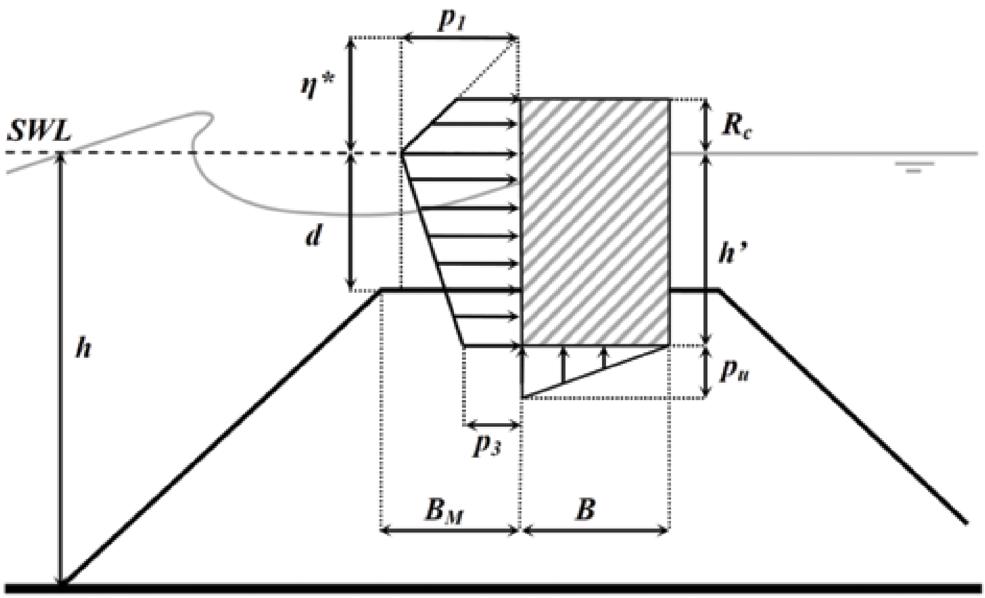


Figure 8.124 Nomenclature used in Goda's wave load prediction method (Goda, 1985)

The most robust (and most widely accepted) prediction method for wave loads on vertical and composite walls is that developed by Goda (1974 and 1985). This method assumes that wave pressures on the front face can be represented by a trapezoidal distribution, reducing from p_1 at the still water level (SWL) to p_3 at the caisson base, see Figure 8.124. At points above SWL, wave pressures reduce to zero at the notional run-up point given by a height η^* above SWL.

If wave pressures can penetrate under the wall, uplift pressures at the waterside edge might be determined by a separate expression, and may be less than pressures calculated for the toe of the waterside face. In Goda's method, uplift pressures are distributed triangularly from the waterside edge to zero at the rear heel. The method was developed from hydraulic model tests where wave pressures were measured, and from a larger set of sliding tests on model breakwater caissons. The resulting prediction formulae were then calibrated by comparison with field experience. The main response parameters determined in Goda's method are:

$$\eta* = 0.75\,(1+\cos\beta)\,H_{max} \tag{8.219}$$

$$p_1 = \frac{1}{2}\,(1+\cos\beta)\left(\alpha_1+\alpha_2\cos^2\beta\right)\gamma_w H_{max} \tag{8.220}$$

$$p_3 = \alpha_3 p_1 \tag{8.221}$$

where the coefficients α_1, α_2 and α_3 are determined from:

$$\alpha_1 = 0.6+0.5\left[\frac{4\pi H_s/L}{\sinh\left(4\pi H_s/L\right)}\right]^2 \tag{8.222}$$

$$\alpha_2 = \min\left\{\frac{h_b-d}{3h_b}\left(\frac{H_{max}}{d}\right)^2; \frac{2d}{H_{max}}\right\} \tag{8.223}$$

$$\alpha_3 = 1-\frac{h'}{H_s}\left[1-\frac{1}{\cosh\left(2\pi H_s/L\right)}\right] \tag{8.224}$$

where η^* is the maximum elevation above SWL (m) to which pressure could be exerted (taken by Goda as $\eta^* = 1.5\ H_{max}$ for normal wave incidence) β is the angle of wave obliquity in plan (°). The design wave height, H_{max}, is taken as $1.8H_s$ for all positions waterside of the surf zone. In conditions of broken waves, H_{max} should be taken as $H_{max,b}$. The water depth H is taken at the toe of the mound, and d over the mound at the front face of the wall, but h_b is taken $5H_s$ waterside of the wall.

The total horizontal force, F_h, is calculated by integrating pressure p_1 over the height h_f of the front face. Similarly, where appropriate, the total uplift force is calculated by integrating $p = p_u$ at the front edge to p = 0 at the rearward edge, giving a total uplift force: $F_u = 0.5\ p_u\ B$. All force and pressures calculated by Goda's method represent a 1/250 exceedance level, $F_{1/250}$.

For mounds with a relatively large height, the water depth over the mound, d, may be sufficiently smaller than the depth in front of the mound, h, to cause impulsive breaking. Takahashi *et al* (1994) have devised an adaptation of α_1 in Goda's equations, where:

$$\alpha_1 = \alpha_{10}\,\alpha_{11} \tag{8.225}$$

where α_{10} is given by $\alpha_{10} = H/d$ for $H/d \leq 2$, or $\alpha_{10} = 2$ for $H/d > 2$ and α_{11} is given by the diagram in Figure 8.125. Coefficient α_{11} takes a maximum value of 1 when $d/H = 0.4$ and $B_M/L = 0.12$. The impulsive breaking coefficient α_1 takes values between 0 and 2, with larger values giving larger wave forces.

When calculating wave forces using Goda's method modified by Takahashi, α_1 is used in place of α_2 when $\alpha_1 > \alpha_2$.

It should be noted that the Goda formula deals with wave action only. The hydrostatic action of water on both sides of the flood wall has to be added in order to calculate the resultant action of water.

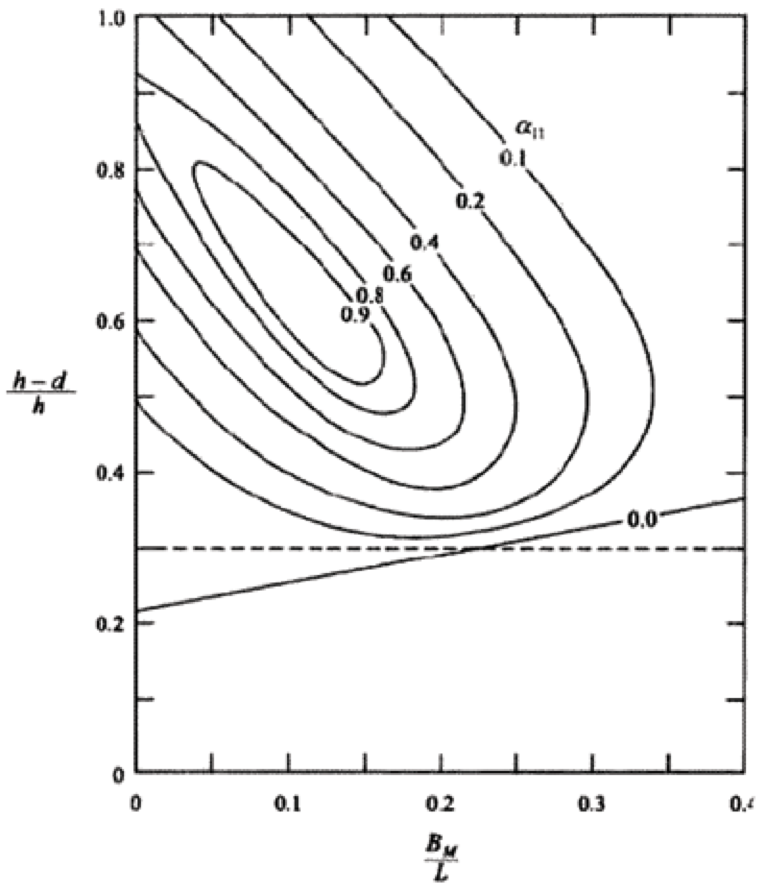


Figure 8.125 Impulsive breaking wave pressure coefficient α_{11} (after Takahashi et al, 1994)

Impulsive wave loads

A simple and robust method to predict wave impact pressures was derived by Allsop and Vicinanza (1996) based on testing by Allsop *et al* (1996a). They noted that for waves close to breaking given by $0.35 < H_{si}/d < 0.6$, other prediction methods underestimate measured forces. Differences are greatest where the incident wave conditions approach the breaking limit, approximated for shallow bed slopes by $H_{si}/H_s \approx 0.55$. A simple prediction curve using Equation 8.226 was fitted to test results for composite walls (vertical wall with a toe berm/mound) for $0.35 < H_{si}/d < 0.6$, see Figure 8.126.

$$F_{h,1/250} = 15\,\gamma_w\,d^2\left(\frac{H_{si}}{d}\right)^{3.134} \tag{8.226}$$

Fortunately, this equation also gives a good description of wave impact forces for walls on low mounds given by $0.3 < h_b/H_s < 0.6$, and higher relative wave heights given by $0.6 < H_{si}/d \leq 1.3$.

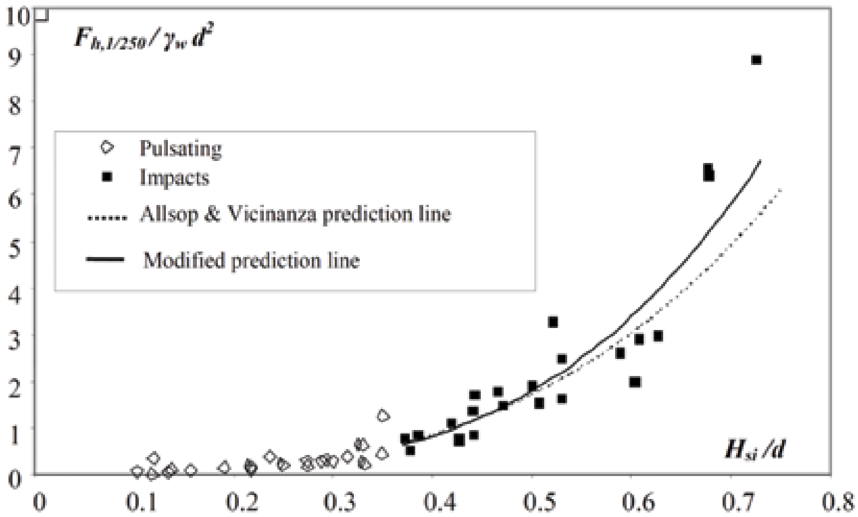


Figure 8.126 Impulsive wave load (after Allsop and Vicinanza, 1996)

Recently Cuomo *et al* (2010a, 2010b and 2011) have improved the prediction of impulsive loads using results from the Big-VOWS large-flume experiments resulting in:

$$F_{h,imp,1/250} = \gamma_w\,H_{m0}\,L_{hs}\left(1 - \frac{|h_b - d|}{d}\right) \tag{8.227}$$

where L_{hs} is the wave length at the toe of the structure, and the water depth at breaking, h_b, is evaluated using:

$$h_b = \frac{1}{k}\arctan\left(\frac{H_{m0}}{0.14 L_{hs}}\right) \tag{8.228}$$

where $k = 2\pi/L_{hs}$.

Broken wave conditions

For many coastal seawalls, and for some breakwaters, the design wave condition may be limited by depth in front of the structure. In these cases, the larger waves at the structure will be broken and it is most unlikely that wave impact loads will occur. A method to estimate an average wave pressure from broken wave loads was developed by Blackmore and Hewson (1984).

$$p_{i,max} = \lambda\,\rho\,T_p\,C_b^2 \tag{8.229}$$

where λ (s^{-1})is an aeration for which values are suggested in Table 8.27, ρ is the water density, T_p is the peak wave period, C_b is the velocity of the breaker, and d is the depth at the wall. The simplest formula for breaker celerity may be given by shallow water wave theory:

$$C_b = \sqrt{gd} \tag{8.230}$$

Table 8.27 Aeration coefficients for broken wave loads (Blackmore and Hewson, 1984)

	Approach slope		
Foreshore conditions	1:5 to 1:10	1:30 to 1:50	1:100
Smooth bed, sand	1.5	0.9	0.7
Rough, rocky	0.5	0.3	0.24
Very rough, emergent rocks	0.13	0.18	0.14

These methods may be used to make an initial estimate of the horizontal wave force under broken waves, $F_{h,broken}$, to be applied only if $F_{h,broken} < F_{h,Goda}$:

$$F_{h,broken} = h_f \, p_{i,max} \tag{8.231}$$

Bore wave conditions

Where the wall (toe) is above the static water level, there is a single method cited in USACE (2006a), which was developed by Camfield (1991) and based on earlier work by Cross (1967) for wave loads on back-beach seawalls. The method requires a wave run-up limit on the beach to be calculated, from which a wave 'surge height' (H_w) at the wall is deduced. Wave run-up levels are subject to significant measurement uncertainties, and to some debate. The classic method for estimating wave run-up on beaches or shallow slopes is that ascribed to Hunt (1959), perhaps as re-stated by Battjes (1974). The 'surge' force, F_{surge}, is calculated from a 'surge height', H_{surge}, by:

$$F_{surge} = 4.5 \, \gamma_w \, H_{surge} \tag{8.232}$$

where:

$$H_{surge} = 0.2 \, H_b \left(1 - \frac{x_1}{x_2}\right) \tag{8.233}$$

where x_1 is the horizontal distance from shoreline to toe of the wall, and x_2 from the shoreline to the notional run-up point without the wall.

In its original application, on shallow beaches, the breaking wave height was approximated to be $H_b = 0.78h_s$, but this would not be a safe estimate of H_b on slopes steeper than 1:50. Camfield (1991) recommends the method for slopes between 1:100 and 1:10, but notes that waves "on composite slopes should be investigated on a case-by-case basis".

This method gives no indication of the height over which the load applies, or of the average pressure, so simple rectangular distribution over the full wall height is generally assumed. The calculation of bore wave load is rather subjective, and it is not known whether it has been validated by any measurements, either field or laboratory, so its reliability is unknown.

Box 8.21 provides an example for the evaluation of wave loads on a reservoir wall.

Box 8.21 Wave loads on a reservoir wave wall

The wave wall

An embankment dam at the western end of a reservoir faces approximately east to south-east. Prevailing winds are generally away from the dam, but waves along the main fetch (650 m) of the reservoir may break directly onto the 1 m high vertical wave wall at the crest of the 1:3 embankment slope.

There is no simple prediction method for wave forces on this wall, which is within range for the particular geometry of dam slope, wave wall position, and water level. None of the usual prediction methods are strictly valid for the particular configuration given. So, it was necessary to apply a number of methods, all involving extrapolating from their original ranges.

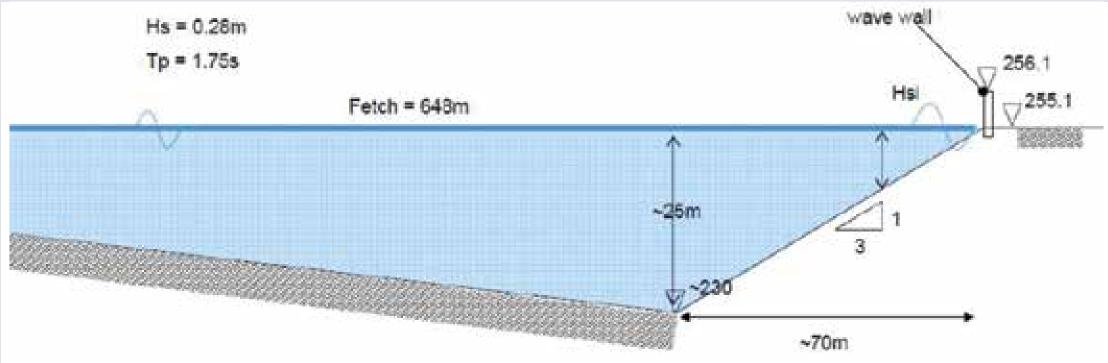


Figure 8.127 Schematic of dam and wave wall – input conditions for the calculations

The key ‘given’ data are summarised in Figure 8.127. The water level coincides with the toe of the 1 m high vertical wall, and the crest of the embankment slope. This coincidence is unfortunate as no generic prediction method for either vertical walls or plane slopes is within range, so there is a need to extrapolate different methods out of their intended range.

The approach to wave load calculations is summarised:

- determine the effective wave condition at $5H_s$ waterside from the structure
- calculate the momentum-driven horizontal Goda load (F_{hGoda}) and pressures
- if the geometry has a noticeable berm, which may cause impulsive breaking, apply the Takahashi modification to Goda’s method to give an enhanced quasi-static load of $F_{hG\&T}$
- if impulsive wave loads are possible, use simple methods by Allsop and Vicinanza (1996), or Cuomo et al, to estimate $F_{impulsive}$ and an impulsive load duration
- if the wave can be broken by the time it reaches the wall, use the method by Blackmore and Hewson (1984) to calculate $F_{hB\&H}$
- if the wall is only reached after a breaking/broken bore has travelled over a slope or beach, estimate the load by Camfield’s method (1991), $F_{hCamfield}$.

The default load should always be F_{hGoda} or $F_{hG\&T}$ either of which may be taken as a quasi-static load. Any impulsive load should be taken as an additional load case, not replacing the default load. High-intensity impulsive loads are limited in duration so have to be treated as dynamic loads.

Assumptions and results

In the first stage, a check was made on wave conditions at positions from the dam toe to a depth of 0.1 m below the wall toe (note extending the calculations to the wall would simply give zero wave height in zero water depth, a pointless calculation). The Goda location of $5H_s$ away from the wall toe was position 8 in these calculations with a ‘bed’ level at 254.6 mODN.

There are no validated methods to predict shoaling and depth-limited breaking on a 1:3 slope. So, calculations of incident wave height in Table 8.28 used a simple depth-limiting check for the steepest slope available at 1:10 to test whether waves will have broken before or at the analysis position.

Table 8.28 Summary wave condition check

Position	Bed level (mODN)	Local depth (m)	H_{si} (m)	H_{max} (m)
6	254.2	0.9	0.28	0.50
7	254.4	0.7	0.28	0.50
8	254.6	0.5	0.28	0.50
9	254.8	0.3	0.24	0.44

Wave conditions in Table 8.28 were then used to calculate Goda momentum-driven wave loads in Table 8.29. These calculations inherently assume that the wall is shifted ‘seawards’ such that the wall toe is below water level. So the wall height used to calculate the total horizontal force will be over-estimated, as will the calculated values of $F_{h1/250}$ itself. The indicative wave pressure at the

Box 8.21 Wave loads on a reservoir wave wall (contd)

waterline, p_1, will not however be significantly distorted by these (slight) changes to the structure geometry.

Table 8.29 Goda wave load check

Position	Bed level (mODN)	Local depth (m)	H_{max} (m)	$F_{h1/250}$ (kN/m)	p_1 (at SWL) (kN/m²)
6	254.2	0.9	0.50	3.6	3.5
7	254.4	0.7	0.50	3.6	3.9
8	254.6	0.5	0.50	3.7	4.7
9	254.8	0.3	0.44	3.4	5.6

It is interesting to note that, while values of the wave pressure at the water line may increase 'landward' of position 8 (eg p_1 = 5.6 kN/m² at position 9), this does not increase the total horizontal force, improving confidence in the calculation of $F_{h1/250}$ = 3.7 kN/m as the representative quasi-static loading at position 8.

As impulsive breaking is likely, the Takahashi extension of Goda's method was applied for an (assumed) berm of 0.2 m height and 0.25 m width. The changes to $F_{h1/250}$ *and* p_1 are however small (Table 8.30).

Table 8.30 Goda and Takahashi wave load check

Position	Bed level (mODN)	Local depth (m)	H_{max} (m)	$F_{h1/250}$ (kN/m)	p_1 (at SWL) (kN/m²)
6	254.2	0.9	0.50	3.80	3.36
7	254.4	0.7	0.50	3.78	3.64
8	254.6	0.5	0.50	3.81	4.11
9	254.8	0.3	0.44	3.12	4.15

In the last set of calculations summarised in Table 8.31, methods by Allsop and Vicinanza (1996) for impulsive loadings, Blackmore and Hewson (1984) for broken waves, and Camfield (1991) for wave bores were applied. The calculation of broken wave loads with Blackmore and Hewson used a coefficient λ = 0.5, and the bore wave load calculated by Camfield used a Hunt (1959) wave run-up limit for H_s.

As expected, the impulsive loads (A&V) increase as the depth decreases, while the broken wave load (B&H) reduces with decreasing depth. Load estimations using Camfield's method are very much lower than Goda's loads, and are not regarded as realistic.

Table 8.31 Impulsive, broken waves, and wave bore load check

Position	Bed level (mODN)	Local depth (m)	Allsop and Vicinanza		Blackmore and Hewson		Camfield
			$F_{A\&V}$ (kN/m)	p_{av} (kN/m²)	$F_{B\&H}$ (kN/m)	p_{av} (kN/m²)	$F_{Camfield}$ (kN/m)
	(mODN)	(m)					
6	254.2	0.9	6.9	4.2	13	7.7	0.45
7	254.4	0.7	8.5	5.8	8.7	6.0	0.45
8	254.6	0.5	11.1	8.8	5.4	4.3	0.45
9	254.8	0.3	11.5	12	2.5	2.6	0.34

Recommendations

Given the unusual configuration (for wave load calculations), and the potential for plunging wave action onto the wall, the minimum load that should be considered is the Goda load of $F_{h1/250}$ = 3.7 kN/m, taken as a quasi-static load. The possibility of two alternative loads should also be considered.

If it can be demonstrated that these waves will break before the wall, then the broken wave load of $F_{B\&H}$ = 6.4 kN/m should be applied, taken as effectively a static load.

However, if the wave can plunge direct against the wall, then impulsive loads should be estimated, eg $F_{A\&V} \approx$ 11 kN/m, $p_{av} \approx$ 9kN/m². This will only be of short duration, so must not be applied as a static load, but as an impulsive load with appropriate duration.

8.9.1.3 Scour depth

Scour in front of vertical walls is more severe than for slopes/mound, driven by enhanced bed pressures/velocities/turbulence. This mechanism uses simple rules to estimate scour depth of granular materials under wave attack.

For normally incident, nonbreaking, regular waves incident upon an impermeable vertical wall (Xie, 1981 and 1985):

$$S_m = 0.4 \frac{H_s}{[\sinh(kh)]^{1.35}} \tag{8.234}$$

where:
S_m = maximum scour depth at node (L/4 from wall) (m)
H_s = incident regular wave height (m)
k = incident regular wave number (-)
h = water depth (m)

For normally incident, nonbreaking irregular waves (Hughes and Fowler, 1991):

$$S_m = 0.05 \frac{\langle u_{rms} \rangle_m}{[\sinh(k_p h)]^{0.35}} \tag{8.235}$$

where:
k_p = wave number associated with the spectral peak by linear wave theory (-)
$<u_{rms}>_m$ = root-mean-square of horizontal bottom velocity

The value of $< u_{rms} >_m$ was given by Hughes (1992) as:

$$\langle u_{rms} \rangle_m = \frac{\sqrt{2}}{4\pi} \frac{g\, k_p\, T_p\, H_{m0}}{\cosh(k_p h)} \left[0.54 \frac{(1.5 - k_p h)}{2.8}\right] \tag{8.236}$$

where:

T_p = wave period of the spectral peak (s)
H_{m0} = zero-th moment wave height (m)
g = gravitational constant (9.81 m/s^2)

8.9.2 Stability of T-walls

In the analysis of T-walls the following limit states shall be considered:

- bearing resistance failure
- failure by sliding
- failure by overturning
- loss of overall stability
- structural failure.

In this handbook, focus is given only on the first three limit states. The overall stability is discussed in Section 8.6, and the reader may refer to relevant national or other design standards for the structural resistance assessment tools.

8.9.2.1 Bearing capacity

General considerations

Depending on the stiffness of the foundation soil and depth of the foundation, three modes of failure may be experienced:

- general shear failure
- local shear failure
- punching shear failure.

Considering that the T-walls are founded on shallow foundations, consideration of the general shear failure pattern may only occur. Therefore, bearing resistance limit state verification is made through the verification that the vertical stress applied by the structure does not exceed the ultimate limited strength of the soil:

$$q \leq q_{ult} \tag{8.237}$$

Bearing resistance can be obtained either by:

- analytical methods
- semi-empirical methods
- numerical models.

Analytical methods

Determination of ultimate bearing capacity (q_{ult}) for shallow foundations on soil is regarded as a problem of equilibrium of rigid-plastic solids. The solutions rely on a physical understanding of the failure mode, which is generally considered under the general pattern described in Figure 8.128.

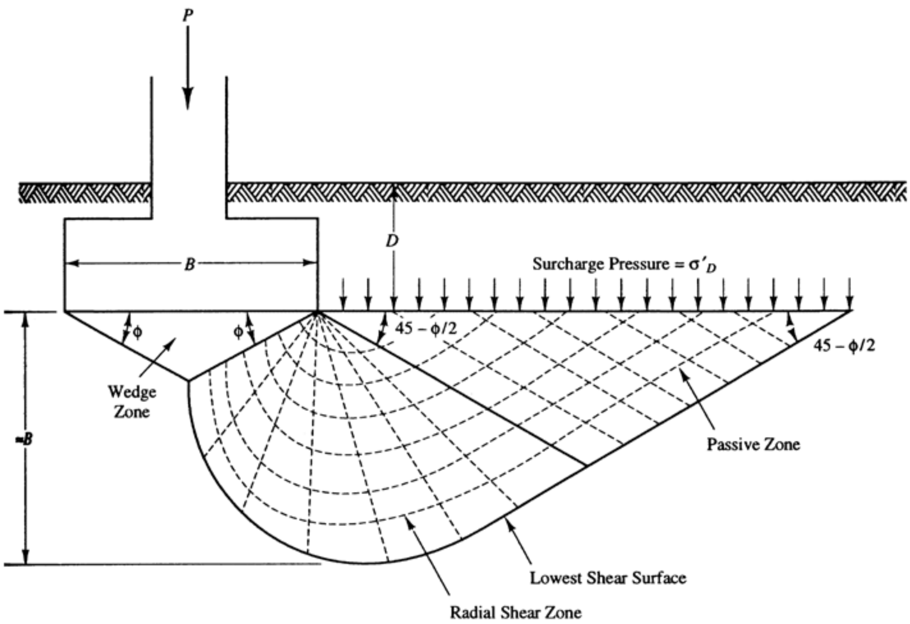


Figure 8.128 General bearing capacity failure pattern (after Terzaghi, 1943)

The subsequent equation of the bearing capacity, first proposed by Terzaghi and extended by several authors (Meyerhof, Hansen, Vesic), may be expressed, for frictional soils, as:

$$q_{ult} = c\,N_c\,s_c\,d_c\,i_c\,g_c + q\,N_q\,s_q\,d_q + \frac{1}{2}\gamma\,B'\,N_\gamma\,s_\gamma\,d_\gamma\,i_\gamma\,g_\gamma \tag{8.238}$$

where:

q = overburden pressure at base of the footing (kPa)
c = average cohesion of the soil (kPa)
B' = corrected width of the footing (m)

The parameters N_c, N_q and N_γ are bearing capacity factors representing the influence of cohesion, soil unit weight and overburden pressure respectively. The other factors can take into account the footing shape (s_i), the footing embedment depth (d_i), the load inclination (i_i) and the sloping ground (g_i). The load eccentricity is taken into account by reducing the dimensions on the footing: $B' = B–2e_B$ and $L' = L–2e_L$ with e_B and e_L as the eccentricity in the minimum and maximum dimensions of the footing respectively. The bearing capacity and other correction factors are detailed in Table 8.32 and Figure 8.129.

Table 8.32 Bearing capacity and correction factors

Bearing capacity factor		$N\gamma$	N_q	N_c
Formula	φ (°)	$N_\gamma = 2(N_q - 1)\tan\varphi$	$N_q = e^{\pi\tan\varphi}\tan^2(\pi/4 - \varphi/2)$	$N_c = (N_q - 1)\cot\varphi$
Values	0°	0.0	1.0	5.1
	5°	0.11	1.6	6.5
	10°	0.5	2.5	8.3
	15°	1.6	3.9	11.0
	20°	4.6	6.4	14.8
	25°	9.0	10.7	20.7
	30°	20.1	18.4	30.1
	35°	45.2	33.3	46.1
	40°	106.1	64.2	75.3
	45°	267.8	134.9	133.9
Shape factor		$s_\gamma = 1 - 0.3\dfrac{B'}{L'}$	$s_q = 1 + \dfrac{B'}{L'}\sin\varphi$	$s_c = \dfrac{s_q N_q - 1}{N_q - 1}$
Depth factor (1)		$d_\gamma = 1.0$	$d_q = 1 + 2k\tan\varphi\cos^2\varphi$	$d_c = 1 + 0.4k$
Inclination factor (2)		$i_\gamma = \left[1 - \dfrac{H}{V + B'L'\cot\varphi}\right]^{m+1}$	$i_q = \left[1 - \dfrac{H}{V + B'L'\cot\varphi}\right]^{m}$	$i_c = i_q - \dfrac{1 - i_q}{N_c}\tan\varphi$
Sloping ground factor		$g_\gamma = (1 - \tan\beta)^2$	$g_q = g_\gamma$	$g_c = i_q - \dfrac{1 - i_q}{(\pi + 2)\tan\varphi}$

Notes :
The load applied to the footing is decomposed in three components : H, V and M which are the horizontal, vertical forces and the momentum acting on the footing.
(1) Values of k parameter depend on the embedment depth D_e as follows : if $\frac{D_e}{B'} < 1$ then $k = \frac{D_e}{B'}$, otherwise $k = \tan^{-1}\left(\frac{D_e}{B'}\right)$.
(2) Values of the m parameter depend on the orientation of the load as follows :

- when H is oriented in the direction of B : $m = m_B = 2 + \frac{B'/L'}{1+B'/L'}$
- when H is oriented in the direction of L : $m = m_L = 2 + \frac{L'/B'}{1+L'/B'}$
- when the horizontal force is oriented with an angle θ : $m = m_L\cos^2\theta + m_B\sin^2\theta$

For purely cohesive soils, the ultimate bearing capacity becomes:

$$q_{ult} = (\pi + 2)\, s_u\, (1 + s'_c + d'_c - i'_c - g'_c) + q \qquad (8.239)$$

where s_u is the undrained shear strength. The correction factors depend on soil characteristics, footing geometry etc. The values of the factors depend on national standards and the reader is referred to those standards.

Figure 8.129 Bearing capacity factors for analytical determination of bearing capacity (from Chai, 2000)

ϕ	N_c	N_q	$N_{\gamma(M)}$	$N_{\gamma(H)}$	$N_{\gamma(V)}$	$N_{\gamma(C)}$	N_q/N_c	tan ϕ
0	5.14	1.00	0.00	0.00	0.00	0.00	0.19	0.00
1	5.38	1.09	0.00	0.00	0.07	0.07	0.20	0.02
2	5.63	1.20	0.01	0.01	0.15	0.16	0.21	0.03
3	5.90	1.31	0.02	0.02	0.24	0.25	0.22	0.05
4	6.18	1.43	0.04	0.05	0.34	0.35	0.23	0.07
5	6.49	1.57	0.07	0.07	0.45	0.47	0.24	0.09
6	6.81	1.72	0.11	0.11	0.57	0.60	0.25	0.11
7	7.16	1.88	0.15	0.16	0.71	0.74	0.26	0.12
8	7.53	2.06	0.21	0.22	0.86	0.91	0.27	0.14
9	7.92	2.25	0.28	0.30	1.03	1.10	0.28	0.16
10	8.34	2.47	0.37	0.39	1.22	1.31	0.30	0.18
11	8.80	2.71	0.47	0.50	1.44	1.56	0.31	0.19
12	9.28	2.97	0.60	0.63	1.69	1.84	0.32	0.21
13	9.81	3.26	0.74	0.78	1.97	2.16	0.33	0.23
14	10.37	3.59	0.92	0.97	2.29	2.52	0.35	0.25
15	10.98	3.94	1.13	1.18	2.65	2.94	0.36	0.27
16	11.63	4.34	1.37	1.43	3.06	3.42	0.37	0.29
17	12.34	4.77	1.66	1.73	3.53	3.98	0.39	0.31
18	13.10	5.26	2.00	2.08	4.07	4.61	0.40	0.32
19	13.93	5.80	2.40	2.48	4.68	5.35	0.42	0.34
20	14.83	6.40	2.87	2.95	5.39	6.20	0.43	0.36
21	15.81	7.07	3.42	3.50	6.20	7.18	0.45	0.38
22	16.88	7.82	4.07	4.13	7.13	8.32	0.46	0.40
23	18.05	8.66	4.82	4.88	8.20	9.64	0.48	0.42
24	19.32	9.60	5.72	5.75	9.44	11.17	0.50	0.45
25	20.72	10.66	6.77	6.76	10.88	12.96	0.51	0.47
26	22.25	11.85	8.00	7.94	12.54	15.05	0.53	0.49
27	23.94	13.20	9.46	9.32	14.47	17.49	0.55	0.51
28	25.80	14.72	11.19	10.94	16.72	20.35	0.57	0.53
29	27.86	16.44	13.24	12.84	19.34	23.71	0.59	0.55
30	30.14	18.40	15.67	15.07	22.40	27.66	0.61	0.58
31	32.67	20.63	18.56	17.69	25.99	32.33	0.63	0.60
32	35.49	23.18	22.02	20.79	30.21	37.85	0.65	0.62
33	38.64	26.09	26.17	24.44	35.19	44.40	0.68	0.65
34	42.16	29.44	31.15	28.77	41.06	52.18	0.70	0.67
35	46.12	33.30	37.15	33.92	48.03	61.47	0.72	0.70
36	50.59	37.75	44.43	40.05	56.31	72.59	0.75	0.73
37	55.63	42.92	53.27	47.38	66.19	85.95	0.77	0.75
38	61.35	48.93	64.07	56.17	78.02	102.05	0.80	0.78
39	67.87	55.96	77.33	66.75	92.25	121.53	0.82	0.81
40	75.31	64.19	93.69	79.54	109.41	145.19	0.85	0.84
41	83.86	73.90	113.98	95.05	130.21	174.06	0.88	0.87
42	93.71	85.37	139.32	113.95	155.54	209.43	0.91	0.90
43	105.11	99.01	171.14	137.10	186.53	253.00	0.94	0.93
44	118.37	115.31	211.41	165.58	224.63	306.92	0.97	0.97
45	133.87	134.97	262.74	200.81	271.74	374.02	1.01	1.00
46	152.10	158.50	328.73	244.64	330.33	458.02	1.04	1.04
47	173.64	187.20	414.32	299.52	403.65	563.81	1.08	1.07
48	199.26	222.30	526.44	368.66	495.99	697.93	1.12	1.11
49	229.92	265.49	674.91	456.40	613.13	869.17	1.15	1.15
50	266.88	319.05	873.84	568.56	762.85	1089.46	1.20	1.19

Note
Subscripts refer to different methods to calculate $N\gamma$: (M) Meyerhof, (H) Hansen, (V) Vesic, (C) Chen

The main issue concerning this approach is that the different methods give a wide range of Nγ values. Also, the choice of the angle of friction (from triaxial or plane strain solicitations) is an important source of uncertainty. This point has been discussed in detail by Droniuc and Magnan (2002). Finally, the controversial aspect of Nγ determination is that it does not include compressibility of the soil. In addition, some comparisons of predicted solutions against model footing test results were found inconclusive.

Semi-empirical methods

Different semi-empirical methods can be used to obtain the bearing capacity q_{ult} of a shallow foundation. Some examples are given in the following paragraphs. The general expression is as follows:

$$q_{ult} = i_\delta \, i_\beta \, K_s \left(B, L, D_e, D_w\right) \; q_e + \sigma'_{v0} \tag{8.240}$$

where:

q_e = averaged soil resistance 1.5B below the shallow foundation
σ'_{v0} = vertical effective stress at the base of the footing
B = width of the footing
L = length of the footing
D_e = embedment depth
D_w = water table depth

The expressions of the correction factor K_s depend primarily on soil type and *in situ* test. The reader may refer to the relevant standards, with an example from France given in Box 8.22.

Two additional correction factors, i_δ and i_β, can be introduced to take into account inclination of the applied load and proximity of a slope respectively. An example of curves allowing selection of the reduction factor i_δ due to inclination δ of applied load is given in Figure 8.130.

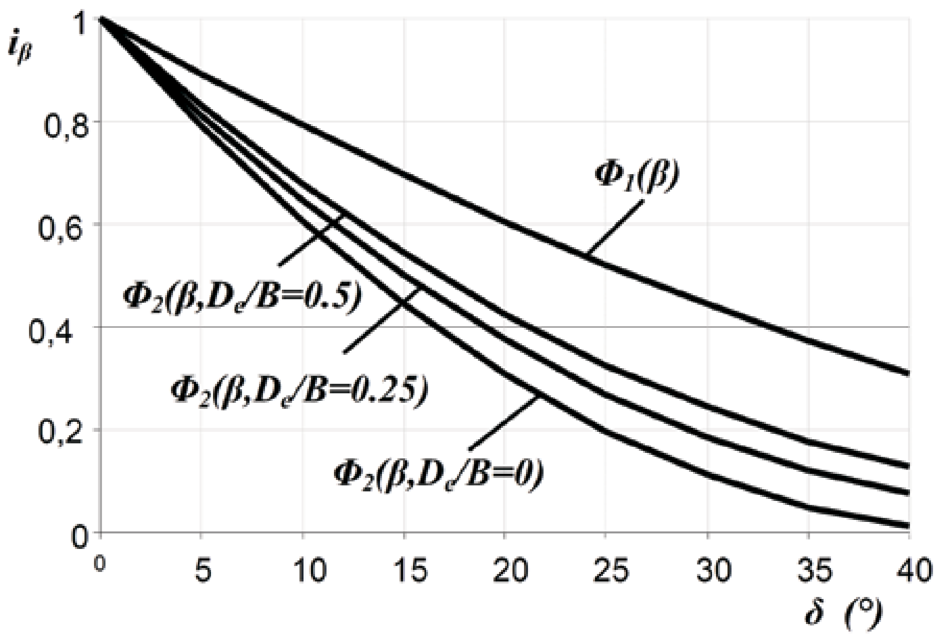


Figure 8.130 Reduction factor i_δ (curves Φ_1 for cohesive soils, Φ_2 for non-cohesive soils)

Values of the reduction factor i_β due to inclination β of a close slope can be obtained through the following formulae (hypothesis shown in Figure 8.131 are used).

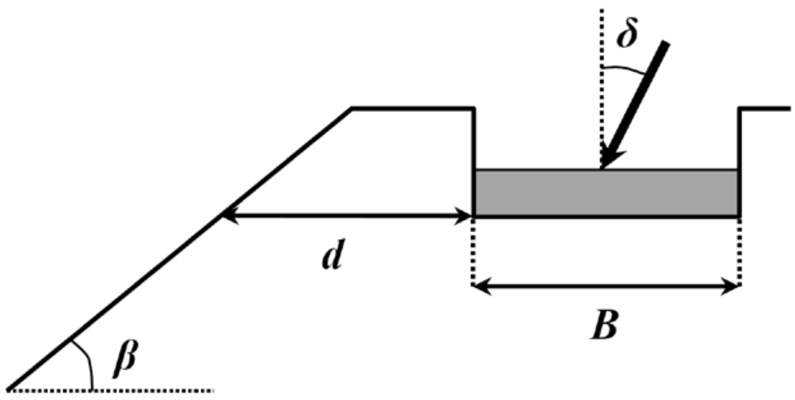


Figure 8.131 Hypothesis used for reduction factor i_β

- for cohesive soils: $$i_\beta = 1 - \frac{\beta}{\pi}\left(1 - \frac{d}{8B}\right)^2 \quad (8.241)$$
- for non-cohesive soils: $$i_\beta = 1 - 0.9\tan\beta\,(2 - \tan\beta)\left[1 - \frac{d + D_e/\tan\beta}{8B}\right]^2 \quad (8.242)$$

Box 8.22 Example of the French standards

Pressuremeter method

Ultimate bearing capacity is directly correlated with the limit pressure p_l measured *in situ* (Bustamante and Gianeselli). For a homogeneous soil layer, q_e, is defined as the interpolation value of net limit pressure (pl-p0) at *2B/3*. For a heterogeneous soil formation, the equivalent net limit pressure is estimated with a geometric mean over the measured values, obtained as follows:

$$q_e = \left(\prod_{i=1}^{n} p_{l,i}\right)^{\frac{1}{n}} \quad (8.243)$$

The correction factor k_p depends on soil under the shallow foundation, on the foundation shape and on the equivalent depth of the foundation, D_e, which is as follows:

$$D_e = \frac{1}{q_e}\int_0^D p_l(z)\,dz \quad (8.244)$$

The correction factor is expressed under the general formula:

$$K_s = \alpha\left[1 + \beta\left(0.6 + 0.4\frac{B}{L}\right)\frac{D_e}{B}\right] \quad (8.245)$$

where α and β depend on the soil type (Table 8.33).

Table 8.33 Limit pressure values to estimate the correction factor K

Soil category	p_l-p_0 (MPa)	α	β
Clay and silts	< 0.7	0.8	0.25
	1.2–2.0		0.35
	> 2.5		0.50
Sand and gravel	< 0.5	1.0	0.35
	1.0–2.0		0.50
	> 2.5		0.80

Cone penetration test based method

Cone static penetrometer can be used to estimate ultimate bearing capacity (Tandetal, Bouafia, and Bustamante and Gianeselli). The equivalent cone penetration resistance is estimated with a geometric mean over the measured values, given as follows:

Box 8.22 Example of the French standards (contd)

$$q_e = \frac{1}{1.5B}\int_D^{1.5B} q_c(z)\,dz \quad (8.246)$$

The correction factor *K* depends on the depth of the foundation, on the soil under the shallow foundation and on the foundation shape. It is generally comprised between 0.31 and 0.44. It can also be obtained based upon the equivalent depth of the foundation D_e, which is given as follows:

$$D_e = \frac{1}{q_e}\int_0^D q_c(z)\,dz \quad (8.247)$$

Table 8.34 gives some examples to determine the correction factor *K*, for different soils and shapes of the foundation.

Table 8.34 Correction factor K for cone penetration based method

Soil category	q_c (MPa)	α	β
Clay and silts	–	0.8	0.35
Sand and gravel	< 5	0.14	0.35
	8–15	0.11	0.50
	> 20	0.08	0.80

An example of determining the allowable bearing pressure from SPT measurements is given in Box 8.23.

Box 8.23 Meyerhof-Bowles method from SPT blow counts

Other methods have been developed from SPT measurements. In the USA, the following method is widely used. The net allowable bearing pressure q_f (MPa), for B ≥ 1.22 m, may be expressed as follows (Meyerhof, 1965, and Bowles, 1977):

$$q_f = \left(\frac{3.28\,B + 1}{3.28\,B}\right)^2 11.98\,N_{60}\,\frac{s}{0.025} \quad (8.248)$$

where:

B = equivalent width of the foundation (m)

N_{60} = normalised blow count for a 60 per cent transferred energy (-)

s = tolerable settlement (m)

In the case of an embedded foundation, the net bearing capacity pressure has to be multiplied by the depth factor $1 + 0.33D_e/B \leq 1.33$.

8.9.2.2 Horizontal sliding

The following inequality shall be satisfied where the loading is not normal to the foundation base, foundations shall be checked:

$$H \leq A_c\,c + (V - U)\tan\frac{\varphi}{k} + R_p \quad (8.249)$$

where *H*, *V* and *U* are the horizontal, vertical and uplift forces, R_p is the resistance caused by earth pressure in front of the foundation. Its value should be related to the scale of movement anticipated under the limit state of loading considered and reflect the anticipated life of the structure. The parameter *k* defining the design friction angle may be assumed equal to $k = 1$ for cast *in situ* concrete foundations, but for smooth precast foundations, it may be equal to $k = 2/3$. For drained conditions, any effective cohesion *C* should be neglected, but for undrained conditions, the cohesion term may be replaced by the undrained shear strength S_u.

8.9.2.3 Overturning

Avoiding failures by overturning is reached by limiting the eccentricity of loadings e (m). For ultimate states and strip footings, the simple following criteria could also be used:

$$e < \frac{B}{10} \tag{8.250}$$

where B is the width of the footing (m). This method requires special care to design values of actions and magnitude of construction tolerances in order to determine the accurate location of the foundation.

8.9.3 Stability of I-walls

In the analysis for stability of I-walls the following limit states shall be considered:

- failure by rotation (overturning)
- loss of overall stability
- seepage and uplift
- structural failure.

An I-wall is defined as a slender cantilever flood wall, deeply embedded in the ground or in an embankment. The wall rotates when loaded and is stabilised by reactive lateral earth pressures. A design goal is to limit wall deflection to tolerable levels during loading. This is typically achieved by designing the wall using the limit equilibrium method based on lateral earth pressures at their limit state, after applying a safety factor to the soil shear strength. Advanced soil-structure interaction (SSI) approaches, such as modelling I-walls using springs or finite elements/finite difference techniques, are available for more rigorous solutions and are discussed briefly in Section 8.9.3.5.

When hydraulic forces are applied to I-walls founded in soils that exhibit cohesion, a gap may form between the I-wall and waterside soil resulting in a loading that exceeds the active lateral earth pressure (Duncan *et al*, 2008). The authors indicate that the formation of the gap is an important feature and so it needs to be considered in all failure modes. This is because wall loading may be increased, thereby reducing stability, and seepage paths may be shortened increasing the potential for heave and uplift. Hydrostatic pressures within the gap are applied to the wall and to the soil face to the full gap depth at a point where the hydrostatic pressure within the gap is equal or less than the total active lateral earth pressure. Gaps are not considered in cohesionless soils as saturated granular soils have no free-standing height and will displace and remain in contact with the wall as it deflects.

8.9.3.1 Overturning

General considerations

It should be demonstrated by equilibrium calculations that embedded walls have sufficient penetration into the ground to prevent rotational failure. In addition to active lateral earth pressure other actuating/driving forces causing rotation towards the landward side include:

- hydrostatic pressure
- hydrodynamic loads
- seepage effects
- vessel or debris impacts
- ice forces.

In the rotational failure mode the wall rotates as a rigid body about a point somewhere in its embedded depth typically near the tip of the wall. Equilibrium is achieved by a balance of driving loads and of active and passive soil pressures that depend on the wall relative deflection. Driving loads are primarily from the water (flood) force, and resisting pressures are the passive pressures near the ground surface on the landward side of the wall and near the tip of the sheet pile on the watersideside of the wall. The following sections are based on information from Dawkins (1991).

Hydraulic loads

Water loads are applied to the wall above and below the ground surfaces on both the water and landward sides. These are applied as pressures due to hydrostatic head on either side of the wall, hydrodynamic distributions from wave impacts, pore water pressure on the embedded part of the wall and seepage induced forces (where applicable) that are incorporated in the computation of lateral earth load.

Hydraulic forces acting on the wall above the ground are discussed in Section 8.9.1. Pore water pressures and effects of seepage below the ground surface are included in a simplified manner and determined separately for the waterside (driving) of the wall and for the landward (resisting) side as shown in Figure 8.132 with associated net water pressure diagrams. Hydrostatic loading within a potential flood side gap must also be considered, and the hydrostatic loading within the gap is used instead of the active lateral earth pressure when it exceeds that value.

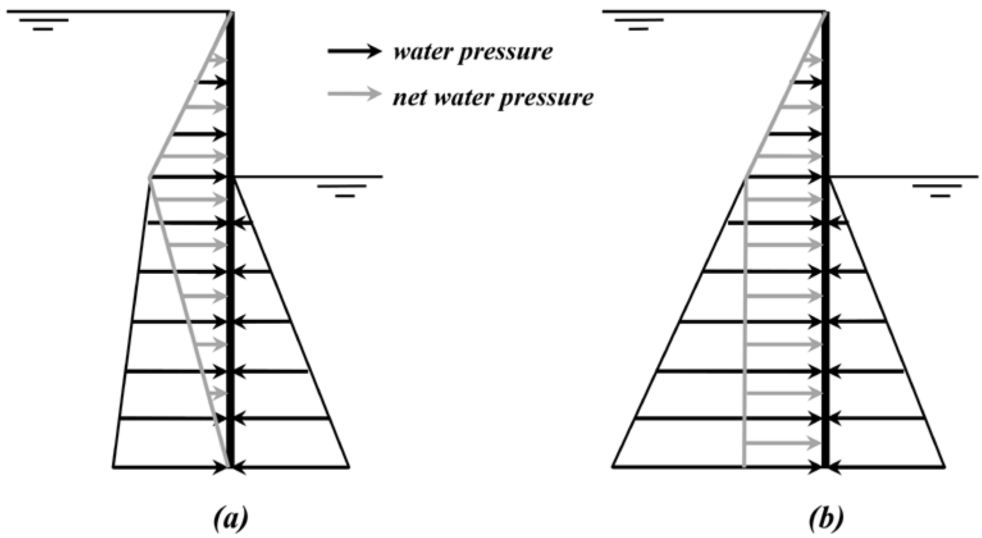


Figure 8.132 Water pressure diagrams and net pressure, sand with seepage (a), and clay without seepage (b)

Earth loads

The pressures on both sides of the wall are computed using lateral earth pressure theory based on mobilised shear strengths (Equations 8.253 and 8.255), and the point of rotation is found that simultaneously provides force and moment equilibrium. Analyses are performed using either effective or total stresses. As the wall is loaded by a flood loading the top of wall rotates towards the landward side and pivots about a point above the sheet pile toe. As the wall rotates away from the waterside active pressures develop, on the watersideside while passive pressures exist on the landward side. Likewise, below the point of rotation passive pressures develop on the waterside and active on the resisting landward side. The differences between passive and active pressures on the waterside and on the resisting side of the wall are computed and these pressure differences on each side of the wall are the net earth pressures that can exist (note that seepage forces tend to increase the effective unit weight of soil on the waterside while reducing the effective unit weight on the landward side resulting in differing lateral earth loads on each side of the wall). With the net water pressure diagrams and the maximum passive pressure diagrams the point of rotation is computed considering horizontal force and moment equilibrium. The lateral earth pressures and embedment depth of wall are computed as follows:

$$\varphi_m = \arctan\left(\tan\varphi / F_s\right) \tag{8.251}$$

$$c_m = c/F_s \tag{8.252}$$

where:

φ = angle of internal friction (°)
φ_m = mobilised angle of internal friction (°)
c = cohesion (kPa)
c_m = mobilised cohesion (kPa)
F_s = given or required factor of safety (-)

Several different methods can be used to determine the limiting values of earth pressures. For a vertical wall with horizontal ground surfaces and soil layers and zero wall/soil adhesion, limit values of earth pressure may be calculated using Coulomb's earth pressure coefficients as follows:

For active limit state:

$$\sigma'_a = \left(K_a\,\sigma'_v - 2\,c_m\,\sqrt{K_a}\right)\cos\delta_a \tag{8.253}$$

where σ'_v is the effective vertical stress (kPa) calculated using the effective soil-unit weight (including seepage effects), δ_a the angle of friction along the wall (°), and K_a the active earth pressure coefficient (-). The active earth pressure coefficient may be calculated from different methods, which have to account for the real geometry of the levee. When the crest is large enough, the following formula may be used:

$$K_a = \left[\frac{\cos\varphi_m}{1+\sqrt{\dfrac{\sin(\varphi_m+\delta_a)\sin\varphi_m}{\cos\delta_a}}}\right]^2 \frac{1}{\cos\delta_a} \tag{8.254}$$

For passive limit state:

$$\sigma'_p = \left(K_p\,\sigma'_v + 2\,c_m\,\sqrt{K_p}\right)\cos\delta_p \tag{8.255}$$

where K_p is the passive earth pressure coefficient (-).When the crest is large enough, the following formula may be used:

$$K_p = \left[\frac{\cos\varphi_m}{1-\sqrt{\dfrac{\sin(\varphi_m+\delta_p)\sin\varphi_m}{\cos\delta_p}}}\right]^2 \frac{1}{\cos\delta_p} \tag{8.256}$$

The calculation using Coulomb's passive earth pressure coefficient (based on a linear failure surface) may overestimate the passive resistance. Log spiral failure mechanisms should be checked as they often return a less passive pressure. When the ground surface is not horizontal or other limiting assumptions are not valid, earth pressures may be calculated using the Wedge method (Section 8.6.3.1) where the active or passive load is either analytically or numerically optimised.

Hydrostatic water pressures may be altered by seepage. When seepage effects are included, the excess hydrostatic head is assumed to be dissipated by vertical flow downward on the waterside and upward on the landward side. The seepage gradient i (-) is assumed to be constant at all points in the soil on either side of the wall. Under this assumption, the effect of seepage is to alter the effective unit weight of water (and the unit weight of soil) in the region of flow. On the waterside of the wall:

$$\gamma_{we} = \gamma_w\,(1-i) \tag{8.257}$$

and on the landward side:

$$\gamma_{we} = \gamma_w\,(1+i) \tag{8.258}$$

where:
γ_{we} = effective unit weight of water used to calculate σ'_v (kN/m^3)
γ_w = unit weight of water (kN/m^3)
i = seepage gradient (equals zero under hydrostatic conditions) (-)

Figure 8.133 shows the maximum earth pressure diagrams for the landward side and waterside of the wall and the resulting net pressure diagram needed to achieve horizontal force and moment equilibrium about a point of rotation denoted as Point O. Solving for the location of Point O to achieve equilibrium is an iterative process that requires varying embedment depth until the required factors of safety are achieved.

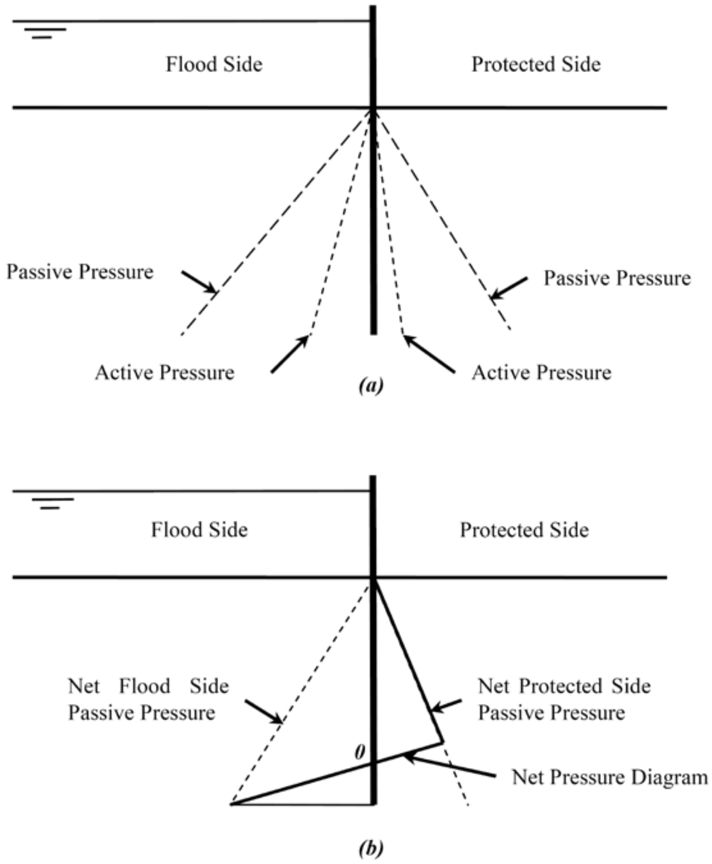


Figure 8.133 Lateral earth pressure diagrams active and passive pressures (a), and maximum passive pressure with resulting net pressure diagram (b)

Vessel or debris impacts and ice loads

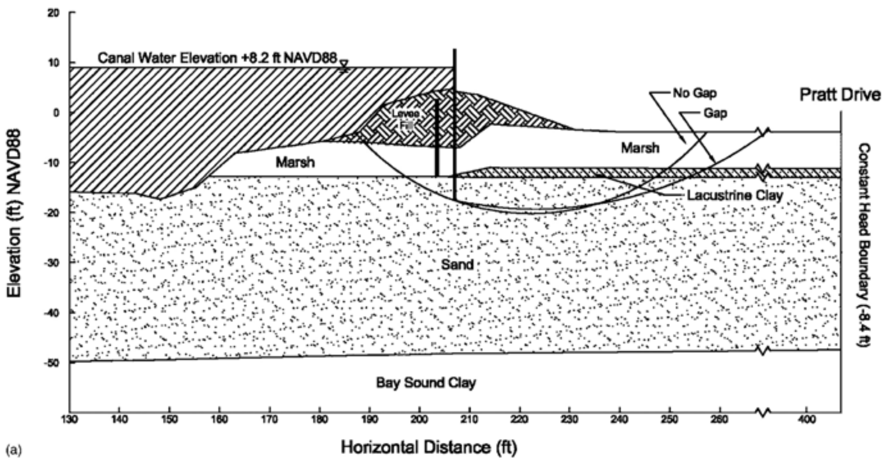
These loads may be estimated and included with hydraulic loads and lateral earth pressures when solving for horizontal force and moment equilibrium. The determination of these loads is project specific and not discussed further.

8.9.3.2 Overall stability

Overall, global, or deep-seated stability are terms used to describe a failure mode where the wall is assumed to displace along with the soil mass in which it is embedded when it slides or rotates under a slope stability type failure mechanism. This failure mode is most likely to be critical when I-walls are located within levees in very soft soils. Global stability is evaluated using typical slope stability software for the gap and no-gap analyses as presented in Figure 8.134. The methods used to evaluate global stability shall satisfy all conditions of static equilibrium. The no-gap condition is evaluated using typical or routine slope stability analysis procedures but incorporation of the gap can add complexity and is discussed in more detail as follows. A waterside water-filled gap can be included by removing the waterside soil to the bottom of the gap and replacing it with a mechanical pressure to represent the hydrostatic water load against the wall. Tension crack options in software packages can be used but should be checked for correctness regarding the treatment of submerged tension cracks.

Methods for determining gap depths are considered approximate, so global stability needs to be checked for the no-gap and full-gap conditions and possibly the partial gap condition. Under the no-gap and full-gap conditions stability is performed assuming either that no waterside gap develops or that a gap will extend to the bottom of the sheet piling. Because saturated granular soils will not sustain a gap, a gap is not presumed to develop in these materials and a gap is not included in modelling. When cohesive soils overlie granular soils, the gap depth may propagate to the top of the granular layer but no deeper (Figure 8.134). The condition where cohesive soils underlie granular soils is not fully understood. However, the previous assumption that the gap will extend to the bottom of the sheet pile or to the bottom of the fine-grained material is recommended.

Figure 8.134a presents the gap and no-gap critical slip surfaces for the composite levee system shown. The no-gap slip surface is constrained below the wall toe, preventing potential slip surfaces from passing through the wall. In soft soils or where high pore water pressures in sand result in low shear strength, this is a reasonable assumption as the stiffness of the wall is expectedly higher than that of surrounding soil. Figure 8.134b presents the partial gap stability model showing soil removed to the toe of the wall and the slip surface initiating at that point. In this analysis the gap did not extend completely to the wall toe but instead to the top of the sand strata, and hydrostatic water pressure is included to this depth. Seepage and associated head loss within the sand layer is shown as less than hydrostatic from the top of sand to the toe of the wall. In this analysis an active effective horizontal lateral earth pressure is calculated and applied to the model in the sand strata (Brandon *et al*, 2008).



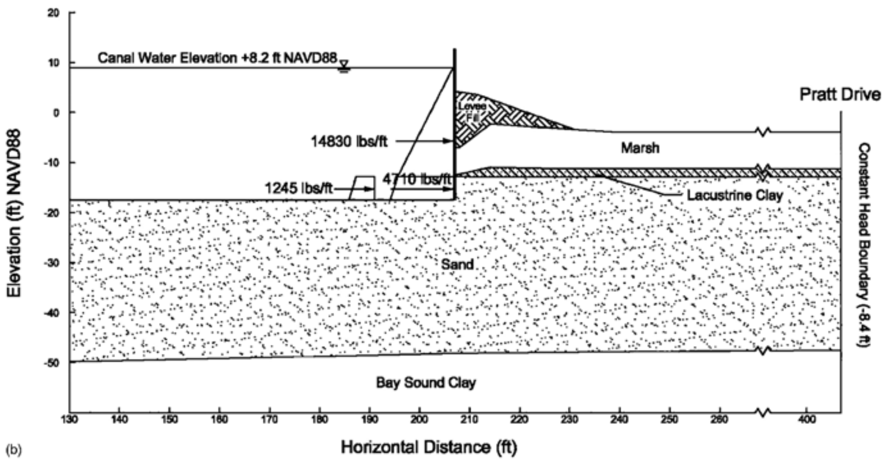


Figure 8.134 Slope stability analyses with and without gap formation (Brandon et al, 2008)

8.9.3.3 Seepage and uplift

Seepage and associated heave or uplift for levees is described in Section 8.6. The same underlying principles apply to seepage around I-walls with the additional concern of gap formation. The formation of a waterside gap in cohesive soils adjacent to I-walls can create a direct connection to underlying sand layers (Duncan *et al*, 2008), and increase the potential for heave or uplift.

Several procedures are available to analyse seepage and uplift. Graphical methods (flow nets), analytic or closed-form solutions that have been solved for specific conditions, method of fragments, and finite elements are common tools. Advances in hardware and software associated with modern computers have greatly reduced the time and effort to perform numerical analyses. Also, analysis of seepage by finite elements has become routine for many designers. Finite elements are often used where the substrata system is considered too complex for generalised characterisation, and the waterside gap for seepage analysis is easily incorporated with this method. Several computer programs couple results from finite element seepage analysis with limit equilibrium slope stability programs to aid in estimating pore water pressures for global stability analyses.

8.9.3.4 Structural failure

Steel sheet pile and reinforced concrete elements of the I-wall are designed to resist the strength limit states of bending and shear. Bending and shear forces in the I-wall are determined from the limit equilibrium analysis for rotation. However, the designer should recognise the factors of safety that are included in the analysis. Moment and shear forces produced from the limit equilibrium analysis to determine required tip depth using mobilised soil strength values already include a factor of safety. The design must consider the total factor of safety when combined with live load factors or allowable stresses for structural strength design from design codes. Typically limit equilibrium analyses are performed without including a factor of safety in order to determine moment and shear for the design of the structural elements.

I-walls are often constructed with a reinforced concrete wall above the ground surface and steel sheet pile driven below the ground surface. Besides designing for bending and shear forces in the sheet pile and reinforced concrete portions alone, the wall must be designed to transfer moment and shear from the reinforced concrete section to the sheet pile section. Methods for designing this connection vary but normally depend on a length of sheet pile to extend into the concrete wall (typically a metre or more) with reinforcing bars passed through holes cut into the sheet pile or shear studs welded to the flange of the sheet pile.

8.9.3.5 Advanced soil-structure interaction methods

Advanced soil-structure interaction (SSI) methods may be used for the design of I-walls but it is recommended that simpler limit equilibrium methods also be performed to complement the more advanced methods. Finite element/difference methods have been used to perform complete SSI analyses. Modelling the soil as a continuum requires the characterisation of stress-strain behaviour in addition to soil strength. Foundation investigation and testing must consider the parameters needed to support the constitutive model selected for performing the analysis. The simplest constitutive model considered acceptable for I-wall analysis and design is the Mohr-Coulomb (linearly elastic, perfectly plastic) model. In addition to using traditional Mohr-Coulomb shear strength properties this model incorporates properties for linear elasticity, such as Young's modulus and Poisson's ratio or shear and bulk modulus.

When constructing the FEM model it is important to include appropriate interfaces that allow slip and separation at the wall/soil contacts, but in order to capture potential overall stability concerns it is also important to allow the toe of the wall to move, such as by extending an interface below the wall or horizontally at the wall toe. Loading is applied as mechanical pressures acting normal to the ground surface and normal to the wall face. When a gap is included between the soil and I-wall a horizontal mechanical pressure is added to both the soil and the wall to the depth of the gap. Gap development is modelled following the procedure used in a report by USACE (2006b). The total horizontal stress in the element adjacent to the wall is compared to the hydrostatic pressure that would exist if a gap were present. If the hydrostatic water pressure exceeds the total horizontal stress it is assumed that a gap would form. Starting at the flood side ground surface, each underlying element is checked as water levels are incrementally raised. Water levels are raised in small increments (ie 0.3 to 0.6 m or 1 to 2 ft) and the gap is deepened in small steps as needed. The protocol for loading and gap initiation and progression is as follows:

- the model is brought into equilibrium and nodal displacements and velocities are reset to zero at the initial water level
- the water level is incrementally raised until loading is applied to the wall (ie no gap is allowed to form until water levels reach the wall)
- when water loads the wall the gap criteria is checked and the gap is deepened in small increments until the horizontal stress exceeds the hydrostatic water pressure that would exist at that depth
- the water level is incrementally raised and the need for extending the gap is checked
- once a gap has been extended to depth it is assumed the gap will not close (ie the depth of gap never reduces).

When using FEMs for evaluation or design the criteria for acceptable performance includes allowable stress in the structural elements but also allowable deflection of the wall. Confidence in calculated deflection is a concern and performing field load tests may be useful for calibrating models on critical structures. In lieu of field load testing conservative stress-strain parameters can be assumed from *in situ* and laboratory testing. Also, factor of safety calculations can be performed using a strength reduction technique and maintaining a reasonably high factor of safety as used in limit equilibrium analyses for wall rotation. The allowable amount of wall deflection should be selected based on a serviceability limit state.

Software involving reactive loads from springs rather than a soil continuum has been developed and is available for evaluation and design. When using these tools designers are encouraged to perform limit equilibrium analyses for comparison purposes.

Box 8.24 presents an example of gap analysis for an I-wall.

Box 8.24 Example of composite levees, New Orleans, USA

As reported by Duncan et al (2008), failures of I-walls during Hurricane Katrina were responsible for many breaches in the flood protection system in New Orleans, USA. An important factor in all of the cases investigated was development of a gap behind the wall as the water rose against the wall and caused it to deflect. Formation of the gap increased the load on the wall, because the water pressures in the gap were higher than the earth pressures that had acted on the wall before the gap formed. Where the foundation soil was clay, formation of a gap eliminated the shearing resistance of the soil on the flood side of the wall, because the slip surface stopped at the gap. Where the foundation soil was sand, formation of the gap opened a direct hydraulic connection between the water in the canal and the sand beneath the levee. This hydraulic short circuit made seepage conditions worse, and erosion due to under-seepage more likely. It also increased the uplift pressures on the base of the levee and marsh layer on the landward side of the levee, reducing stability. Because gap formation has such important effects on I-wall stability, and because gaps behind I-walls were found in many locations after the storm surge receded, the presence of the gap should always be assumed in I-wall design studies.

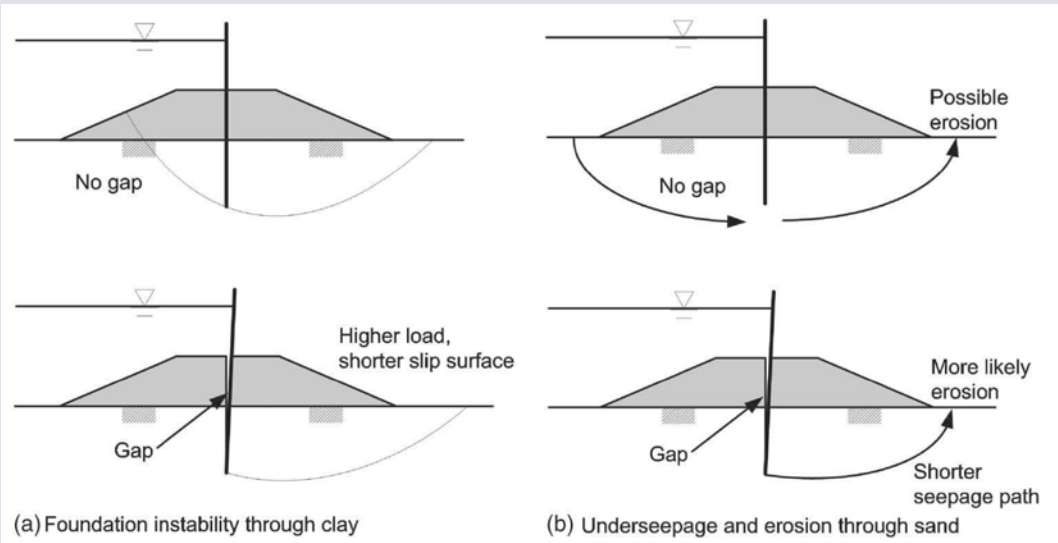
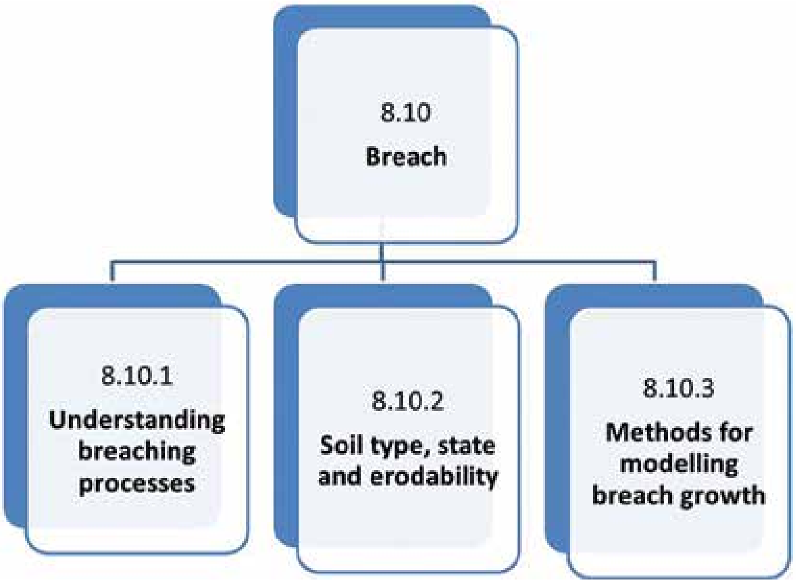


Figure 8.135 Potential I-wall failure mechanisms (from Duncan et al, 2008)

8.10 BREACH



The reliable prediction of breach processes (rate at which a breach forms, volume and rate of release of floodwater) is fundamental to many activities such as flood risk assessment, emergency planning and flood event management. There are a range of issues to be taken into account when considering breach prediction.

Uncertainty

The degree of uncertainty associated with breach prediction methods can be large in comparison to, for example, the prediction of river flood levels through numerical modelling. Significant uncertainty exists in the prediction of breach because:

- uncertainties and variabilities exist in natural and constructed soil conditions
- breach processes involve a complex interaction of hydraulic, soil and structure behaviour
- a single prediction of breach may only be one possible scenario within a wider range of possibilities

Understanding the uncertainty within breach prediction should help to determine which methods of breach prediction are appropriate for use in a given situation. Where a large degree of uncertainty may be acceptable, it is likely that the simpler, rapid methods of breach estimation may be acceptable. Where a greater degree of certainty is required, more complex methods of analysis, perhaps combined with data collection, might be appropriate. Sensitivity analysis to provide a distribution for potential breach conditions can assist by narrowing the limits of uncertainty.

Relevance of different aspects of breach prediction to different end users

Different methods for predicting breach, and in particular simplified methods, ignore certain aspects of the overall process. So, it is important to appreciate which processes are of priority to end users when considering how to predict breach. For example:

- in a **high level flood risk assessment**, the focus is on establishing indicative areas at risk of flooding, and so an estimate of breach flood volume distributed across the inundation area may be acceptable. Details of the breach itself would not be as important as an estimate of the flood hydrograph
- in **flood risk assessments for local development control decisions**, exact boundaries for flooding become important and so does reducing any uncertainty within the flood hydrograph prediction
- for use in **emergency planning**, determining the approximate timing, duration and peak flood conditions becomes relevant

- in **emergency event management (evacuation and repair)**, exact timing, extent of flooding and rates of draw-down after the flood becomes of interest. Where breach is taking place and needs repair, a reliable prediction of breach dimensions, including rate of growth and maximum size, becomes important.

8.10.1 Understanding breaching processes

Three steps can be identified for levee breaching, which apply to breach growth through both headcut and surface erosion processes.

1. **Breach initiation:** with overflow or overtopping, surface protection measures, such as grass or rock cover, fail and soil starts to erode from the surface of the levee. Inside the levee, seepage flow increases slowly as material is gradually eroded from the body or foundation. Outflow remains relatively small but increases slowly as sediment is eroded. This stage can last for hours, days, or months if load conditions are not extreme, but can occur quickly if load conditions continue to increase.
2. **Breach formation:** occurs once erosion under the initiation stage starts to affect the hydraulic control. For breach overflow or overtopping this is when surface erosion starts to lower the crest or when headcut cuts back through the upstream slope. During breach formation, both the flow and erosion increase rapidly often resulting in catastrophic breach. For internal erosion, this stage occurs once a pipe is formed and flow and erosion increases rapidly. As the pipe grows, the roof collapses resulting in open breach formation.
3. **Breach widening:** once breach formation occurs, erosion typically cuts down to the base of the levee very quickly and flow through the levee increases rapidly. Breach widening then occurs, where the focus for erosion is undercutting and removal of material from the sides of the breach. Breach widening will continue while there is sufficient flow through the breach to erode material from the sides. Flow and erosion will slow and eventually stop as the flow subsides, either because flood loads reduce or because the breach becomes drowned by floodwater from the downstream side.

Inundation hydrograph and breach growth

The two main factors that dictate the type of hydrograph are:

- soil erodibility
- stage duration relationship for the hydraulic loading (or in the case of a reservoir, the stage-area or stage-volume relationship).

Resulting hydrograph shapes include:

- **low peak discharge:** where the upstream water level can drop at the same rate as soil erosion lowers the levee crest, the flood hydrograph will be relatively slow and long duration
- **high peak discharge:** where the release of water does not immediately affect the upstream water levels, the rapid increase in discharge being associated with rapidly increasing size of breach.

Where there is an ability to control the 'soil erodibility' and/or the upstream load conditions, this can be used to ensure that, in the event of a breach, the speed and peak of the flood hydrograph can be reduced to a minimum, even though the overall flood volume released may remain the same. In the area close to the breach this is likely to reduce the risk of damage and loss of life.

The impact of drowning on breach flow and formation can be significant and, where likely, should be considered as an integral part of the breach analysis. This is because when water levels downstream of a breach rise they act to drown the flow through the breach and so reduce discharge. This reduction in discharge in turn reduces the rate of erosion and breaches growth. Drowning of the breach will typically occur when downstream levels raise above two-thirds the depth of the upstream level relative to the breach invert level.

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Breach initiation timing

At the breach initiation stage overflow, overtopping or seepage flow starts to erode the soil, removing any surface protection if present. When the erosion is sufficient to significantly affect the hydraulic control of the levee (for example, loss of crest elevation or an increasingly large pipe through or under the levee) then the increase in flow starts to accelerate and breach formation occurs. The timing at which breach formation occurs in relation to the timing of the peak of the flood event is significant. If the timing is coincident, then worse flooding from the breach may arise than if the breach formation occurred after the peak of the flood.

Factors affecting size and location of a breach

Factors that affect breach location include:

- variation in the crest level of the levee – low points in a crest, whether as a result of construction, settlement or erosion through animal or human use, will provide a focal point for overflow driven surface erosion
- variations in the quality of surface protection, especially of grass cover and/or of more erodible areas of soil
- structures through or over the levee (transitions), which often provide an opportunity for seepage at interfaces or a focal point for surface erosion
- transitions in surface protection measures, which also provide a focal point for erosion.

For coastal levees, additional factors affecting breach location include:

- local focusing of wave action due to wave refraction processes (Section 7.4)
- steep bathymetry in front of the levee increasing the force of breaking waves.

A study of historical breaches may provide useful guidance on making assumptions on breach location. Historical breach analysis in a catchment is also useful for assessing breach size, as the size depends upon the soil erodibility and hydraulic load conditions. Within a natural river system, where levee construction may have used broadly similar soils, analysis of historic breach events may show increasing sizes of breach down through the catchment. As the size of the catchment upstream increases, so will the volume of floodwater that might pass through a breach during a given storm event.

Box 8.25 gives an example of historic breach analysis to determine breach location.

Box 8.25 ***Historic analysis of breach location, River Loire, France***

The location of potential breaches initiated by overflowing can be easily located by comparing a longitudinal profile of the crest elevation to a longitudinal profile of the water elevation along the levee for various different flood events. However, there are many other factors that affect breach location, and analysis of the history of a levee will help to detect some of these weaker points in the levee system.

An analysis of historical failures of the levee will provide information on both the breach location and their characteristics, including the main cause, breach dimensions etc. For example, an analysis of the historical breaches along the levees of the River Loire, France (Gaullier and Piney, 2011) identified from archives details of most of the breaches that occurred during the three major floods of the 19th century. Also, some recurring features could be identified including:

- overflowing was the main cause of breaches on the Loire but that internal erosion was also identified as the cause for some breaches
- the widths of breaches varied from a few metres to several hundred metres (600 m for the widest).

Figure 8.136 ***Extract of a historical map of the River Loire levees (first edition in 1850) showing positions of breaches (arrows), infiltrations and limits of flooded areas (dotted and yellow lines), occurred during the main 19th century flood events in 1846, 1856 and 1866***

Historic breach locations can be considered as preferential locations for future breaches because of repairs that provide weaker points within the levee or simply because those locations correspond to points where specific factors make breach formation more likely to occur. These factors could be the location of higher water elevations (relative to crest level), scour at the toe of the levee, high water velocities (corresponding to a narrowing of the river channel, development of vegetation or vegetation jams, the outside bank of a river bend etc). So in addition to historical analysis, morphological and geotechnical analyses can help to identify possible locations of breaches. However, of the many potential breach locations identified, judgement is required to select those associated with higher potential consequences for more detailed consideration. Alternatively, a systematic assessment of flood risk along the entire levee system may be performed. Such an analysis considers the performance of all levees under a range of load conditions, and the likelihood and consequences of failure. By attributing flood risk from each of the thousands of scenarios considered to the breached levee or flood defence being analysed, the system model can build a picture of the flood risk associated with each section of levee or flood defence, providing a valuable tool to assist in asset management.

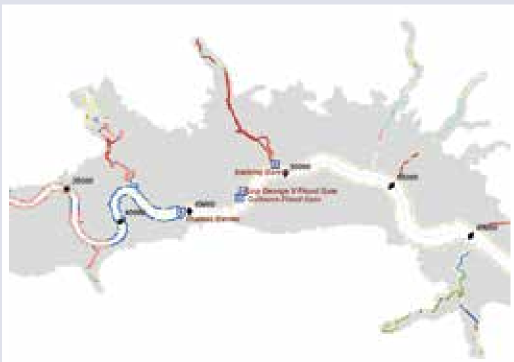


Figure 8.137 ***An example of system modelling showing river channel, flood risk area (shaded) and colour coded flood defences representing flood risk attribution***

8.10.2 Soil type, state and erodibility

As previously discussed, the type and nature of soil within a levee determines the erodibility of that soil and this in turn affects the rate and type of erosion that will occur during breach initiation and formation. So, natural variations within the soil, or variations introduced through construction, can create areas of strength or weakness in relation to erodibility. Since soil erodibility is significantly influenced by compaction energy and soil moisture content during construction, consideration of these parameters will allow more erosion resistant levees to be constructed. However, changes in these parameters over time (deterioration) will also affect erodibility.

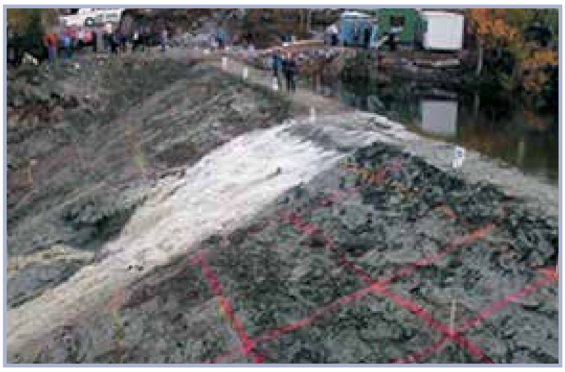
Levee erosion will typically be in the form of surface or headcut erosion, depending upon the nature of the soil (Hahn *et al*, 2000, and Hansen *et al*, 2005a and b). A highly erodible soil, such as a soil with low cohesion and a high sand or gravel content, will erode rapidly and typically through erosion of the exposed surfaces, including erosion of the crest. A highly erosion-resistant soil, such as clay with high cohesive strength, will erode slowly, typically through headcut formation, whereby a step erodes on the downstream face of the levee and recedes towards and through the crest. As the crest of the levee controls the rate of overflow during breach formation, an erosion process that lowers the crest will probably result in catastrophic breach sooner than, for example, a headcut process where catastrophic failure only occurs after the headcut has receded through the crest into the upstream face.

These processes are fuelled by the removal of sediment from the levee body. This can occur via three mechanisms (de Vroeg *et al*, 2002, Mostafa, 2003, Mostafa *et al*, 2008, and Morris, 2009):

1 Sediment erosion.
2 Mass erosion.
3 Soil wasting.

Sediment erosion occurs when sediment is removed from the surface of the embankment and held in suspension by the flow. Mass erosion occurs when small lumps of soil, rather than individual particles, are removed from the embankment surface by the flow. This process is particularly affected by the structure of the soil, including any fissuring that may have occurred. Soil wasting occurs when large blocks of soil are undercut and collapse into the breach flow. These are then quickly removed via a mixture of sediment and mass erosion.

a **Sediment erosion by turbulent flow along base of breach sides**



b ***Mass erosion – small lumps of soil/clay being removed***

c **Soil wasting – block failure on left face of breach**

d ***Soil wasting – block failure on left face of breach 2s after failure of block breach (ie block has been removed)***

Figure 8.138 Small scale erosion mechanisms (courtesy M Morris)

These processes can be seen in different scales of embankment or levee; for example, headcut and block failure during failure of the El Guapo dam (Figure 8.139) shows similar processes to those seen during tests on five to six metre high levees (Figure 8.138).

Figure 8.139 Failure of the El Guapo Dam, Venezuela, December 1999 (courtesy M Morris)

Until recently (~2005), a majority of levee erosion models have used equilibrium sediment transport (EST) equations. The problem with the use of these equations is that they have been developed for the long-term prediction of river bed morphology rather than the prediction of short-term, dynamic conditions typical of catastrophic levee erosion. EST relationships have typically been established by studying equilibrium sediment transport conditions in a flume, where sediment is fed into and collected from a flume under steady state flow conditions in order to establish what bed material load and wash load transport rate occurs for a given sediment and flow condition. This process relies upon a balance being established between sediment inflow and outflow. It is also based upon flow over a sediment bed, rather than flow across a levee or through a breach, where erosion may occur along the breach sides resulting in soil wasting, where a mass of sediment is injected into the flow.

Critically, the rate of levee erosion towards breach can be seen to be highly dependent upon soil state, for example, a highly compacted soil as compared to a loosely placed soil, will take much longer to erode (Hanson, 1992, and Hanson, *et al*, 1997). EST equations do not offer the flexibility of allowing for soil state, because the equations are based upon the soil being in flux along a river bed. So, the use of erosion equations rather than EST equations for simulating levee erosion offers a better solution that more closely represents the physical processes that occur. Erosion equations relate the rate of sediment removal to the shear stress applied by the surface flow and are applicable to non-equilibrium conditions.

A common form of erosion equation as used by Chen and Anderson (1986) and Hanson *et al* (2005b) is given here:

$$E = K_d b(\tau - \tau_c)^a \qquad (8.259)$$

where:

E = erosion rate, bulk volume hence rate of bed elevation change or retreat ($m^3/s/m^2$)

K_d = erodibility or detachment coefficient (-)

τ = effective shear stress (kPa)

τ_c = critical shear stress (kPa)

a, b = empirical coefficients dependent upon soil properties (-)

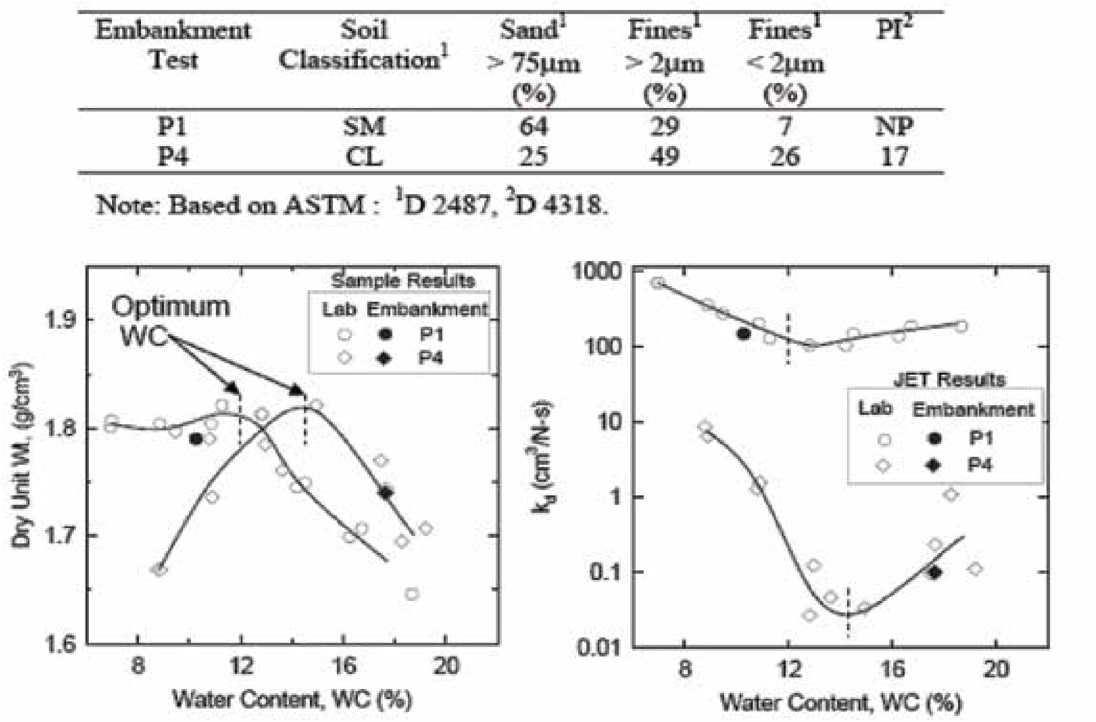
Hanson assumes that a = b = 1. The only variables in calculating the rate of erosion are the critical shear stress (τ_c), and the erodibility of the soil (K_d).

The use of such an erosion equation has two advantages:

1 The equation reflects a dynamic erosion process and is not based upon steady state equilibrium conditions, which clearly do not apply.

2 The erodibility parameter, K_d, can be used to reflect variations in erosion as a function of soil state (compaction, moisture content etc) (Hanson and Hunt, 2006).

It can be seen that soil erodibility is highly dependent upon soil compaction and moisture content, Figure 8.140.



Embankment Test	Soil Classification[1]	Sand[1] > 75µm (%)	Fines[1] > 2µm (%)	Fines[1] < 2µm (%)	PI[2]
P1	SM	64	29	7	NP
P4	CL	25	49	26	17

Note: Based on ASTM : [1]D 2487, [2]D 4318.

Figure 8.140 Example analyses showing relationship between soil erodibility (K_d) and soil type, density and water content (Hanson et al, 2010)

The drawback to using an equation based upon an erodibility coefficient, such as K_d is the need to define a value for K_d. To date this has been undertaken through laboratory or field testing (Hanson *et al*, 2005a) but there are several different methods by which this might be done and results are not yet consistent between approaches (Regazzoni *et al*, 2008b, Wahl, 2008, and Wahl *et al*, 2009). The two main approaches are jet testing (JET) (Hanson) for erodibility relating to surface or headcut erosion, and hole erosion testing (HET) (Fell) for internal erosion erodibility.

Simple guidance on the likely range of erodibility for a given soil and state is available, but this is indicative and care should be taken to assess the impact of uncertainty in these values on any particular study. Temple and Hanson have undertaken programmes of research into soil and vegetation performance at the USDA Agricultural Research Service centre in the USA. As part of this work they have produced some indicative and qualitative descriptions of soil erodibility, as shown in Equation 8.260 and Tables 8.35 and 8.36. Equation 8.260 provides an approximate method for estimating erodibility (K_d) based upon percentage clay content and soil density (Temple and Hanson, 1994).

$$K_d = \frac{10\gamma_w}{\gamma_d} exp\left[-0.121(C_{\%})^{0.406}\left(\frac{\gamma_d}{\gamma_w}\right)^{3.10}\right] \qquad (8.260)$$

where:

K_d = erosion rate (cm³/N-s)
$C_{\%}$ = per cent clay
γ_d = dry unit weight (mg/m³)
γ_w = unit weight of water (mg/m³)

When using Equation 8.260, a value of τ_c is also required (Table 8.37). An approximation is to assume that τ_c = 0 or to use Equation 8.261 (Hanson and Simon, 2001, and Hanson and Hunt, 2006).

$$K_d = 0.2\tau_c^{-0.5} \qquad (8.261)$$

where:

K_d = erosion rate (cm^3/N-s)

τ_c = critical shear strength (Pa)

Given the uncertainty associated with a clear description and measure of erodibility, an alternative approach is to adopt qualitative descriptions of erodibility and to allow for this uncertainty when considering modelling results such as shown in Figure 8.141.

Table 8.35 *Qualitative descriptions of values for K_d modified from (Hanson et al, 2010)*

Description	K_d (cm^3/N-s)
Extremely rapid	1000
Extremely rapid	100
Very rapid	10
Moderately rapid	1
Moderately slow	0.1
Very slow	0.01
Extremely slow	0.001

Table 8.36 *Approximate values of K_d (cm^3/N-s) relative to compaction and % clay (Hanson et al, 2010)*

	Modified compaction (27.5 kg-cm/cm³)		Standard compaction (6.0 kg-cm/cm³)		Low compaction (kg-cm/cm³)	
Clay (%)	*≥ Optimum water content%*	*< Opt WC%*	*≥ Opt WC%*	*< Opt WC%*	*≥ Opt WC%*	*< Opt WC%*
>25	0.05	0.5	0.1	1	0.2	2
14–25	0.5	5	1	10	2	20
8–13	5	50	10	100	20	200
0–7	50	200	100	400	200	800

Table 8.37 *Approximate values of τ_c (Pa) relative to compaction and % clay (Hanson et al, 2010)*

	Modified compaction (27.5 kg-cm/cm³)		Standard compaction (6.0 kg-cm/cm³)		Low compaction (kg-cm/cm³)	
Clay (%)	*≥ Optimum water content%*	*<Opt WC%*	*≥Opt WC%*	*< Opt WC%*	*≥ Opt WC%*	*< OptWC%*
> 25	16	0.16	4	0.04	1	0.01
14–25	0.16	0.01	0.04	0	0.01	0
8–13	0.0	0	0	0	0	0
0–7	0	0	0	0	0	0

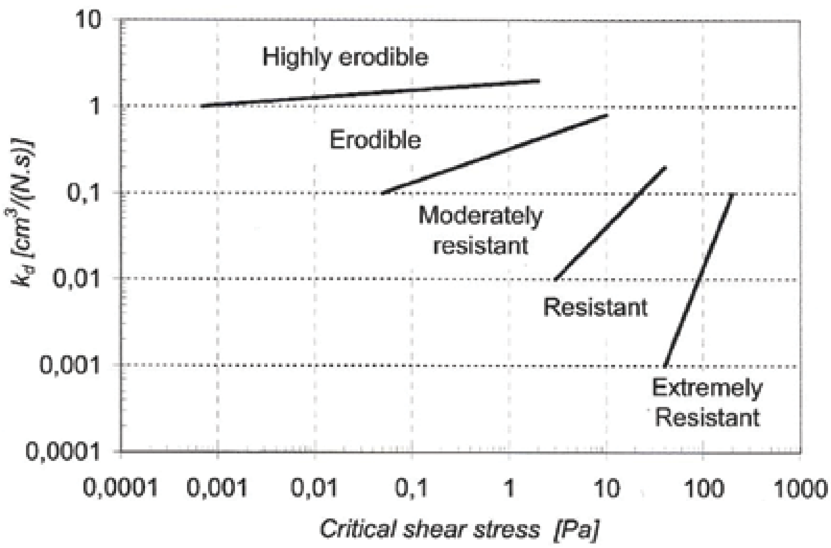


Figure 8.141 Erodibility of soil (from Hanson and Simon, 2001)

8.10.3 Methods for modelling breach growth

There are a variety of different types of model (or method) for predicting breach conditions. These may be broadly categorised as:

- non-physically based, empirical models
- semi-physically based, analytical and parametric models
- physically based models.

Non-physically based or empirical models

Such methods are usually based upon data collected from a series of documented breach events. Breach parameters (eg peak discharge, beach width etc) are estimated from predictor equations, derived through regression analysis using historic case study or laboratory data. The advantage of these equations is their simplicity – there is no need to run computer models. However, this simplicity is also one of their main weaknesses, because there can be considerable uncertainty within the predictions. Users often have little knowledge of the data that the equations were based upon, so any constraints for application and the suitability for application to site specific cases are hidden or unknown. An additional limitation of these equations is that they only predict specific parameter values, for example, peak discharge rather than the whole outflow hydrograph, or final breach width rather than the time varying growth of width. Wahl (2004) provides a review and comparison of these equations, recommending the Froehlich (1995b) equation as the least uncertain:

$$Q_p = 0.607 {V_w}^{0.295} {h_w}^{1.24} \tag{8.262}$$

where:

Q_p = peak outflow (m³/s)
V_w = volume of water stored above breach invert at the time of failure (m³)
h_w = depth of water above breach invert at the time of failure (m).

This equation has been developed by regression analysis against a record of 22 dam failures, so it is unclear how applicable this would be to smaller levees. Note that the key parameters (V_w and h_w) relate to volume and depth of water retained by a dam. Wahl (2004) suggested that the uncertainty in use of this equation was in the order of 0.53 to 2.3 with a hypothetical value of 1.0. Additional equations predicting final breach width and time to failure are also available. The uncertainty in time prediction is greater at 0.38 to 7.3 with a predicted value of 1.0. So applying such an equation to levee conditions offers a quick, simple estimate but with potentially large uncertainties.

All of the equations compared relate to breach formation through dams rather than levees. The suitability for cross application has not been studied and it is likely that considerable errors may be introduced to an already uncertain method of breach prediction. Research work by Verheij (2002) provides a simple relationship between predicted breach width B (m) and time t (s) for sand and clay dikes, based on field and laboratory tests (Figure 8.142),

For sandy dikes: $$B = 67 \log\left(\frac{t}{522}\right) \tag{8.263}$$

For clayey dikes: $$B = 20 \log\left(\frac{t}{288}\right) \tag{8.264}$$

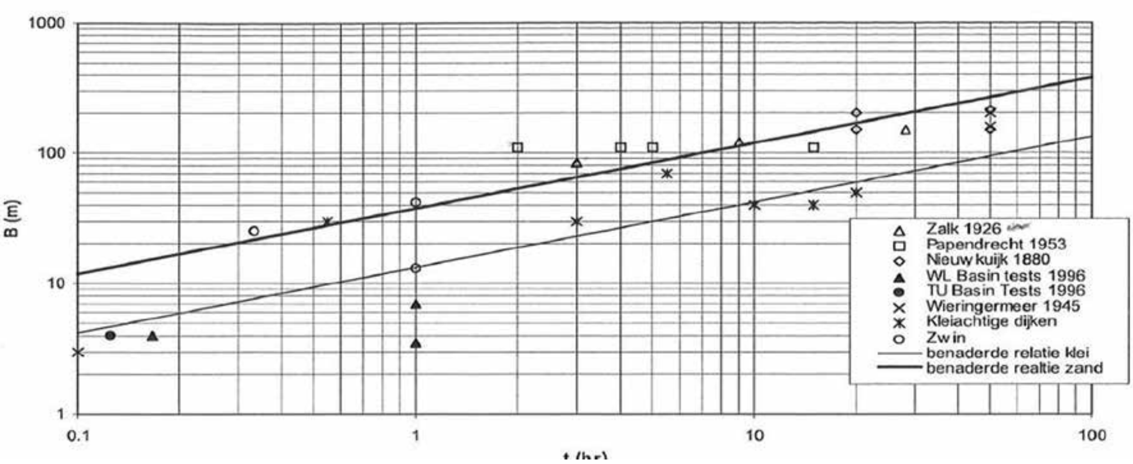


Figure 8.142 Breach width (B) as a function of time (t) and soil type (upper curve: sand, lower curve: clay) (Verheij, 2002)

Semi-physically based, analytical and parametric models

The large range of uncertainty associated with the non-physically based methods prompted development of more complex models. These were based on physical processes, but with simplified assumptions to represent the failure of the dam or levee so as to not unduly complicate the calculation process. Assumptions usually include use of a weir equation to represent the flow over the embankment, so that critical flow conditions exist on the embankment crest. However, these models often also require the input of erosion rate for the growth of breach, or the time taken to form the breach, and the final dimensions of a breach shape. Some models then simply predict a growth pattern to fit these parameters and subsequently produce a flood hydrograph. So their values may vary and are highly dependent upon the user. While these models appear to provide a more accurate prediction of the flood hydrograph in comparison to empirical equations, they simply reflect the data provided by the user, and can also include large degrees of uncertainty.

Examples of this approach can be seen within the original DAMBRK code and the Hydrologic Engineering Center River Analysis System (HEC-RAS) (USACE, 2011). Sensitivity analysis is usually performed using several methods of parameter estimation to develop an expected range of breach behaviour, and then try to determine the range of uncertainty within the approach.

Physically based models

Physically based numerical models simulate the failure of embankments based on the processes observed during failure, such as the flow regimes, erosion and slope instability processes. In the last four decades many models have been developed to simulate the failure of dams and levees. These models differ in their complexity, assumptions involved, and techniques used. Morris (2011) provides a summary of these models.

The CEATI Dam Safety Interest Group funded a research project to review and evaluate breach models for industry use (Morris *et al*, 2012b). After an international review, this research focused upon the

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performance of two models – the HR BREACH model (Mohamed, 2002, and Morris *et al*, 2012b) and the SIMBA model (Hansen *et al*, 2005c). The SIMBA model is now integrated into the WinDAM B software (USDA, 2013) while the 2008 HR BREACH model is integrated into the InfoWorks®RS (Innovyze, 2013) flow modelling package. SIMBA simulates breach formation through headcut, while HR BREACH simulates breach initiation and formation through surface erosion. However, a version of the SIMBA headcut process is also included in HR BREACH, along with prediction of breach growth through pipe formation.

Both SIMBA and HR BREACH are physically based models although they adopt different approaches to simulation. SIMBA runs very quickly, but achieves this by predefining the erosion failure process. HR BREACH takes minutes to run, but allows the model to predict how erosion develops through the levee.

Later developments of HR BREACH (Morris *et al*, 2012b) introduce the ability to predict breach formation processes through zoned or layered levees, where layers of different soil, or soil erodibility, can be seen to significantly affect some breaching processes. Development of a new simplified model called AREBA (van Damme *et al*, 2011) takes a similar approach to SIMBA in predefining the failure process, but includes failure (of homogeneous levees only) by considering surface erosion, headcut or piping. AREBA runs in less than one second, making it a useful tool to aid understanding of how a levee might fail under varying conditions and failure modes.

Although the WinDAM (SIMBA), HRBREACH and AREBA models are some of the most recent physically based breach models, it should be recognised that there are a variety of other models in existence, often developed as part of research programmes. When choosing a particular breach model to use it is important to understand what processes are simulated and what simplifications or assumptions have been made and how these affect your particular case.

The EU FLOODsite project included a substantial programme of research into modelling breach initiation and growth. Conclusions from this work, including guidance on breach models and modelling can be found online (Oumeraci, 2005). A range of associated reports also provide supporting information and offer a good starting point for anyone interested in understanding more about breaching processes. In particular, Oumeraci (2005) provides frame by frame images of various levee failures, highlighting the different processes that occur.

Selecting breach model input parameters

The model input parameters required will depend upon the model being used, ie the simpler the model (or equation) the fewer parameters are required but the greater the uncertainty in prediction. Since breach processes depend upon the hydraulic load, combined with soil erosion and structure response it would be reasonable to assume that parameters reflecting these processes are required in order to model these processes. A key parameter affecting erosion is the soil erodibility, which is affected by parameters such as soil water content, compaction etc. However, in practice, these parameters are often not taken directly into account, instead being reflected by judgement as to the soil type and of potential erodibility. Although this approach offers a practical approach to breach modelling, care should be taken to include consideration of how erodibility might vary for a given case. Formal sensitivity analysis using a range for key modelling parameters is strongly recommended.

It should be noted that early breach models tended to use sediment transport equations to predict erosion within the breach. In recent years there has been a move by many modellers towards the use of erosion equations, which derive the rate of erosion from the flow stress and soil erodibility, rather than simply particle size. This better reflects the dynamic, rapidly changing conditions within a breach and allows consideration of soil state as well as type.

Integrated breach and flow modelling

To correctly simulate breach conditions within a levee system it is often necessary to integrate the breach and flow modelling together. Where downstream floodwater levels can exceed about two-thirds the depth of upstream water levels on the breach invert, then the release of water through the breach and the rate of breach growth will be affected. In these situations, the correct prediction of conditions requires a step-by-step analysis of water levels and breach growth throughout the levee system. Very few breach models are truly integrated with flow models to provide breach predictions throughout a levee system. In some situations the effects of drowning on overall flood conditions will be significant.

Examples of different models for breach analysis are given in Boxes 8.26 to 8.27.

Box 8.26 ***Example of non-physically based or empirical models***

For some studies a simple assumption that breach has occurred is made, and flood conditions are then simulated. While simple, this can be unduly pessimistic for assessing the extent and magnitude of flood risk. Figure 8.143 shows an inundation plan generated from such an assumption. These results can be quickly misinterpreted because the degree of detail from the inundation mapping masks the crude assumptions made for breach modelling, which ultimately dictates the volume and rate of floodwater released into the inundated area.

Figure 8.143 ***Example of zoning of water depths 30 minutes after a levee breach in an urban area (Th. Monier, Sogreah, 2011)***

An example of the differences that might be found through predicting breach rather than assuming instantaneous breach are shown in Figure 8.144. The left plot shows breach growth with time, the right plot shows the difference in predicted flood hydrograph. The example was computed by calculating in advance the breach evolution with the code Rupro, developed by Irstea in France (this code is included in the simplified breach modelling code CastorDigue), then by using this evolution in the hydraulic modelling code. This assumption requires that the breach does not drown during formation, because the breach growth and flow modelling are undertaken independently.

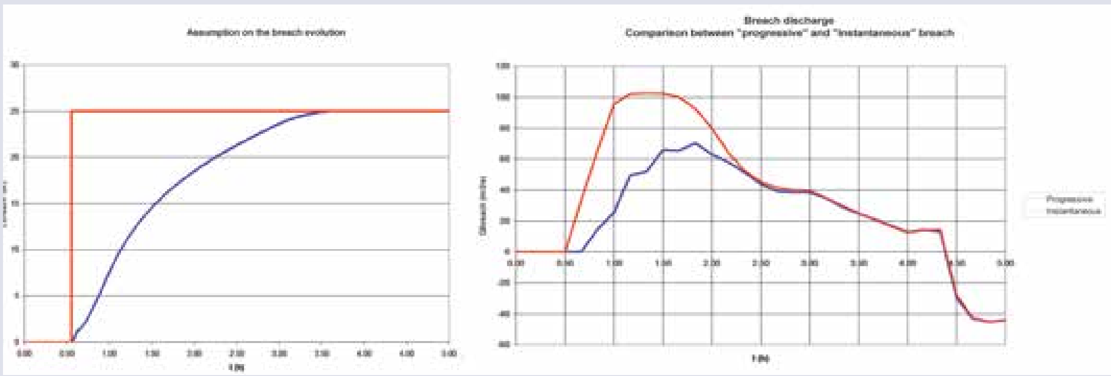


Figure 8.144 ***Example of breach prediction assuming instantaneous breach (red) or progressive breach growth (blue)***

Box 8.27 *Examples of physically based models*

The HR BREACH model was originally developed by Mohamed (2002), and integrated with the InfoWorks®RS flow modelling package in 2008, and extended to simulated breach through zoned embankments by Morris (2011). The model requires a range of parameters to define the hydraulic boundary conditions, embankment structure and soil type and state. Breach simulation takes into account surface protection (grass, rock etc) and simulates breach formation through surface or headcut erosion, and piping. The model predicts conditions at sections through the embankment, uses a soil erosion equation to predict erosion section by section and allows for discrete block failure and removal during the process.

The integrated version of the breach model interacts with a 1D/2D flow modelling package at a time step level so that conditions within the breach and the associated flood cells update interactively (Figure 8.145). Multiple breach units can be run within the flow model simultaneously allowing prediction of multiple breaches within a levee system within a single simulation.

The extended (Morris, 2011) version of the model includes the ability to simulate breach formation through zoned embankment structures. So, where a levee has been constructed using different material in different areas, or where a levee has been extended so that there are layers of different soil, or different soil states (and also erodibility) the model simulates how the rate and shape of breach growth is affected by the various zones. Zones of different material within a single levee can significantly affect the way in which a breach forms.

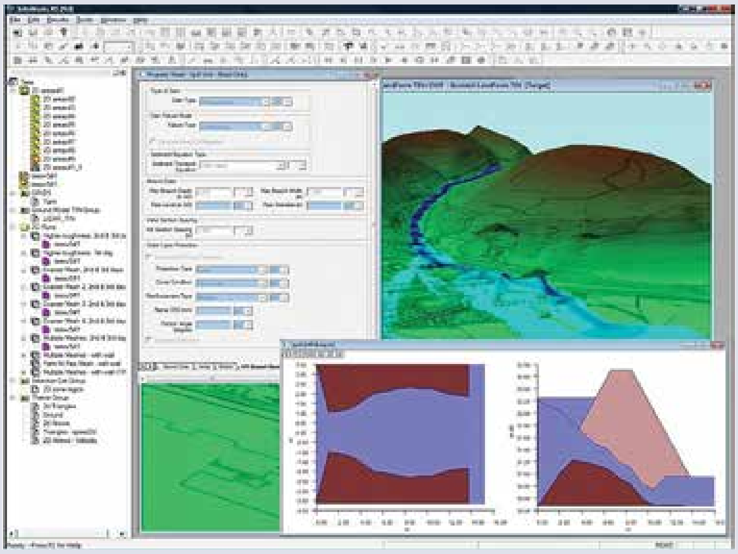


Figure 8.145 Fully integrated breach and 2D flow model

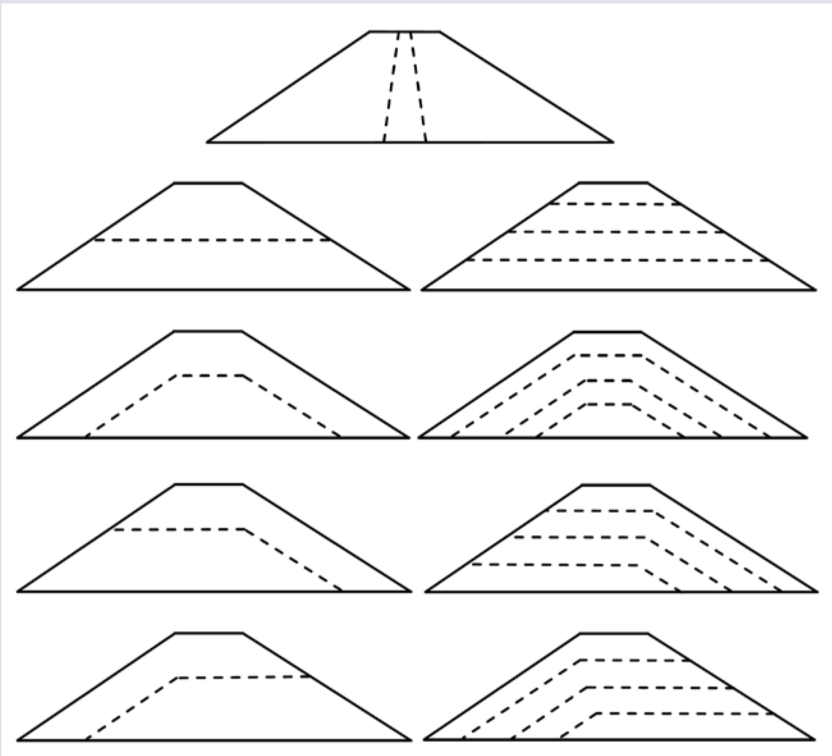


Figure 8.146 Zoned approach to breach modelling (HR BREACH, 2011)

Box 8.27 Examples of physically based models (contd)

The WinDAM B code (USDA, 2013) incorporates the SIMBA headcut model. SIMBA simulates headcut erosion through a levee or earthen dam by assuming a predefined failure process. By making these assumptions the model can simulate breach very quickly (a few seconds). The WinDAM B package incorporates the SIMBA model within a framework that allows simulation of a reservoir, including grass resistance to overflow.

Figure 8.147 WinDAM code for estimating erosion of earthen embankments and auxiliary spillways of dams (from USDA, 2013)

AREBA is a new model that adopts a similar approach to SIMBA in predefining the way in which breach formation occurs, but allows the user to simulate erosion through surface erosion, headcut or internal erosion (pipe formation). The model takes less than a second to run and has been designed for use either within system risk models (ie simulation of flood risk for levee systems) or as a 'standalone' model. At the time of writing (2012), the model was being finalised.

Example of current practice

The second type of model is often used in operational studies because it remains a simple tool but avoids fixing the evolution timescale in a quite arbitrary way. For instance, for the case of La Faute sur Mer (France), the breach model Rupro, which is encapsulated in CastorDigue software (Irstea, 2012) was used (Box 8.28).

Box 8.28 Rupro model

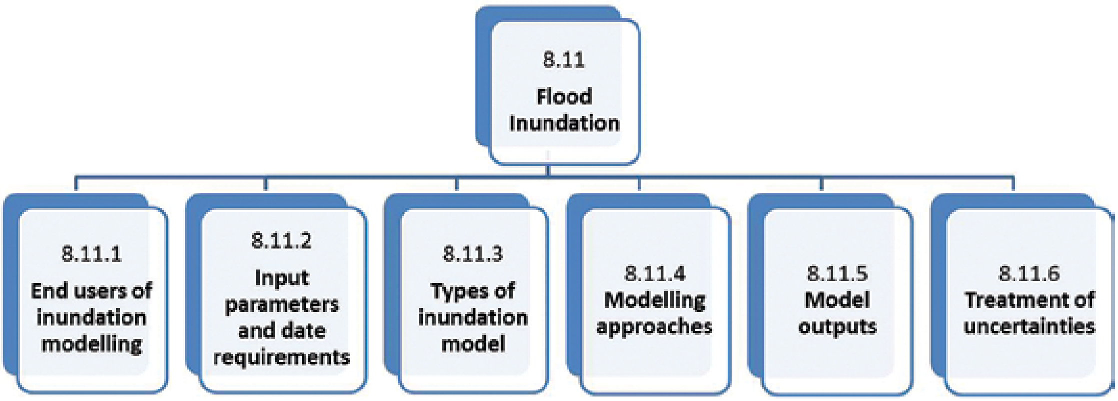
This model relies on the assumption that the breach cross-section can be represented by an average value and does not have to be explicitly defined, which helps to evaluate the linear loss of head along the breach channel. Then, the discharge hydrograph is obtained using the Bernoulli equation while the rate of erosion comes from the sediment discharge calculated using the Meyer-Peter and Müller (1948)'s equation:

$$Q_s = 8\,(\beta\psi - 0.047)^{3/2}\sqrt{\left(\frac{\gamma_s}{\gamma_w} - 1\right)\, g\, d^3} \qquad (8.265)$$

The primary shape of the representative breach cross-section is either circular (such as occurs due to piping) or rectangular (such as occurs due to overtopping). Results from the Cadam and Impact European research projects(blind tests on controlled experiments both field and laboratory) showed that such a simplified model can provide suitable estimates of breach discharges but less reliable results on breach shape development (Paquier and Recking, 2004). From the Impact European project, Paquier (2007) showed that the model can be improved by reducing the erosion rate during the widening step in order to obtain nearly perfect agreement between measured and predicted results (error is of the same order as measurement uncertainty). The limits of such a model and also of most of the models to field applicability are the assessment of model parameters (which are easier to establish in controlled experiments with homogeneous materials). During the Impact European project, the uncertainty assessment of a well-known dam break event (the Tous dam failure in Valencia, Spain) provided a 30 per cent uncertainty for peak discharges at 90 per cent, due to the uncertainty parameters. If the failure scenario is not known, uncertainty will be much higher. Therefore, to reduce uncertainty of breach modelling results, it is important to consider the following factors:

- location of the breach
- estimation of the time for breach development
- assessment of the levee material parameters.

8.11 FLOOD INUNDATION



This section provides guidance on how to relate accuracy of modelling results to the end user and be clear on limitations of existing methods, and under which circumstances these methods may be applicable. The section highlights, as appropriate, current efforts undertaken by groups or organisations across the world, especially during the FLOODsite project (Oumeraci, 2005).

8.11.1 End uses of inundation modelling

8.11.1.1 Land use planning

A critical component of risk reduction is minimising the consequences that could occur as a result of a flooding or storm event. A large component in minimising the consequences is increased awareness of all stakeholders (population at risk in addition to any federal, state, and local government entities). So, making information such as flood maps that incorporate breach scenarios available to stakeholders is a prudent step. Flood mapping should be made available to the public and accompanied by information explaining the risk linked to these breaching processes (for example, the flooding of the town of Toulouse in the South of France)

8.11.1.2 Risk analysis

In several countries, national policies are imposed on owners of levees to assess the risk induced in the flooded area by a failure or a breach that may occur in the levee. To do so, 2D inundation modelling is used to determine and localise versus time, maximum velocities and water depths in the flooded area. For life safety, these results are compared to criteria chosen generally to enable safe evacuation (an example of limits is shown in Figure 8.148).

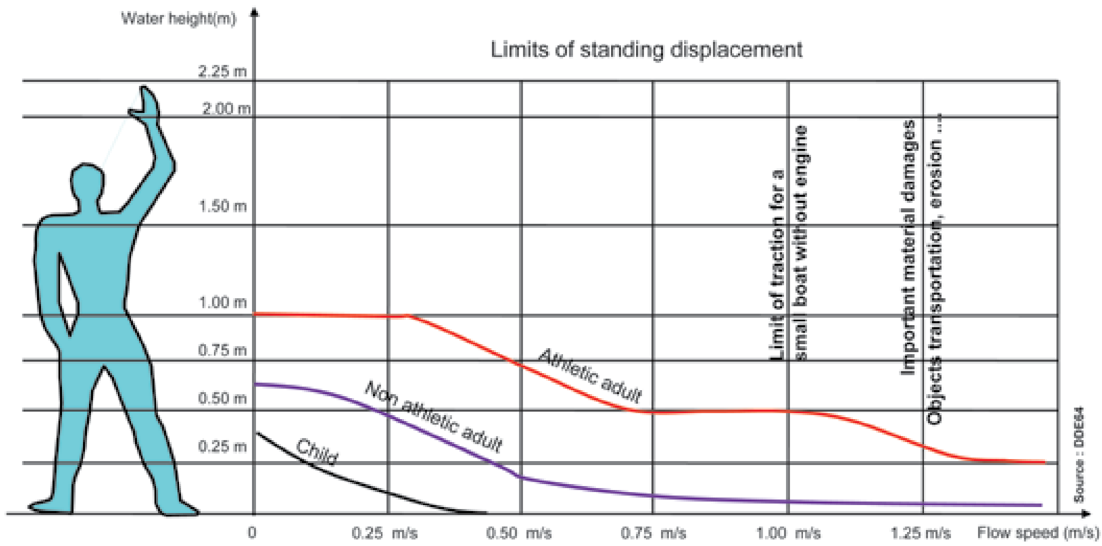


Figure 8.148 ***Limits of standing evacuations for a child (lower curve), a non-healthy adult or a stressed healthy adult (upper curve) regarding water depth or flow velocity***

This kind of criteria can be used to estimate casualties and material damage. An example of a map used to communicate flood inundation is given in Box 8.29.

Box 8.29 ***Example of flood map from a 2D diffusive wave model***

Figure 8.149 ***Example of a map of maximum depth generated with RFSM, showing the inundation resulting from a breach in a levee protecting an urbanised area in the floodplain***

Planning authorities often use inundation mapping to evaluate the potential risk of areas considered for new developments. Insurance companies also use inundation mapping techniques to calculate insurance premiums for individuals and businesses.

8.11.1.3 Flood and risk management

Another type of end user is the emergency responder that should organise the evacuation of the population in advance. In this context, modelling could be used to highlight the areas with the greatest risk of casualties. Box 8.30 illustrates the use of flood mapping in the case of emergency preparedness.

Box 8.30 Inundation mapping for evacuation planning

A flood inundation map was used for preparing the evacuation plan for a population (provisional results for a municipality in the South of France). Because of the high probability of overtopping in cases of extreme flood and the short period for evacuation (flood peak can occur within one hour from the start of rainfall), the municipality proposed to evacuate the population, starting with the people in the more exposed locations (red, orange, and yellow grids on Figure 8.150). Using a 2D model for the whole area and simulating breaches in various locations, hazard classes due to levee failure were derived from classes for peak water depth and peak velocity. Due to the low water head upstream the levees are located very close to the river where most of the more risky areas are, within 100 m of the levees. The computational mesh is shown in Figure 8.150.

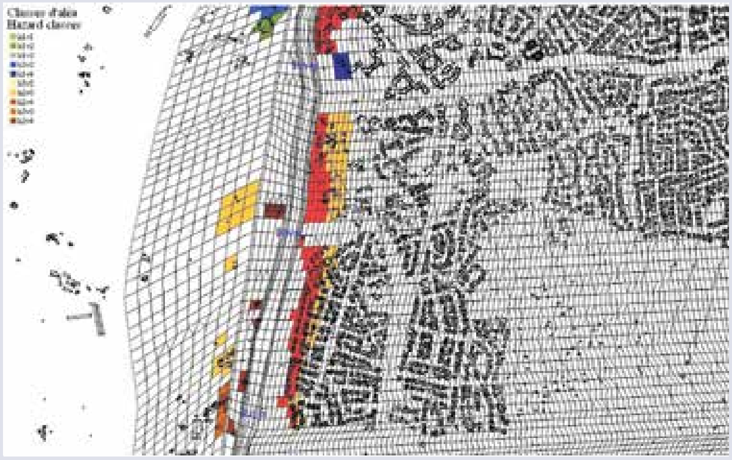


Figure 8.150 Flood map showing high risk areas due to levee failure calculated from 2D hydraulic model

8.11.2 Input parameters and data requirements

8.11.2.1 Input data

The primary data representation for inundation modelling is the digital terrain model (DTM). It gives a numerical representation of the topography and is usually acquired from the air (airplane or satellite). The most common format is a regular grid, but triangular irregular networks (TIN) also exists. LiDAR (Light Detection and Ranging) is currently the most accurate type of DTM, with horizontal resolution as low as 0.5 m (or even lower), and vertical accuracy of 0.1 m. But this means that the amount of topographical data available can be much greater than the data the inundation model can take into account. Other types of DTM available include SAR (Synthetic Aperture Radar) and ASTER (Advanced Spaceborne Thermal Emission and Reflection Radiometer), which are more widely available, cheaper to purchase (or free), and cover extensive areas. A DTM can also be created by digitising points using a detailed topography (paper) map if no DTM can be sourced at the required resolution. Alternatively, depending on the type of model used, cross-sections can be given to represent the topography. These will usually be measured perpendicular to the river and cover the river and the floodplain.

A land use dataset is useful to help the modeller to assign the friction coefficient values. In urban areas, the network of streets and the location of buildings have a key influence on the propagation of the flood. Being able to use a polygon dataset indicating the contours of the buildings is important to build a detailed inundation model, and can be obtained from national cartographic institutes or from the internet. Bridges are not captured properly in a DTM, as this will show only the top of the bridge. Manual editing might be needed to restore the terrain elevation under the bridge and avoid artificial blockages. Also it is possible in some software to insert a 1D structure within the 2D mesh to represent the bridge opening.

The flow model should include the whole area that is influenced by the breaching process. In the case of breaching a levee along a river during a flood, breaching will influence the whole flooding process downstream from the breach, so the extent of the breach model including breaching will be similar to the extent of the general flood model. Due to the high velocities close to the breach, any obstacle in the floodplain involves a rise of the water elevation upstream and a drop downstream, so it is important to describe the floodplain in detail near the breach. In the case of a flood flowing through an urban area, the flow will be concentrated in the streets and straight streets can generate high velocities. The location of the breach in relation to the street directions influences the flood results (El kadi Abderrezzak *et al*, 2009).

Initial conditions (level and velocity) in the river and the floodplain are needed for the numerical model to start the simulation, although the floodplain is usually assumed to be dry at the start. Boundary conditions are also needed at the upstream and downstream end of the river (upstream discharge, downstream level or rating curve) and at the limits of the floodplain (no flow, free flow).

8.11.2.2 Model assumptions

Other parameters influencing the inundation cannot always be measured and some assumptions need to be made. The main assumptions are:

- location of the breach in the dike
- moment of the beginning of the breach (if the evolution of the breach is modelled) or moment of the breach (if an instantaneous breach is assumed)
- maximum depth of the breach
- maximum width of the breach.

Some of these assumptions can be helped by using existing studies (eg hazard studies required for some dikes in France). The number of assumptions can be large and these can influence the results such as water level or velocity in the floodplain or the flooded area. For this reason it is often considered that a precise modelling of the breach evolution is not always needed.

8.11.3 Types of inundation models

8.11.3.1 Model requirements

The aim of a flood model consists in providing the time evolution of water depth and flow velocity in any point of the floodplain. Flow coming from a breach failure, such as flash-floods in urban areas, is generally characterised by high velocity and high water level. Both factors lead to an increased risk compared to an assessment using water depth only. In terms of constraints for modelling, the fast flows imply that the numerical model needs to cope with the changes of regime from subcritical to supercritical and conversely.

If all the physical processes are considered, the flood model should also consider sediment transport, sediment coming from the breach and sediment eroded downstream of the breach. Even if sediment transport is not included in the modelling, it is better to integrate the breach model with the flood model in order to have the right upstream condition for the flood model (breach discharge hydrograph), but above all in order to obtain the right upstream and downstream hydraulic conditions for the breach model. Also, such integration or coupling is necessary in cases where the flow is going from upstream to downstream of the levee by other means or processes than a breach (overflow, piping, connecting hydraulic structure etc).

8.11.3.2 Choice of the hydraulic model

First of all, the modeller has to choose a hydraulic model that is suited to the considered inundation. Because of the configuration of a breach and, usually, an extended floodplain, the flow is essentially 2D horizontal. This means that 2D models are relevant (or 3D if the vertical distribution of velocity is considered) in most cases. In the case of flood wave propagation due to dam break, a 1D model is acceptable, but this is not the case for flood wave propagation due to a dike breach, because the flow is spreading in the floodplain and no preferential direction can be assumed. 3D models are more expensive to create than 2D models, and are not always necessary. Indeed, in some situations a model may not be needed at all. Given gauged water surface elevations along a reach, or water surface elevations predicted, based on flood frequency analysis, a similar interpolation to that used by Werner

(2001 and 2004) can be created. This estimates the flood wave as a plane (or series of planes), which is intersected with the DEM to give extent and depth prediction.

Models that solve the shallow water equations (Bates *et al* 2010), (either 1D or 2D), are preferred as they can represent both subcritical and supercritical flows. Advanced models have shock capturing capabilities to represent more accurately the flow regime changes (hydraulic jump). Infoworks (Innovyze, UK), Telemac and Mascaret (EDF, France and consortium), Rubar20 (Irstea, 2013) are a few examples. Although when the levee failure is progressive, the flood hydrograph is less extreme and a simplified flow algorithm can be used. Examples are Lisflood at the University of Bath (Bates *et al*, 2010) and RFSM-EDA from HR Wallingford (Jamieson *et al*, 2012a and b), based on the diffusive wave approximation with a local acceleration term.

Examples of flood maps produced with three different models are shown in the Boxes 8.31 and 8.32.

8.11.3.3 Computation set-up

Advances in computational hardware in recent years have led to a reduction of model run-times. Faster processors and large amounts of RAM contribute to model acceleration, but parallel processing is the main factor. Parallel processing involves splitting the computation between multiple concurrent processes, and reducing the total time needed for the whole process. There is a small overhead in doing so, meaning that the total reduction in run-time is not equal to the number of processes, as the calculations from each process need to be merged at the end of a time step. Three computation approaches are commonly used to achieve parallel processing:

- multi-core
- multi-computer distributed
- graphics processing unit (GPU).

Usually model software supports only one parallel processing approach. Some inundation models can run in parallel such as Telemac and Infoworks.

8.11.4 Modelling approaches

Different options are available when modelling the flood caused by a dike breach:

- model the breach evolution and the flood wave propagation in the floodplain simultaneously by coupling a breach model and a hydraulic model
- estimate a realistic hydrograph at the breach, by means of a separate calculation, and use this hydrograph as an inflow into the floodplain
- assume an instantaneous breach and propagate the flood wave in the floodplain.

The first solution is more elegant from a scientific point of view, but it requires combining a soil or breach erosion model with a hydraulic model, and as mentioned previously, the uncertainty of the sediment or soil parameters is high in these models. Also, the physical phenomenon of breach formation and evolution are not completely understood. Research on this issue is still ongoing (ie research programmes FLOODsite and ERINOH in Europe, or LEVEES and DOFEAS in France).

The second solution is more practical, but the disadvantage is that only a part of the hydraulic system is integrated in the model. This solution does not take into account the interactions between the river, the dike and the floodplain. These interactions can influence the breach discharge, water level and velocities in the floodplain, which cannot be taken into account if the hydrograph is set at the breach.

The third solution is easier to implement, because the breach is assumed instantaneous. This assumption is acceptable especially in the case of concrete or masonry structures. In the case of earth levees, the breach is generally not instantaneous, but it is not obvious that this assumption has great influence on the impact in the floodplain, in particular on the flooded area at the end of the simulation. Also, this assumption is favourable from a safety point of view, because the water levels and velocities should be overestimated compared with a progressive breach assumption.

It is preferable to model the whole flood system including the river and the floodplain to its left and right, the dike, and the landward zone, which could be flooded in case of breach in the dike, in order to capture all the processes involved and their interaction.

8.11.4.1 Model coupling

Coupling can be achieved by four methods:

- linking breaching and flow software by external coupling through the upstream and downstream water elevations.
- linking breaching and flow software by an exchange protocol such as the OpenMI Association, allowing a dynamic interaction between the two models
- using flow software (generally solving the 2D shallow water equations) that treats the breach as a hydraulic structure evolving in time:
  - **Rubar 20** software (Paquier, 2009 and 2010), developed by Irstea (2013) that integrates the simplified breach model Rupro (the parameters of the hydraulic structures representing the breach are assigned *a priori*)
  - **Infoworks®RS** software, developed by Innovyse (2013) that includes HR BREACH developed by HR Wallingford.
- using a sediment transport model in which the embankment is treated as an ordinary bed area. Generally, this kind of model is based on 2D shallow water equations with additional equations to simulate bed load or suspended load. This kind of modelling allows assessment of the erosion and deposition processes downstream of the breach, which can influence the water elevation. Alternatively models including the sediment as a fluid phase can be used. Although they bring some advantages in terms of coupling various very unsteady processes, these models are not fully operational, or still need an empirical parametrisation and a sensitivity analysis when used in operational situations. A benchmark of some of this software was performed at a PIRE workshop in Belgium (Soares-Frazaoa, 2012), which allowed evaluation of the corresponding uncertainty for further spreading of sediment.

8.11.4.2 Multiple breaches

For operational purposes, it should be considered that flooding can be caused either by one or several breaches at the same time. A first step should be to select the scenarios corresponding to the aim of the study. If there are many possible scenarios, this step is not obvious and a simplified model can be used (such as CastorDigue developed by Irstea, 2013, or AREBA developed by HR Wallingford and University of Oxford) to evaluate which scenarios should be studied in a detailed way. This selection can be based on a comparison of the breach outflows. Alternatively, select the breaches that will start first.

8.11.4.3 Specific modelling of urban areas

Buildings and streets have a great impact on flood propagation in urban areas as they create preferential flow directions. Urban areas can be represented in various ways by inundation models. The three following approaches are the most common and are used in both consultancy and research applications:

1. **Raised ground:** the ground elevation in the footprint of the building is raised, to the real elevation of the top of the building or to a generic value, such that water never flows through the building footprint. This can be done as a direct use of a digital surface model (DSM) or by modifying the DTM elevations using the dataset of building polygons. However, using a DSM can be a challenge as it will also show the elevation of the top of the trees rather than the ground. This approach can cause stability problems in some models if computational cells straddle the edge of the building, as they will have very steep slopes.

2 **Increased friction:** here the individual buildings are not represented in the computational mesh, but the whole urban area is represented by an extremely high friction coefficient to account for the reduction in conveyance through the urban area (low Strickler coefficient or high Manning coefficient). However, this approach does not account for the reduction in storage in the urban area.

3 **Voids:** the building footprints are used to create holes or voids in the computational mesh. The effect is similar to the raised ground approach, but this avoids issues with steep slopes at the building edges. It also requires a flexible meshing technique (unstructured mesh usually).

Sanders *et al* (2008a) describe a comparison between the raised ground and voids approaches. The increased friction and voids approaches are compared in Soares-Frazao *et al* (2008).

The following approaches are also possible but are less common:

- **porosity:** some models use a modified version of the shallow water equation that includes a porosity coefficient. This porosity coefficient can be different in each cell, it can be isotropic or anisotropic (Guinot and Soares-Frazao, 2006, and Sanders *et al*, 2008b). It is usually constant with time and with the water depth. This approach means that buildings are not represented individually in the mesh, reducing the number of computational cells and the run-time. Both conveyance and storage reduction are represented correctly. Schubert and Sanders (2012) present a comparison of the three approaches previously mentioned, with the porosity approach.
- **sub-cell topography:** instead of having one average ground elevation, each computational element is assigned a range of ground levels that captures the topography inside the element (Hartnack *et al*, 2009, and Jamieson *et al*, 2012b). Both conveyance and storage reduction are represented correctly, and this approach is also useful outside of urban areas. This allows use of large computational elements while still capturing accurately the topography, reducing the run-times.
- **multi-layer:** in this approach complex building footprints are finely captured using simple polygons contained in multiple layers (Chen *et al*, 2012). This allows use of a coarse mesh while still accounting accurately for the buildings, hence reducing the run times.

8.11.5 Model outputs

Water depth, level and velocity are the variables calculated by all models. Time series of water depth and velocity are produced by the models and allow understanding and visualising the evolution of the flood. Maximum depth and maximum velocity are often used for the production of flood maps.

Hazard to people is usually considered to be related to the product of flow depth and velocity (Ramsbottom *et al*, 2004). This can be calculated by the inundation model at each time step and saved with the other outputs. It is more accurate to calculate the maximum hazard as the maximum in the hazard time series, rather than as the product of maximum velocity and maximum depth. This is because the timing of the peak depth and peak velocity can be completely different, and the product of the maximum values can greatly overestimate the maximum hazard. Hazard can then be considered alongside vulnerability to estimate the risk to people. Similarly, building failure can be estimated by the inundation model using flow depth and velocity, plus some parameters describing the resistance of the buildings.

The outputs from the inundation model can then be used as inputs to an evacuation model (for example, life safety model (LSM) from BC Hydro – Canada, and the Utah Water Research Laboratory model). Using a description of the population (eg age, location, transport mode, decisions) and the road network, the arrival of the flood wave triggers the evacuation of the population. The evacuation model routes people through the road network towards 'safe havens', and estimates the number of casualties from drowning, cars being washed away and building collapse. The LSM model has been applied to various study cases in Europe and North America and proved to give reasonable estimates of the number of casualties (Lumbroso *et al* 2010 and 2011). An example output from LSM is shown in Box 8.33.

8.11.6 Treatment of uncertainties

In order to develop the data required to understand probability of occurrence and uncertainty, techniques such as the Monte Carlo simulation are used. In a Monte Carlo simulation the variability of the various input parameters are represented and a large number of model runs are carried out with each input parameter sampled from its underlying distribution, so the data generated by the total set of model runs can be analysed probabilistically. In this way, the sensitivity of the overall outputs (such as inundation depth and timing) to specific parameters can also be evaluated, and the different components of risk can be assessed. A significant amount of effort has been undertaken recently to refine Monte Carlo simulation techniques to allow more complete and more complicated evaluation of input parameters. HEC-RAS (USACE, 2011) and similar hydraulic models provide deterministic results for specified input conditions, ie a single set of input (flow, channel conditions, breach formation parameters) is provided and the characteristics of flow are generated for that specified condition.

A Monte Carlo approach can also be incorporated in breach software. HR BREACH can give a distribution of likely outcomes (fail/not fail) and a range of shapes and peak values for the hydrograph in failure cases, depending upon the knowledge of the embankment properties and a given probability distribution for the input parameters.

Monte Carlo simulation can be applied not only to the hydrologic and hydraulic modelling, but throughout the flood assessment process wherever appropriate understanding of uncertainty is required. Froehlich (2008) presents a method to use Monte Carlo simulation to evaluate the effects of breach parameter uncertainty within an inundation analysis. Determination of the nature of the expected distribution, and possible spatial correlation of relevant input parameters, is an important consideration when conducting failure assessments. Levee fragility analysis incorporates Monte Carlo simulation of structural soundness as reflected in 'fragility curves'. Fragility curves display the probability of failure of a levee segment due to one or more mechanism over the full range of loads it is likely to experience. Curves can be developed based on analyses of specific locations, but general curves can also be developed based on generic levee type where more specific information is not available. Such an approach has been used for the UK National Flood Risk Assessment (NaFRA). Monte Carlo simulations for flood breach analysis would apply probabilities to each reach of levee, which would be converted into an elevation where failure will occur for that run. Unsteady floodwater level profiles would be input and breach locations for each run would be determined. The summation of output from a large number of runs would indicate which reaches of levee are most likely to breach under the chosen flood conditions.

Box 8.31 Example of flood map from 2D finite element software

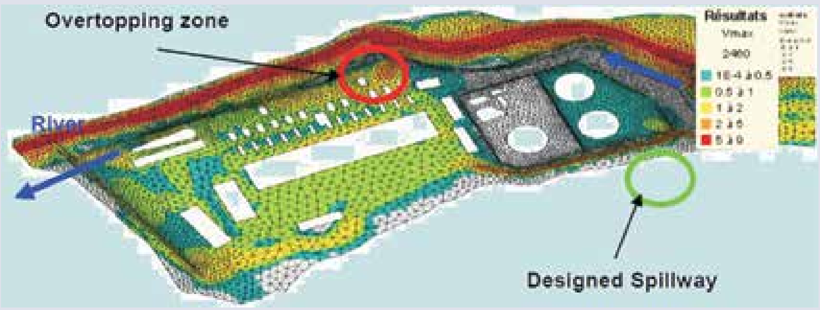


Figure 8.151 Example of flood map showing maximum flow velocities due to levee overtopping calculated from 2D finite element software (Telemac) for which the computational mesh is shown (EGIS Eau for EDF PEI, 2009)

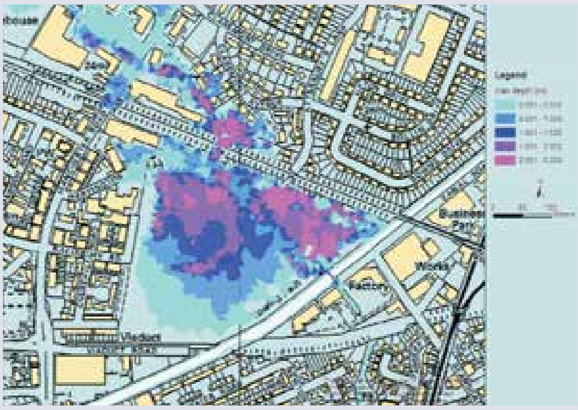


Figure 8.152 Example of a map of maximum depth generated with InfoWorks®RS, showing the inundation resulting from a breach in a canal embankment (from Innovyze, 2013)

Box 8.32 Example of implementation of a 2D hydraulic modelling of the flood wave due to a breach in a dike

In the case of an open source code, ie when the user can modify subroutines to model a breach, the instantaneous breach modelling is rather easy to do. The user has only to locate the nodes (in the case of a finite elements mesh) or the elements (in the case of a structured mesh) and to modify the field elevation when the breach occurs. Figure 8.153 illustrates the modelling of breach in a sea dike, before the breach, just after the breach, at the maximum level of the sea and at the end of the event.

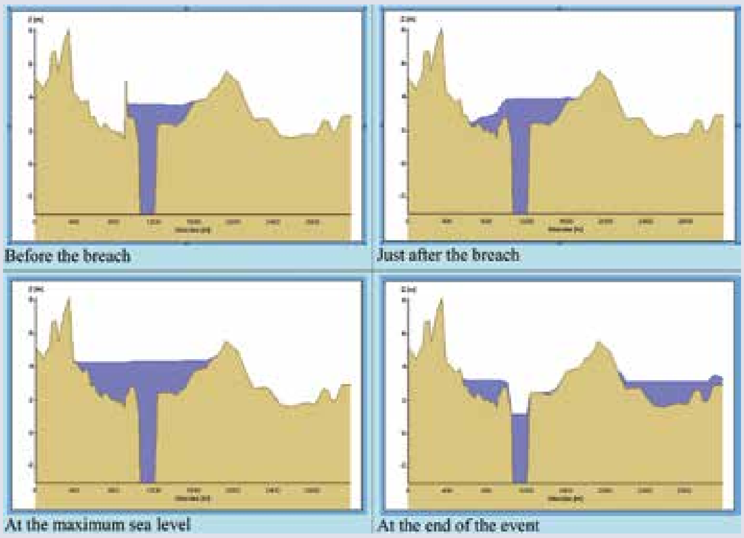


Figure 8.153 Breach modelling in a sea dike with the 2D code Telemac

Progressive breach modelling is also possible, but its utility has to be compared to the aims and precision needed. Use of such a model is more difficult to do than an instantaneous breach model and sometimes not justified. The following example has been developed by first calculating in the breach evolution using Rupro, developed by Irstea in France (this code is included in the simplified breach modelling code CastorDigue, Irstea, 2012), then by using this evolution in the hydraulic modelling code. Figure 8.154 presents the breach evolution calculated with the code Rupro.

Box 8.32 Example of implementation of a 2D hydraulic modelling of the flood wave due to a breach in a dike (contd)

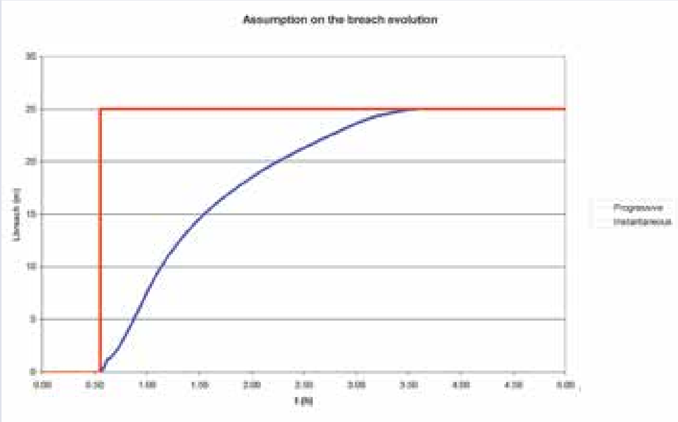


***Figure 8.154** Assumption on the breach evolution*

Figure 8.155 presents the calculated hydrograph at the breach in the case of a progressive breach evolution (calculated using Rupro) and in the case of an instantaneous breach.

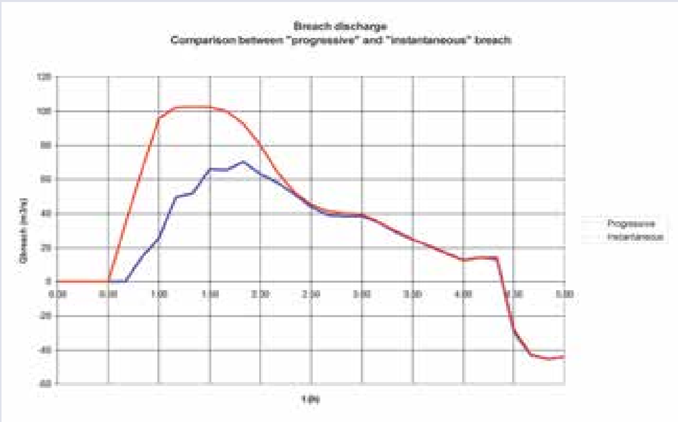


***Figure 8.155** Breach discharge*

In this example, the breach discharge with an instantaneous breach is higher than with a progressive breach, which is favourable from a safety point of view. The question is to know if this assumption is acceptable or not. By considering the hydraulic conditions downstream of the breach, especially the maximum water level or velocity, it can be noted that this assumption is conservative. Figures 8.156 and 8.157 present the maximum water level and the maximum velocity downstream of the breach.

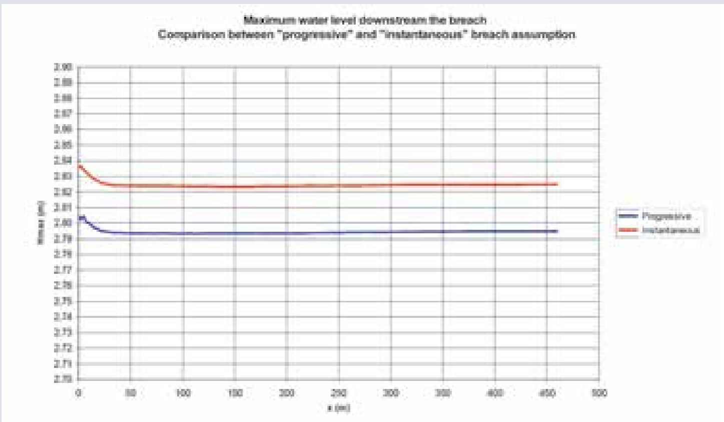


***Figure 8.156** Maximum water level downstream the breach*

1 2 3 4 5 6 7 8 9 10

Box 8.32 Example of implementation of a 2D hydraulic modelling of the flood wave due to a breach in a dike (contd)

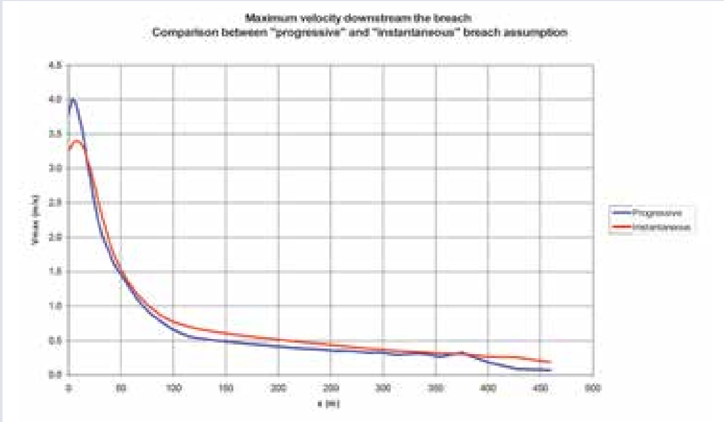


Figure 8.157 Maximum velocity downstream the breach

From these figures, it can be noted that the water level differs by a few centimetres and that the velocities are very close.

Other case studies available such as TMFlood Inundation modelling, River Durance – 2D flood wave Modellisation of St Jacques Levee in Cavaillon (France).

Box 8.33 Example of outputs from an evacuation model

Figure 8.158 Output from the life safety model showing people evacuating the flooded area and travelling towards the safe havens

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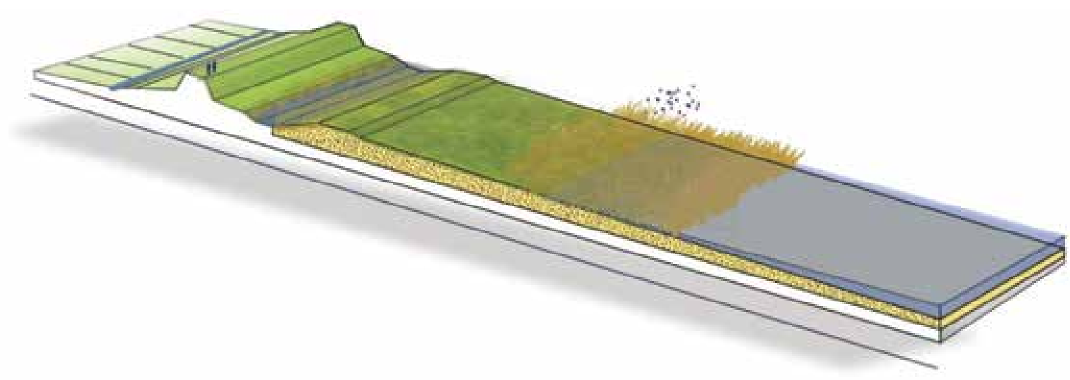
9 Design



Courtesy T GA Cents, ARCADIS and Hoogheemraadschap Hollands Noorderkwartier Vliet

CHAPTER 9 CONTENTS

9 DESIGN

Chapter 9 sets out procedures and good practice for design and detailing of levee interventions.

Key inputs from other chapters

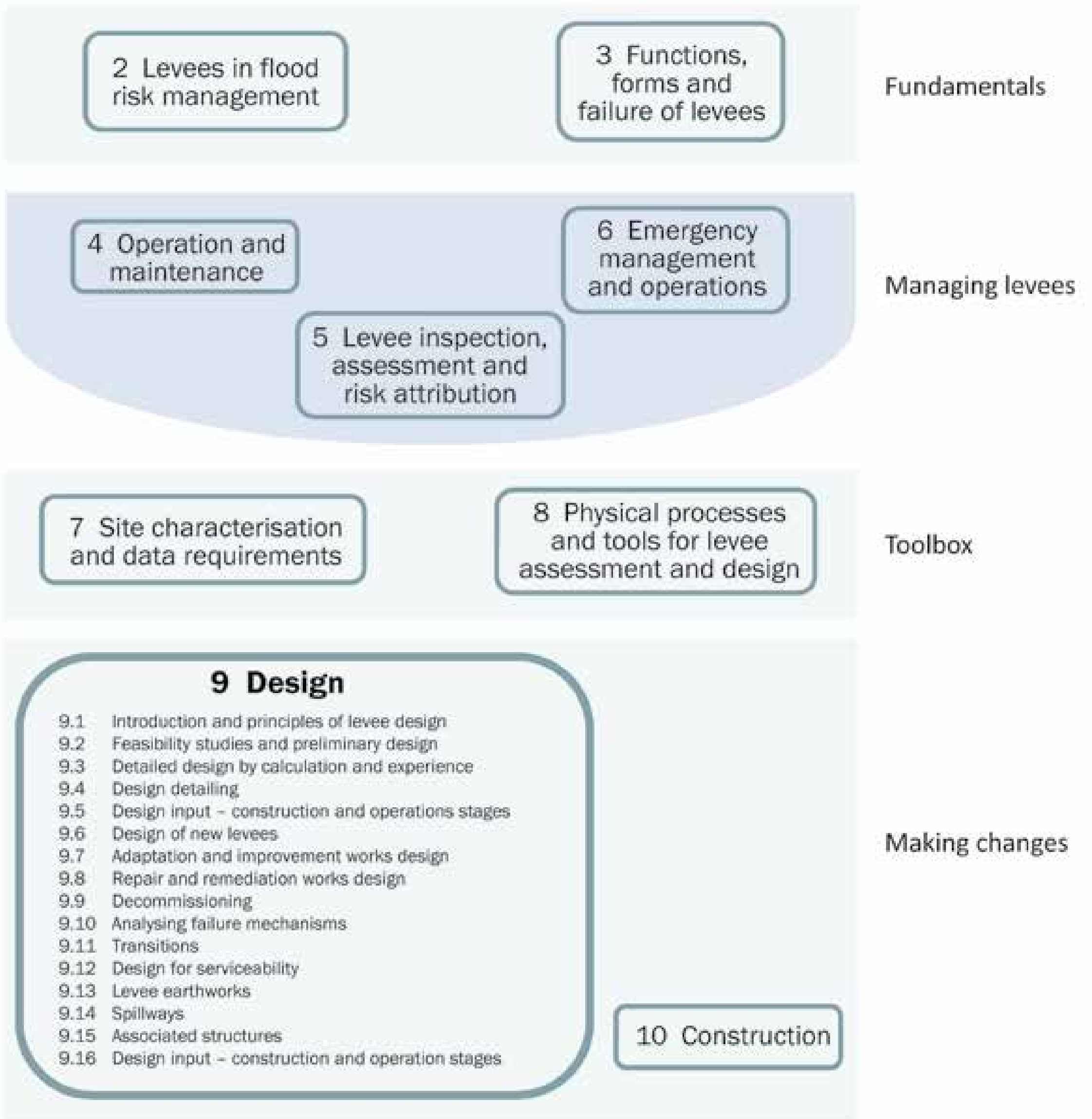
- Chapter 2 ⇨ **basic concepts for option selection**
- Chapter 3 ⇨ **forms, functions and failure mechanisms**
- Chapter 4 ⇨ **requirements from operations and maintenance**
- Chapter 5 ⇨ **levee performance assessments and flood risk analysis**
- Chapters 7 and 8 ⇨ **toolbox (data and models)**

Key outputs to other chapters

- design requirements ⇨ Chapter 10
- operations and maintenance ⇨ Chapters 4 and 6

Note: The reader should revisit **Chapters 2 and 3** throughout the levee life cycle for a reminder of important issues.

This flow chart shows where to find information in the chapter and how it relates to other chapters. Use it in combination with the contents page to navigate the handbook.



CHAPTER CONTENTS AND TARGET USERS

This chapter is divided into 16 sections which focus initially on giving a general introduction to levee design principles and processes, moving on to discuss the setting of levee alignment and geometry, dealing with designing against various modes of failure. The final parts of the chapter discuss more specific aspects relating to transitions, earthworks, spillways, associated structures and construction issues. The focus is on identification of key issues; design approaches are illustrated but are seen to vary considerably from one country to another. A number of ancillary matters (walls spillways, pipes etc) which touch on levee design are outlined but with the main focus being on the interaction with the levee earthworks.

Levee design principles

Section 9.1 sets out the overall principles and engineering considerations which should govern all levee design, and additional general considerations that apply to adaptation, improvement repair and decommissioning.

Levee design process

Section 9.2 explains the overall levee design process both from a sequential point of view, from concept to construction, and from the perspective of the technical process. The issues of roles and responsibilities and design verification are also discussed.

Reporting and documentation

Section 9.3 describes the various reports and documents that are relevant to the different stages of levee design.

Levee layout and alignment

Section 9.4 sets out the principles for setting the alignment of a levee, the width of the levee corridor and interactions with interior drainage issues.

Levee geometry

Section 9.5 explains how levee crest levels and cross-section should be established.

Surface protection measures

Section 9.6 discusses a range of measures that can be adopted to protect levees against external erosion, from grass systems for riverine levees through to much heavier forms of protection required in coastal applications.

Control of seepage and uplift

Section 9.7 describes how to assess and calculate seepage and uplift problems. It also provides key inputs into the following two sections on internal erosion and stability.

Control of internal erosion

Section 9.8 summarises the basic processes of internal erosion and identifies measures to deal with the problem.

Mass stability throughout levee life

Section 9.9 describes the identification and assessment of both landside and waterside instability that may arise during the life of a levee, including during construction, flooding and post construction.

Analysing failure mechanisms

Section 9.10 describes how the various failure mechanisms discussed in previous sections should be analysed, given the background of guidance documents and standards in Europe and the USA.

Design of transitions

Section 9.11 sets out the key issues that must be addressed when designing transitions, principally between levees and hard structures.

Design for serviceability

Section 9.12 explains how to design for the management of settlement, desiccation and animal burrowing.

Levee earthworks

Section 9.13 explains procedures for selecting, managing and controlling earthworks materials and their compaction, and discusses levee raising, the use of geofabrics and innovative ground improvement methods.

Spillways

Section 9.14 discusses the role and function of spillways for mitigating flood risk and levee damage and describes the various types that can be adopted.

Associated structures

Section 9.15 provides information on the design of crest and embedded walls in levees and pipe crossings, mainly from the perspective of their interaction with the levee earthworks.

Design input construction and operations

Section 9.16 explains the ways in which design input should continue into the construction and operation phases of a levee project.

9.1 PRINCIPLES OF LEVEE DESIGN

This section sets out the common design principles and considerations, including failure processes, which are applicable to all levee design, including the following:

- **new build –** a new levee built on previously unprotected land or a realignment of an existing levee
- **adaptation –** the modification of an existing levee to change the degree of risk reduction provided by the structure such as by raising the crest or improving the resistance of the levee against failure, or by incorporating a new structure such as a spillway
- **repair –** non-emergency reinstatement of a levee to its original condition or to an improved state but without increasing the level of protection intended
- **decommissioning –** the safe removal of a section of a levee, for example as part of a scheme of levee realignment.

After describing some overall guiding principles, the section moves on to describe the main engineering considerations and principles that apply to all levee works. Additional issues that relate to adaptation improvement remedial or repair works are then described. The section concludes by outlining the challenges of balancing conflicting requirements and whole-life planning, sustainability and eventual decommissioning of levees.

9.1.1 Overall guiding principles

Levees retain, channel or control water

As explained in Section 3.1, a levee's primary function is to retain water, channel it or control the passage of water over, through or under the levee under the design conditions. When a levee is retaining or controlling water, the designer must consider the potential pathways for that water and must consider both the direct consequences of the passage of water (flooding) as well as the structural issues (external erosion, internal erosion, instability). Resisting flow through the levee can be delivered in various ways, including surface sealing and cut-off walls; impermeability of the core material of the levee is not necessarily required.

Levees are part of a flood defence system

Levees are constructed to improve public safety and/or limit property damage during flood events, but, as explained in Chapter 2 and Section 3.1, a particular levee segment may be only one of many features within a flood defence system. A typical flood defence system will also include other levee segments and various man-made and natural structures, all working together to help defend the leveed area from inundation. The structural performance of the whole flood defence system is only as strong as its weakest part. The alignment (Section 9.4) and cross-section (Section 9.5) of the levee should be designed accordingly.

Levees should have resilience

The resilience of a levee can be seen as its ability to retain and recover functional performance under the stress of known and unknown adverse events (Schultz *et al*, 2012).

In this regard, a levee should have sufficient *capacity* to accommodate the loading situations for which it is being designed, without damage and breach due to potential failure mechanisms (Sections 9.1.3 and 9.6 to 9.9) Uncertainty in the estimation of loadings and capacity, means that ideally there should be some *redundancy* to accommodate overload conditions without overtopping or breaching.

It should also have sufficient *robustness* to accommodate situations in which the levee is overtopped, ideally for a longer period than the likely extreme flood. Ideally, levee failure due to external or internal erosion or instability (Section 3.5) should not occur if the levee overflows or is subject to wave

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overtopping. The levee should normally be designed so that it does not fail for an extended period of time that reflects the time required to evacuate or provide safe shelter for those within the leveed area, especially for more vulnerable persons. For lower-risk scenarios, the requirement for robustness may not be as important. In addition to surface protection measures (Section 9.6) in some situations this may require the introduction of spillway sections (Section 9.14) or of berms into the cross-section (Section 9.5) providing additional material. All of these factors are affected by the way in which the earthworks (Section 9.13) of the levee and any associated crest or other structures (Section 9.15) are designed and constructed. Furthermore care is needed to avoid weaknesses in the levee at the locations of any structural transitions (Section 9.11). Improved robustness to failure during overtopping or overflow can also be introduced by design, for example by introduction of lowered and strengthened spillway sections (Section 9.14).

In the event of any breach (including at fuse plug locations) the levee should be readily *repairable*.

The levee should also be designed to allow the required level of erosion resistance, impermeability and stability to be *maintained* over its design life, taking two types of *deterioration* into account:

- over a period of time, unrelated to overtopping (eg due to settlement of the levee crest, desiccation, vegetation growth on or near the levee, animal burrowing, internal erosion or seepage) and the appropriate *serviceability* design (Section 9.12)
- related to multiple or prolonged overflow and/or overtopping events.

Tiered approach or level of detail

The level of detail in the design process may vary, depending on the size and nature of the envisaged works and the flood risk associated with the levee. For example, for a simple repair, the preliminary design may consist of a few sketches (of a couple of options) and a few notes, put together by an experienced individual following a site visit. In contrast, for a large scheme involving, among other options, a new levee system through the centre of a town, it would be usual for the design process to be much more extensive. Such a situation may require the consideration of a range of options, flood risk assessments, environmental impact assessments and detailed drawings and specifications supported by potentially complex calculations.

Selecting and optimising solutions

Solutions must be technically and economically feasible, stable and constructible, and the selected option must be capable of being optimised to provide a balance between costs, risks and benefits. Furthermore, the level of investigation and study that may be carried out should be proportional to the degree of risk associated with the levee. Further details are provided in Sections 7.1 and 9.2.

9.1.2 Main engineering considerations

Despite their apparent simplicity, levees can be surprisingly complex structures. Given their function and location, they are often built on soft alluvial soils which can exhibit low strengths (potentially leading to instability), high compressibility (causing settlement) and/or high permeability (seepage). Furthermore, they are prone to variability, imperfections and deterioration with time, not least because they are constructed with natural materials which are largely sourced from the immediate locality. Table 9.1 gives a summary of the main considerations that help to address the complexities that arise during the design process.

Table 9.1 Main engineering considerations in levee design

Main design consideration	Section	Summary of main design considerations
Levee alignment	9.4	Levee alignment is the first aspect of design to be considered and resolved, as it will control both the characteristics of the environment, including the hydraulic conditions, and the ground conditions.
Levee crest level	9.5	The crest level should be set to provide the relevant level of flood risk reduction throughout the design life, recognising that the levee may be overtopped in more extreme flood events and may even breach (unless strengthened for this eventuality). For this reason, there may also be a need for spillway sections with slightly lower crest elevations, for the event where the levee is overtopped.
Levee cross-section	9.5	The selection of the levee cross-section – in particular the crest width, gradient of the side slopes and the location and width of any berms and any landside drains – will be affected by a number of factors. These include: • mass stability, given the height of the levee and the ground conditions • resistance to internal erosion • requirements for operation and maintenance of the levee (eg grass cutting) • any requirements for dissipation of wave energy. In addition, consideration should be given to incorporation of low points (such as a reinforced spillway) or a fuse plug (a weak point which would collapse or be breached more easily than the surrounding levees).
Ground conditions	7.1, 7.7, 9.12	Levees are often built on relatively flat, alluvial or estuarine plains in which the natural soils, such as peat, are soft. These soils limit the height to which the levees can be constructed safely at one time and can undergo significant settlement (eg of the order of 1 m) over the design life of the levee. This, in turn, may result in the need to raise levee crests in the future. Thought should be given to building levees with a wider crest than initially required, in order to accommodate subsequent raising of the crest after settlement. The presence of sands can cause seepage through the foundation soils, if there is hydraulic connectivity between the waterside of the levee and the floodplain. Design must take account of the variability of ground conditions along the alignment of the levee and not assume the same conditions apply to all levee segments.
Materials	9.13	Levees are often constructed out of locally available soil, regardless of its quality. However, levees constructed out of clays will be prone to cracking, and those constructed out of sands may be too permeable. If the primary construction material cannot deliver all functions (stability, impermeability, erosion protection, filtration, drainage – see Section 3.2), then additional components using alternative materials may need to be added. The choice of construction material may involve a compromise between suitability, proximity, cost, environmental impact and sustainability.
Deterioration and serviceability	9.12	Levees can be badly affected by problems such as animal burrowing, seasonal desiccation and the unmanaged growth of vegetation. These processes of deterioration will create local weaknesses which will lead to an increased risk of erosion, seepage, failure from overtopping or instability during an extreme event.
Transitions and other points of weakness	9.11	Local imperfections or weaknesses can also be caused by poor design detailing or poor construction, which then attract further deterioration through processes such as scour and internal erosion during a flood event. Good levee design must anticipate and avoid these faults, because such points of weakness can compromise the integrity of the entire flood defence system. For situations where a single organisation does not have responsibility for the entire levee system, the various operating authorities need to understand this risk and agree the levels of effective operation and maintenance.
Designing for human-induced impacts	7.6	• vandalism • accidental impacts • encroachments • terrorism.

Table 9.1 Main engineering considerations in levee design (contd)

Reliability of existing levees	Chapter 5	Many levees are decades or even centuries old, but may not yet have been subjected to the extreme events for which they were originally designed, or which it is hoped they will resist. When they do experience these extreme loadings, they do not always perform as anticipated. Adequate or untested performance in the past does not guarantee acceptable behaviour in the future.
Levee construction	9.15 and Chapter 10	It is important that the designer considers how the levee will be constructed, adapted or repaired. Among other things, this may require early constructor involvement to consider how the works will physically be constructed. It may also include a consideration of issues such as stability during construction (eg there may be a need for staged construction to avoid construction-related failures).

9.1.3 Principles related to modes of failure

Evaluating the likely failure modes of a levee under various potential design loading scenarios is a critical part of the design process. This section sets out some of the possibilities. However, it is worth devoting some effort to visualising realistic extreme events which can pose a much more threatening situation than may be indicated by neat straight lines on an engineering drawing (Figure 9.1).

Figure 9.1 Localised overtopping (courtesy Defra)

External erosion

An essential and major component of levee design is the consideration of the vulnerability to external erosion of exposed fill materials within the levee, or soils in the natural ground to the landside or waterside of the levee. Account should also be taken of *morphological change* (discussed in Sections 7.2 and 7.3). For the design of coastal levees, the potential impact of coastal sediment movements should be considered, as this will control the ground level on the seaward side of the levee and thereby modify the wave conditions. For fluvial levees, the geomorphological impacts both upstream and downstream of a new scheme of levees should be considered.

Once the erosive events have been characterised, assessments must be made for resistance to wave attack and overtopping for coastal levees and/or resistance to channel scour and overflow for fluvial levees. Guidance on the selection of appropriate surface protection measures is given in Section 9.6 and these can be sized using the calculation techniques set out in Section 8.4. In designing any surface protection system, it is important to recognise that resistance to external erosion can be compromised by local weaknesses. For example, local surface unevenness of a layer of pre-cast concrete blocks caused by local settlement or poor construction can cause localised turbulence which triggers the loss of the protection in that area and may result in rapid erosion, and eventually in levee breach.

Seepage

The levee should provide a barrier of controlled permeability between the floodwater and the leveed areas. Any water able to pass beneath or through the levee during the design flood situation should not be large enough to cause:

- flooding
- creation of a hazard
- *internal erosion* to reach a critical state
- reduction in the mass stability of the levee
- significant deterioration in the ability of the levee to perform adequately during future flood events.

Practical ways of controlling seepage are described in Section 9.7 and are supported by tools for seepage analysis given in Section 8.3.

Internal erosion

Assessment of the potential for internal erosion may be made using the tools in Section 8.5. This includes assessing hydraulic gradients and seepage velocities. These characteristics can then be used to assess the potential for fill materials within the levee, or the natural soils beneath the levee, to form sand boils. This process is called suffusion (the gradual loss of finer material from a given soil or fill material), which can lead to the formation of voids and eventual collapse of soils and fills. This is why the arrangement of the various materials within the levee section is important. Soils immediately adjacent to structures built into levees (eg bedding materials) can become a focus for internal flows and hence suffusion. Internal erosion can be controlled by ensuring that hydraulic gradients are not too high, controlling seepage (Section 9.7) and including appropriate filters. Filter design is described in Section 9.8.2 while tools for seepage analysis and filter analysis are given in Section 8.5.

Mass instability

The levee should not become unstable allowing levee slopes to collapse under the design conditions. This includes during the construction stage, during or after extreme flood, ice or seismic events. Tools for stability analysis are given in Section 8.6. A full discussion of the approach to stability analysis is given in Sections 9.9 and 9.10.

Settlement

Excessive settlement of the levee structure after construction, adaptation or repair can create the potential for excessive overtopping and consequential damage. Designs should check for this possibility and also for the potential for differential settlement, as this can cause cracking or other problems that could compromise performance. Assessment of settlement can be made using the tools in Section 8.7.

- the issue of settlement is likely to be of critical importance for *new levees* that are to be constructed on an alluvial floodplain, particularly if the levee is underlain by soft clays or peats. In such situations it is not uncommon for levee settlement in excess of 1 metre (3 feet) to be observed over the lifetime of large levees. Furthermore, differential settlement may cause additional deterioration of the levee through cracking etc. Part of the design process is therefore to anticipate and calculate the magnitude of potential settlement and to accommodate it within the final plans (eg constructing the levee to a high enough level to accommodate the settlement, or designing the levee crest so that it is sufficiently wide to enable raising it just by the placement of additional fill materials)
- for levees undergoing *adaptation or raising*, differential settlement between new works and existing structures is likely to be the important issue. Existing structures will typically have already undergone most of their settlement, but new adjacent structures could undergo significant deformation, and so cracking could result. Details should be designed to accommodate these potential changes

- repairs to damage caused by historical settlement are commonly carried out. An assessment of the cause of the settlement may help the designer to identify a suitable method for controlling the potential for settlement in the future.

Burrowing animals

Assessments should be made as to whether the fill materials used in the levee are vulnerable to penetration and removal by burrowing animals and whether details can be incorporated into the design to deter such animals. Fuller discussion of this subject may be found in Chapter 4 and in Section 9.12.3.

Seismic loading

Although the probability that a major seismic event will coincide with a serious flood event is extremely low, seismic events can cause extensive damage to levees and leave them vulnerable if they are not repaired to an adequate condition before the next major flood event. Levees should therefore be assessed for their resistance to earthquakes in combination with a relatively frequent flood event (eg a five-year return period).

Both mass instability and liquefaction should be considered when assessing seismic effects, since liquefaction of loose granular soils beneath or adjacent to a levee may cause uncontrolled settlement or lateral displacement. Seismic design calculations can usually be carried out using the same analytical software as is used for static design analyses. This can be done either through pseudo-static calculations or by time-domain analyses. The choice of approach will depend on the relevant seismic codes of practice and the capabilities of individual software packages. Guidance on approaches that can be adopted is given in Section 9.9.4, with tools for assessment of earthquake-induced instability and liquefaction presented in Section 8.8.

9.1.4 Adaptation and improvement works

Many more levees are adapted or improved than new levees constructed, typically involving measures to raise the levee crest or to increase resilience. Specific issues that may constrain design solutions (in addition to the general principles discussed in the previous sections) include:

- accommodating the water and other environmental actions (physical processes) throughout construction – this may involve temporary works and modifications to the hydraulic operating regime
- uncertainty as to the condition of existing levees – materials, consistency or underlying ground conditions will be variable and there will always be a risk of unforeseen or unforeseeable conditions, but a programme of geotechnical investigations (Chapter 7) can help mitigate these risks
- differential settlement or differential bearing capacity – historical consolidation and strength gain of soft alluvial soils under existing levees means that new structures or levees built adjacent to them may experience greater future settlement than existing structures
- restrictions on construction activity as a result of adjacent structures such as houses or commercial buildings
- increased health, safety and welfare risks for those involved in construction and for the public.

Design options for crest raising that can accommodate the above issues include raised and widened earthworks. However, in many cases space constraints mean that alternatives must be considered such as:

- the use of lightweight or recycled materials or improving the bearing capacity of the ground to allow the height of existing levees to be increased with smaller increases in width
- the use of a crest structure such as a gravity wall or an embedded retaining wall, where there is insufficient space for full levee widening
- the use of temporary or demountable crest structures.

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Design options that can provide increased resilience to overtopping include:

- improved surface protection
- techniques to reduce the flow velocity down the landward face.

Crest raising and resilience to overtopping can interact with one another. For example, when a vertical crest wall is overtopped, it can increase the amount of turbulence and erosion on the landward side of the wall, which can lead to premature failure. This phenomenon was observed along many of the I-walls on the New Orleans levees during Hurricane Katrina in 2005 (Figure 9.33).

9.1.5 Remedial or repair works

Levee repair (Chapter 4) is generally carried out in response to a problem rather than as a result of strategic planning. Typical repair works (Sections 9.11, 9.12, 9.15) might include:

- reinstating levee slopes (Section 9.5) taking account of any mass instability (Section 9.9)
- repairing damage from surface erosion and scour (Section 9.6)
- repairing damage from animal burrowing(Section 9.12.3)
- managing existing woody vegetation (Section 4.5).

In the case of external erosion, for example, repair work might range from remediating slowly propagating scour around a pipe or a culvert, to urgent and extensive work to restore part of a levee washed away during a flood. The selected form of repair and the speed and programme of construction activity will influence the robustness of the solution.

When considering design solutions for the repair of existing levees, the following issues (in addition to those raised in Section 9.1.4) need to be taken into account:

- the need for good design thinking, despite the time constraints which often prevail; wherever possible the philosophy should be to 'build back better', rather than leaving or introducing points of weakness into the levee which may lead to subsequent damage or breach
- the difficulties of access to the area(s) requiring repair, for example due to local obstructions, structures or buildings, or because access along the river or coast is restricted
- the risk of levee failure during the repair work
- early involvement of contractor(s) (Sections 9.2.5 and 10.1.1) in order to reduce the risks to the health and safety of construction personnel, and to the project budget and programme.

The only distinction between large/urgent repairs and emergency remediation works (discussed in Chapter 6) may relate to the time that is available to carry out the repair (eg before the next potential flood). For this reason, temporary repair works should always be checked after the emergency event is past to ensure their long-term adequacy. Additional investigations and studies may be required.

9.1.6 Balancing conflicting requirements

Selecting the best alternative and juggling constraints can make the design process very difficult. A balance must be found between conflicting engineering, environmental, social and economic factors at the earliest possible stage in the development of a levee. Examples of conflicts to be resolved might include the following:

- raising riverine levees in an area to protect the people or physical assets at risk may well improve the degree of flood risk reduction in that area. However, this may come at the cost of increasing flood risk to residents or landowners both upstream and downstream of the improved area. These consequences and impacts should be assessed and considered
- the role of woody vegetation (such as trees and shrubs) on levees is a complex subject (Section 4.4).

Well-vegetated levees may offer a habitat for animals or plants, but the growth of large trees or shrubs may have an adverse effect on the ability of a levee to resist high water levels or overtopping events. From a flood defence point of view, woody vegetation should ideally be removed, but this may be in direct conflict with the need to provide habitat for rare species

- economic development groups may find it attractive to locate a new enterprise in an area that could encroach on potential side slope extension areas, ponding areas etc
- providing aesthetic enhancements to an existing levee may require the addition of walking paths or ramps up the sides of the levee and could introduce new failure points and maintenance concerns
- the requirements of existing utilities may have to be met and there may be significant costs of utility diversions associated with levee construction
- appropriate storage or pumping systems have to be provided to allow interior drainage systems to continue to operate during flood events when gravity drainage through the levee is unavailable
- legal and other environmental requirements (eg of the Water Framework Directive in Europe) for the catchment or coastal cell have to be met.

Balancing conflicting objectives is always a challenge, particularly where some of these may relate to legal requirements. However, if the primary functions of a levee are to retain or channel water or to control its release (Section 3.1), then this main objective must be strictly maintained, as well as meeting other objectives, such as environmental or leisure enhancements, accommodation of individual buildings and third party access.

The process of information exchange and deliberation to balance these conflicting requirements during the design process is susceptible to misinformation and/or loss of information. Efforts should be made to ensure appropriate *communication* between disciplines, across agencies and with the public. Effective engagement with local authorities, residents, businesses and interest groups throughout the design process to seek local views and local support are important in achieving a successful outcome. As part of this process, the flood risk remaining after construction should be carefully communicated (Section 5.2.9). A residual risk mitigation plan should be developed during design.

An example of balancing conflicting requirements is given in Box 9.1.

Box 9.1 ***Balancing conflicting requirements when strengthening a weak link, Noordwijk, the Netherlands***

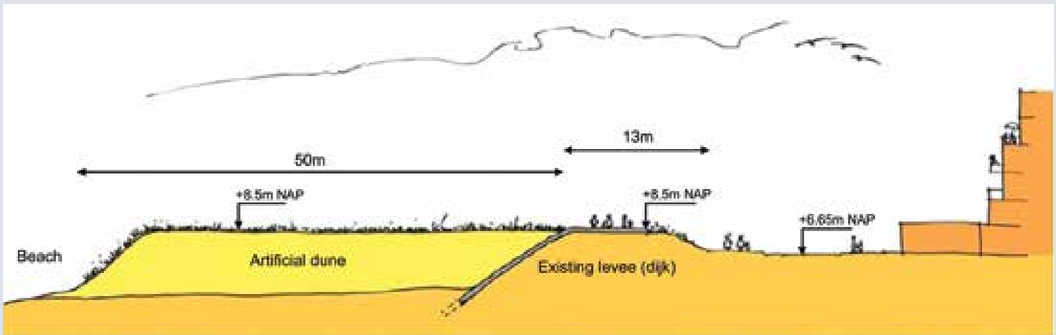


Figure 9.2 ***'Levee in dune', constrution to facilitate spatial development of the boulevard in Noordwijk, the Netherlands (courtesy Marco Veendorp, Arcadis)***

Within the Dutch national programme, *Weak Links*, the sea coast of the tourist town of Noordwijk was strengthened by a so-called 'hybrid' construction ('soft' sandy solution combined with the 'hard' levee structure) shown in Figure 9.2 (Lange Voorhout, 2009) . The Weak Links programme aims to reduce the risk of flooding over the next 50 years, associated with coastal structures, and at the same time improve the human and natural environment and enhance the economic potential of the location.

The selected technical approach for the design and construction of the strengthened levee at Noordwijk was constrained by a number of factors and functional requirements. The complexity of the issues and involvement of several stakeholders meant that stakeholder management and communication had to be an integral part of the project.

At Noordwijk, the pre-existing coastal protection comprised a narrow and relatively low dune row. Just behind the dunes the Wilhelmina Boulevard, a popular promenade in Noordwijk, was situated. This is a one lane road, flanked by two pavements, which is the only access to a dozen luxury hotels. In the summer season, various temporary pavilions are located on the beach, including a bar, restaurant, sailing club and police post.

Box 9.1 Balancing conflicting requirements when strengthening a weak link, Noordwijk, the Netherlands (contd)

The levee structure chosen for the strengthening works consisted of a sandy core with so-called Basalton (basalt concrete) block armour on a filter construction with a geotextile base and a granular filter layer. This characteristic permeable construction was chosen to accommodate the dynamic loading of the structure by waves. To preserve the view from the boulevard the increase of levee height was restricted to 1.5 m. The levee was covered with approximately 0.5 m sand on top and in front of the outer slope, creating an artificial dune (sandy buffer) of approximately 50 m. This provides sufficient safety against flood, and preserves the natural dune-like look of the levee. Complementary beach and foreshore sand replenishment was carried out to maintain the pre-existing beach width.

The (seemingly incompatible) constraints on the construction activities included the following:

- construction activity was not wanted during the summer season (April to September) because of the touristic importance of the coast and beach to the town
- the original dune area is a very valuable flora and fauna location with several protected species of thistles and sand lizards. As a result, excavation works were prohibited between October and April and between May and July (hibernation and hatching seasons respectively)
- there is a general Dutch law prohibiting works on the flood protection structures in the storm season (October to March).

The compromise decision was to carry out the construction work from the end of August to March of the following year, subject to the following conditions:

- before any excavation work, all sand lizards from the area were captured and moved to another suitable habitat
- during excavation and profiling of the outer slope of the levee in the dune, the excavated material was used to construct a protection dune (designed with significant protection level of 1/100). This dune was finished before the storm season, with additional replenishment of sand from the sea
- the Royal Dutch Rescue Brigade required access to the sea for their rescue boat at all times. In the northern part, a temporary access road was created over the protection dune to allow the rescue boat to access the seafront at any time
- road transport of construction materials and resources via the 'boulevard' road was strictly forbidden. Transport of levee material by ship was not possible because of the relatively shallow water with tidal movements and construction of a temporary berth at reasonable cost was not possible. The only permitted access to the construction area was by road from the northern end of the 'boulevard'
- along the coast were remains of the so-called Atlantic Wall, a German defence line constructed in 1943, comprising a chain of concrete bunkers and other structures in the dunes. Because of very good (original build) documentation and verification of the position by proof trenches of these structures, it was possible to adjust the position of the 'transition' parts of the levee to avoid any interference with these Second World War artefacts
- short concrete sheetpiles (resistant to saline environment and visible in Figure 9.2) were chosen for the construction of the scour resistant toe of the levee so that the filter and armour layer could be placed in the dry. Only during high tide was a limited drainage system required to permit the works on the toe to continue
- the original dune and the subsoil mostly consisted of fine and coarse sand with only occasional lenses of (medium soft) clay. As a result, the anticipated settlement of the new (higher and heavier) structure was negligible and there were no negative implications (deformations) for the armour layer
- the anticipated rapid erosion of the sand in the new dune was prevented by planting sea oat directly after the completion of the sand layer deposition.

Information on the quality procedures for this project are described in Box 10.3

9.1.7 Whole-life planning and sustainability

Consideration should always be given at some level to 'whole-life' planning and sustainability (including future decommissioning – see Section 9.1.8). Specific issues of change that might be taken into account for earthen levees include, but are not limited to:

- the need to modify the defence system in response to changes such as:
    - settlement and subsidence (extraction of water from underlying aquifers or minerals from substrata may cause subsidence of the levee foundation; some areas are even prone to subsidence without extraction activities)
    - increases in hydraulic loading due to climate change (eg increases in sea level, rainfall and river hydrographs, storm surge levels)
    - changes in river or coastal morphology (which may cause changes in flow velocities), changes in river bed or seabed levels or increases in loading on the levee, or increased scour potential (which may lead to destabilisation)
- the impact of ongoing levee deterioration (as offset by any planned maintenance activities), for example, a period of drought may reduce the resilience of a levee (desiccation cracking, loss of mass of levee body or berm), so the risk of this should be taken into account in selecting the types of materials used for levee construction

- the introduction of embedments or encroachments into the levee (Section 4.4) or changes already there
- changes in the use of the levee (eg a greater leisure use)
- changes in land use and associated risk to life and property.

Where there remains doubt, or where the initial cost of designing the levee to be resilient to every realistic form of future change or risk is unaffordable, a scheme of 'no-regret' measures that allows adaptation in the future should be adopted.

9.1.8 Decommissioning levees

Historically, most levees have been constructed as permanent structures, regardless of their size or the quality of their design and construction. There is a growing realisation that this is not necessarily the case. Changes in policy mean that decisions not to defend the coastline for example, or to make more space for the water in river valleys can and will be made. The possibility and process of levee decommissioning should ideally be considered at the time of the original design so that:

- risks associated with decommissioning can be identified
- suitable modifications can be incorporated into the design
- works can be carried out in a safe and effective manner.

For any levee to be decommissioned in a safe manner, a decommissioning design will be required and may include the following:

- insertion of a breach or of gravity flow pipes into the existing levee (perhaps as part of a scheme using set-back defences). Such a design will require knowledge of the existing levee and the active selection of the location of the breaches or pipes on the basis of the flood modelling and a consideration of safety issues
- temporary repair or raising of the existing levee to reduce the flood risk in the short to medium term so as to allow permanent set-back defences to be constructed farther inland
- decommissioning of associated mechanical and electrical equipment (pumping equipment etc)
- consideration of the impact of ceasing maintenance of the levee and allowing it to deteriorate naturally (the challenges associated with predicting failure scenarios for a non-maintained levee system should not be underestimated).

9.2 THE LEVEE DESIGN PROCESS

Figure 9.3 outlines the design *process*, commencing with a decision that change is required. It is presumed that other options have been considered and rejected before the decision to construct or adapt a levee was taken. The figure shows that it is the designer's responsibility to understand the levee owner's intentions and to turn them into performance-related goals for retaining, channelling or controlling the release of water, and for design life. Such goals could include:

- reduction of flood risk for the majority of affected parties and/or up to a fixed water level
- integration with local/regional development plans
- managing changes to the leveed area, including provision of secondary benefits for recreation and environment.

These goals are subject to physical, material, social, environmental, cultural and economic constraints and require the achievement of a balance between risk and cost. The levee owner is likely to require technical input from designers in setting these goals and in turning them into construction drawings, specifications and design reports. In achieving the end goal, the levee owner should endeavour to be aware of the nature of the design process and its iterative cycles of optimisation. In this regard, an understanding of the principles underlying good practice in levee design, as set out in Section 9.1, will serve the levee owner well in delivering or maintaining flood management systems based on levees. Having said this, designers should also prepare their designs in a manner that is consistent with their relevant national codes of practice.

Figure 9.3 demonstrates that it is also the designer's responsibility to understand the physical environment in which the levees are (or will be) located, to be aware of the constraints and to be able to predict how the materials of which the levees are (or will be) built, will perform during design events.

Within the context of this handbook, the start of the design process can be traced to procedures described in other chapters. The following are of particular concern.

- at a strategic level (Chapter 2), the introduction of a levee system may have been identified in a strategic options analysis as one of the measures and instruments required to reduce flood risk.
- a levee assessment (as described in Chapter 5) might identify that a section of existing levee requires repair or adaptation.
- following an emergency or severe flooding event (Chapter 6), a deficiency in an existing levee may be identified. In this case, it is possible that both immediate emergency repairs and a longer-term solution are required.

Fuller details of the design process are given in Section 9.2.

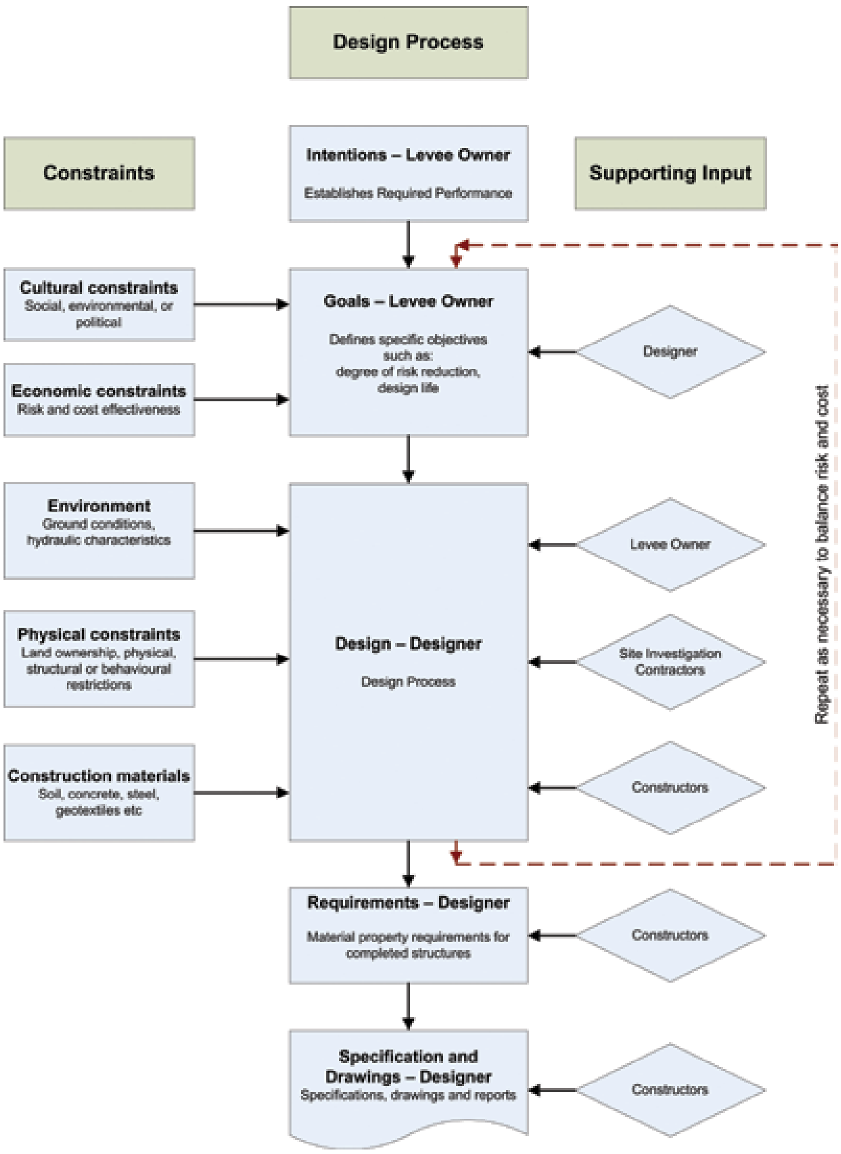


Figure 9.3 The design process for levees

9.2.1 Phasing of design

Regardless of the extent or nature of the works, the typical design process is generally the same, but varies in its effort according to the scale and risk associated with the levee. Three broad phases can be identified (Figure 9.4).

1 **Identifying the need (reconnaissance):** the site and its environmental setting are characterised, and the hydraulic conditions are established. For existing levees, this stage may include inspections, desk studies and/or initial investigations and diagnoses or assessments of an existing levee or levee system (see Chapter 5 for details).

2 **Conceptual design stage (outline or feasibility design):** options for construction or adaptation are evaluated on the basis of simple calculations, judgement and experience. Sufficient topographic, environmental, hydraulic, geomorphological, geological and geotechnical site data should be gathered at this stage to allow meaningful cost/benefit comparisons of the options to be made. Where required, consent will be sought for a preferred option from the relevant statutory authorities and this will usually be supported by an environmental impact assessment (EIA). Issues commonly considered include:

   a For new levees:

      i a range of potential levee alignments

      ii the use of discrete elements such as spillways, channel diversions or temporary flood storage at strategic locations

      iii a range of different levee cross-sections (channel depths, levee geometry, levee crest structures etc)

   b For existing levees:

      i local levee raising or repair, if required to bring a short section of levee up to the same standard as the rest of the levee system

      ii general levee raising or strengthening, if required to deal with a perceived increased risk of flooding (either as a result of increased likelihood or greater consequences)

      iii repairs, if required following damage or breach which occurred during a flood or other damaging event.

3 **Detailed design stage, including:**

   a finalisation of the design criteria, further site assessment and modelling to establish hydraulic conditions for design, and the gathering of sufficient site data including ground investigation works to establish a conceptual site model and geotechnical parameters (Sections 7.1, 7.7, 7.8) for the full detailed design of the levee

   b use of judgement, calculation and engineering experience to finalise the design using knowledge embodied in engineering codes and guides

   c production of drawings and specifications to allow the works to be constructed.

These initial three phases should run in parallel with the equivalent phases of site characterisation and investigation outlined in the next section (9.2.2) and discussed in much more detail in Section 7.1. Two further phases then follow:

4 **Construction stage:** the design is required to ensure that construction is carried out in compliance with specifications. Design input may be required, for example, to react to alternative proposals from constructors, unforeseen ground conditions or to analyse monitoring data obtained during construction.

5 **Operations and maintenance (design input):** to be successful, this requires that the essence of the design concept must be captured for the levee owner/operator. This is frequently done within an operations and maintenance (O&M) manual – a ‘living’ document, which is first created by the scheme designers, updated after completion of the construction works and then passed on to the levee owner. Interaction with the designer is also desirable during O&M, to assist in predicting areas of poor levee performance and in improving subsequent designs (see Chapter 5 for further details).

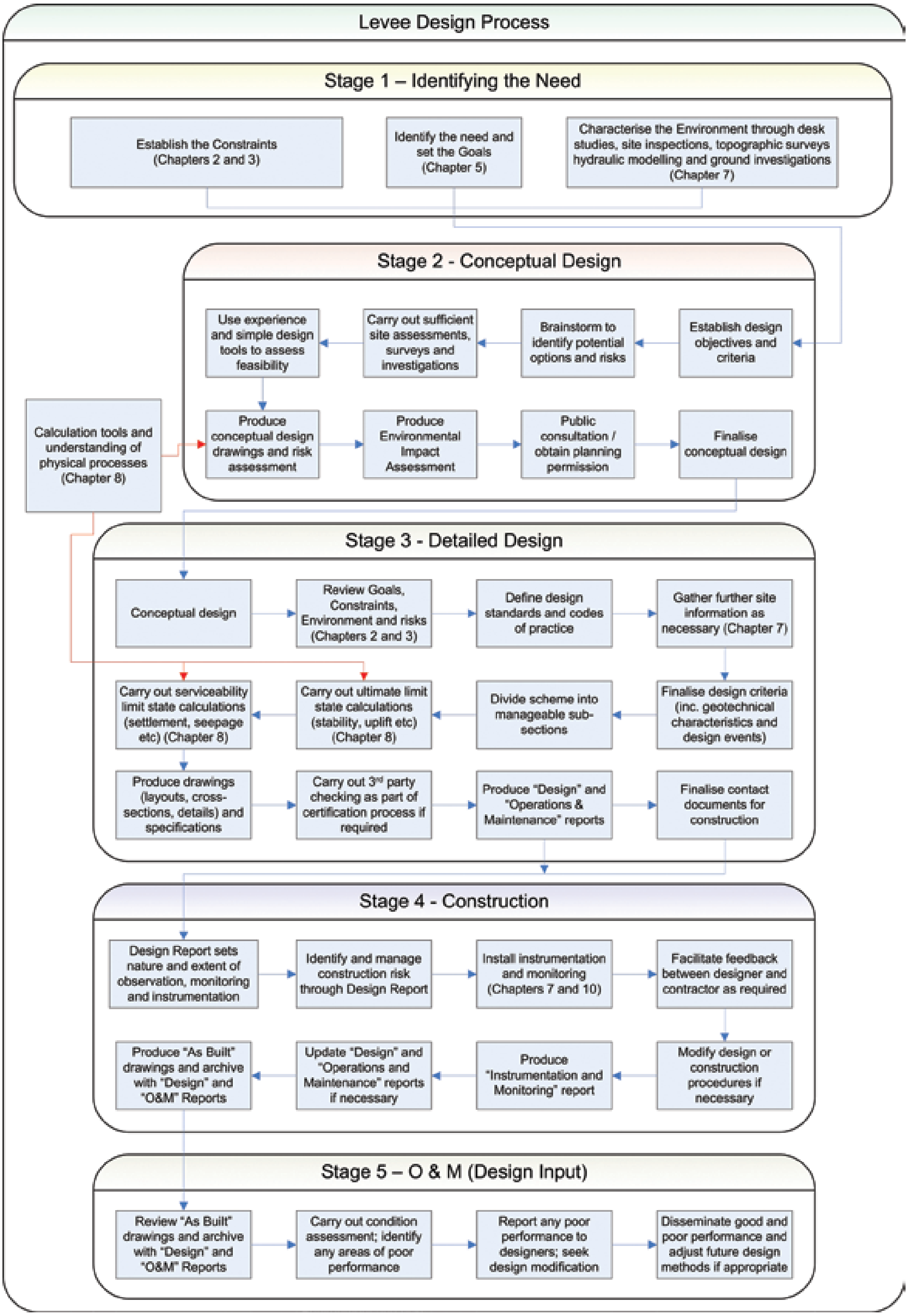


Figure 9.4 The levee design stages

9.2.2 Phasing of site characterisation for design

The primary focus in the early stages of a design is the characterisation of hydraulic and morphological site conditions. Various options are then evaluated to estimate levels of protection and risks, along with the cost and benefits for each option. The site characterisation involves, for example:

- for riverine locations, the catchment characteristics, stream and floodplain topography and morphology
- for coastal locations, shoreline topography and morphology, tidal, current and wave climates
- for estuarine locations, an appropriate combination of the riverine and coastal characteristics.

Geotechnical input is relatively limited in the earlier design phases and may only need to be of sufficient detail to screen the options. However, as the levee design moves towards a recommended option, the amount of hydraulic and hydrologic evaluation is reduced and efforts towards defining geotechnical, structural and cost engineering parameters increase.

A careful and logical thought-process for data collection for each of the three main design stages of levee development discussed in the previous section can be identified. An indication of the possible steps in the hydraulic, morphological and ground characterisation processes relative to the levee design stages are summarised in Table 9.2. If adopted, these should assist in a more efficient characterisation and should help prevent any possible oversights. The details of the main activities in Table 9.2 are discussed in more detail in Section 7.1.

Table 9.2 *Logical process of data collection for each levee design stage*

Levee design stage	Hydraulic data	Morphological data	Geotechnical data
Need identification/reconnaissance	• assemble available data • identify gaps in data • collect new data if required to fill data gaps • minimum required to inform assessment • calculate basic statistics from available stream gauge data (discharge probabilities: max/min/ave and durations) • assess water levels, waves and currents using historic data or rudimentary modelling.	• assemble available data • assess stream type in the case of rivers • identify near shore and beach characteristics for coastal zones • identify type of estuary and describe depositional tendencies between coastal and riverine settings • rudimentary assessment of historic channel and/or shore/ beach behaviour • identify controls such as bedrock outcrops and structures • description of bed and bank sediments or beach sediments • estimates of long-term system response • locations of sediment sources and sinks.	• assemble available data • assess geology, groundwater and geotechnical properties • determine impacts of ground conditions on levee performance for option screening • identify gaps in data • identify potential source of borrow material • assess potential for special issues related to contamination, archaeology and ordnance.

Table 9.2 Logical process of data collection for each levee design stage (contd)

Feasibility/outline design	• collect new and updated data, and detailed information • assess water levels, waves and currents using refined numerical models for higher resolution and accuracy • check impact of presence of existing or proposed levees on hydraulic conditions • develop uncertainty relationships for discharge-probability, water level-discharge functions.	• collect new and additional detailed information • prepare sediment budget for system • assess channel migration potential and beach lowering, and estimate rates of change • check influence of channel adjustment and beach erosion on levee alignments • develop sediment transport models to assess long-term system response • develop uncertainty relationships for channel, shore or beach erosion/deposition • identify locations where bank and/or beach stabilisation features are required to insure levee integrity • estimate local scour effects • detailed evaluation of levee interaction with morphological processes.	• site survey • collect some new data • conduct non-intrusive investigations (geophysics) and limited intrusive investigations • consider including data requirements for other elements needing non-intrusive/intrusive investigations, contamination, archaeology and unexploded ordnance • level of information may be the minimum required to inform assessment of options • evaluate potential sources of borrow, if known • rudimentary quantitative evaluation of credible levee failure and deterioration modes to outline a credible levee profile.
Detailed design	• update data if necessary • assess local currents and their interaction with levee • assess wave actions on levee and potential overtopping • if relevant to design of levee system, use physical scale models to further refine local assessment of hydraulic conditions (and the response of the levee to those conditions).	• update data, if necessary, for localised areas • develop models to identify system response at specific erosion control features • develop models to assess landside effects from spillway discharges for rivers.	• collect new detailed data in sufficient detail to allow the design concluded • conduct comprehensive intrusive investigations, possibly complemented by non-intrusive investigations • consider including data requirements for other elements needing detailed intrusive investigations, contamination, archaeology and unexploded ordnance • define properties of borrow material • detailed quantitative evaluation of credible levee failure and deterioration modes to define levee profile.

9.2.3 Step-by-step technical design process

A technical process flow chart, which sets out the details of the geotechnical and hydraulic/morphological design processes, is presented in Figure 9.5. This should be read in conjunction with Figures 9.3 and 9.4. Figure 9.5 sets out the design process in a step-by-step manner, consistent with most modern international design codes and practices. Further information on the data determination and calculation approaches identified in these flow charts is given in Chapters 7 and 8, including assessment of resistance to the three main groups of failure mechanism (external erosion, internal erosion and mass instability) discussed in Chapter 3.

The flow chart demonstrates how the hydraulic/geomorphic and geotechnical designs are interdependent processes. Particular points of interdependence between the processes include:

- determination of levee alignment (Section 9.4), including identification of vulnerable levee segments

- determination of levee crest level and cross-section (Section 9.5)
- determination of impacts of future morphological change, including influence on:
  - river design water levels
  - adjacent river bed or shoreline beach levels, which might arise from global morphological change or local morphological change induced by the levee itself (eg scour from increases in bed velocities due to river flow concentration or wave reflection)
- stability against all three main groups of failure mechanisms, ie, external erosion, internal erosion and mass instability.

When considering the hydraulic design of new or modified levees, the options appraisal should assess the system characteristics (Chapter 7) for behaviour over the entire design life of the levee system. The level of investigation for each case must be determined by considering the level of risk associated with the levee and how rapidly change is anticipated. Full analysis by hydraulic modelling may only be needed for the shortlisted options or the final option recommended for detailed design. Cases to be considered include:

- present conditions without levee system
- present conditions with levee system
- future conditions (taking account of changes in hydrology, hydraulics and morphology) without levee system
- future conditions with levee system.

Uncertainties mean that it is advisable to consider more than one set of future conditions and to consider the resilience of the levee to change beyond the end of the nominal design life.

9.2.4 Design verification, approval and certification

Levees are very important structures as they are designed to protect individuals, businesses and large areas from flooding. However, if they do not perform to expectations, lives, livelihoods and environments are often put at greater risk. Despite their importance, in many countries, levees have not always been subjected to the same level of rigorous review and control as other elements of critical infrastructure such as dams, power stations or transport infrastructure.

In contrast to levees, some authorities responsible for other elements of critical infrastructure adopt a rigorous approach to the control of the quality and safety of the designs, insisting on a well-defined design process or design review at key stages of the work (see the example in Box 9.2). Historically, these controls have been put in place to manage financial and programme risk of construction as well as to manage safety. Such controls normally adopt a risk-based approach where the nature and extent of the reviews reflects the likelihood and consequences of failure.

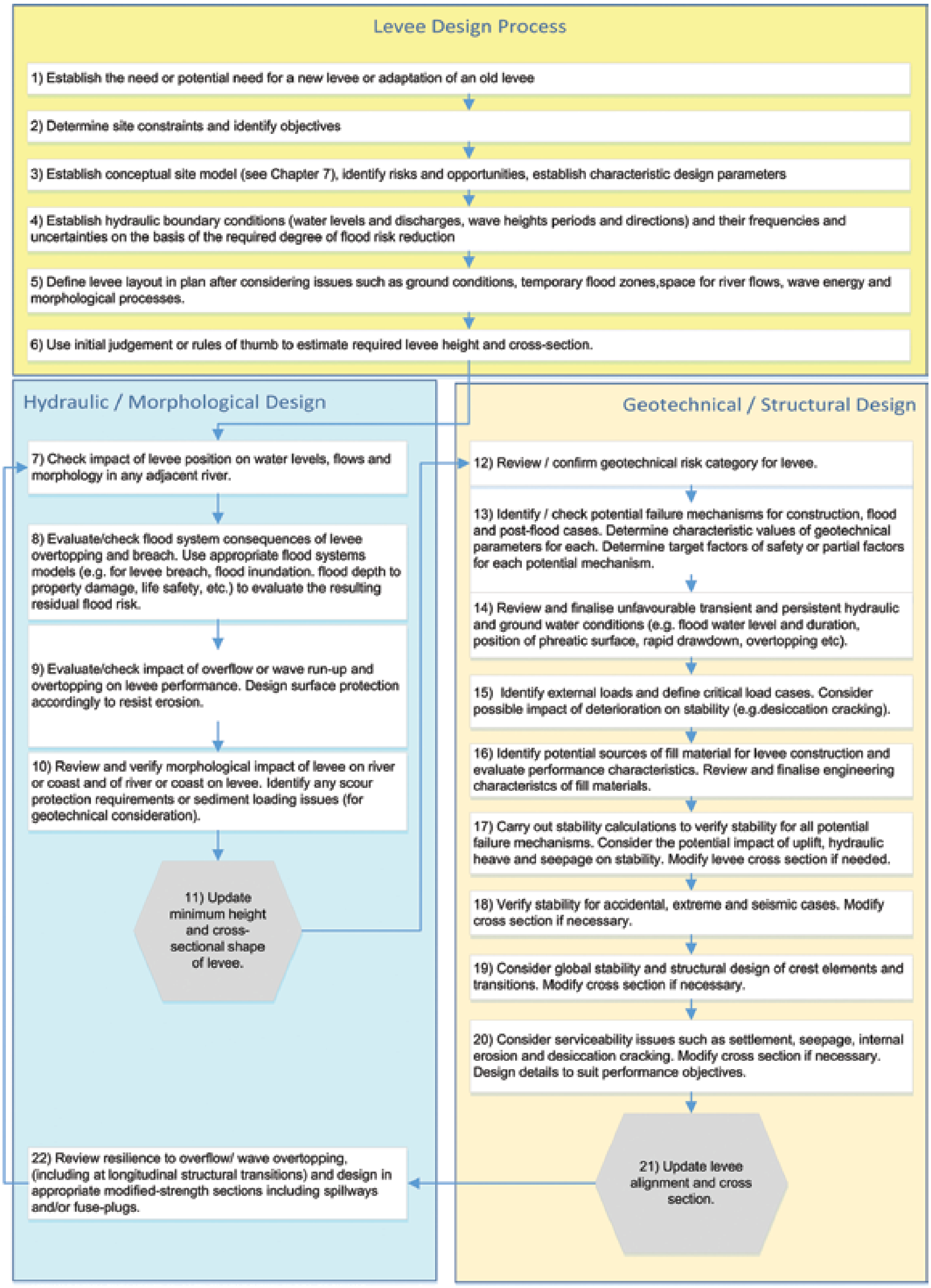


Figure 9.5 Technical process chart for levee design

Box 9.2 ***Key stages in the geotechnical certification process for design of Highways Agency works in the UK***

Through its standards, the UK Highways Agency aims to provide a clear and consistent framework to record the management of the geotechnical risk involved in any particular scheme. The Highways Agency terms this quality assurance process 'geotechnical certification' (Highways Agency, 2008). The process of geotechnical certification is used to ensure that geotechnical risk is managed throughout the lifetime of a scheme. The process is applied to all schemes that involve geotechnical activities and that may pose a risk to the highway asset, to the highway asset owner or to the general public.

Certification is carried out by a nominated individual, who makes statements based on an evaluation as to whether appropriate risk management processes have been applied at four key stages of projects, which are related to the key stages of decision-making within the lifetime of a scheme:

Key stage 1

Initial review of project and geotechnical risks to determine its *geotechnical classification* and so the requirement for geotechnical certification – this stage ensures that potential geotechnical risks are identified at project inception. The requirements for specialist geotechnical processes are also assessed at this stage. The document required from the designer at this stage is the statement of Intent.

Key stage 2

Preliminary assessment including preliminary certification – this stage contributes to the preparation of the outline design and, where necessary, the requirement for land acquisition and orders preparation. The documents required from the designer at this stage are the preliminary sources study report (desk study) and the ground investigation report.

Key stage 3

Geotechnical design and construction certification – this stage provides the information for the detailed design and for the contractor to prepare and carry out construction. The output required from the designer at this stage is a geotechnical design report with all sections completed, prior to construction of relevant areas.

Key stage 4

Geotechnical feedback – this stage reports on all construction work and particularly any unexpected ground conditions requiring changes to the design that occurred. This key stage is a requirement in contracts let by the highway asset owner. The output required from the designer at this stage is the geotechnical feedback report.

For critical levee schemes that carry a high level of risk, independent review of the preliminary designs may be advisable or necessary. Given the potential risks associated with the failure of a large levee or a levee system, a tiered system of quality assurance similar to that used for other types of infrastructure could also be applied by levee owners or regulators to determine the level of review required (see the example in Box 9.3). Any such a system would need to be established and agreed upon at a national level (or within individual contract documentation).

Box 9.3 *Example method for determining risk categories for levees and associated investigative and design effort*

Clear risk categories for levees do not exist in many countries at present. However, by an extension of processes used for risk categorisation of dams, it would be possible to assess each levee against various key issues and determine for each issue the resulting score and, from this, determine the appropriate investigative and design effort.

An example of this approach is shown in Tables 9.3 and 9.4. If this were adopted, the resulting scores arising from the use of Table 9.3 would be totalled, and from Table 9.4 the levee risk category identified. For example, a levee 2.5 m high experiencing a flood hydrograph of two days with 90 people at risk and low potential other damage would score 4 + 2 + 4 + 4 = 14, which would make it a Levee Category II.

Table 9.3 *Issues and associated scores*

Issue	Possible categories and associated classification scores			
Flood duration	> 1 week (6)	1 day to 1 week (4)	1 hour to 1 day (2)	1 hour (0)
Maximum height of levee (related to level of potential impact in the event of breach)	>6 m (6)	3–6 m (4)	1–3 m (2)	<1.0 m (0)
Number of people at risk	>1000 (12)	100–1000 (8)	100–10 (4)	10 (0)
Potential damage to buildings, infrastructure, environment, etc	High (12)	Moderate (8)	Low (4)	Negligible (0)

Table 9.4 *Levee risk category from classification scores*

Total classification score	Levee category
0–6	I
7–18	II
19–30	III
31–36	IV

The subsequent level of investigative and design effort could be determined in the following manner:

- for the simplest and least risky structures, the designs could be checked and reviewed by an individual from the same design team as the designer but who was not responsible for the design. The calculations could be checked directly or by comparison with experience of similar structures nearby
- the designs for more significant structures could be checked and reviewed by an individual from the same company as the designer but who was not responsible for the design or in the design team. The calculations could be checked directly or by comparison with independent calculations
- For levees in situations of highest risk, designs might well be independently checked by a third party organisation. Such an organisation would be expected to prepare its own independent calculations to check the acceptability of the structures shown on the construction drawings.

For example, in the German national guidance (DWA, 2011), a risk categorisation process determines under which Eurocode geotechnical category the levee lies, according to Eurocode 7, and this in turn is used to determine the extent and cost of site investigation.

The investigative, checking or verification level might also be adjusted depending on the nature of work performed on the levee. In the case of existing levees, simple repairs or modifications may not represent a substantial change to the morphology, hydraulics or geotechnics and so may need less checking than when designing a new levee.

If such a system were to be adopted, good documentation (Section 9.3) of the design process would be an essential component of the checking and validation process, as it clearly sets out the design intentions, the assumptions and the approach. Sometimes such a checking or verification process is termed 'certification', but this should not be taken to imply that the process would provide guarantees of levee performance to third parties. At present, various approaches to checking, verification and certification are being adopted in countries who participated in the production of the handbook, and these are illustrated in Box 9.4.

Box 9.4 Approach to levee design or designer certification in various countries

France

In France, there is no certification as such for levee designs or engineers. Nonetheless, for some types of actions related to levees (and dams), engineering firms have to be 'approved' by the Ministry of Environment through a formalised procedure. Also, levee (and dam) projects, whether new-build or modifications, have to be authorised by the local state authorities, according to the Water Law. This authorisation includes a technical review by the relevant administration, assisted for the largest structures by national technical advisory boards.

The French national regulation on dams and levees was introduced by decree 2007-1735 of 11 December 2007. The Ordinance of 29 February 2008, modified 16 June 2009, defines and describes different elements that levee managers have to perform and communicate to the state control services. Some of these elements can only be realised by approved engineering firms, independent companies or levee manager specific departments. These elements include assessment and risk analysis studies and works design and control. The same approved engineering firm has to realise the design studies and control the actual works.

Approvals of engineering firms are associated with five different categories of levees and dams, according to the type of structure and its class (small dams and levees or all types of hydraulic structures including large dams) and the type of action (monitoring and monitoring analysis, studies, studies and works). The Ordinance of 18 February 2010 details the categories and the approval procedure. Subsequent ordinances, of a roughly annual period, list the approvals themselves (engineering firm, type of approval and time length). Approvals are granted for a maximum period of five years, and can be suspended and revoked in case of deficiency.

United Kingdom

Statutory checking of designs for large reservoir dams (dams impounding more than 25 000 m^3) takes place through a government-regulated system of approved 'panel engineers'. There is no certification of levee designs undertaken in the UK, as these are seen to be of lower risk. However, firms appointed to carry out designs of levees are selected through careful procurement processes, such as those for the Environment Agency's framework agreements, which check the technical capabilities of the appointed firms.

The Environment Agency constructs, maintains and manages a significant number of levees and other flood defences, as well as having a lead role in supporting local authorities. The flood defences managed include tidal barriers, pumping stations, flood walls and levees. The Environment Agency monitors and reports, using a risk-based approach, on the condition of its assets and third party assets against a target condition. The experience in England is that there have been few structural failures of levees.

United States of America

There is no legal requirement or statute for certification of levees in the USA. However, Code of Federal Regulations 44 CFR 65.10 (FEMA, 2000) specifies the minimum design, operation and maintenance standards that levees must meet, and continue to meet, in order to be recognised as providing protection from the base flood (also known as one-per-cent-annual-chance flood) on Flood Insurance Rate Maps (FIRMs). The Federal Emergency Management Agency (FEMA) recognises (accredits) levees based on data and documentation provided by a community or other responsible party. Certification, as defined in 44 CFR 65.2(b), is a statement that the submitted information is accurate and in accordance with sound engineering practice.

Data submitted to support a given levee must be certified by a registered professional engineer, along with certified as-built plans for the levee. The submitted documentation must include:

- freeboard design, including that for riverine and coastal levees
- closure designs showing that all openings have closure devices, and that closures are designed according to sound engineering practice and are a structural part of the levee during operation
- embankment protection, demonstrating no appreciable erosion of levee embankment during the base flood
- embankment and foundation stability analyses evaluating expected seepage during base flood loading conditions, including flooding depth, duration, penetrations and other seepage and stability factors
- settlement analysis assessing potential freeboard loss due to settlement, showing that minimum freeboard will be maintained
- an interior drainage analysis identifying the source(s) and magnitude(s) of interior flooding
- operations and maintenance (O&M) plans including evidence that this is being carried out under the jurisdiction of an approved agency and officially adopted by that agency. Official adoption generally requires a vote by a governing body.

The FEMA review is solely for establishing flood hazard zones and does not constitute a determination as to how a levee will perform during a flood event. If a levee is accredited, FEMA will reflect the levee as providing protection from the base flood on the FIRM.

9.2.5 Roles and responsibilities

In this section, the discussion focuses on roles and responsibilities rather than organisations, because many variants are possible within individual organisations. For example:

- the organisation which owns the levee may also employ the designers and project managers
- the construction company may take responsibility for the design and the project management as well as for the construction itself.

Clearly defined roles and responsibilities are fundamental for seamless and efficient delivery of levee design and construction. Owners should evaluate project needs, clarify roles and responsibilities and establish effective lines of communication. In general:

- the *levee owner* will need to work closely with the designer and project manager to keep the public appraised of funding, programme, and overall project status
- the *designer* will normally be responsible for the technical elements of the project, and will work closely with the levee owner so that public aspirations are properly addressed in the design process; the designer may also have responsibilities for checking that the constructor is complying with the contractual requirements including the design drawings and the specifications
- the *project manager* must have sufficient knowledge and experience to manage a wide variety of disciplines; project managers should have an understanding of levee construction, risk identification, analysis and management and may have to manage conflicting requirements
- the *constructor* is responsible for adhering to the design and specifications provided by the project team; good overall project management is crucial to the timely delivery of projects, but all team members must understand their roles and contribute accordingly to achieve success.

Table 9.5 provides an example of levee project roles and responsibilities. The size of design and construction teams will vary depending on levee size and complexity. In addition to the usual personnel skilled in geotechnical engineering, hydraulic engineering, structural engineering, construction expertise and project management, other specialists that are commonly required include geologists, hydrogeologists, seismologists and geomorphologists. Other specialists that may be involved include landscape architects, archaeologists, environmental engineers and various ecological disciplines. The extent of the involvement of these specialists will depend on the size, complexity and nature of the levee.

The owner/levee design team should also consider involving suitably experienced constructors in the design stages of levee construction projects. This is increasingly recognised to be of value in the case of levees because of issues of access, proximity to water affecting plant choices (land-based vs. water-based), sources of fill materials and levee-specific constraints and risks (Section 10.1.1). Early involvement of an experienced constructor also helps the team estimate and compare approximate costs for proposed options, identify construction-specific safety or environmental-impact concerns, and evaluate construction site access, which all help limit the number of surprises during construction (Box 9.5).

Box 9.5 Designer using early involvement from a constructor

A significant number of levee construction projects have suffered from inefficiencies, design modifications and rising construction costs that can be tied back to the lack of interaction between designers and constructors. In one US public works project, early constructor involvement was successfully used to ensure on-time delivery within budget. The contractor assisted the designer in choosing alignment and materials to take advantage of the local market. The contractor was also able to set up materials handling and receiving operations to expedite production, provide near-site storage and reduce the possibility of logistics-related issues. Without early access to design direction, this would not have been possible.

Table 9.5 *Levee project roles and responsibilities*

Project responsibilities	Project roles			
	Levee owner	*Levee designer*	*Project manager*	*Constructor*
1 Pre-design (identifying the need)				
Establish project goals and performance standards that meet the project requirements and aspirations of the public	✓			
Account for sustainability and life-cycle costs, including management, operations and maintenance	✓			
Conduct public meetings and respond to public concerns	✓		✓	
2 Conceptual design				
Manage project funding and assess cost-effectiveness	✓		✓	
Accept the project goals and performance standards presented by the stakeholders (could be private and public)		✓	✓	
Identify a range of feasible alternatives and options for consideration	✓	✓	✓	
Characterise the site, develop design parameters and prepare preliminary design report		✓		
Prepare flood risk assessment and environmental impact reports		✓		
Prepare preliminary design and assess alternatives		✓		
Identify a preferred option or options		✓		
Conduct public meetings and introduce preferred solution	✓	✓	✓	
Complete statutory processes, if necessary	✓	✓	✓	
3 Detailed design				
Undertake further site investigation for detailed design if necessary		✓		
Solicit input from competent and experienced contractors	✓	✓	✓	
Prepare basis of design report		✓		
Prepare design calculations and document design by experience		✓		
Prepare design drawings, specifications, documentation and reports		✓		
Procure construction contract	✓	✓	✓	
4 Construction				
Adhere to legislation and statutory processes	✓	✓	✓	✓
Supervise and monitor construction		✓	✓	✓
Deliver project on time and within budget		✓	✓	✓
Create as-built drawings and construction records		✓	✓	✓
Create operations and maintenance manual		✓	✓	
5 Operations and maintenance				
Finalise and update operations and maintenance manual	✓	✓		
Monitor long-term performance with project objectives	✓	✓		
Maintain project per operations and maintenance manual	✓			

Notes

1 Not every stage will be required on every project.

2 The table is indicative of roles. Actual roles may differ depending on the specifics of particular design and construction situations.

9.3 REPORTING AND DOCUMENTATION

Information management throughout the life of a levee, or a levee system, is a critical element in the long-term performance of these structures. Historically, however, this information has not always been acquired, recorded, retained or archived properly. As a result, pertinent information is not always available, and this can lead to inefficiency or disaster. For example, a levee breach could occur at a location where previous events had threatened to destabilise the levee but had gone unrecorded, or the records were lost.

Records management is a major consideration in the levee design process. Table 9.6 provides a sample of the reporting that could be required in each of the design stages, and it references the relevant chapters where each topic is covered in detail.

Table 9.6 Reporting by design stage

Design stage	Reports used during the design stage	Discussed in section	Produced/updated products (all discussed in 9.3)
Pre-design (identifying the need)	Flood history records and reports	7.1.6	Flood risk analysis reports
	Condition assessment reports (CAR)	5.3	Basis of design report (BoD), including design criteria such as hydraulic conditions, ground conditions, geotechnical parameters etc
	Operations and maintenance manual	4.1	Risk register
	Performance records	5.6	
	Archived reports and records	5.6	
Conceptual design	Flood history records and reports	7.1.6	Updated BoD
	Desk study report and risk register	7.1.6	Preliminary design report (often called options assessment report)
	Preliminary ground investigation report (GIR), if required	7.1.6	Updated risk register
	Condition assessment reports (CAR)	5.3	
	Risk analysis reports	9.3.1	Flood risk analysis reports
	Basis of design (BoD) report	9.3.4	
Detailed design	Updated BoD report	9.3.2	Design report
	Preliminary design report	9.3.4	Drawings
	Ground investigation report (GIR), including desk study report and geotechnical interpretation	9.3.3	Technical specifications
	Risk register	9.3.1	Updated risk register
Construction stage	Design report	9.3.4	As-built updates of…
	Drawings	9.3.5	design report
	Technical specifications	9.3.5	technical specifications and drawings
	Operations and maintenance manual	9.3.6, 4.1	daily reports (including the risk register)
	Specification for instrumentation and monitoring	9.3.5, 9.16.2	construction monitoring report
	Ground investigation report (GIR), including desk study report and geotechnical interpretation	7.1.6	requests for information (RFIs)
	Risk register	9.3.1	change order reviews
O&M stage (design input)	Operations and maintenance manual	9.3.6, 4.1	
	Condition assessment reports (CAR)	5.3	Archive of historical reports
	Archived and as-built reports	9.3.5, 9.3.6, 9.16	Regular updates of O&M manual including records of repairs etc

Table 9.6 Reporting by design stage (contd)

O&M stage (design input)	GIR (desk study through interpretation)	7.1.6	Performance records
	Design report	9.3.4, 10.1	
	Drawings	9.3.4, 10.1	
	Technical specifications	9.3.4, 10.1	
	Construction monitoring report	10.1	

The design process starts when a decision is made that change is required, or when a decision is made to establish whether change will be of overall benefit. This decision will normally be made on the basis of information gathered and provided in various reports. As the design process matures, additional information is gathered, scrutinised and added to existing information. This succession of reporting will highlight the importance of detailed records management and the overriding principle that no information should be lost. All projects should establish records management directories and procedures early on in the design process to ensure consistency and continuity.

Not all of the reports outlined in Table 9.6 will be available or necessary for every project. In some cases, particularly situations where only minor works are required, not all of these documents will be available or necessary. Similarly, while some of the reports are required for compliance with statutory design codes (such as Eurocodes) or because their production is considered to be good practice, they can be simple, short reports.

This section describes some elements of good practice in reporting and documentation including:

- risk register
- basis of design report
- site investigation
- design reports
- detailed design outputs
- operations and maintenance manuals.

9.3.1 Risk register

The risk register is a living document that lists the risks (time, cost, quality, safety, environmental) to the levee project, including construction risks, which is developed throughout both the design and construction processes. Construction risks and their representation in a risk register are described in detail in Section 10.1.3. One key to managing risk during design is to appreciate the importance of keeping team or discipline leads appraised of changes in risks or the addition of new risk categories. This is especially true if the risks cross disciplines and require co-ordination. Furthermore, identified risks should be eliminated, mitigated or minimised in accordance with good practice. A risk register facilitates this process in a manner that avoids risks being ignored, overlooked or forgotten.

9.3.2 Basis of design report

The basis of design report is a preliminary report (known by different names in different countries and situations, eg design criteria report) which compiles the available data, identifies the constraints, sets the goals, characterises the site and defines the design criteria. If initial reconnaissance investigations and appraisals were conducted, the basis of design report will summarise their findings. The preliminary basis of design report is a living document, which can be updated as new constraints and data become available.

The basis of design report is important for consistency and makes checking and reviewing the input data efficient and straightforward. This report makes it easy to check pertinent levee project information at any design stage, use consistent information for each proposed alternative and update data for any subsequent detailed design stage. The basis of design report avoids uncertainty over design criteria and clarifies individual responsibilities. It reduces errors, abortive work, extra costs and subsequent delays, and also reduces the chance of subsequently forgetting design situations and risks considered early on in the design process.

When the site investigations and assessments have been completed and fully interpreted, the preliminary basis of design report should be updated and finalised before the design team starts detailed design calculations. This reduces the risk of subsequent abortive work, because the final basis of design report is an effective sign-off of the key inputs and objectives for the calculations. Careful version control of the document is therefore required, to avoid the design being carried out on the basis of old or superseded information. The basis of design report will only be successful if it clearly defines the design criteria, is accessible, and is easy to update and put into effect if situations change during the design process. See Table 9.7 for the basis of design report checklist.

9.3.3 Site investigation reports and the designer

Site investigations provide input data for the design process. Ground investigation data will usually be provided in desk study reports (collated historical data), factual reports (factual data from field or laboratory sampling and testing, usually supplied directly by the ground investigation contractor) and interpretative reports (interpreted ground conditions and geotechnical design parameters, provided variously by the ground investigation contractor, by a specialist geotechnical adviser or by the designer). The factual and the interpreted data may be combined into a single report. Similarly, hydraulic studies will provide hydraulic loads for design purposes. This data can come from the analysis of historic information or as the output from hydraulic modelling exercises. The 'characteristic' loads and parameters can then be used in combination with factors of safety or partial factors during the design process. Details of the entire process of site investigation to develop a conceptual site model are described in Chapter 7, covering investigation, sampling and testing, analysis and modelling methods.

Designers often commission a specialist contractor to conduct site characterisation works on behalf of the levee owner. These are commonly dominated by geotechnical investigations, but may also involve collecting topographic, bathymetric, hydraulic and geomorphological data. The designer will commonly interpret the factual data recovered by the investigations or other characterisation. If this is not the case, those responsible for the scoping and interpretation of site characterisation should fully understand the objectives, scope and purpose of the levee project. The designer should supervise the site investigations, field trials and installation of monitoring equipment. Apart from anything else, such investigations can be useful prototypes for construction and can highlight site-specific issues. If unforeseen conditions are encountered during these field works, the designer should modify the scope of the investigative works. The designer should check that the interpreted characteristic parameters are appropriate for the design calculations. This process of interpretation requires experience, knowledge and judgement: the resulting ground models and design parameters will form the basis of the design calculations and will directly affect the final configuration and the eventual performance – or failure – of the levee structure.

Box 9.6 Importance of datum validation

Many deltas or estuaries around the world are prone to flooding. Part of the reason for this is the ongoing natural settlement across these vast areas. In this situation, settlement over extensive areas can lead to survey datum levels becoming unreliable. This can lead to uncertainty surrounding the degree of risk reduction provided by any particular levee.

During the course of levee reconstruction in the aftermath of Hurricane Katrina in the USA, the US Army Corps of Engineers put special emphasis on validating the vertical data. These investigations proved critical to efforts to maintain or enhance authorised levels of protection.

9.3.4 Design reports

Design report – preliminary version

The preliminary version of the design report should refer to the basis of design report (assuming that it has been prepared) and should incorporate its findings, either as an appendix or as part of the opening sections of the report. The preliminary version of the design report should include any further assumptions relating to constraints, site characteristics or objectives that are not included in the basis of design report but form an important part of the design. The preliminary version of the design report will generally include a detailed summary of the preliminary design outputs and calculations, simple drawings showing layouts and typical cross-sections and simplified technical specifications highlighting particular or unusual requirements for construction procedures and materials or construction sequences. It is also important to include recommendations for further site investigations (if required) and a list of issues that need to be considered carefully during the detailed design stage. These recommendations could include a scope for further detailed topographic or bathymetric surveys, for hydraulic modelling or monitoring, for additional ground investigations or for further environmental background studies etc. A checklist for contents of the preliminary version of the design report is included as part of Table 9.7.

Again, it is stressed that the level of detail provided in the preliminary version of the design report should reflect the size and complexity of the levee project and the associated level of risk. For a major levee project, the preliminary version of the design report, the environmental impact assessment and the flood risk assessment report could all be substantial documents. In contrast, for a simple repair, the preliminary version of the design report might be limited to a few photographs of the problem area coupled with some simple sketches of solutions based on experience (eg of a solution that has worked successfully elsewhere) and a consideration of construction risks.

However, the design team should produce a preliminary version of the design report (and, following the completion of the detailed design, the design report) for each situation where a design is eventually constructed. Without such documentation, the rationale behind an element of construction may not be clear or may be forgotten with the passage of time, which could lead to inadvertent damage or neglect in the future.

Design report – update after detailed design

A common requirement of engineering design codes such as the Eurocode 7 (European Committee for Standardization, 2004a) is that a design report is produced at the end of the design process. This report documents the assumptions, data, methods of calculation and the verification of safety and serviceability of the structures designed. The design report should reflect the nature and extent of the work to be constructed. The level of detail can vary greatly and will depend on the type of design.

The design report should be an extension of the previous reports produced, including the preliminary design report, and should cross reference the basis of design report, relevant site investigation reports, such as the ground investigation report (including interpretative reporting). The design report should normally provide a detailed description of the site, surroundings, site characteristics, ground conditions and proposed construction. The report should include all calculations, design values, drawings, specifications, codes and standards. A suggested checklist for contents of the design report can be found in Table 9.7.

Table 9.7 Design reports checklist of contents

Design report sample checklist	Issues to consider including/updating at following stage:			
	Basis of design report	*Preliminary design report*	*Design report – construction stage*	*Design report – O&M stage*
1 Introduction				
General introduction	✓	✓	✓	✓
Levee project background	✓	✓	✓	✓
Scope and design objectives	✓	✓	✓	✓
2 General site description				
Site location and layout	✓	✓	✓	✓
Layout of existing levees and other works	✓	✓	✓	✓
Description of leveed area and stakeholders	✓	✓	✓	✓
Historical performance data	✓	✓	✓	✓
Present the interpretation of the data from the site investigations and the site characterisation exercises	✓	✓	✓	✓
Site constraints and existing usage	✓	✓	✓	✓
3 Environmental conditions				
Topography and bathymetry		✓	✓	✓
Historical wind, wave, tides and current data			✓	✓
Historical temperature, salinity and humidity			✓	✓
Historical rainfall, snow and ice data			✓	✓
Historical contamination data			✓	✓
Information on protected habitats and species			✓	✓
4 General design criteria and performance requirements				
Identification of protected physical assets	✓	✓	✓	✓
Confirm the required level of flood risk reduction	✓	✓	✓	✓
Summarise characteristic loads, parameters and resistances for detailed design	✓	✓	✓	
Review and confirm other inputs and constraints	✓	✓	✓	
Confirm the design codes of practice	✓	✓	✓	✓
Design codes, standards, guidelines and manuals			✓	✓
Return periods of design loads			✓	✓
Design life			✓	✓
Performance requirements (including allowable settlement, resilience etc)			✓	✓
Confirm the risk category of the structures to be designed	✓	✓	✓	
Detailing to provide resilience			✓	
Review and confirm likely failure mechanisms that need to be considered during the detailed design stage	✓	✓	✓	
Highlight any particular calculation of analysis techniques required	✓	✓	✓	
Confirm the anticipated alignment and structural form (simple levee, complex structure with berms and/or crest structures etc)	✓	✓	✓	

Maintenance objectives (eg ease of access, frequency of grass cutting)				✓
External loads (eg vehicle loadings, seismic design conditions, loadings during emergency works)			✓	✓
Performance requirements for structures (eg resistance to erosion for given flood duration)			✓	✓
Confirm the design requirements for accidental, extreme (flood fighting) and seismic design conditions	✓	✓	✓	✓
Durability of structural materials		✓	✓	✓
Sustainable construction operations		✓	✓	✓
Secondary benefits		✓	✓	✓
5 Hydraulic design criteria				
Confirm hydraulic actions for design (retained water levels, wave heights etc)	✓		✓	
Design water levels, velocities and hydrographs			✓	✓
Design wave and current conditions			✓	✓
Design overtopping flow conditions			✓	✓
Design rainfall, snow and ice			✓	✓
Geomorphological design criteria	✓	✓	✓	✓
6 Geotechnical design criteria				
Design ground models		✓	✓	✓
Design groundwater levels			✓	✓
Geotechnical design parameters		✓	✓	✓
Design requirements for non-seismic conditions			✓	
Design requirements for seismic conditions			✓	
Identification of fill sources and characteristics			✓	
Selection and characterisation of other materials such as geofabrics			✓	✓
Identification and management of ground risks		✓	✓	✓
7 Construction considerations				
Identify potential health and safety issues that may affect construction. Establish principles for the identification and management of health and safety risks to construction workers	✓	✓	✓	
Health and safety	✓	✓	✓	
Safe access routes			✓	
Design for safe construction			✓	
Assumptions made about potential construction methodologies and techniques, stated in such a way that they can be scrutinised by others	✓	✓	✓	
Sourcing and transportation of materials to site			✓	
Identify potential borrow areas, if the use of fill materials is required for construction	✓	✓	✓	
Confirm characteristic parameters for fill materials	✓	✓	✓	
Flood risk during construction			✓	
Update the risk register to include design, construction, health and safety and programme risks	✓	✓	✓	

8 Operation and maintenance considerations				
Identify potential health and safety issues that may affect operation, and establish principles for the identification and management of health and safety risks to operatives working for the end-user	✓	✓	✓	✓
Safe access routes			✓	✓
Design for safe operation and maintenance			✓	✓
Maintenance required to optimise the balance between deterioration and the need for maintenance work			✓	✓
Design for safe emergency access			✓	✓
9 Reporting requirements				
Basis of design report	✓	✓	✓	
Preliminary design report		✓	✓	
Design report			✓	
Construction documentation (technical specifications, BoQs, drawings)			✓	✓
As-built documentation (technical specifications, drawings)				✓
Operations and maintenance manual				✓

The design report should also include a plan for supervision and monitoring during construction, as appropriate, and should state the purpose of each set of observations or measurements. Further updates to the design report with the field observations should be added as an addendum to the report, after the required checks have been carried out at the construction stage. The design report requirements for monitoring during construction could include:

- the specific parts and locations of the structure to be monitored
- the frequencies for reading monitoring instruments and taking measurements
- methods to be used to evaluate monitoring data
- the range of expected results
- clear definitions of stop points or hold points
- a list of the parties responsible for the monitoring and interpretation of results.

9.3.5 Detailed design outputs

Drawings and specifications should contain sufficient detail to adequately plan and price the construction work. The documentation should set out any constraints on the phasing, nature or extent of the construction works (such as environmental requirements or regulations, limitations on working hours etc) and the risks to construction identified during the design process, but which have not been eliminated as part of that process.

Design drawings

The information normally supplied to construction companies as part of a tender package for levee works is generally based on a series of drawings and a technical specification. Table 9.8 provides a summary of the types of drawings that are commonly required for levee-related construction projects.

Table 9.8 Checklist for levee drawings

✓	site location plans, layout, facility plan drawings
✓	the alignments of existing levees or other structures with layout of crests, slopes, berms and other major levee components – these plans should also show the locations and layout of any known constraints on construction.
✓	the location of allowable corridors for construction traffic access routes and staging areas, the location of potential borrow areas and the location of any specific leveed areas.
✓	cross-sections showing the detail of existing levees and other structures, with clear notes on the drawings to state whether the detail provided is known or inferred.
✓	plans showing the alignments of new levees and other structures with layout of crests, slopes, berms and other major levee components – these plans should also show the locations and layout of any constraints on construction, demolition plan, erosion plan, drainage plan and any paving plans.
✓	cross-sections showing the detail of the new, adapted or repaired levees, together with sufficient detailed information for the structures – these should be provided at a sufficient number of locations for cost estimating and construction safety.
✓	drawings presenting typical architectural, landscape architectural, structural, process, mechanical, electrical, instrumentation and control or miscellaneous details, if required.
✓	typical construction details in plan, cross-section and elevation to convey the nature and the detail of the work required – these will include cross-sections and details showing dimensions, material types and layouts, details of interfaces between material types, structural information such as steel grades and section details for embedded retaining walls, concrete grades and reinforcement details for concrete crest structures etc. These drawings are particularly required at transitions between new and existing structures and at interfaces between different structural types.
✓	cross-section drawings with sequence and timing of construction wherever appropriate, for example, a retaining wall may need to be constructed in a certain manner or sequence to match the requirements of the design calculations. Similarly, a large levee may need to be constructed in stages (lifts) to avoid over-stressing the foundations soils during construction.
✓	the locations of further ground investigation and/or monitoring works.
✓	plan and cross-sectional drawings showing typical monitoring details and layouts.

Technical specifications

On small levee projects such as repairs, it may be possible to present all of the specification information required for construction as notes to the drawings. In most cases, however, technical specifications for levee construction projects will be stand-alone documents. They may be adapted from standard specification documents (for example, Box 9.7) but, as these may not necessarily address levee related issues, the designer will need to append or modify clauses as necessary.

The technical specifications and drawings are usually supplemented within the contract documents by requirements on the contractual procedures for the control of construction, including key approval hold points or milestones during construction. These can be written on a project specific basis but, generally, it is better to make use of standard specifications and adapt these to the requirements of the project. An example of this process is given in Box 9.7).

Box 9.7 *Standard specifications in the USA (from USACE/NVAFAC/AFCESA/NASA (2011)*

In the USA, there are standard specifications that have a focus on earthworks, earthwork-related structures and long linear embankments such as levees. An example is provided in USACE/NVAFAC/AFCESA/NASA (2011). These standard specifications are well understood by designers and construction companies alike – both are familiar with the standard material types, the standard methods of deposition and the standard methods of compaction covered by such specifications. Furthermore, these standard specifications can usually be adapted to particular levee projects through the use of particular or site-specific clauses. Table 9.9 summarises a sample table of contents of the technical specifications given in USACE/NVAFAC/AFCESA/NASA (2011).

Table 9.9 *Checklist of contents for technical specifications*

✓	procurement and contracting requirements – list of drawings, instructions to bidders, bid schedule, contracting conditions.
✓	general requirements – summary/scope of work, payment procedures, cost and performance report, schedule, submittal procedures, special levee project procedures, safety and health requirements, quality control, environmental management, materials management.
✓	existing conditions – operations and maintenance of existing conditions, removal of materials and remediation of site, materials control.
✓	materials – concrete, earthwork, geofabrics, masonry, metals, wood, plastics, composites, insulation, moisture protection, fixtures, finishes, specialities, equipment, furnishings, special construction, conveying equipment, fire suppression, plumbing, HVAC, electrical, communications, electronic safety and security, utilities, transportation.
✓	waterway and marine construction – machinery for locks, gate lift systems, tainter gates, vertical lift gates, dredging, channel protection, levee construction, reinforced soil slope, pilings, embankments.
✓	process integration – pipelines, fibre optic lines.
✓	material processing and handling equipment – cranes, hoists, hydraulic fluid and power systems.
✓	process heating, cooling, and drying equipment.
✓	process gas and liquid handling, purification, and storage equipment.
✓	pollution and waste control equipment.
✓	water and wastewater equipment.
✓	electrical power generation.

9.3.6 Operations and maintenance (O&M) manual

The final stage of the design process should be to produce the O&M manual. Its purpose is to inform those responsible for operations and maintenance of the assumptions incorporated into the design of the levee in its current configuration and the envisaged operations and maintenance requirements throughout the life of the levee. The document is a critical link between the design process, construction and the long-term operation of the levee. Its importance and typical scope and contents are discussed in detail in Section 4.1.4. O&M manuals should be updated periodically, and especially after any significant works by the levee owner and those responsible for operations and maintenance. Input will be required from the designer whenever works are carried out by design. Further, the O&M manual should be used by designers if improvements, extensions or other significant modifications are deemed necessary.

9.4 LEVEE LAYOUT AND ALIGNMENT

Levee alignment is the first aspect of design to be considered and resolved as it will determine the characteristics of the environment, including the hydraulic conditions and the ground conditions. Careful early thinking about the levee alignment and footprint may avoid locked-in problems when future adjustments are needed.

It should be noted that this section does not discuss crest elevation and geometry, which is the subject of Section 9.5. However, it is likely that preliminary ideas about geometry will be developed at the same time as considerations of alignment.

9.4.1 Principles for selecting levee alignment

The levee alignment needs to be reviewed throughout the design process, as changes may be required as a result of further ground investigations or due to the output from some of the detailed design calculations. For example, a thorough ground investigation may encounter an area of soft or permeable ground not previously identified, and it may be found to be more cost-effective to avoid that ground rather than trying to engineer a solution to cross it.

Often the final decision is a balance or trade-off between the positive and negative aspects of each issue, including cost, as illustrated in Boxes 9.8 and 9.9. The following issues should be considered when examining options and making decisions on levee alignment.

- **alignment of existing levees:** careful consideration is needed as to whether any new, adapted or repaired levee is restricted to the existing alignment
- **geomorphological processes:** rivers and coasts change their alignment naturally (Sections 7.3 and 7.4). Allowance should be made for these changes throughout the design life of a levee (Figure 9.6). In the case of existing defences, one option to be considered will be managed realignment or set-back of defences to make room for the river or for the increasing energy of the sea. For set-back defences, consideration should be given to non-structural alternatives, such as making use of natural high ground. Thought can also be given to locating non-critical amenities such as parklands or sports fields in front of the levee.

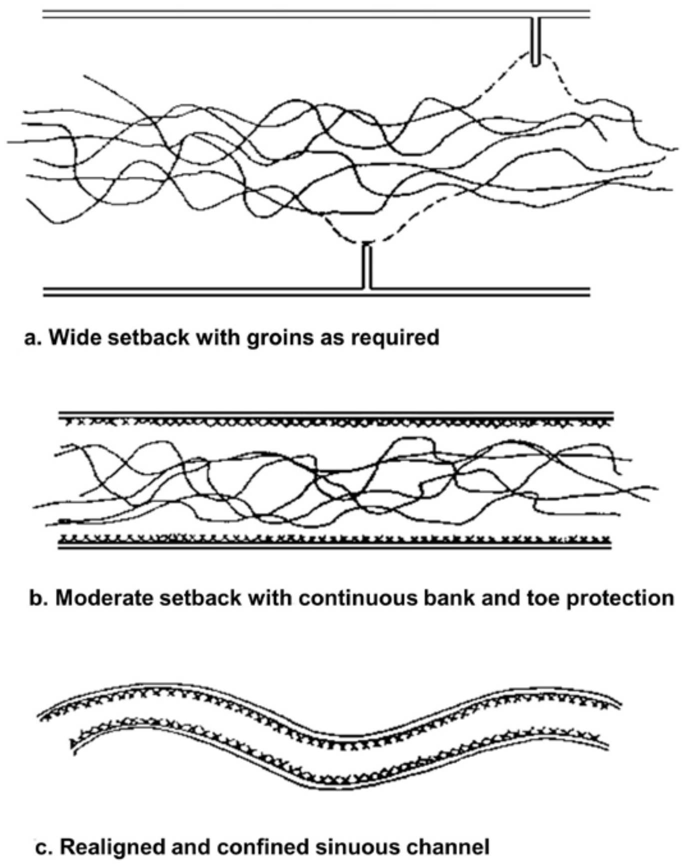


Figure 9.6 Geomorphic considerations in aligning levees (USACE, 1994)

- **potential hydraulic impacts:** if the levee alignment narrows the river corridor, this may encourage increased velocities and upstream water levels. Similarly on the coast, levee alignments too close to the sea may reduce the available width on the coast for wave energy to be dissipated. In these cases increased flow velocities or wave action adjacent to the levees may lead to the need for scour protection measures to control erosion of the levee, particularly on the outer bends of rivers or at denuded or over-steepened coastal beaches. In such situations, modelling will probably be required to predict how floodwater will interact with the different potential locations of the levee.

- **environmental benefits:** locating the levee in a set-back position and allowing the river or the coastline to develop naturally without adversely affecting the levee, may allow habitats to develop or be retained which may be crucial for bio-diversity and ecosystem services. These might include vegetated strips alongside a low-water river channel, or an area of salt marsh between a coastal levee and the sea
- **underlying ground conditions:** soft or permeable soils can have a significant impact on levee performance. It is therefore important to consider all available information (Sections 7.7 and 7.9) on underlying ground conditions at an early stage when comparing and selecting possible alignments, and to be prepared, where possible, to change these decisions as and when additional information becomes available. Ideally the selected alignment should avoid weak and compressible ground or naturally permeable soils, but this is not always possible
- **proximity of existing high ground:** for the purposes of alignment selection, high ground is that which is above the relevant flood levels under consideration. Often levees can be tied into high ground to minimise length and cost. If this is the case, the nature of the high ground should be considered (eg it should not tie in to highly permeable ground or into a structure not built for water retention such as a road embankment)
- **construction materials:** potential sources of earthworks materials for the possible alignments will need to be examined. In addition to cost, ease of access to the site for construction traffic and building materials must be considered
- **existing and future land use:** the proximity of existing development may restrict the possibilities for alignment. This is particularly so if there are heritage sites to be considered or avoided. Furthermore, land use behind the levee may change in the future and the potential for this will need to be considered in the flood risk analysis (Chapters 2 and 5)
- **interaction with levee geometry (Section 9.5) and construction safety (Chapter 10):** if the levee to be constructed, adapted or repaired is positioned close to the water, early consideration should be given to how the work can be executed safely, quickly and cost-effectively
- **location and use of spillways and temporary flood storage areas** will also need to be considered in fluvial situations. These can interact with the other issues discussed above such as geomorphology, flow constriction etc
- **location and nature of existing utilities:** existing utilities (surface, buried or aerial) may need to be relocated as part of any levee construction or realignment.

Box 9.8 Examples of balancing conflicting requirements for levee alignment

Levee constructed close to a river

A levee constructed close to a river will protect the maximum number of properties in the vicinity of the levees but will not leave much room for the floodwater. Such levees may need to be of a considerable height to act as a barrier to the floodwater, and this will both represent a significant construction cost and have a major visual and environmental impact. It will also potentially increase upstream water levels and hence increase risk.

Set-back levees

Set-back levees will allow more space for the water (creating, for example, room for a river to meander naturally and allowing space for floodwater to accumulate). They can also incorporate environmental enhancements or benefits (and so may not need to be so high or so intrusive). However, set-back levees will increase the level of flood risk on the unprotected floodplains and may leave some dwellings or properties undefended.

Cost implications may be significant in comparing the above two options, particularly once the cross-sections of the levee in the two cases is known. For example, is the additional cost of purchasing more land and constructing a new levee balanced by having a levee of smaller height and cross-section which is easier to build located further from the water's edge.

Box 9.9 *Setting of levee alignment for the Feather River set-back levee, Sacramento, CA*

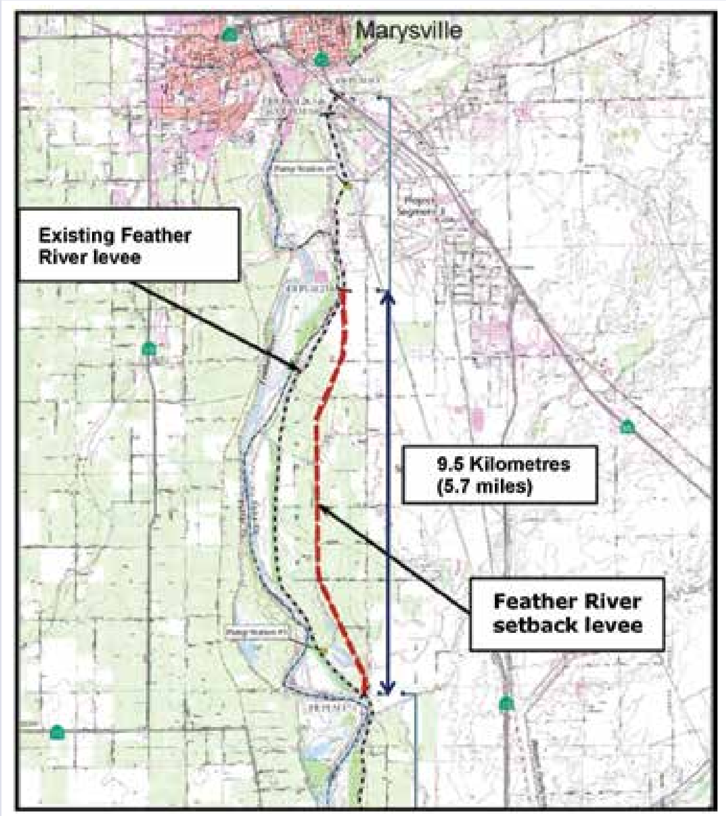


Figure 9.7 *Realignment of levee on Feather River (courtesy Three Rivers Levee Improvement Authority, Yuba County, California, USA)*

The Feather River Set-back Levee in Yuba County, California, is an excellent example of a levee improvement that provides multiple benefits. This set-back levee is approximately 9.5 km long and set-back from the almost 100-year-old existing levee along the Feather River by generally about 1 km. It was constructed between 2008 and 2010 at an approximate cost of $165m using local and state funding. It was designed to provide a 200-year level of flood protection and included the following benefits:

- new engineered levee built on a good soil foundation
- widens Feather River channel to allow better flow and eliminates water flow choke point
- replaces current old and defective levee
- moves new levee away from eroding face of Feather River
- expands floodplain by 1500 acres, benefiting several threatened and endangered species including Swainson's hawk, yellow-billed cuckoo, valley elderberry beetle and Chinook salmon.

The design included 6 m to 12 m high earthen levee, approximately 8 km of soil-bentonite cut-off walls up to 22 m deep, 33 relief wells, approximately 1600 m of landside stability berms and about 1300 m of waterside stability berms.

9.4.2 Width of levee corridor, including no-construction zones

At the same time as setting the levee alignment, it is highly desirable to identify (and where possible purchase or obtain licences for) the full levee corridor, and to understand the potential consequences of compromise (Box 9.10). The width of the levee corridor can be assessed as explained in Section 9.5. The designer should keep the following issues in mind:

- if, because of a levee's alignment or location, it has to be constructed on poor foundation soils or out of poor quality (but locally available) construction materials then a wider corridor may be required to provide space for shallower side-slopes to improve stability or a greater crest width to facilitate easier subsequent raising of the crest level
- restrictions on corridor width can impose subsequent restrictions on levee design, and may mean that adequate stability with an earthen levee cannot be achieved, and may therefore necessitate

more expensive composite structures (cross-sectional options are set out in Chapter 3). This may require (Section 9.5):

- acceptance of steeper than desirable side-slopes with reduced mass stability during extreme events or during rapid draw-down situations after the flood recedes (note that this may conflict with national standards or codes of practice)
- the use of structural components to increase the effective crest level (sheet piling, concrete walls etc)
- the levee slopes being closer to the riverbank/beach than desirable from a scour perspective and hence requiring surface protection layers on the waterside of the levee.

Box 9.10 ***Care needed in identifying land-take for levees***

A major levee project in the UK was granted planning permission on the basis of levees that had been sketched onto a drawing during a feasibility study rather than being the result of calculations. These levees had over-steep side slopes and a narrow crest. As a result, insufficient land was purchased for the scheme and this lack of space subsequently created major problems for the scheme.

As part of establishing the corridor width for new levees or levee modification projects, consideration should be given, where practical, to establishing *no-construction zones* that extend beyond the levee toes. These zones protect the levee from incursions and damage, make the process of levee inspections easier and provide easier access and working conditions in the case of an emergency. In these zones, prohibited activities may include:

- excavation that might destabilise the levee or increase internal erosion
- construction work
- access and storage
- the growth of trees or shrubs (Section 4.5).

The width of such zones should be large enough:

- to allow access and movement of equipment for surveillance and O&M activities
- to provide a contingency for future levee modifications that may be necessary as a result of changes (Section 9.1.7), such as climate change related increases in loading, or requirements to update and improve design standards (eg as a result of changes in land use, population or protected physical assets)
- to prevent ingress by aggressive root systems of trees.

In practice, there are many locations in existing levees where such a no-construction zone has not in fact been maintained. In levee rehabilitation or modification projects, it may be impractical to retrospectively introduce such a zone, given that, in the area that would be allocated for no-construction, many buildings roads etc do already exist and even in places now form part of the levee. In the Netherlands, the compromise adopted on this issue is to allow such buildings to remain so long as they are completely outside of the minimum levee cross-sectional prism. The French and US approaches are described in Boxes 9.11 and 9.12 respectively.

Box 9.11 ***French approach to no-construction zones***

In France, it is common to create no-construction zones on the landward side of levees. A common rule of thumb along the river Loire is that no construction should be allowed within a distance of 7 times the levee height from the most landward point of the levee structure (including any ditch on the landward side of the levee). In reality, this is not always achieved for historic or existing structures, but every effort is made to apply it to new construction works.

Box 9.12 *US approach to no-construction zones*

In the USA, the recommended minimum distance by which the set-back zone should extend beyond the levee and any appurtenant structures should ideally be the greater of:

- the distance from the outermost point of the levee cross-section or appurtenant structure to the limits of the critical stability failure surface, plus sufficient room to make repairs (Figure 9.8). However, while this is a reasonable objective, it is noted that it can be practically impossible to be obtained in a heavily urbanised area, with structures in close proximity to the levees, either for historical reasons or because of political, social, environmental etc reasons. In this case, the designer must not accept these other constraints without designing an alternative way to manage levee stability, levee performance and levee safety

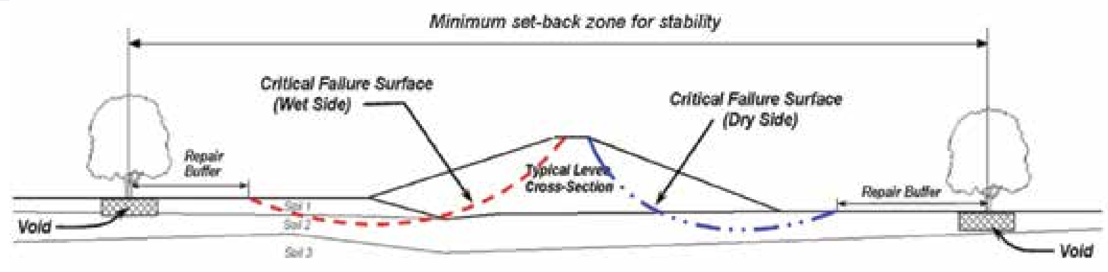


Figure 9.8 ***Stability evaluation for determining limit of minimum set-back zone***

- the location closest to the levee cross-section where the critical seepage exit gradient, determined by seepage analysis (Sections 8.3 and 9.7), is less than the target design value, for example 0.5, as shown in Figure 9.9, plus sufficient room to make repairs (Figure 9.9).

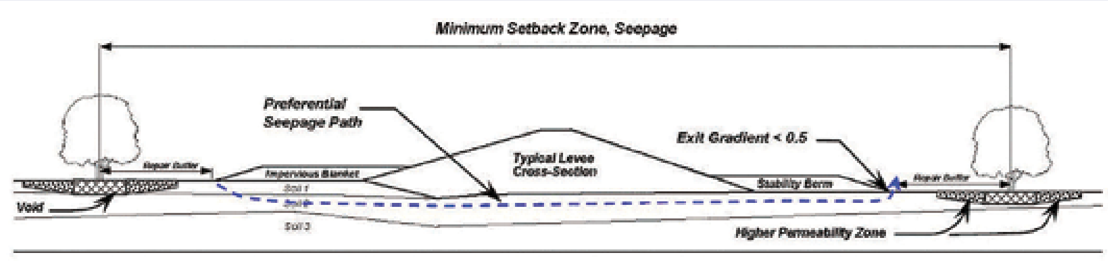


Figure 9.9 ***Seepage evaluation for determining limit of minimum set-back zone***

9.4.3 Interior drainage systems

Detailed guidance on interior drainage systems associated with a levee – including reservoirs, channels, pipes, gates and pump stations (Section 3.1.1.3 and Sections 3.4.1.4 to 3.4.1.8) is beyond the scope of this handbook. However, in setting levee layout and alignment, it is important to recognise that areas behind levees must often accommodate drainage of water resulting from seepage through the levee, or from rainfall which drains directly into the channel system to the landward side of the levee (Figure 9.10). Regardless of the type of structure used to convey water across/through the levee, adequate channels must be constructed to convey the water to the outlets or control structures to avoid localised flooding. Furthermore, because a levee creates a barrier during flood events, water coming from the landward side is either stored for later gravity discharge or is pumped across the levee. The capacities of the pipes and pumps (mean and peak flows and durations) required to return this water back to the river or sea across or through the levee are determined from:

- studies of runoff from storm rainfall that might reasonably occur during flood (not discussed in this handbook)
- through-seepage or under-seepage studies (Sections 8.3.1 and 9.7) of conditions during high water levels on the waterside side of the levee.

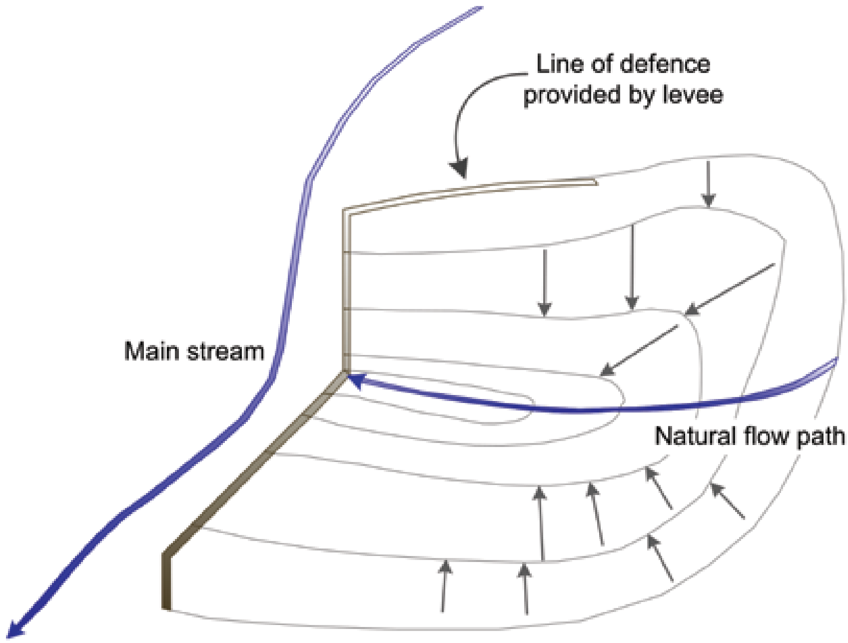


Figure 9.10 Relationship of levee to the interior drainage system

9.5 LEVEE GEOMETRY

The identification of levee alignment (or a number of potential alignments) will also have established some preliminary ideas about levee crest level and geometry. The next stage is to investigate in more detail and determine the **crest level** and **typical cross-section** of the levee. The process of establishing levee geometry on the basis of the hydraulic loads is a starting point in the design process. Resulting geometries will then have to be checked for stability, settlement, seepage etc, and these calculations may require subsequent geometrical changes and close interaction between the hydraulic and the geotechnical designers. The design process may require a number of iterations to achieve a balance between levee alignment, geometry, stability and resilience, and this will require consideration of the issues discussed in Sections 9.6 to 9.10.

An appreciable amount of wave action can affect the design of not only the crest level but also the cross-section because of the potential for the waterside slope and berms to affect the level of wave run-up. For this and other reasons the setting of crest levels of coastal and riverine levees are discussed separately in this section, although geotechnically there are many common features between them.

A further decision is whether the levee should be a pure earthworks structure or whether it should have composite features (eg where there are constraints on the total width or footprint of the levee). A range of possible options are available and have already been described in detail in Sections 3.2 and 3.3.

9.5.1 Setting crest levels of riverine levees

The setting of **crest levels** for riverine levees should be based on delivering an appropriate level of flood risk reduction, as established by flood risk analyses (Sections 2.1 and 5.1), along with meeting all other relevant constraints (Section 9.1).

9.5.1.1 Setting minimum crest elevation of levee

The setting of minimum crest elevations requires a series of calculations and hydraulic modelling of flow rates and water levels in a river for a range of return periods, using the tools provided in Section 7.3. Once this information is available, there are two approaches to the determination of minimum levee crest elevations:

Deterministic: a deterministic design event may be established by National or Regional policies and depends on characteristics and factors within their jurisdiction. Height is set based on a design water level determined using discharge–probability and water level–discharge relationships for a specific event probability, typically a design event such as the 100-year return period (one per cent chance event) or higher.

Probabilistic: a target crest level for the levee is typically first established using a deterministic method, but based on the water surface profile associated with the 90 or 95 per cent assurance level for the critical flood event (say the one per cent chance event) derived from only hydraulic and hydrologic factors. The height is then confirmed or adjusted using a risk analysis (Section 5.1) of the flows and water levels in the river in relation to the crest levels and the fragility of the levee. (The analysis may use some form of Monte-Carlo simulation, and in countries such as the UK and the USA it is linked to some form of cost–benefit analysis).

9.5.1.2 Additional allowances for crest elevation

Once the minimum crest elevation has been determined, various allowances should be added. Figure 9.11 shows a typical plan and profile view of a levee, illustrating the relationship between the calculated water surface profile from hydraulic models and possible levee crest elevations. It shows that the following other expected influences should be considered when setting crest elevation:

- morphological changes (Section 7.3) over time in bed elevation along the river thalweg, which can result in changes in water levels for the same flow conditions
- changes in water surface elevations for any given return period event, arising from changes in river flows, owing to climate change
- allowances for factors such as run-up due to local wave action (Section 3.1.3.1 for more details)
- provision for settlement of the levee or its immediate foundation (related to applied loads or levee self-weight) or subsidence beneath the levee foundation (eg due to abstraction of groundwater or minerals or due to decomposition of peat) – *settlement* (Section 9.12.1) will be heavily influenced by the ground conditions and the levee material and, in general terms, the lower the levee the smaller the settlements will be, so lower crest heights will be more useful for a levee on soft soil than on hard stiff soil
- local and/or national (minimum) requirements for *freeboard*, taking care that these allowances do not duplicate any calculated allowances (Box 9.13).

Box 9.13 ***Freeboard***

Historically, many levees have been designed to incorporate an additional height, called freeboard, over and above the minimum crest level. Definitions for freeboard vary from country to country, because it is a matter of design and not just (for example) the difference between crest level and the flood level in a given design flood event.

In Germany, freeboard is the vertical distance between the crest on the waterside and the design flood level resulting from the protection level plus wind set-up plus wave run-up plus super-elevations due to settlement, subsidence, road construction, cover of sealings etc. Hydrologic or hydraulic uncertainties are enclosed in the design water level.

As a general rule, in the USA, freeboard includes two elements. The first is to provide an uncertainty allowance of confidence factor in the determined design water level. The second is to compensate for physical effects that can be quantified by calculation or measurement (such as settlement, depth of desiccation cracks, wave heights for rivers etc).

For example, historically, in the USA, urban levees have been required to have a minimum of 3 ft (1 m) freeboard, compared to agricultural levees. Commonly in the UK, a nominal freeboard of between 300 mm and 1 m has been applied to cover “wave height and settlement”.

However, this simple requirement is potentially vague and may discourage the determination of the final design crest level in a rigorous manner. Furthermore, it may complicate proper assessment of the requirements for levee superiority (discussed in this section). For these reasons, and as explained in this section, the requirement for a freeboard to be incorporated into a design is gradually being replaced by a requirement for designers to set construction crest levels from a risk-based assessment of the likelihood and consequences of levee overflow and realistic predictions of levee behaviour and performance (such as settlement of the crest with time).

9.5.1.3 Superiority considerations

Levee **superiority** is a principle of adjusting levee crest elevations along river reach to ensure that initial overtopping of levees at flows above the design flow is avoided in hazardous locations (by having 'superior' crest elevations) and instead takes place in the least hazardous locations (USACE, 1986) or at designated spillways. The application of this principle may simply mean providing higher levee crest elevations at all points except where initial overflow is desired. However, more complex situations may arise, as in the following examples:

- where two separate levee systems exist across a river from one another, one protecting highly urbanised areas, the other mostly agricultural areas, first stage analysis is likely to have determined that both levees should have similar minimum levee elevations. However, when determining additional allowances, a value judgement could be made to allow overflow to the agricultural area before the urban area. The additional storage of floodwater so generated could also provide a higher degree of protection to the urban area by preventing water levels from rising quite as high for a given river flow
- where there are adjoining but independent levees, it is possible that a chain failure may arise with failure of one levee successively rupturing the next and so on. Levee superiority can be used to reduce this potential
- where there are embedded structures such as pumping stations, gravity drains and closure structures, the levee may be made a little higher on the upstream and downstream sides of the embedded structures to avoid overflow around, and potential damage to, the potentially expensive embedded structure. The USACE (St Louis District) typically adds this kind of levee superiority for lengths of 30 m to 50 m in the vicinity of such structures.

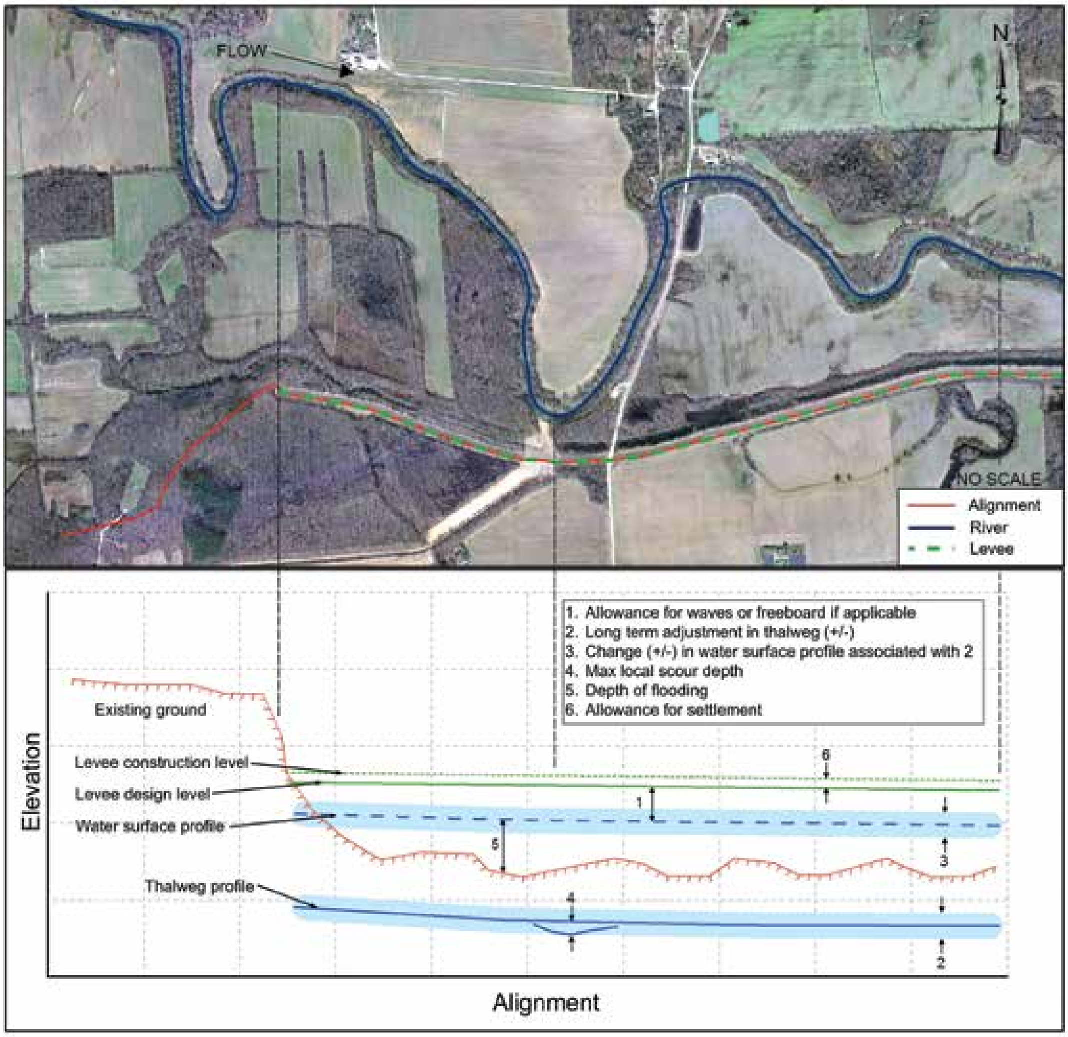


Figure 9.11 Example of components to be taken into account in setting riverine crest elevations (courtesy B M Hall, USACE)

It is important to recognise that unexpected water surface profiles can result for flows that are only slightly above the design discharge can do suprising things (Box 9.14). So, good practice (Box 9.15) is to perform additional hydraulic model runs with events that produce water surface profiles at intervals higher than the design event and ensure that the levee crest elevations are then set to ensure that water only overtops at the least hazardous location. However, in such locations, the robustness of the landward slope will need to be carefully considered, as this part of the levee will be critical in providing resilience during periods of overtopping, where appropriate levee spillway segments (Section 9.14) could be introduced at such a location.

Box 9.14 US example of steep water surface profile for flow above the design discharge (USACE, 1986)

In one large flow event, a water surface profile was measured at the downstream end of the levee 0.6 m above that for the design discharge. However, the flood produced a steep longitudinal water surface profile down the river and, at the upstream end of the levee, created water levels 3 m higher than those predicted for the design discharge. Had the levee project been designed with a constant 1 m freeboard rather than a varying freeboard, a flood only slightly above the design event would have overtopped the levee at the upstream end, flowed at high velocity through the town, filled the leveed area and run back over the top of the downstream portion of the levee.

Box 9.15 Use of multiple water surface profiles to determine superiority in levee crest profile (USACE, 1986)

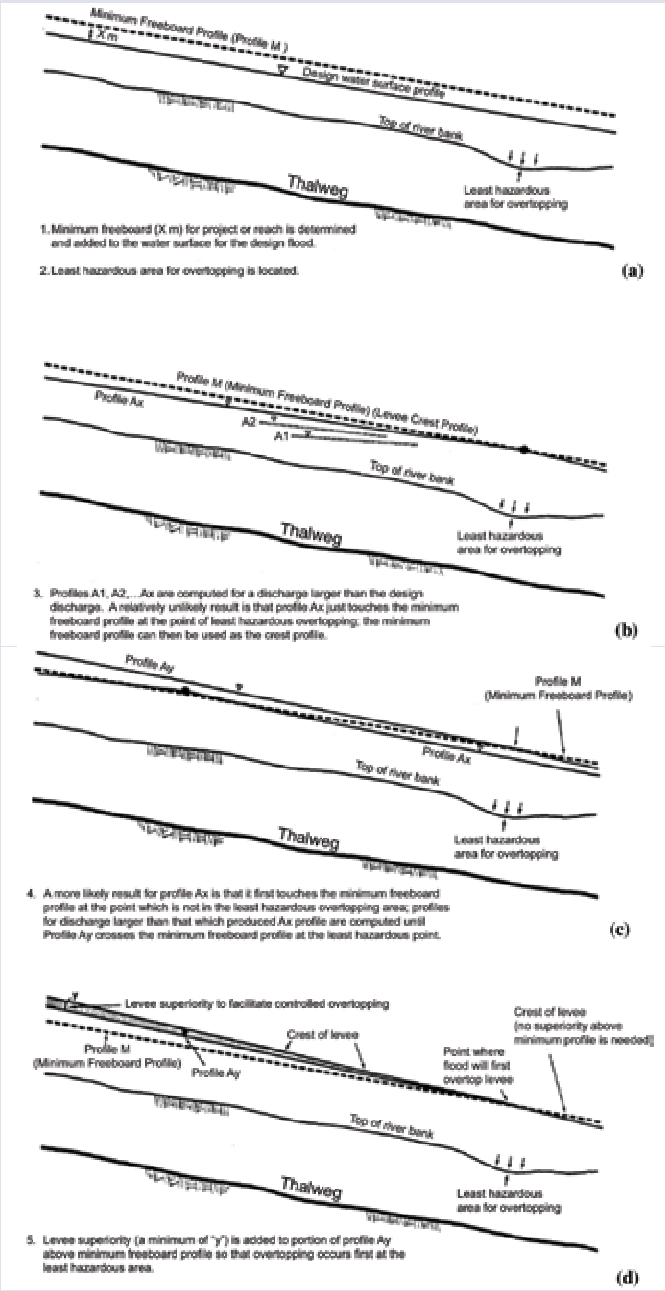


Figure 9.12 Levee crest level superiority determination (from USACE, 1994)

Profile M in Figure 9.12a shows the minimum levee crest profile with constant freeboard and the least hazardous area for levee overflow. Once these have been established, a series of water surface profiles (profiles A1, A2, Ax, Ay in Figure 9.12b) above the design discharge should be computed. One of these profiles, profile Ax, will just touch profile M. If this point coincides with the desired overflow location, and all other points along profile M are above profile Ax, then profile M is set as the levee crest. If profile Ax touches profile M at a location other than at the desired site, additional profiles with increasing discharges are considered until profile Ay crosses profile M in the reach of desired overtopping (Figure 9.12c). The portion of profile Ay that is higher than profile M represents a levee crest which is at exactly the same level as the incipient overflow. Levee superiority (Figure 9.12d) is then added to the portion of profile Ay above profile M to allow initial overflow only at the desired location. The added height should not be in the form of abrupt jumps in levee height, but should have a gradual increase.

9.5.1.4 Spillways

A spillway on a riverine levee is of one of two types:

- security spillway, designed to help control extreme situations in which a levee is overtopped by ensuring that the water overtops in the least vulnerable area
- bypass spillway, designed to divert high river flows into an alternative bypass channel or a safe area for temporary storage.

The details of these types of spillways and their design are set out in Section 9.14.

As with levee superiority design, discussed in the previous section, when selecting the location of any security spillway, consideration should be given to the consequences of flooding deriving from flow over the spillway. Typically the spillway will be located so that the water overtops into a flood expansion zone (Figure 9.13a) with low population and low economic value, typically surrounded by areas of natural high ground or other levees. Flood expansion zones are often areas that have flooded historically and naturally and they include sensitive areas such as housing or critical infrastructure; these elements can be protected by secondary defences such as short local levees (Figure 9.13b).

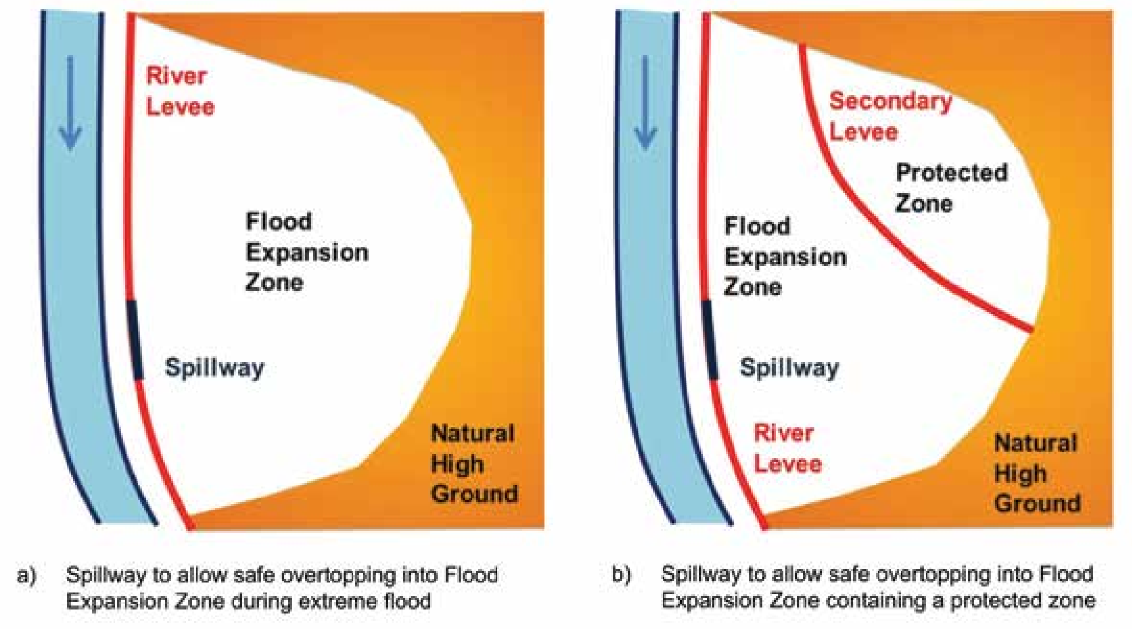


Figure 9.13 Use of spillways to divert flood flow into a flood expansion zone (Degoutte et al, 2012)

In order to maximise the effect of either type of spillway, it is important to consider the hydraulic impacts of the spilled volume of water on the overall flow of water in the river system, both upstream and downstream of the spillway. This typically necessitates computerised flood modelling to determine both the impact of the floodwater diversion on the flow in the river (Section 7.3.9) and the flow of the spilled water over the ground in the flood expansion zone (Section 8.11).

Geomorphological conditions will also influence the location of a spillway, as it is usual to locate a spillway on a straight section of a river to avoid the effects of riverbed erosion or deposition at the spillway location. This is particularly important with rivers that carry high sediment loads.

The location choice also needs to take into account the requirements of the hydraulic and civil engineering design of the spillway, as explained more fully in Section 9.14.

Further discussion on the issues related to selection of the spillway location, particularly in relation to managing flood risk, is given by Hall *et al* (1993) and Degoutte *et al* (2012).

9.5.1.5 Final setting of river levee crest elevations

The finally selected levee crest profile should be a combination of the minimum crest profile together with the additional allowances and superiority adjustments discussed above. An example of a design of a levee/flood wall system modification following these principles is given in Box 9.16.

Given the foregoing comments, consideration should be given to incorporation of low points (such as a reinforced spillway) or a fuse-plug (a weak point which would collapse or be breached more easily than the surrounding levees).

Box 9.16 ***Analysis of impact of levee raise on downstream flooding, Wyoming Valley Susquehanna River, Pennsylvania (USA)***

An analysis was performed to determine the impact of a levee raising project on flooding at downstream locations. The Wyoming Valley levee system in north-eastern Pennsylvania was overtopped by tropical storm Agnes in 1972. A levee raise was designed in the 1990s, with construction completed in 2003. Approximately 24 km of levee and flood walls were raised 0.9 to 1.5 m. During the design, it was determined that the proposed levee raise would increase flooding at downstream locations for discharges greater than the existing levee-top capacity at Wyoming Valley and smaller than the levee-top capacity of the proposed levee system. Although impacts were determined for multiple downstream communities, this case study focuses on the impacts at Sunbury, Pennsylvania. Sunbury is 112 km downstream of Wyoming Valley and has a levee/flood wall system.

A 1D unsteady flow model was used to model the reach of river from Wyoming Valley to Sunbury, Pennsylvania (112 km) (the DWOPER model, developed by the National Weather Service, was used.) Modelling was performed for the hydrograph for a reoccurrence of tropical storm Agnes, for both the existing levee conditions and the proposed project. The modelling showed that the levee raise would significantly increase peak flows downstream at Sunbury. Specifically, it indicated that a flow of 18,600 m^3/s (existing levee top capacity) would increase to a flow of 19 620 m^3/s after the levees in the Wyoming Valley were raised to contain a design discharge of 9555 m^3/s.

Further work included developing a one-dimensional, steady-state, step-backwater model (USACE, 1990) for the Sunbury levee/flood wall system. This was run for a flow of 19 620 m^3/s. The computed water surface profile was compared to the top-of-protection profile for the Sunbury system to evaluate whether modifications were needed to contain this flow, and increases in flood wall height were recommended.

Figure 9.14 ***A portion of a flood wall decorated by Citizens of Sunbury, PA, after it successfully protected them and their homes from the flooding caused by tropical storm Lee, September 2011 (courtesy Baltimore District, USACE)***

9.5.2 Setting crest levels of coastal levees

For coastal levees, the crest level will be set on the basis of hydraulic studies which consider the joint effect of wind, water level and wave climate (Section 7.4), together with run-up and overtopping calculations and criteria (Section 8.2.1). As for river levees, the setting of *crest levels for coastal levees* should be based on delivering an appropriate level of flood risk reduction, as established by flood risk analyses (Sections 2.1 and 5.1), along with meeting all other relevant constraints (Section 9.1). Minimum crest level(s) for levees are typically designed to limit wave overtopping (calculated as set out in Section 8.2.1) to an acceptable level under a specific return period event (eg a 1 in 100 year coastal storm). Account must be taken of circumstances throughout the design life. Indicative design return periods may be agreed on a national or regional basis but may vary depending on the importance of any individual levee. In setting the design crest level, it should be recognised that there remains a significant probability (Box 2.8) that more extreme events will occur during the design life and these may cause wave overtopping at rates higher than the design rate.

Box 9.17 ***Typical steps for establishing levee height for coastal levees***

Step 1: Calculate water levels and waves at the levee toe

a Using information and procedures from Section 7.4, assess the following:

- i mean sea level (local datum)
- ii tidal variations in water level
- iii storm surge and wind set-up
- iv waves (including any tsunami potential) and wave set-up.

b Apply joint probability method or other statistical analysis to evaluate the joint probability of water levels and waves.

Step 2: Evaluate overtopping criteria (involves interaction with waterside levee slopes and berms)

a For initial estimate of crest height, estimate overtopping rates from numerical models or empirical equations (using procedures in Section 8.2.1).

b Compare overtopping rates with limiting criteria based on grass cover and/or armouring of the levee and adjust levee elevations/armouring as required.

Step 3: As required, adjust crest height and waterside levee slopes and berms (Section 9.5.3)

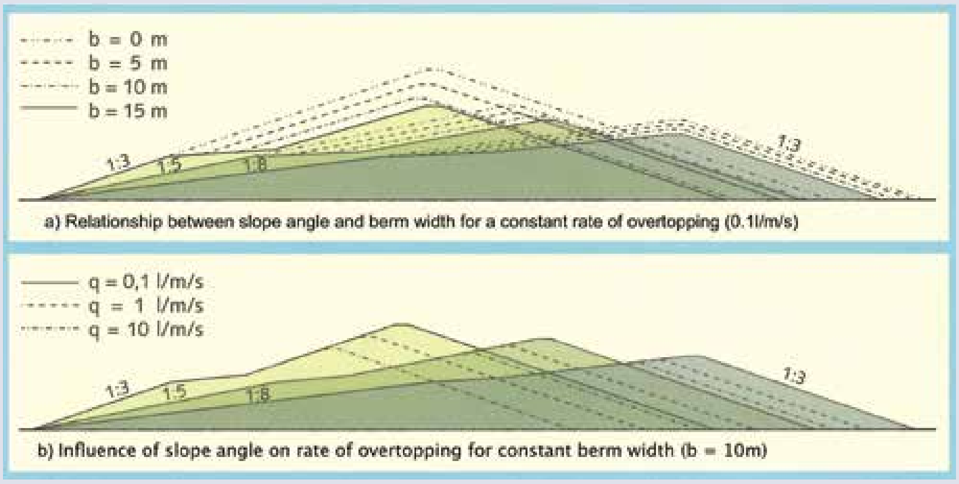
Step 4: Determine final levee crest level

Final levee crest level = calculated still water level + tide effect + surges + wind set-up + appropriate allowance for wave run-up/overtopping (including any embedded wave set-up calculation).

The crest level of coastal levees will also generally contain a freeboard allowance (for the same reasons as those given for riverine levees). There may also be additional requirements for slopes and berms to dissipate the incoming wave energy. Use of the tools in Section 8.2.1 will reveal that, from a hydraulic perspective, there is a complex interaction between the selection of the crest level and the selection of the waterside slopes and berms. Decisions about the adopted side slopes will also interact with the selection and design of the surface protection system (Section 9.6). A typical process followed for determining the crest level and cross-section of a coastal levee is set out in Box 9.17. Box 9.18 then shows an example of how decisions about levee crest height are, in the case of coastal levees, interactive with decisions about the waterside slope.

Box 9.18 ***Example of interaction between waterside slope and crest level of a coastal levee***

Figure 9.15 presents a theoretical example of the influence of the geometry of the waterside slope on the required crest level and width of a coastal levee. Note that this example does not consider the influence on the design of the external erosion revetment protection (shallow slope means that the revetment can have a smaller thickness) and on stability (less height may reduce the dimensions of required stability berms).



Notes

1 The top of Figure 9.15 shows the influence on crest height and levee width for varying waterside slopes (1:3, 1:5 and 1:8) and berm widths b, varying between b = 0 m and b = 15 m), for a constant overtopping discharge of q = 0.1 l/m/s.

2 The bottom of the figure shows the influence on crest height and levee width of (waterside) slope (1:3, 1:5 and 1:8) and overtopping discharge q = 0.1, 1 and 10 l/m/s) for a constant berm width b = 10 m).

Figure 9.15 ***Theoretical coastal levee profiles (from TAW, 1999, courtesy Rijkswaterstaat)***

Additional considerations

For riverine levees there are some additional considerations for setting crest levels:

- changes in water surface elevations for any given return period event arising from changes in mean sea levels, and storm surges, associated changes in tidal propagation and wave conditions due to climate change (see also the discussion in Section 3.1.3.1)
- provision for settlement of the levee or its immediate foundation (related to applied loads or levee self-weight) or subsidence beneath the levee foundation (as discussed for riverine levees in Section 9.5.1 and in Section 9.12.2)
- local and/or national (minimum) requirements for *freeboard*, taking care that these allowances do not duplicate any calculated allowances (Box 9.13).

9.5.3 Establishing levee cross-section

Determination of cross-section of levees including crest width, gradient of side slopes and location and width of any berms and any landside drains is an optimisation process which will be affected by the available space (footprint of the levee) and a number of factors related to resistance to failure mechanisms.

External erosion (Sections 8.4 and 9.6)

In the case of coastal levees, any requirements (discussed in Section 9.5.2) for dissipation of wave energy determined in conjunction with the assessment of crest elevation can have a significant impact on levee cross-section. But it is important to appreciate that limitation of wave run-up is not just a matter of shape but also of the porosity and effective roughness of the erosion protection materials used on the waterside face. Since the external erosion protection system must also be stable itself, there is an interaction between its design and that of the overall levee cross-section. Note that it is also possible to use wave attenuating measures (eg emergent Bor submerged breakwaters) in front of the levee to limit wave run-up and overtopping (for more information see CIRIA; CUR; CETMEF, 2007). An example of optimisation of a coastal levee cross-section is shown in Box 9.19.

For both coastal and fluvial levees, there is an interaction between the design of the cross-section and the design of the external erosion protection to the crest and landside slope, against overtopping or overflowing water. The velocity of any overflowing or overtopping water over the landward face can be reduced by flatter landward slopes and the introduction of berms. So it is possible to optimise control of the erosive forces by the gradient and profile of the slopes against the resistance of the protection material (grass, stone, concrete etc). A reduction of the area of landward revetment requiring special external erosion protection measures (eg of armourstone or special concrete units) will be especially valuable in situations where there is limited availability of such materials.

In the most extreme conditions, the size of the landward slopes and berms will also affect the time it takes for any back-cutting erosion that does take place to reach and start cutting through the levee crest (Section 8.10).

In all cases the thickness of the erosion protection measures (Section 9.6) must be taken into account in building up the overall levee shape. In the case of coastal levees these thicknesses can be significant.

Internal erosion (Sections 8.5 and 9.8)

The various widths of the levee, its slopes and landside berms will affect its resistance to internal erosion. In general terms, the wider the levee, the lower the hydraulic gradients across the levee and hence the lower the probability of internal erosion starting. There is also a strong interaction with the permeability and susceptibility to internal erosion of the levee material (Section 9.13.2), of the underlying ground conditions (Sections 7.1 and 7.7 to 7.9) and of any interfaces between materials. Improvements of the soil/foundation beneath the levee is one option that can be considered to limit the width of the levee cross-section.

Internal erosion processes are also affected by the duration of any river flood hydrograph, which will affect the extent of water pressure and *seepage* flow build-up (Section 9.7) within the levee during flood conditions.

Mass stability (Sections 8.6 and 9.9)

The height of the levee and the form of the slopes and berms will also affect the overall stability of the levee. As for internal erosion, stability will also be affected by the underlying ground conditions and the material selected for the levee. Stability is often at its lowest during construction (Sections 9.5.5 and 9.9.1), when particularly high internal pore water pressures can be present.

Transitions (Section 9.11)

Transitions between various cross-sections along the levee should be as smooth as possible because discontinuities will disturb the flow of water along or across the levee and cause potentially damaging turbulence. Sudden changes in cross-section should be avoided.

Spillways for riverine levees (Section 9.14)

As discussed in Section 9.5.1, spillways may be considered at specific locations along riverine levees to allow preferential overtopping into a designated area where the risk to life or property is low. Section 9.14 provides information on dimensioning such a spillway and designing an appropriate amount of resilience for it. The provision for safe evacuation of spillway discharge on the landside of the levee will also require appropriate analyses (Chapter 6) (spillways offer no advantage for coastal levees as there is no possibility of reducing the quantity of water likely to overtop the levee).

Box 9.19 ***Example from the Netherlands of determination of the cross-section of a levee under wave action***

The example considers a levee along a large lake on extremely soft (organic) soil. Rehabilitation of the levee required raising of the crest, which subsequently required additional height to compensate for settlement and large stability berms.

To optimise the design, a very flat waterside slope (Figure 9.16) was chosen to reduce the wave run-up, permitting the required height to be reduced by approximately 3 m. Because of the reduced height and increased stability of the flatter slopes, the width of the stability berms was also diminished. This also improved the constructability of the levee. A further advantage of the very shallow waterside slopes was that slope protection against wave attack was not needed. (In a soft soil environment with considerable settlement which often disrupts external erosion protection systems, this was a significant advantage both for construction and maintenance.) Furthermore, the absence of structural elements made it easier to be able to raise the levee in the future, if required due to climate change, sea level rise and increase of wind speed and waves.

However, this design introduced new challenges, particularly in relation to the presence and location of the existing historical levee, which dated back to the 12th century. Removing a major part of the existing levee was not permitted, and therefore the optimised design involved the construction of a completely new levee in front of the existing levee.

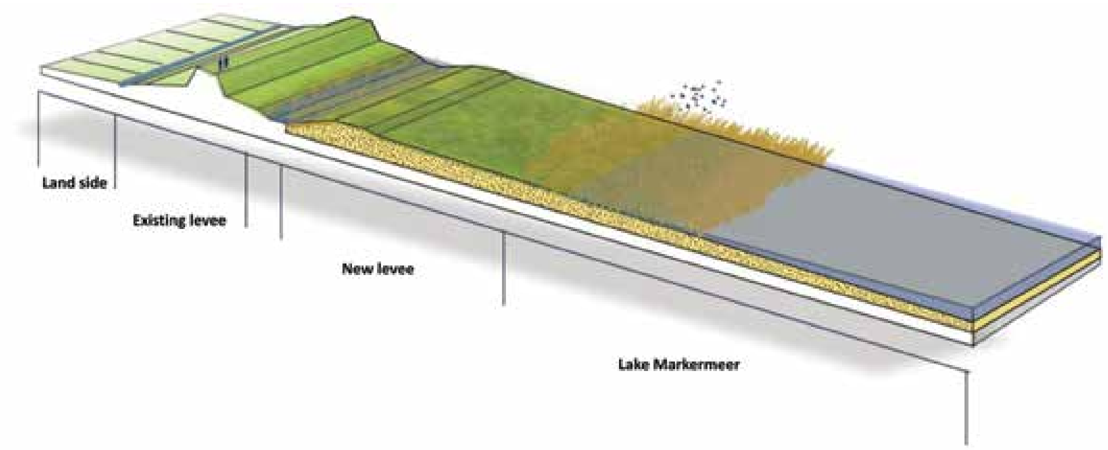


Figure 9.16 ***Cross-sectional profile of old and new levees (courtesy T GA Cents, ARCADIS and Hoogheemraadschap Hollands Noorderkwartier Vliet)***

Relationship to minimum requirements for O&M and construction (Section 9.5.4)

Minimum requirements for crest and berm widths and side slopes for operation and maintenance (Chapter 4) and construction (Chapter 10) are described in Section 9.5.4. The adoption of standard levee cross-sections should not imply that calculations to assess stability and under-seepage analyses are not required. The use of standard sections should generally be limited to levees of moderate height (not more than 5 m) in reaches where there are no anticipated under-seepage problems, soft or weak foundation soils or undesirable borrow materials (high moisture contents or high organic contents). However, when hydraulic and ground conditions are similar to those of existing levees nearby, analyses of failure mechanisms may only be required to the extent necessary to confirm satisfactory behaviour.

In addition to being used in levee design, standard levee cross-sections can be very useful in establishing an initial cost estimate, or quickly generating a suitable basis for emergency measures or urgent maintenance repairs.

An example of the development of a levee cross-section in a complex situation is given in Box 9.20.

Box 9.20 Major levee modification project, Natomas, California, USA

The Natomas Levee Improvement Program (NLIP) 2006–2013 was a multi-phase flood system infrastructure project, initiated by the Sacramento Area Flood Control Agency (SAFCA) to correct major weaknesses in the levees that protect the Natomas Basin of Sacramento, California. The Basin has been rapidly urbanising and is a critical part of the metropolitan Sacramento regional economy, containing 100,000 residents, hundreds of local businesses, a key transportation hub, including the Sacramento International Airport, and Interstate 5, the main interstate highway on the west coast of the USA. The Basin is adjacent to the Sacramento River, the state's largest river, and the managed floodway draining the Great Central Valley of California.

Recognising that the consequences of a flood in the Basin would be catastrophic, and could happen any time, SAFCA started a planning and design process to construct levee improvements to a 200-year level of flood protection as quickly as possible. Early on in the planning phase, managers recognised that swift and successful execution of a programme of this magnitude required implementation approaches that advance the achievement of multiple objectives, while simultaneously meeting multiple state and federal mandates.

The NLIP was funded through bond funds secured by the California Department of Water Resources (DWR) and monies raised from local property benefit assessments levied and collected by SAFCA.

In 2006, SAFCA, with support from its multiple federal and state partners (US Army Corps of Engineers, California Department of Water Resources and the California Central Valley Flood Protection Board), began planning and design of improvements to the 42 mile (68 km) levee system and 53 000 acre (21 000 ha) Basin to generate a 200-year level of protection (see Figure 9.18). The socio-economic and technical challenges included:

- 19 miles (30 km) of deficient levees with deep foundation instability (under-seepage), inadequate freeboard or levee height and riverbank erosion concerns
- US Army Corps of Engineers (USACE) and Federal Emergency Management Agency (FEMA) analyses that significantly downgraded the level of flood protection of the levees in the Basin, requiring homeowners to purchase costly, mandatory flood insurance, and a moratorium on commercial and residential construction
- a new USACE federal mandate requiring removal from levees of all waterside encroachments (fences, driveways, etc) and levee vegetation greater than two inches in diameter
- federal aviation safety standards, regulated by the US Federal Aviation Administration (FAA), to reduce wildlife attractants, given the high rate of aircraft bird strikes that are a hazard to flight safety at Sacramento International Airport
- federal and state habitat protection mandates regulated by the US Fish and Wildlife Service (USFWS) and the California Department of Fish and Wildlife, to ensure measures to protect habitat for federally and state-listed endangered species were adequately addressed
- redesigning and relocating major infrastructure facilities, including electrical transmission lines, major irrigation and drainage canals, water pump plants and roads located along the 19 mile levee footprint to accommodate the new landside levee
- the use of statutory 'eminent domain' powers to seize private property, necessary for the public good, to expand the levee footprint
- historic Native American tribal lands and significant cultural resources in the identified levee improvement area.

Early in the planning phase of the NLIP, SAFCA assembled a team of expert engineers, planners, ecologists and environmental specialists to formulate a programme implementation approach that could be achieved in a few short years. This led to the formulation of co-planning and design teams that integrated engineering and habitat function objectives. Collaboration and partnerships were developed with state and federal flood management and natural resource agencies, the Airport and FAA, and a number of key local agencies including tribal leaders, city, county, levee maintenance, habitat protection and utility operating organisations.

Ultimately the selection of the preferred construction alternative required SAFCA to balance the socioeconomic and technical factors in a cost-efficient, minimally disruptive and environmentally responsible way. SAFCA and its partners chose an innovative, landside 'adjacent' levee design (Figure 9.17), instead of expanding the existing riverside levee in place, to avoid the loss of several hundred acres of mature riparian forest and fish habitat, and to avoid large-scale removal or condemnation of residential landscaping and structural encroachments on the waterside of the 'Garden Highway' levee.

Levees improvements in the Basin have consisted principally of an 'adjacent' landside set-back levee (enlarged levee embankment), and as local geology dictated, a combination of deep seepage cut-off walls and the construction of extensive seepage berms (Figure 9.17). These improvements required significant quantities of geotechnically suitable soil material in close proximity to minimise haul distance and optimise overall programme cost efficiency.

A number of innovative and complementary approaches were employed to identify the sources of soil required for the NLIP – approximately 8.6 million cubic yards (7.2 million m^3). These included the Sacramento International Airport's need to modify poor drainage conditions and to manage surrounding buffer lands to reduce wildlife/waterfowl aviation hazards, and borrow sources on agricultural properties that could be graded and subsequently reclaimed for cropland and other compensatory habitat purposes needed by the project.

Box 9.20 Major levee modification project, Natomas, California, USA (contd)

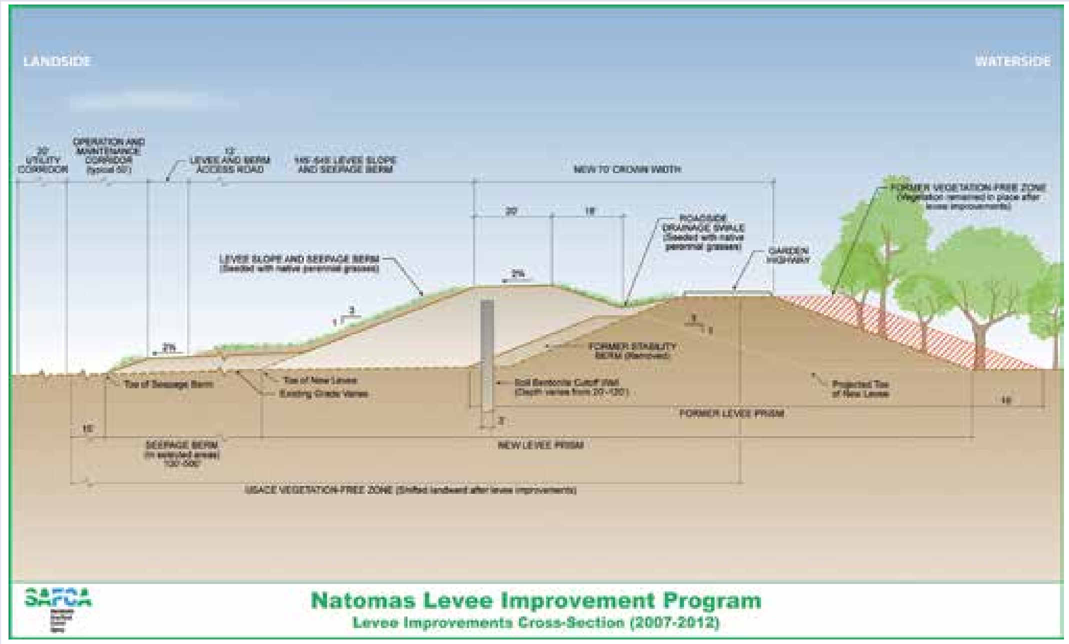


Figure 9.17 Typical cross-section, Natomas Levee Improvement Program (courtesy Peter Buck, Sacramento Area Flood Control Agency)

The NLIP conservation strategy encompassed multiple objectives with the overarching goal of increasing the extent and connectivity of habitat lands in the Basin, and offset pre-project habitat losses for federal- and state-listed endangered species. Three basic elements encompassed the conservation strategy rubric: connection, consolidation and expansion of habitats.

Primary habitats and species of concern were mature woodlands, the giant garter snake (GGS), valley elderberry longhorn beetle (VELB) and Swainson's hawk. All were subsequently enhanced through permanent preservation of existing and/or establishment of new habitat areas. Overall the NLIP has resulted in:

- preservation and planting of 135 acres (55 ha) of new and mature woodlands, including transplanting 1200 valley oak trees up to 20 inches in diameter
- construction and planting of an 8.5 mile (14 km) canal to provide a migration corridor linking population clusters of the GGS in the north and south part of the Basin, additional drainage capacity and allowing elimination of a flight safety hazard parallel to the airport runway system by dewatering and filling a canal
- creation of 135 acres (55 ha) of specially designed managed marsh to benefit GGS and compensate for impacts to GGS and wetland habitats
- creation of approximately 165 acres (67 ha) of high quality foraging habitat bordered by woodland nesting habitat for the state endangered Swainson's hawk
- approximately 600 acres (245 ha) of managed native perennial California grasslands for erosion protection on levee slopes, seepage berms and adjacent O&M corridors.

With the exception of ongoing compensatory habitat projects (as identified above), after six years of construction the NLIP levee improvements were largely completed by 2012, at a total cost of approximately $410m. Additional levee improvements, under the leadership and direction of the USACE, are anticipated to occur in upcoming years as funding becomes available through US Congressional authorisations expected in 2013–2014. Once completed, the levees protecting the Natomas Basin will arguably be the strongest in the western United States.

In 2011 the Natomas Levee Improvement Program was recognised by the ASCE for outstanding flood management in california.

Box 9.20 Major levee modification project, Natomas, California, USA (contd)

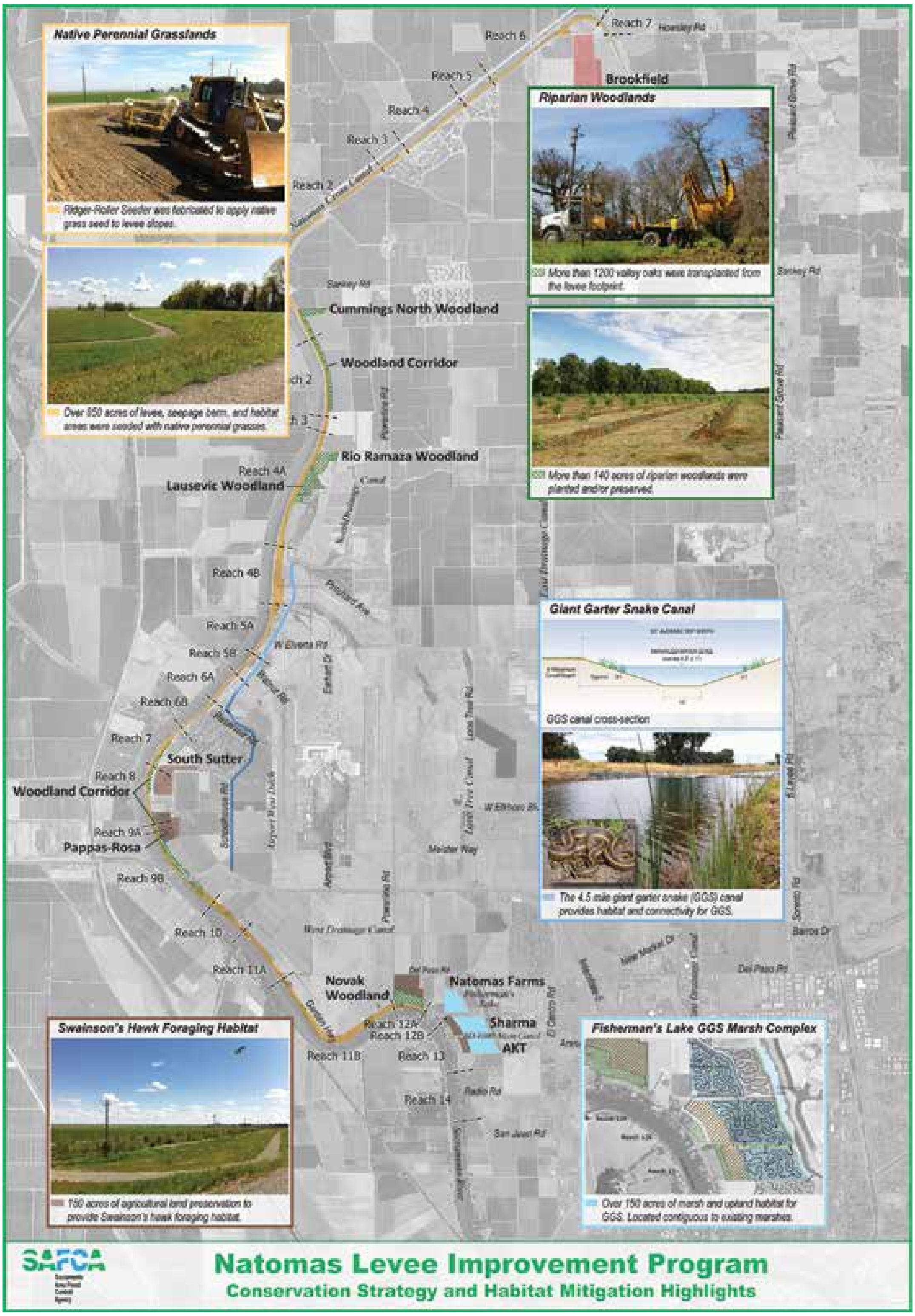


Figure 9.18 Plan of levee alignment around Natomas basin also illustrating environmental works (courtesy Peter Buck, Sacramento Area Flood Control Agency)

Source: www.safca.org/programs_natomas.html

9.5.4 Minimum levee geometries

Levee geometry (Sections 3.2 and 3.3) is commonly controlled by minimum safe operational requirements for emergency access, maintenance and rehabilitation activities such as grass cutting (Section 4.5) and for construction. The critical geometrical features are the slopes (both waterside and landward side), crest width, crest level (or height above the surrounding land) and the dimensions of any berms (berm width, berm level, berm slopes etc). These should be established at an early stage by:

- communicating with the levee owner/operator
- liaising with relevant individuals and authorities
- application of the appropriate regional or national guidelines.

Minimum crest widths do not include any allowance for future raising of levees, a topic discussed in Section 9.5.5.

9.5.4.1 Minimum levee geometries for operations and maintenance

Examples of minimum levee geometries for fluvial levees are given in Box 9.21 for the UK and in Table 9.10 for the USA. These illustrate the following:

- for vehicular access for both normal maintenance operations and emergency operations during floods, minimum crest widths of 3 m to 4 m are required
- side slopes ideally should not have slopes steeper than 1V:3H. One reason for this is that steeper slopes make it more difficult to safely maintain good grass cover by mowing.

Where crest walls are present, minimum crest widths to the landward side of the wall may have to be enhanced in order to avoid difficulties with opening vehicle doors and passing other vehicles.

Box 9.21 Example of operational minimum cross-sectional dimensions (Environment Agency, 2007)

Guidance by the Environment Agency (2007) provides minimum cross-sectional dimensions for the safe operation and maintenance of fluvial levees in England and Wales. For levees set back from the immediate vicinity of the water, these are a minimum crest and berm width of 4 m and slopes no steeper than 1V:3H as shown in Figure 9.19.

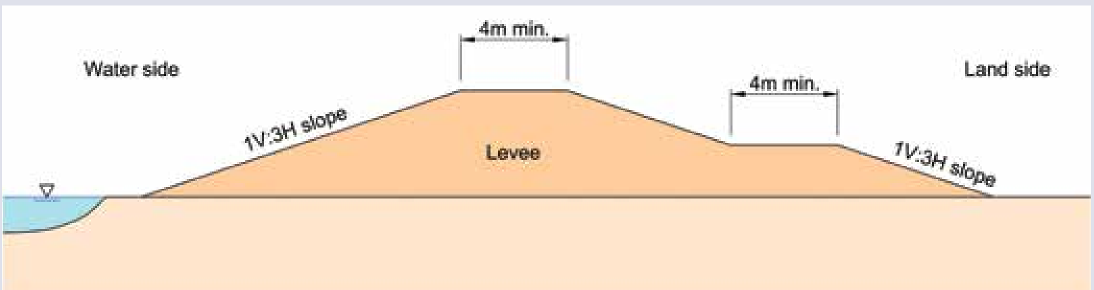


Figure 9.19 Typical minimum levee dimensions (after Environment Agency, 2007)

Table 9.10 Various US guidelines for minimum dimensions for fluvial levees

	USACE (2000)	**Central Valley Flood Protection Board (2010)**	**USACE (2008)**
Minimum crest width	10 ft (3.0 m)	20 ft (6.1 m) (major stream levees) 12 ft (3.7 m) (minor stream levees)	20 ft (6.1 m) main line, major tributary and bypass levees) 12 ft (3.7 m) (minor tributary levees)
Minimum waterside levee slope	1V:2H*	1V:3H (generally) 1V:4H (bypass levees)	1V:3H (generally)
Minimum landside levee slope	1V:2H*	1V:2H (generally) 1V:3H (bypass levees)	1V:3H (new levees) 1V:2H (existing levees with good performance)

Note

* Current standard of practice issued by US Corps of Engineers calls for 1V:3H.

For a given levee system, it is common in the USA to establish several different standard sections (Box 9.22) depending on the degree of risk (agricultural vs urban areas being defended) and the type of construction to be used (compacted, semi-compacted, uncompacted or hydraulic fill).

The implications of using semi-compacted or uncompacted fill for levee construction should be carefully reviewed before adoption. In Europe, higher population densities mean that it is unusual to adopt such an approach to levee construction as these materials will be vulnerable to settlement (particularly on first hydraulic loading), and may exhibit low strengths, high permeability and low resilience to overtopping or deterioration.

Box 9.22 US standard levee cross-sections

In the USA, many districts have established standard levee sections for particular levee systems (Figure 9.20). These have proven satisfactory for the general fluvial regime, the foundation conditions prevailing in those areas and for soils usually available for levee construction (in many cases, the standard levee sections have more than the minimum allowable factor of safety relative to slope stability, side slopes being established primarily on the basis of construction and maintenance considerations).

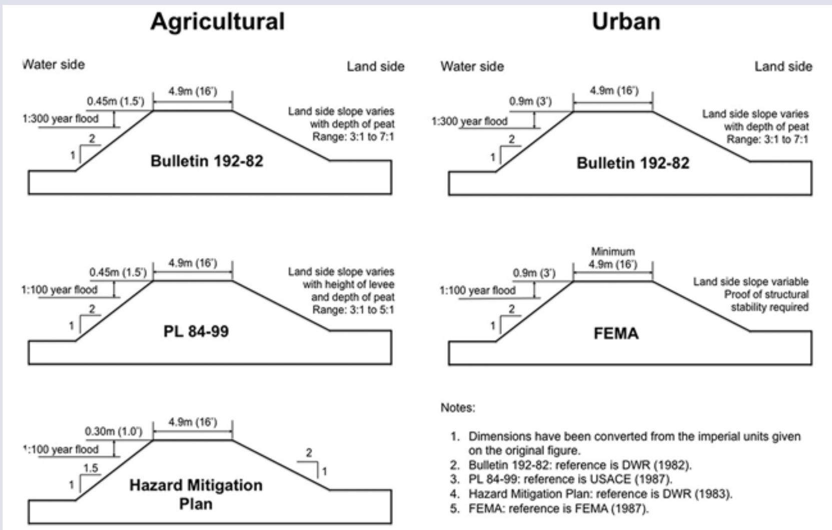


Figure 9.20 Example of minimum standard levee geometries from USA (after CALFED Bay Delta Program, 1999)

1 2 3 4 5 6 7 8 9 10

Access roads

Access roads should allow access to the levee at any time of the year and in most weather conditions for the purpose of inspection, maintenance, flood management and emergency works. The spacing of access roads to the levee should be set in such a way as to minimise the amount of traffic along the crest (which could cause progressive damage) and to provide a redundancy of access locations in case of an emergency during poor weather conditions. The selected spacing will be a compromise between cost, flexibility and emergency access requirements.

Requirements for access roads, turnouts, toe roads and ramps will dictate minimum dimensions and maximum gradients for such features which, in turn, will control crest and ramp dimensions (Box 9.23).

Box 9.23 Typical US access zones and features

In the USA, it is a common requirement for major levees that a minimum of 20 ft (6 m) beyond the landside toe of the levee (or other similar flood protection system) must be acquired for right-of-way purposes. In the case where stability berms and/or relief wells are present, the measurement of the minimum 20 ft wide zone should be beyond the limits of those features (including seepage collection ditches).

Typical levee features that accommodate common O&M requirements are shown in Figure 9.21.

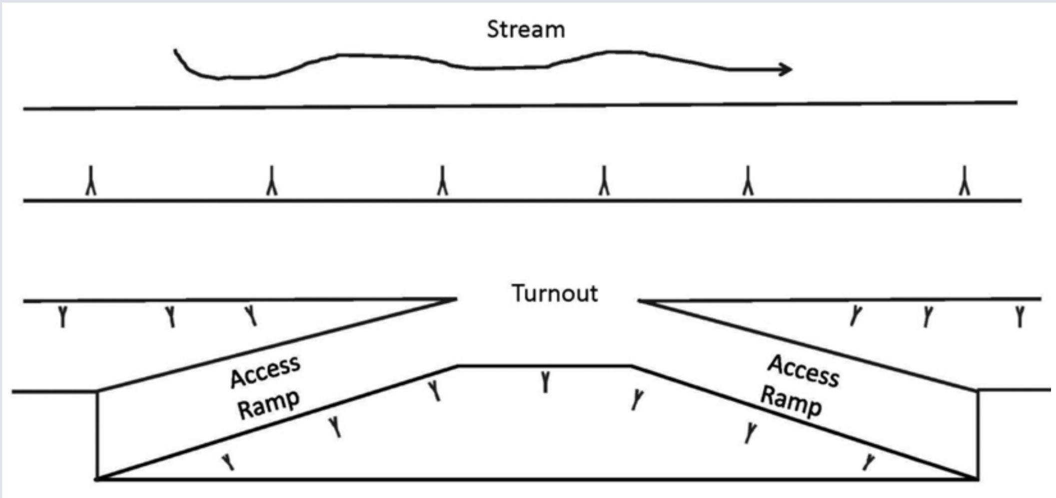


Figure 9.21 Geometry requirements for operations and maintenance (USACE, 2000)

9.5.4.2 Minimum levee geometry for constructability

The early involvement of contractors (Box 9.5), or those with construction experience, may help to identify ways in which best use can be made of existing structures or permanent works to mitigate the overall cost of construction. Levee dimensions should facilitate and accommodate access to construction vehicles, in terms of both space and bearing capacity. Crest widths of 3 m to 4 m are considered to be the minimum feasible for construction using modern heavy earthmoving and compacting equipment, such as rollers; to avoid safety problems, these should not be reduced. Temporary haul roads also require space and materials for their construction. Local construction practices and materials (eg the use of high-plasticity clays) may also require the levee to have flatter slopes.

Earth levees constructed on soft alluvial, estuarine, deltaic or marine sediments may be limited in height to about 3 m to 4 m, depending on the shape of the levee and the thickness of any surface crust. To build to greater heights requires the use of ground improvement techniques (Section 9.13).

The historical approach to this problem has been to construct in a series of lifts or raises and, after each lift, allowing the ground to consolidate naturally over a long period of time before commencing the next. The alternative, if a high level of defence is required immediately, is to adopt much more expensive foundation solutions (such as deep cement mixing) or to construct a composite structure including, for example, a central sheet pile wall. Many of the innovative techniques commonly used for ground improvement beneath embankments built for transport corridors (such as the use of lightweight fill or the use of prefabricated band drains in conjunction with a highly permeable drainage layer) are not suited to levees, but some options are discussed in Section 9.13.7.

9.5.5 Geometry defined by requirements for future levee raising

Future raising of levees to accommodate settlement or increased hydraulic loading will affect or be affected by the initially constructed levee dimensions. For example, if the levee side slopes are 1:3, raising the crest level by just 0.5 m, using conventional earthworks, will mean that the crest width is reduced by 3 m. Options available, if such a change is to be anticipated in the original design, include:

- reduction of crest width if acceptable for operational purposes
- construction of hard crest structures
- building the levee with a crest width wider than the minimum to allow for future raising
- widening the whole levee at the time of the levee raising, which may require the purchase of additional land.

An example of planning for future levee raising is given in Box 9.24. Because of the uncertainties associated with the future requirements and the high costs of providing an initially over-wide levee, the following alternatives might be considered:

- provision of a land corridor which is wider than the minimum requirement (Section 9.5.5) – as well as allowing the crest level to be raised more easily in the future, this approach should avoid difficulties with buildings or infrastructure subsequently being constructed to the immediate landward side of the original levee (see also the discussion in Section 4.2)
- widen the levee but only to the level of an intermediate berm – as well as ensuring the required land-take, this approach will also provide consolidation and strength gain in soft ground conditions to allow future raising of a levee to a height that could not otherwise be achieved in one lift. The settlement of the levee crest that goes hand-in-hand with the consolidation of the alluvial soils can also be remediated after each construction stage, as the crest is automatically re-levelled each time it is raised.

When planning such large and potentially adaptable levees, it is important to consider all aspects of levee performance and behaviour at each of the stages of construction (Section 9.6) as this may significantly control both the geometry and the alignment of the levee.

Careful consideration should be given to the issues of constructability and performance as set out in Section 9.11 and Chapter 10. Appropriate instrumentation and monitoring is discussed in Section 7.9.8 and the use of construction monitoring as part of the design process in Section 9.16.

Box 9.24 *Recent proposals for raising levees on the Thames Estuary*

Proposals for the Thames Estuary 2100 project (TE2100) suggested that new levees incorporate wider cross-sections to deal with uncertainties associated with climate change and the need for future raising. Hydraulic assessments based on future climate change predictions have indicated that a substantial increase in storm surge level is possible for the Thames Estuary by the year 2100. However, the range of possible storm surge levels is considerable. Strategic studies were carried out to investigate and develop ways of managing these uncertainties. One solution considered was the use of new set-back levees constructed some distance (many km) behind the existing defences along the outer estuary of the Thames. However, stability analyses showed that such levees could not be constructed to the target crest level in one lift as they would fail during construction.

A process of stage construction was developed, whereby the levees could be raised in stages over the decades between the present and 2100. This allowed the natural process of consolidation to strengthen the ground beneath the levees, thereby making it possible for subsequent raising to be carried out safely. The stages of this construction process are shown in Figure 9.22. The benefit of this approach is that the process allows future adaptation without the need for immediate construction of major flood defence structures.

In the example, the complete process of initial levee construction and subsequent raising after 15 years and then 30 years are considered. This process has been designed to provide a satisfactory level of safety against failure during construction and during a flood within this time period and the magnitude of settlement anticipated has been calculated and incorporated into the design. It is noted that the purpose of the large berms is to provide an acceptable level of stability both during construction and during the design flood situation.

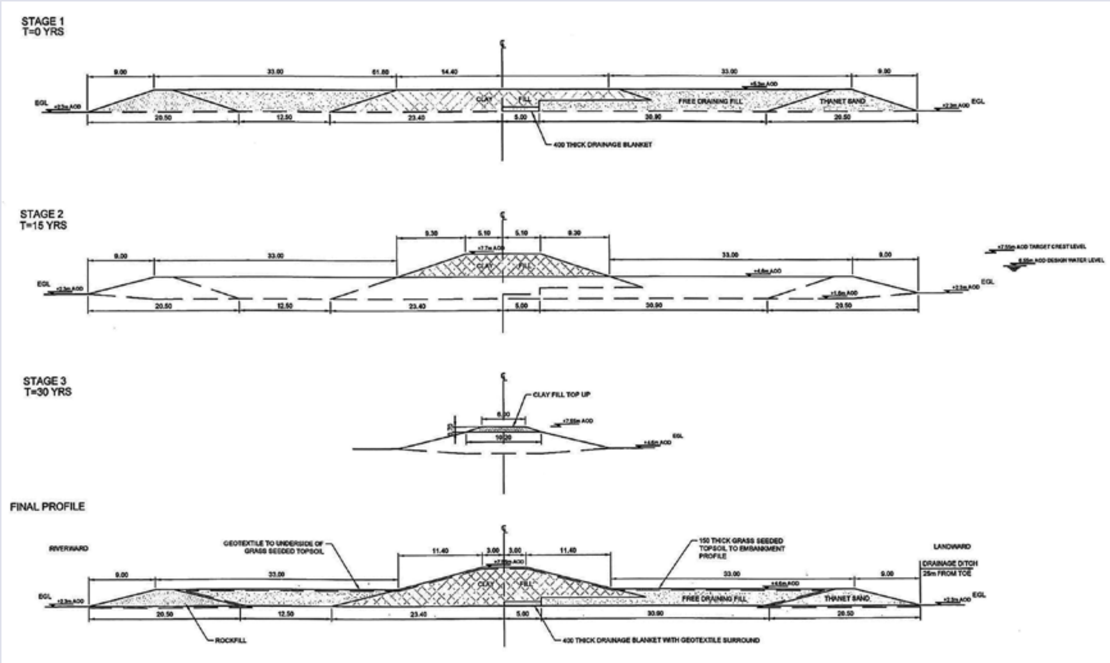


Figure 9.22 *Staged construction of levee on Thames Alluvium (courtesy Environment Agency and CH2MHill)*

9.6 SURFACE PROTECTION MEASURES

The potential for surface erosion or scour of a levee is determined by:

- calculating current velocities and/or wave action on the waterside levee face and overflow velocities or wave overtopping characteristics on the landward face (using the tools in Section 8.2)
- comparing these values to allowable limits for the materials; movement (erosion) can be expected if calculated values exceed allowable limits for the levee material or protection system.

Once it has been determined that erosion and/or scour is a concern for levee safety, it is necessary to consider measures that can reduce or mitigate the effects. This section outlines basic principles associated with selecting appropriate measures to reduce the threats of erosion and scour on levee projects. Sufficient detail is included such that the reader can understand the key levee-specific factors that must be evaluated, but complete coverage of all possible surface protection measures is beyond the scope of this handbook, and suitable references (eg CIRIA; CUR; CETMEF, 2007) should be consulted for further information.

As well as direct protection of the surface (including the toe) of levees, river channels and coasts may change their position or cross-sectional profile over time due to natural morphological processes, and this may impact on levee stability. Methods such as spur levees (Box 9.25) and longitudinal peak stone protection can be adopted to reduce local flow velocities and prevent undermining. Similarly on the coast, methods such as groynes, detached breakwaters and beach nourishment are commonly adopted (as part of overall beach management) to retain materials and/or reduce current velocities and wave action. These topics are not dealt with further in this handbook and the reader is referred to other sources for information on this topic, for example Biedenharn *et al* (1997) McConnell (1998), Rogers *et al* (2010), and CIRIA; CUR; CETMEF (2007).

Box 9.25 Spur levees

Where analysis indicates that extensive reaches of a levee may be exposed to erosive velocities it may be more cost-effective to use one or more spur levees/groynes to deflect those currents away from the levee. Spur levees or groynes are levee segments constructed riverward at an angle to the main levee alignment, protecting the main levee by forcing potentially damaging currents away from the levee. Spur levees are typically used where there are significant changes in the main levee alignment, where the main levee encroaches on the river channel or where changes in encroachment coincide with a change in levee alignment (Figure 9.23).

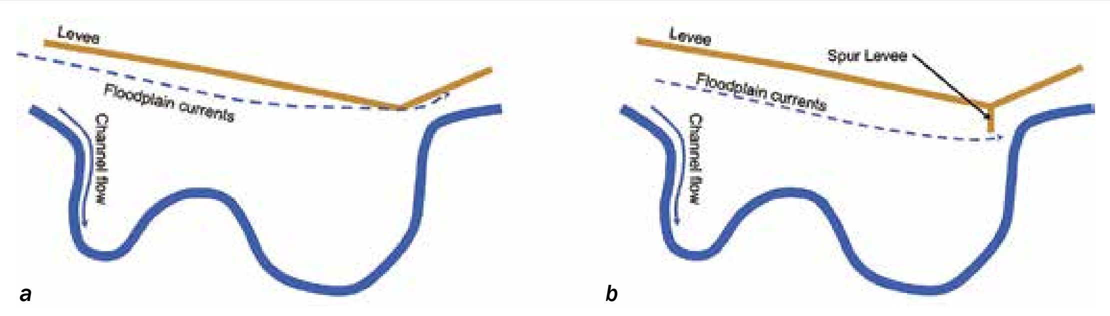


Figure 9.23 Typical use of spur levee or groyne, currents at levee without spur (a), currents at levee with spur (b)

Little quantitative guidance is available on the design of spur levees or groynes to mitigate erosion of levees. The recommended methodology for assessing the effects of spur levees/groynes on velocities is to use numerical models that can be modified to include various configurations (length and profile) of spur levees. Model results will be in the form of calculated velocities and water surface elevations along the main levee. Two-dimensional numeric models can also provide velocities along and around the end of the spur levee which are needed to assess where rock armour is required at the riverward end of the spur.

General guidelines for spur levee design are as follows:

- The **length** and **spacing** of spur levee segments may be determined using numerical hydraulic models to assess the effect of various spur levee configurations on current magnitude and direction.
- The spur levee riverward end should angle upstream (Figure 9.23b). This is because, if the river overflows the spur, flow will be directed perpendicular to the spur alignment. Any angle that would direct flow and currents toward the main levee should be avoided.
- The spur levee **crest elevation:**
  - adjacent to the main levee, should be at the same elevation for a sufficient distance riverwards along the spur to prevent overflow at the spur from impacting on the main levee
  - may be constant or taper down towards its riverward end. A tapered crest permits progressively increasing overflow lengths as water levels increase. This provides opportunity for some tailwater to develop downstream of the spur.
- The spur levee may be constructed of the same or different material used in the main levee.
- Erosion protection in the form of rock armour may be included along the spur, or the spur may be permitted to fail during extreme events.

9.6.1 Alternative surface protection systems

In addition to grass, a variety of different materials, including concrete, stone and asphalt, may be used for the protective surface layer. The surface of the protection may be smooth, rough or stepped. The revetment may be of rigid or flexible construction. Types of protection available for levee projects are shown in Table 9.11. The mass and shear strength of surface protection systems should be taken into account in geotechnical mass stability calculations (Sections 8.4, 9.8 and 9.9).

Table 9.11 General characteristics of surface protection systems (adapted from Pilarczyk, 1995)

Type of cover layer	Critical failure mode	Determinant wave loading	Strength
Sand/gravel	• initiation of motion • transport of material • profile formation.	• velocity field.	• weight, friction • dynamic 'stability'.
Clay/grass	• erosion • deformation.	• maximum velocity • impact.	• cohesion • grass-roots • quality of clay.
Armourstone	• initiation of motion • deformation.	• maximum velocity • seepage	• weight, friction • permeability of sublayer/core.
Gabions or mattresses, including geotextiles	• initiation of motion • deformation • rocking • abrasion/corrosion of wires • UV light.	• maximum velocity • wave impact • climate • vandalism	• weight • blocking • wires • permeability including sublayer.
Placed concrete blocks, including tied block mattresses	• lifting • bending • deformation • sliding.	• overpressure • impact.	• thickness, friction, interlocking • permeability including sublayer/geotextile • cabling/anchor pins.
Continuous concrete or asphaltic paving	• erosion • deformation • lifting.	• maximum velocity • impact • overpressure.	• mechanical strength • weight.

Multiple factors affect the selection of a particular method of surface protection, whether it is to be applied to the waterside face, the crest or the landward slope. These factors include:

- the frequency of action of hydraulic forces on both landward face, crest and rear face, including overflow and/or overtopping situations (infrequent from rare extreme events or more frequent as a means of relieving flood volumes to predefined areas of low risk)
- the likely long-term performance of the levee, including risks of failure under likely normal and extreme future scenarios (acceptability of failure or some damage)
- the nature of the levee and of the foundation soils
- the constructability of the option in the particular site circumstances
- capital and maintenance costs
- availability of the necessary materials and their suitability for the site conditions
- the ability of the levee owner to carry out the necessary operations and maintenance activities
- the appearance of the levee (some materials such as reinforced turf will be hidden by the grass itself, while a more structural solution can either be hidden beneath a sacrificial layer of topsoil and turf, or exposed at the surface to give confidence to local residents that the levee is resilient).

Grass

Grass is the simplest and most common measure used on levees around the world, particularly riverine levees, to protect against erosion. The roots of the grass penetrate the surface of the levee and provide a dense mass of interlocking turf that can resist higher velocities than bare soil alone. In most cases, grass is an ideal material, and as long as it is cut relatively regularly and well maintained, it will:

- bind the topsoil together to resist surface erosion due to laminar flow, wave action, overtopping and precipitation
- reduce the effects of desiccation
- repair itself when damaged
- provide a relatively cheap and relatively robust protection system.

Grass species should be selected to maximise root development and density of root mass (Hewlett *et al*, 1987, Hemphill and Bramley, 1989 and USACE, 2012a). In this context, 'enhanced grass' is a term sometimes used for species that are established, nurtured and maintained in accordance with strict and well-defined regimes that encourage healthy growth.

To ensure a suitably robust grass turf, the following steps are advised:

1. Sample and test the soil where grass turf is to be established to determine its suitability for various grass species.
2. Choose the grass mixture based on soil conditions, climate and management requirements.
3. If necessary, modify the soil before sowing to ensure that it will support good turf development.
4. Decide on the method of sowing and establishing the grass turf.

Further information on maintenance of grass turf is given in Section 4.4, and guidance on calculating the resistance of grass slopes is given in Section 8.4.2.

When grassed surfaces alone are not sufficient to resist erosive forces, turf reinforcement should be considered. Turf reinforcement used a geotextile mesh or turf reinforcement mattress (TRM) to provide additional resistance to soil movement. A high performance turf reinforcement mattress (HPTRM) is a TRM which exhibits a significantly greater ultimate tensile strength (UTS) and a higher resistance to ultraviolet light. HPTRMs are generally thicker and denser than first generation (medium grade) TRMs. Although these parameters do not directly translate into higher erosive resistance, they do have the advantage of providing greater resistance to wheel loads, and have a longer design life. However, the density of the HPTRM is such that it offers the additional protection of a physical barrier between the foundation and the flowing water – leading to much increased erosive resistance. Reinforced grass protection is discussed in more detail in Box 9.26.

Box 9.26 High performance turf reinforcement mattresses

Mattress material (HPTRM)

HPTRM is typically formed of a woven polypropylene composed of non-degradable synthetic fibres, monofilaments, mesh and/or other elements, processed into a 3D, dense, closely woven homogeneous matrix capable of supporting the dense growth of grass roots through the material. Minimum tensile strengths of 150 kPa in both the machine direction and in the cross direction are usually required. The fabric should not be composed of layers of discontinuous material and it should not be held together by stitching or glued netting.

Grass

Consideration should be given to using turf rather than grass seed in conjunction with the HPTRM. Turf has the advantage of preventing erosion from rainfall until the grass roots penetrate the mattress, protecting it from turning wheels and promoting rapid penetration of the roots.

Design and construction details (Figure 9.24) should include:

- good surface preparation including tilling, and removal of stones and previous vegetation
- placement of the HPTRM, taut over the length of the levee crest, rear slope and berm and anchored to the levee surface
- fixing of the HPTRM in anchor trenches at two locations: the upstream end (on the waterside slope, 600 mm down the slope from the crest) and at the downstream extremity of the armouring, with trenches backfilled to a density that matches the soil in the levee section
- careful placement the turf, to avoid damage during selection, preparation, transportation, placement and early life

Box 9.26 *High performance turf reinforcement mattresses (contd)*

- careful transitions with hard points, by anchoring the HPTRM into 150 mm wide by 300 mm deep trenches adjacent to the hard points
- continuation of the reinforcement system "well upstream to a point where the flow is sub-critical before being terminated" (Hewlett *et al*, 1987), and typically over the crest of the levee – this also provides protection against damage by wave splash and spray (Bureau of Waterways Engineering, 2011)
- addition of asphalt or concrete crest roads, if required, after placing of the HPTRM over the crest and designed to resist the lateral forces associated with wave impacts and to support normal O&M traffic.

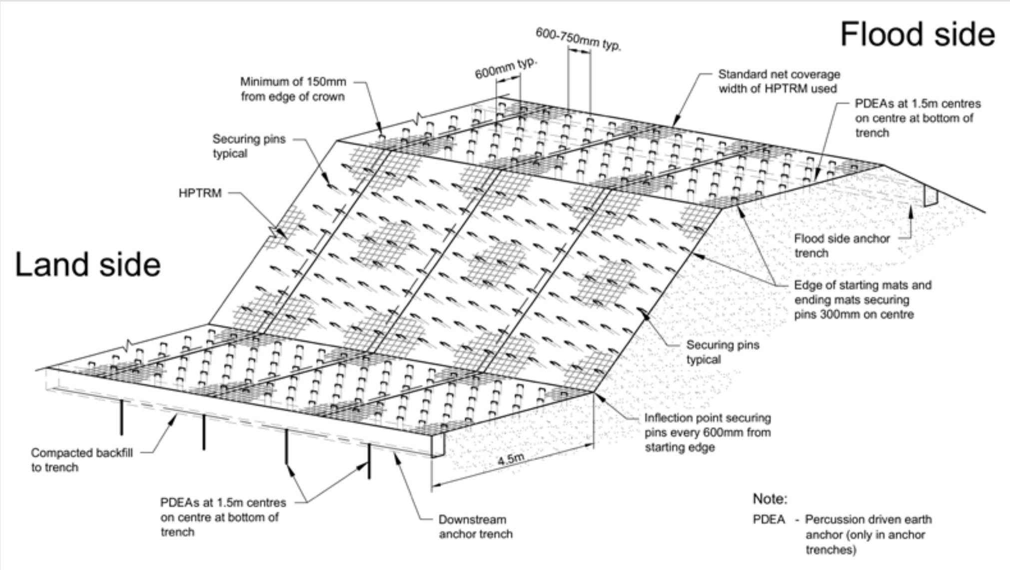


Figure 9.24 Recommended HPTRM trenching and anchoring requirements (from USACE, 2012a)

HPTRM has been applied over significant lengths of the new levees in the New Orleans area to improve resilience during overtopping events (Figure 9.25).

Figure 9.25 Turf reinforced mattress (from USACE, 2011)

Armourstone

Armourstone (sometimes described as rip-rap) is widely used and comprises natural or artificial rock (Figures 9.26 and 9.27) laid over a granular or geotextile filter layer on the levee slope. Its permeable nature means that it is both able to absorb external wave and current energy and also to allow drainage of internal pore water from the levee. The thickness of armourstone layer required can be large and may need to be taken into account in geotechnical stability calculations. O&M requirements for armourstone (Section 4.13) should be taken into account in the design, to reflect the maintenance capability of the levee management authority. Although armourstone may not have significant environmental impacts, the uneven surface and voids within it may pose some health and safety risks when used in recreational or residential areas and so this issue does need to be managed.

Design calculation methods for armourstone are given in Section 8.4.4, and further extensive armourstone design guidance is available elsewhere (CIRIA; CUR; CETMEF, 2007).

Figure 9.26 Armourstone revetment (courtesy William Allsop, HR Wallingford)

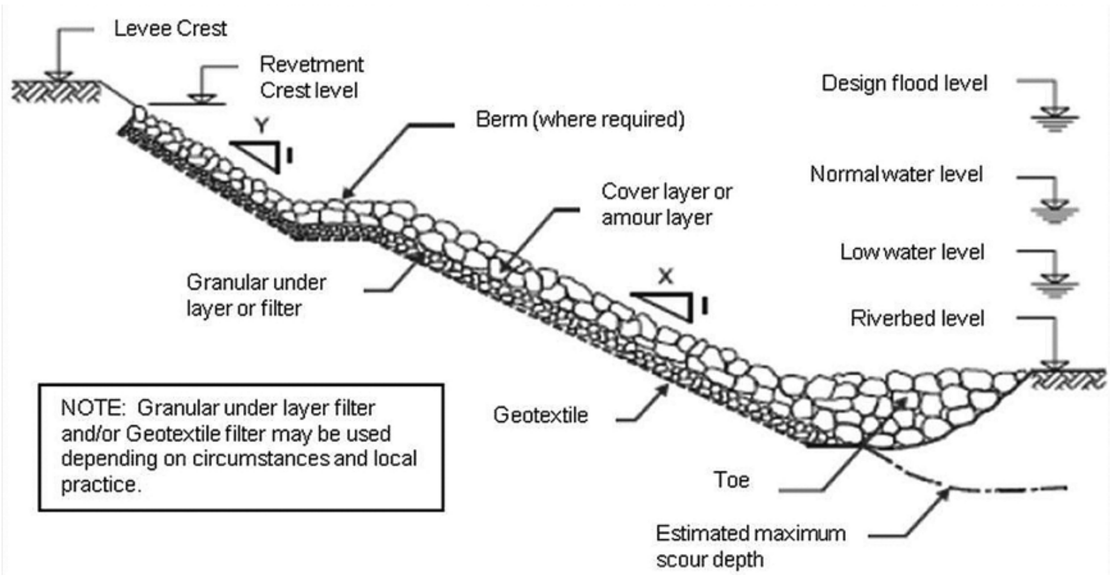


Figure 9.27 Components of a typical armour stone revetment (CIRIA; CUR; CETMEF, 2007)

1 2 3 4 5 6 7 8 9 10

Gabions

Gabions and mattresses are basket containers formed from steel or synthetic mesh that are filled with armourstone, typically of mean size 100 m to 150 mm to form a gravity structure, which can resist scour, overturning and sliding. Their advantages include:

- increased stability for armourstone of a given size
- permeable nature, which means that they do not impede pore-water dissipation
- flexible geometry of the resulting structure that allows them to conform to the natural profile of the protected surface and makes them popular for riverbank stabilisation.

Disadvantages include:

- corrosion of the tie wires and damage by sediment and woody vegetation, which means that they are susceptible to significant deformation and loss of their armourstone contents
- possible sliding of gabions on steep slopes
- potentially increased maintenance requirements compared with plain armourstone.

Concrete block flexible revetment systems

Alternative forms of armouring for levees are provided by close-fitting blocks or concrete slabs (Figure 9.28), placed by hand or with mechanical assistance. These blocks may be laid closely on a bedding layer of relatively low permeability, with sufficient gap between blocks to allow drainage of internal pressure build-up. They are designed such that the individual block can resist uplift forces, principally through their mass, but additional resistance is provided by:

- friction between blocks and filter layer, and between adjacent blocks
- relative thickness and permeability of the protection layer and underlayers
- revetment slope angle
- soil tightness and erosion resistance of the filter layers.

Some designs (eg wedge-shaped blocks on spillways) take advantage of flow patterns to increase downward forces on the blocks to increase stability (Hewlett *et al*, 1997). Concrete block or slab systems in fluvial situations are typically 100 m to 250 mm deep, but in coastal situations 150 mm to 400 mm deep. Variants used on coastal revetments and levee slopes include blocks joined by an overlap (shiplap blocks), columnar blocks placed in a pattern, or plain/tapered blocks grouted with bitumen.

***Figure 9.28** Concrete block protection (courtesy John Harris, HR Wallingford)*

For applications in more turbulent conditions, increases in block thickness are reduced by tying the blocks together in mattresses. This provides extra stability, while maintaining some flexibility. Several such proprietary systems have been introduced in recent years, using cables of steel, nylon or polypropylene. These mattresses can be transported to site on flatbed trailers and installed by crane, using special spreading beams to give very rapid protection. Such systems are commonly used in situations where high shear stresses are predicted, such as at spillways. Further increases in stability, avoiding initial uplift of the mattress, can be achieved by anchoring selected concrete blocks within the mattress to reduce the potential for uplift of the system, but caution is advised in view of some high profile failures of such spillways.

The main advantages of concrete block mattresses are:

- maintenance of contact with the underlayers as they settle
- reduced overburden pressures on the levee
- ease of access and maintenance.

The disadvantages of concrete block mattresses are:

- heavy plant (equipment) required to install the mattresses
- if a block is lost, failure of the mattress can be rapid
- the blocks in such mats are more widely spaced than close individual blockwork, and often require the addition of gravel 'grouting' to increase stability.

In general their permeability to waves is low, the surface is relatively smooth and the overall hydraulic performance of these revetments is close to that of impermeable and smooth slopes.

Continuous concrete or asphaltic slope paving is an alternative to the revetment systems discussed above. The stability of this armouring requires that up-lift pressures acting across the concrete are balanced by net weight force. The simplest revetment armouring is *in situ* concrete, cast in slabs generally 75 mm to 200 mm deep. This is particularly common on inland reservoir dams where waves will not attack the slope until after construction is complete.

The advantage of continuous paving is:

- a very robust protection, which can withstand turbulent hydraulic loading conditions such as those associated with wave overtopping.

Disadvantages of continuous paving include:

- lack of permeability which means that high pore water pressures can build up inside the levee, creating risks of instability
- potential for cracking and deformation arising from erosion of underlayers or (differential) settlement of the levee and/or levee foundation combined with the effect of wave impacts
- inaccessibility of the levee surface for inspection, eg voids that might lead to levee damage cannot be readily observed.

Soil-cement (including roller-compacted concrete)

Use of soil-cement for levee protection in semi-arid regions where grass turf is not feasible was prompted by the abundance of sandy soils in these areas (Box 9.27). Cement added to the sand, with water at optimum moisture content, and compaction of the resulting soil-cement mixture produces a durable erosion-resistant material (soil-cement). In arid areas, low elevations of the water table allow excavation down to the designed scour depth without the need for dewatering. Ready availability of the sand on site reduces costs and avoids the environmental impact of importing materials over large distances.

For slopes exposed to moderate to severe wave action, the soil-cement is usually placed in successive horizontal layers, 2 m to 3 m wide and 50 mm to 225 mm thick, adjacent to the slope (Figure 9.29). This is referred to as 'stairstep' protection. In coastal situations, the steps can help to dissipate wave energy and reduce the height of the wave run-up. The technique also creates resilience to lateral erosion during flood events.

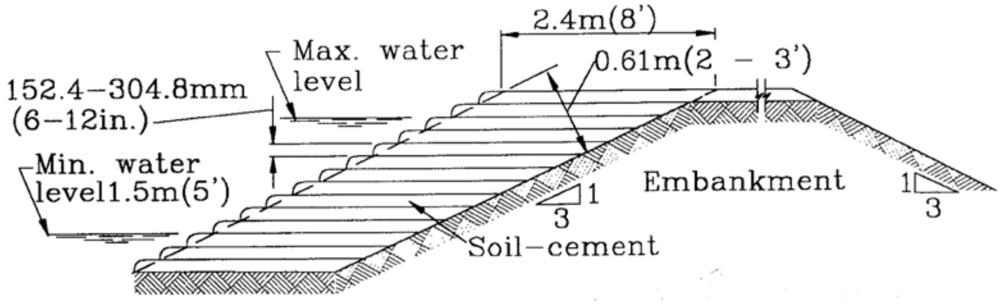


Figure 9.29 ***The use of soil-cement for strengthening levee and dam slopes (from USACE, 2000)***

For less severe applications, slope protection may consist of a 150 mm to 300 mm thick layer of soil-cement placed parallel to the slope face. This method is referred to as plating. It uses less soil-cement than the stairstep method but cannot be successfully placed on slopes steeper than 2.5:1, and it also provides little resistance to wave run-up.

As stated by Richards and Hadley (2006), a typical section consists of approximately 2.5 m to 3 m wide horizontal layers, placed in stairstep fashion along the levee or riverbank slopes. If the design is used in conjunction with a natural river bed, then the base of the soil-cement stabilisation should be installed to a depth equal to the maximum scour depth that could be expected over the design life of the levee. At the end of the soil-cement reach, the soil-cement protection should be turned perpendicular to the channel and extended approximately 15 m into the natural ground to prevent head-cutting erosion behind the soil-cement.

Having used soil-cement, there are a number of options for finishing the slope surface of the levee: it can be trimmed smooth, left natural with loose overbuild soil-cement remaining in place or rough steps can be created without any formwork. To withstand the abrasive force of stormwater flows at velocities up to 6.0 m/sec, the soil-cement is usually designed to achieve a minimum 7-day compressive strength of 5 MPa. The site-specific ground conditions and the nature of the soil used in the soil-cement will control the composition of the mix; trial mixes should be undertaken. If fly ash is used in the soil-cement, a 28-day requirement for compressive strength is usually specified.

Further information on using soil-cement mixes for improving the resilience of levee slopes is given by Richards and Hadley (2006). The use of roller compacted soil-cement (RCC) mixes for the protection of spillways is discussed in Section 9.14.4.1.

Box 9.27 ***USA use of soil-cement for levee slope stabilisation***

The first use of soil-cement mixtures to protect the slopes of levees and riverbanks in the south-western states of the USA can be traced back to the mid-1960s (Hansen *et al*, 2011). Hansen *et al* (2011) describe how soil-cement protected banks have performed well during five significant floods between 1983 and 2006, including two significant flood events in Tucson, Arizona.

In Albuquerque, New Mexico, soil-cement was used on both the San Antonio and the Calabacillas arroyos, where sensitivity to the environment was an important consideration. Special artwork was used at Calabacillas. Coloured shotcrete was used above the soil-cement, and pre-cast dinosaur bones were placed into the shotcrete. The side slopes at Calabacillas and San Antonio arroyos were stepped to both provide an east exit from the channel and to mimic a layered stone formation.

9.6.2 Erosion protection for coastal levees

In selecting an appropriate protection system for a coastal levee, the location of the levee in relation to the beach and active coastal zone is important. Two broad situations can be distinguished:

1. The **levee is set back** from the active coastal zone, away from dynamic coastal sediment movements and only subject to wave action in the most extreme events. In this situation, *grass* is often the most economical revetment material in terms of installation, maintenance and, in many cases, performance. Where grass on its own is not strong enough to resist the erosive forces at the site, or it will not form a dense stand of turf in the material used to construct the levee, the use of a reinforced grass system can be considered (Box 9.26).
2. Where the levee is **located in the zone of more regular wave and sediment action**, physical chemical and biological actions can make maintenance of grass cover unsustainable. Other systems such as rock (unbound or grouted with asphaltic or colloidal concrete grouts), articulated mats, interlocking blocks, gabions or concrete paving may well be required. These will need to be carefully designed on a case-by-case basis. It is particularly important to make sure that the toe of any protection system is secured, especially if it lies within the active coastal zone. Examples are given in CIRIA; CUR; CETMEF (2007) of how to secure the toe of rock armouring given a wide variety of beach situations. For proprietary systems, advice should be sought from the supplier of the system.

Wave and current conditions acting on the levee should be determined according to the guidance in Chapter 7. It is important to bear in mind:

- wave and current action on the front face of the levee (Section 7.4)
- potential toe scour (Section 8.2.5)
- overtopping wave action (Section 8.2.1) and resulting flows down the landward face of the levee.

The selected surface protection system must be capable of resisting this wave and current action, given its known performance characteristics. When comparing and selecting from the alternative systems set out in Section 9.6.1, initial design calculations should follow the procedures set out in Sections 8.2.6 and 8.2.8. Once selected, the detailed design of the appropriate system may well require reference to other guides (eg CIRIA; CUR; CETMEF, 2007).

9.6.3 Erosion protection for riverine levees

Riverine levees need protection from the erosion of surface material, induced by the stress of water flowing along them. Erosion typically occurs during both flood flows that inundate the floodplain and which just begin to contact the levee, as well as during the more severe events that have been used to establish the crest level of the levee. Critical situations for erosion are when:

- floodwaters are high enough to inundate a portion of the levee
- the stream channel threatens to undermine the levee
- floodwaters overtop the levee and risk eroding the rear face.

Any design requires careful evaluation of the range of velocities, directions and durations of the current (acting parallel to or impacting at an angle to the levee alignment). From these, the most severe situation that could be expected should be determined, bearing in mind that velocities having longer durations have a greater capacity to mobilise soil. This evaluation should include consideration of:

- currents parallel to the levee axis as well as perpendicular flows
- currents that may impinge on levee slopes, particularly at bends or changes in river alignment
- the additional erosive stirring effect of wave action generated either by the action of wind or caused by vessel navigation

- high velocity currents and erosive turbulence caused by bridge abutments and piers, gate structures, ramps and drainage outlets
- potential levee overflow situations (Figure 9.30).

Assessment of these situations should follow the guidance set out in Section 7.3.

The selected surface protection system must be capable of resisting this wave and current action, given its known performance characteristics. When comparing and selecting from the alternative systems set out in Section 9.6.1, initial design calculations should follow the procedures set out in Sections 8.2.6 and 8.2.7. Once selected, the detailed design of the appropriate system may well require reference to other guides (eg CIRIA; CUR; CETMEF, 2007).

a Overhead view showing overtopping in sacrificial levee section

b Oblique view showing erosion of downstream levee slope following floodway operation plan

Figure 9.30 Overtopping of levee and resulting erosion without protective measures, USACE, Birds Point/New Madrid Floodway operation during the 2011 flood, USA (courtesy USACE Memphis District)

Special consideration should be given to the protection of the landside slopes of low segments of the levee, introduced to allow overtopping when discharges in the river are higher than the design height. These are often selected to force the initial overflow to the least hazardous location (Figure 9.30). Design of these *spillway* sections is discussed in Section 9.14, along with methods to ensure that the potential for erosion and subsequent breach is minimised at such sections.

Temporary protection with polyethylene sheeting on the surface (Box 9.28) is an approach which is useful if breaks in construction activities are necessary or during emergency situations.

Box 9.28 *Temporary protection measures*

Following activation of the Birds Point/New Madrid floodway during the 2011 Mississippi River flood, segments of the sacrificial levee had to be restored. The extensive work required to reclaim the levee alignment and to reconstruct the levee section could not be accomplished within a single construction season. So, reconstruction could only be completed to an interim elevation, prior to the 2012 flood season. The largest levee segment being restored included use of Hesco bastions to achieve the desired interim elevation. Concerns were raised about vessel- and wind-induced wave attack and also the anticipated current action during the 2012 high water season on the newly constructed levee section. As a result, polyethylene sheeting was placed on the riverward side of the embankment and Hesco bastions to protect the embankment surface from currents. Note that sufficient ballast on top of the polyethylene sheeting is necessary to prevent the current from rolling it up during a flood event. Also, polyethylene sheeting had to be layered so that upstream sheets overlapped on top of the adjacent downstream sheet (Figure 9.31).

Figure 9.31 *Polyethylene sheeting placed over newly constructed levee to provide temporary erosion protection (courtesy USACE Memphis District)*

9.6.4 Detailing surface protection systems

Issues to be considered when detailing surface protection systems include:

- **anchorage:** defining the requirements for securing the system in place
  - at the edges of all reinforcement systems
  - within concrete systems
- **filter or base layer** (Section 9.7.4) defining:
  - the base layer requirements if reinforced concrete is to be used
  - the filter requirements where porous armour will be used
- **crest details:** completing the detailed design of the crest to show transition from armour layer to soil, and any anchorage required to prevent movement resulting from erosion upstream of the armour
- **channel details:** defining cross-sections both down the levee slope and perpendicular to the flow, indicating flow depth with careful detailing at any transition between two or more plane surfaces
- **toe details:** completing the detailed design of the toe to prevent undercutting by scour (Section 9.6.4.1)
- **construction details:**
  - joints in geotextiles or concrete reinforcement
  - preparation of levee prior to placing armour
  - temporary restraint of geotextile reinforcement.

9.6.4.1 Toe and scour protection

Adequate protection of the toe of a slope or bank is essential for its stability. Many of the failure mechanisms result from reduced strength at the base of the slope. Estimation of scour (using the guidance in Sections 8.2.4 and 8.2.5) can therefore be an important design step. The toe of any protection system should be secured to take account of the assessed risks of general scour, local scour or channel movement undermining the levee.

Figure 9.32 indicates some example solutions for how to secure the toe of rock armouring given a wide variety of riverine situations. Further details of how to design and secure the toe of protection systems are given in CIRIA; CUR; CETMEF (2007). For proprietary systems, advice should be sought from the supplier of the system.

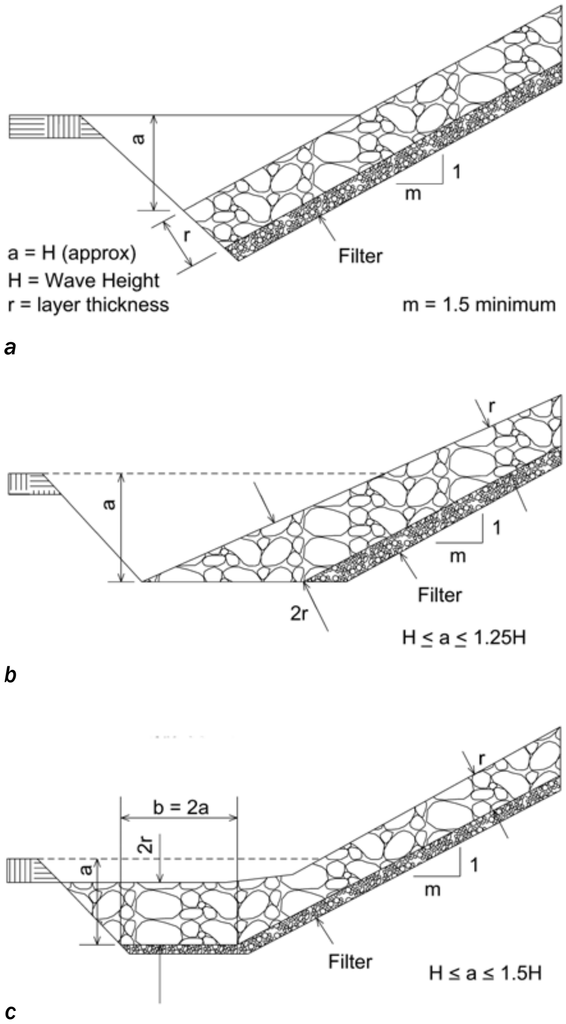


Figure 9.32 ***Toe armour details: low scour potential (a), low to moderate scour potential (b), moderate to severe scour potential (c) (USACE, 1995a)***

There are two main ways of ensuring toe protection:

- by providing sufficient material at sufficient depth to account for the maximum scour depth predicted
- by provision of a flexible revetment that will continue to protect the toe as the scour hole develops. The principal issue for toe protection is that a sufficient quantity of armour material must be placed such that stone can settle into the scoured area as it develops without jeopardising the stability of the remaining bank or slope protection.

In riverine situations, the stability equations used for the design of bed and slope protection works will still be applicable to the design of the toe protection. Differences mainly arise in practice due to construction aspects such as the thickness of the armour layer provided at the toe, the depth at which it is built and the way in which it is constructed (underwater or in the dry).

In coastal situations, the nature of wave action (especially breaking waves) and the many possible combinations of wave height and water depth mean that specialist calculations may be required. Further details are provided in detailed design guides such as CIRIA; CUR; CETMEF (2007) and McConnell (1998).

9.6.5 Protection for flood wall overflow/overtopping

Flood walls that might be overtopped by rising water should be designed with erosion protection on the protected (dry) side because failures can occur as a result of the loss of lateral support due to erosion of the supporting material. The erosion protection should be capable of resisting the force of the free-falling water jet overtopping the wall. The plunging jet penetrates any standing water on the dry side and creates large eddies that erode material from the unprotected soil surface. The same mechanism will scour foundation material when there is no standing water on the protected side of the flood wall (Figure 9.33). Failure occurs if the remaining, undamaged portion of the foundation adjacent to the wall cannot withstand either the shear force or the overturning moment exerted on the flood wall.

The designer should consider the following issues:

- height of water above the top of the flood wall
- height of the flood wall above the ground surface on the protected side
- velocity of the flow approaching the flood wall (waves or surges increase velocity over the side weir flow that occurs when river flow is predominantly parallel to the flood wall)
- duration of overtopping flow
- the size and type of armour.

The location, angle and force of the overtopping plunging water jet can be estimated based on the equations in Section 8.2.2.3. Where surges or tsunamis are the cause of the overtopping discharge, there is a significant horizontal velocity added to the approaching flow. This horizontal velocity alters the jet trajectory and may cause oscillations in the jet.

Designing resistance to overtopping flows

For flood walls, the general process for designing resistance to overflow/overtopping flows is as follows:

1. Determine if wave surges are applicable. If so, estimate the approach velocity due to surge.
2. Calculate the overtopping or overflow discharge, and velocity conditions using the guidance in Sections 8.2.1 and 8.2.2.
3. Estimate the size of protective material required to resist uplift forces and horizontal currents where the jet impacts natural ground. The use of flexible armour to resist the significant uplift force caused by the jet is not recommended. However, concrete aprons or grouted rock have been found to be effective measures.
4. Extend the armour from the flood wall face downstream past the outer trajectory limit. The distance to extend the armour beyond the calculated trajectory should be determined from consideration of:
   - i the jet trajectory
   - ii the proximity to adjacent structures or facilities.
5. If a concrete apron is selected, design it as a reinforced beam to resist the force calculated for the overflowing/overtopping water impacting the surface.

Figure 9.33 Scour trench at a T-wall on the east side of the IHNC following Hurricane Katrina, New Orleans, USA (courtesy USACE New Orleans District)

Caution

Significant external and internal erosion problems can occur at various kinds of transitions. This topic is addressed in Section 9.11.

9.6.6 Surface protection to resist ice

The design of most levees will not require a consideration of ice-related loads. However, for some areas (particularly subarctic regions such as Alaska, Canada, Scandinavia and northern Russia or areas on the fringes of large mountain chains) a consideration of the various impacts of ice on the performance and integrity of levees is important. In general, when deciding whether ice needs to be considered during the design process, past experience should be taken into account, and local codes of practice should be consulted.

Brown and Clyde (1989) identify the fact that ice can affect surface protection systems in a number of ways including:

- moving surface ice can cause crushing and bending forces and large impact loadings
- the tangential flow of ice along a protected levee can cause high lateral shearing forces
- the thawing of upstream ice jams can cause a rapid release of water and blocks of ice, leading to flooding and possible overtopping of water and ice.

Brown and Clyde (1989) and Colorado Department of Transportation (2004) note that historic observations of ice flows in New England rivers indicate that rip-rap, sized to resist design fluvial flow events, will also resist ice forces. They suggest that ice forces should be evaluated on a case-by-case basis. In most instances, ice flows will not be of sufficient magnitude to warrant detailed analysis. Where ice flows have historically caused problems, Brown and Clyde (1989) and Colorado Department of Transportation (2004) suggest that a stability factor of 1.2 to 1.5 should generally be used to increase the design diameter of the rock protection. However, they note that the selection of an appropriate stability factor to account for ice-generated erosive problems should be based on local experience. They also suggest that where significant impact from floating debris and/or ice is anticipated, the stability factor for the design of rip-rap should be increased to values in the range 1.6 to 2.0 (Table 9.12).

Table 9.12 ***Guidelines for the selection of stability factors for rip-rap design (Brown and Clyde, 1989)***

Condition	Stability factor* range
Uniform flow: straight or mildly curving reach (curve radius/channel width > 30), impact from wave action and floating debris is minimal, little or no uncertainty in design parameters	1.0–1.2
Gradually varying flow: moderate bend curvature (30 > curve radius/channel width > 10), impact from waves or floating debris moderate	1.3–1.6
Approaching rapidly varying flow and sharp bend curvature (curve radius/channel width < 10): **significant impact potential from floating debris and/or ice**; significant wind and/or boat generated waves (0.3 m to 0.6 m), high flow turbulence; turbulently mixing flow at bridge abutments; significant uncertainty in design parameters	1.6–2.0

Note

* Stability factor is the number by which the design rock diameter for hydraulic design should be multiplied to take account of ice effects.

Vaughan *et al* (2002) carried out research to investigate the appropriateness of the Brown and Clyde (1989) recommendations in more severe ice situations. They considered five relatively severe ice-related scenarios for the design of rip-rap, carrying out independent assessments and calculations for each to check the guidance:

1 Anchor ice rafting and rip-rap specific gravity reduction.
2 Raft ice impact damage.
3 Raft ice pushup onto shore.
4 Ice jams causing velocity increase.
5 Increased longitudinal effective tractive force imposed by stream ice cover.

They concluded that, for the scenarios investigated, the higher stability factors in the above table (ie in the range 1.6 to 2.0) were still relevant.

Generally, the effect of ice (Section 7.3.3) on the levee can be mitigated by adopting flatter levee slopes or increasing the size of the rip-rap armour (see Box 9.29 for an example).

Further information is given in CIRIA; CUR; CETMEF (2007).

Box 9.29 ***US practice for levee slopes prone to ice action***

> General practice in the Midwestern US is to keep slopes at 1V:4H or flatter. If use of a 1V:4H slope is not an option, there is a need to increase the size and use of rip-rap armour.
>
> The practice adopted by the Omaha District Corps of Engineers is to extend rip-rap protection up to the 10 per cent event ice water surface profile which can be estimated by numerical modelling.

9.7 CONTROL OF SEEPAGE AND UPLIFT

9.7.1 General

The primary function of a levee is to inhibit the passage of water during a flood. To do this effectively and safely, the levee must, to an acceptable degree, control:

- seepage through the levee (along seepage paths such as permeable layers, cracks and fissures or animal burrows)

- seepage beneath the levee (again along seepage paths such as permeable layers or animal burrows)
- seepage along transitions and interfaces with levee structures such as crest walls or pipes as a result of **hydraulic separation**. Hydraulic separation is the process by which a flow path is created between a rigid structure and poorly compacted or low strength fill material by the action of the pressure of the floodwater (Johnston *et al*, 1999). It is a dangerous phenomenon because, like hydraulic fracture, it can happen suddenly, when a critical pressure head is reached. It can cause rapid deterioration and can ultimately lead to breach.

Each of these features has the potential to cause deterioration in the performance of the levee or, ultimately, to trigger a sudden failure which could lead to a breach.

Seepage considerations

The impact of seepage needs to be considered from two main aspects – sudden failure and ongoing deterioration.

First, hydraulic and phreatic pressures can directly trigger a **sudden failure:**

- The hydraulic action could apply a direct horizontal load onto a levee, sufficient to cause a translational failure.
- High phreatic pressure within the body of the levee could cause sudden failure of the levee.
- Groundwater pressure acting in an aquifer beneath the landward levee toe, and linked to the flood level, could create an uplift which could initiate instability of the levee or create a sufficiently adverse hydraulic gradient to cause sand boils.
- Hydraulic separation caused by poor detailing or poor construction could cause immediate failure or could trigger rapid deterioration through internal erosion.

Secondly, seepage can cause **ongoing deterioration** and problems such as leakage, crest settlement and minor flooding. If this deterioration is not managed, it can create internal erosion (Section 9.8) which could lead to breach.

The issue of designing to avoid hydraulic separation is discussed in Section 9.15 (as it is usually associated with hydraulic fracture between the levee and stiff embedded structures).

9.7.2 Understanding seepage through and beneath levees

The factors affecting the seepage through the levee, stability of the levee slopes and seepage through the levee foundation are all interrelated. Important features that affect seepage through a levee and its foundation include:

- the hydraulic load (particularly water level)
- the structure and composition of the levee (particularly any potential flow paths such as permeable layers or interfaces between earthfill and rigid structures)
- the nature of the underlying geology, particularly the hydraulic connection between permeable soils and the source of the water (the sea or the river)
- the condition of the levee (particularly the presence of animal burrows and desiccation cracks)
- the hydraulic conductivity (permeability) of the materials in the levee and the underlying soils
- the hydraulic gradients in different materials resulting from the application of the hydraulic load for the period of the flood
- the flow velocities in different materials resulting from the application of the hydraulic load for the period of the flood
- the grading (particle size distribution) of relevant materials and the related critical tractive stress (the shear stress required for flowing water to dislodge a soil particle) for that material
- the shear strength (particularly the angle of shearing resistance) of the levee and foundation materials.

9.7.2.1 Permeability

The determination of permeability of both the fill material and the underlying soils is clearly an important issue for levees. From a design point of view, it is critical that measurements or assessments are carried out on materials in a condition representative of their actual or eventual state in the natural ground or levee. The ratio between vertical and horizontal permeability of the different materials is also important.

The measured permeability of natural soils is known to be dependent on many factors including:

- the geomorphological conditions of the natural ground
- the anisotropy and stratification of the ground
- the disturbance of the ground caused by drilling and installation of *in situ* testing equipment
- the size of the sample or zone tested (for example, the testing of a small reconstituted sample in a laboratory in comparison with a single borehole test or a pumping test involving a central pumping bore and an arrangement of surrounding observation points)
- the disturbance or uncertain reconstitution of samples tested in the laboratory
- the method of testing
- the effective stress levels and changes in these during the measurement process.

It is important that the designer consider the above issues in determining the characteristic values that are to be used in calculations of seepage velocities, volumes and gradients, and for modelling processes such as internal erosion are discussed in Chapter 8. As permeability is a particularly difficult geotechnical parameter to assess, it is usual for the designer to consider a range of possible values for design, rather than just a unique value.

The determination of *in situ* permeability and the characterisation of the permeability of fill materials is discussed in Chapter 7.

9.7.2.2 Steady-state and transient seepage conditions

Unlike dams, levees may not necessarily reach a steady-state seepage condition. Whether they do or not depends on many factors including the duration of the flood hydrograph (or the coastal storm surge) and the permeability of the levee materials and of the natural ground.

The assumption of steady-state conditions is normally a conservative approach to design, as predicted seepage pressures, volumes, velocities and gradients are all likely to be overestimated. Steady-state seepage analyses (described in Section 8.3.1) are also simpler and easier to validate than transient analyses. Steady-state seepage analyses, however, are likely to be conservative, and in some cases they could be highly conservative.

Transient seepage analyses that predict how pore pressures within the levee will rise and fall with the flood level will usually provide a more realistic prediction of behaviour. Such calculations are normally more complex and time-consuming than the steady-state analyses.

Output from seepage analyses should include seepage volumes, so that the requirements for drainage can be considered if these volumes are significant. The output should also identify the critical phreatic surfaces within the levee for use in stability calculations (as discussed in Section 9.9). Finally, the output should identify the critical hydraulic gradients and flow velocities within the levee for use in the uplift, hydraulic heave, internal erosion (Section 9.8) and filter design calculations using the methods set out in Chapter 8.

Given the problems and uncertainties often associated with the determination of permeability (either in the field or in the laboratory), it can be helpful to compare computational predictions with observed behaviour through a process of the back-analysis of levees for which monitoring data is available. Sensitivity assessments can also be helpful, as it can be difficult to verify the performance of levees until the occurrence of the rare design flood event. Sensitivity analyses performed by California Department of Water Resources (Chowdhury *et al*, 2012) indicated, for the case investigated, that varying the permeability of the levee material had little effect on the height of the phreatic surface exit elevation on the landside levee slope.

9.7.2.3 Permeability of the levee materials

Permeability of the levee materials is important as:

- higher permeability leads to an increase of the flow quantity through the landside slope face
- permeability affects soil strength parameters and hence the stability of the levee slopes during steady-state conditions. According to Duncan and Wright (2005), reductions in levee stability occur during a flood as a result of the decrease of the effective shear strengths of the materials caused by the increase in pore pressure in the levee and the increase of the soil weight due to the increase in water content.

In some geographical areas, the choice of fill materials for levee construction is limited, and designers are forced to use soils such as silty sands, sands and silty, sandy gravels, which would not normally be considered suitable for the construction of impermeable barriers. The use of such materials creates immediate problems:

- granular materials can be of relatively high permeability, and hence the volume of water passing through a levee during a flood can constitute a nuisance level of flooding
- granular cohesionless materials in levees are susceptible to both internal and surface erosion
- during a flood, high permeability granular soils throughout a levee will potentially allow piezometric levels to become elevated, adversely affecting levee stability
- high flow velocities caused by seepage through permeable fill materials can cause suffusion (a form of internal erosion – see Sections 8.5.3 and 9.8), potentially leading to piping or breach.

During periods of flooding, particularly events when the levees are subjected to high water levels over an extended period of time, seepage through levees constructed out of higher-permeability materials can be expected (as can seepage through the underlying soils if they are granular cohesionless materials).

Permeable granular fills provided in discrete zones of a levee can have a beneficial effect. For example, placing granular material on the landward side of a levee will provide better drainage of the landward slope. Similarly, the use of permeable fill on the water side will facilitate post-flood drainage and reduce the possibility of failures following rapid draw-down. Sections 9.7.3.4 and 9.7.3.5 discuss the use of such permeable material for internal and toe drains.

9.7.3 Design to manage and control seepage and uplift

Having evaluated the likely seepage conditions, it may be that one of the following undesirable situations is identified:

- seepage discharge rate above acceptable limits
- groundwater pressures needing to be controlled
- uplift pressures acting beneath the landward slope of a levee needing to be reduced.

In such situations control measures should be incorporated into the levee (see also discussion in Sections 3.3 and 3.4.), options being:

- the use of stabilising berms
- the construction of impervious layers on the levee
- the construction of seepage cut-off walls through the permeable layers

- the use of toe drains that penetrate into the permeable layers
- the installation of relief wells.

The relative advantages and disadvantages of these methods are summarised in Table 9.13.

Table 9.13 Relative advantages and disadvantages of measures to control seepage and uplift

Seepage control measure	Notes	Advantages	Disadvantages
Stabilising berms	• requires seepage analysis to determine the dimensions of the berm	• robust measure, as it relies on mass to counteract uplift • adds resilience to overflow	• may require considerable additional land-take • may require considerable additional fill material
Impervious layers or barriers	• requires seepage analysis to determine dimensions and characteristics of layer	• physical barrier that can be verified during construction • flexible material that will move with the levee	• may require flat slopes to avoid rapid draw-down failures • may be prone to desiccation • burrowing animals may compromise performance
Cut-off walls	• requires seepage analyses if cut-off wall does not reach the base of the permeable layer	• physical barrier that can be verified during construction	• stiffness of cut-off does not necessarily match stiffness of the levee and may therefore attract additional load
Toe drains	• requires seepage analysis to determine the volume of water passing so that the waste water systems can be designed	• relatively simple to construct in short sections	• collected water may require pumping to deal with anticipated volumes; this is not a fail-safe solution as the pumps could fail
Relief wells	• requires seepage analysis to determine the volume of water passing so that the waste water systems can be designed	• relieves water pressure in sub-surface layers beneath the levee, controlling groundwater pressures	• collected water may require pumping, to deal with anticipated volumes. This is not a fail-safe solution as the pumps could fail • increased maintenance

9.7.3.1 Stabilising berms

Berms constructed out of permeable materials can be used to manage seepage and reduce uplift pressures acting on levees. This was shown to be the case for levees on the Crayford Marshes (Box 9.30) and also for rural levees along the Missouri River in Missouri and along the Feather and Sacramento Rivers in California. The seepage berms may need to be wide, so can require considerable land-take (Figure 9.34).

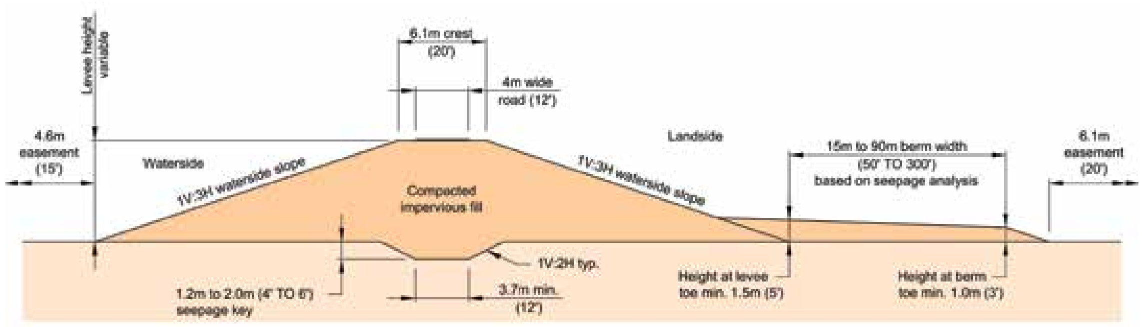


Figure 9.34 ***Levee with seepage berm on the Feather River, California, USA (courtesy Mary Perlea, USACE)***

Seepage berms are used extensively across the continental USA to control under-seepage. The design and construction of seepage berms is described by USACE (2000).

Box 9.30 ***Stabilising berms used in levees at Crayford Marshes, Thames estuary, UK***

Marsland and Randolph (1978) describe how pore pressures in a layer of sandy gravel beneath the alluvial peats and soft clays in Crayford Marshes were measured with an array of piezometers over a large number of tidal cycles. They used these observations to develop an analytical model to reproduce the response of these piezometric pressures to the tidal variations. This model was then used to predict the increase in pore pressure in the sandy gravel layer during a storm surge event, and these estimates were then incorporated into the design of the levee. As a result of the calculations, it was recommended that the levee across Crayford Marshes should be reinforced on the landward side with a berm of 'heavy fill'. The resulting cross-sections through the levee are shown in Figure 9.35.

In this case, the berms were constructed out of sandy gravel, and this material has the benefit of being relatively free-draining negating the need for specific drainage details. However, such material is not particularly resilient to overtopping erosion. For the case of the Thames levees at Crayford, as the available freeboard was in excess of a metre, the likelihood of overtopping was considered to be low and no special resilience to overtopping erosion (other than grass) was incorporated into the design. If overtopping had represented a greater risk, then surface protection measures of the landward slope would also have been considered necessary.

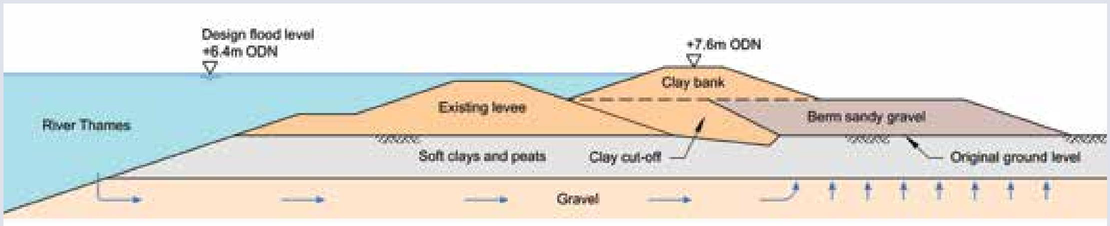


Figure 9.35 ***Landward berms designed to resist uplift of alluvial soils (after Marsland and Randolph, 1978)***

9.7.3.2 Impervious layers

Impervious layers work by reducing seepage through the levee itself and into the ground at the waterside toe of the levee, thereby increasing the length of the seepage path. However, low-permeability materials in this location may increase the risk of the rapid draw-down type failures, as low permeability soils tend to have low angles of internal friction. An example impervious layer constructed on the waterside slope of the levee on Yuba River, California is shown in Figure 9.36.

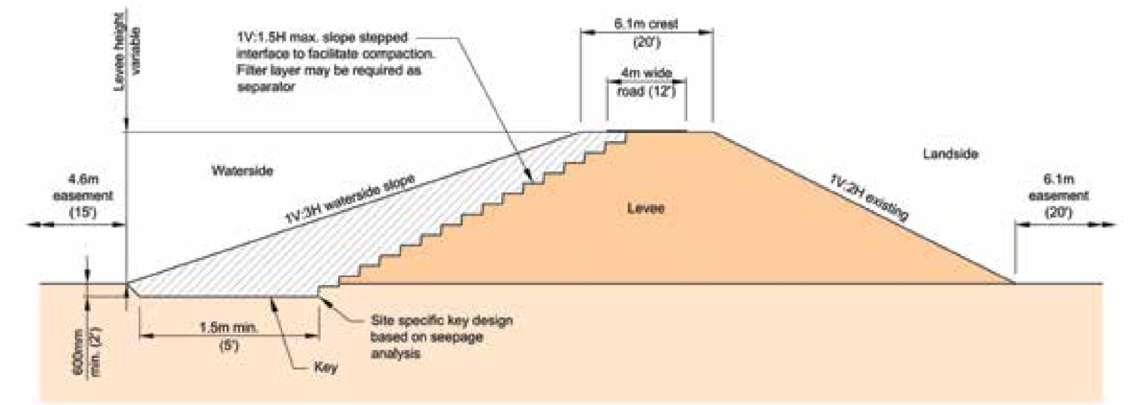


Figure 9.36 The construction of an impervious layer to reduce pore pressures within a levee (courtesy Mary Perlea, USACE)

9.7.3.3 Cut-off barriers

Cut-off barriers such as sheet pile walls or slurry trenches act by creating a vertical or near vertical barrier to the passage of groundwater (Figure 9.37). Ideally, they are installed fully through any permeable layer beneath the levee but if the permeable layer is thick, then it may be sufficient for the cut-off to partially penetrate that material. As a rule of thumb, vertical cut-off barriers begin to reduce uplift pressures and seepage flows when they penetrate through 85 to 90 per cent of the permeable layer.

For the case where the impermeable barrier does not fully penetrate the permeable layer, seepage analyses should be carried out to establish whether the lengthening of the flow path around the partial cut-off is sufficient to reduce uplift pressures beneath the landward slope of the levee to an acceptable level.

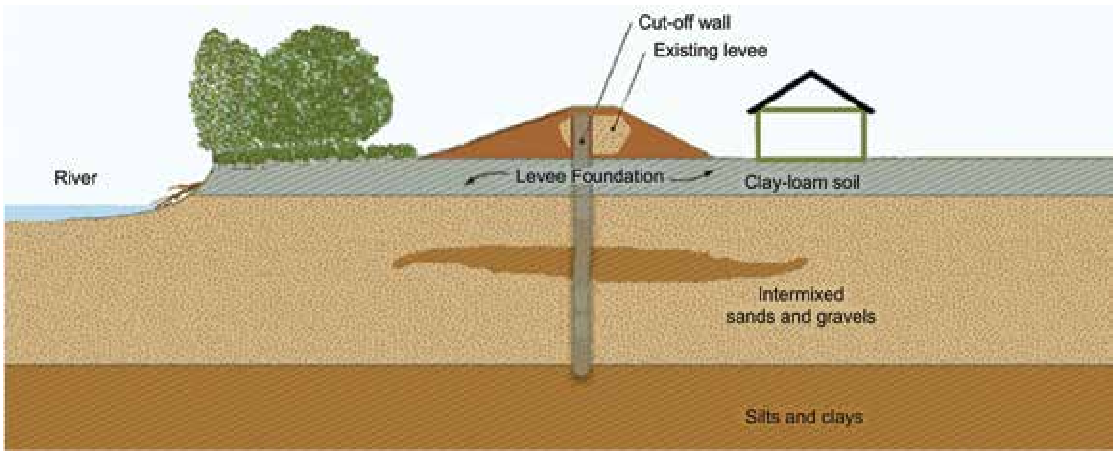


Figure 9.37 Slurry cut-off wall through levee and foundation to prevent seepage-related instability (courtesy Mary Perlea, USACE)

Figure 9.38 presents the results of seepage analyses performed by Chowdhury *et al* (2012) showing that a decrease of pore pressure and water level by construction of a seepage cut-off wall can have positive effects on both slope stability and the hydraulic gradient at the levee toe.

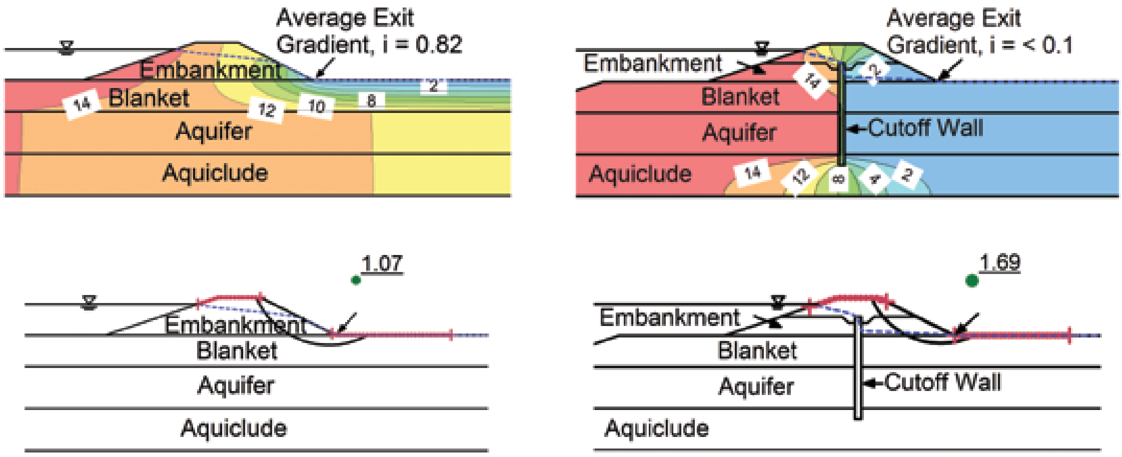


Figure 9.38 ***Effect of a seepage cut-off wall on seepage through the levee and its foundation and on slope stability (Chowdhury et al, 2012)***

9.7.3.4 Internal drains

Internal drains are generally used to control internal erosion rather than to improve levee stability. Cross-sectional details showing methods for controlling seepage through a levee through the use of internal drains are given in Figure 9.39. Further information on simple drainage systems to control seepage is given in USACE (2000).

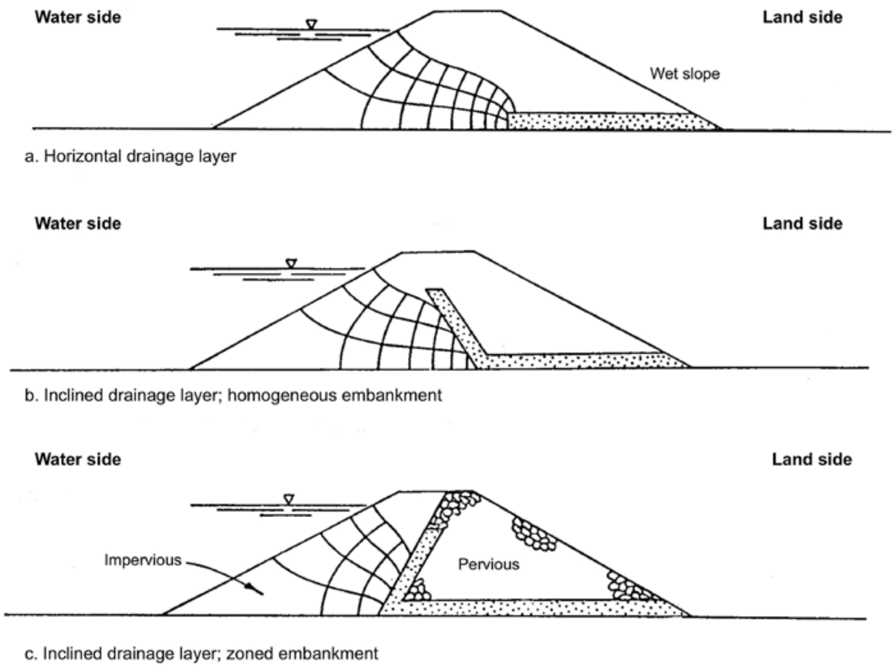


Figure 9.39 ***Control of seepage through a levee by internal drains (from USACE, 2000)***

9.7.3.5 Toe drains

Arrangements for controlling seepage through a levee by the use of toe drains are shown in the levee cross-sections in Figure 9.40. Seepage calculations should be carried out to determine the volume of seepage water that will discharge from the drains during the design flood event, and the drains should then be sized with sufficient capacity for this discharge, and some spare to provide resilience in more extreme events. A suitable means should be provided to carry the seepage water away from the levee (or back into the river/sea) in such a way that flooding is avoided. If an active pumping method is used as part of this process, the designers must consider the operational reliability of the system during the design flood event.

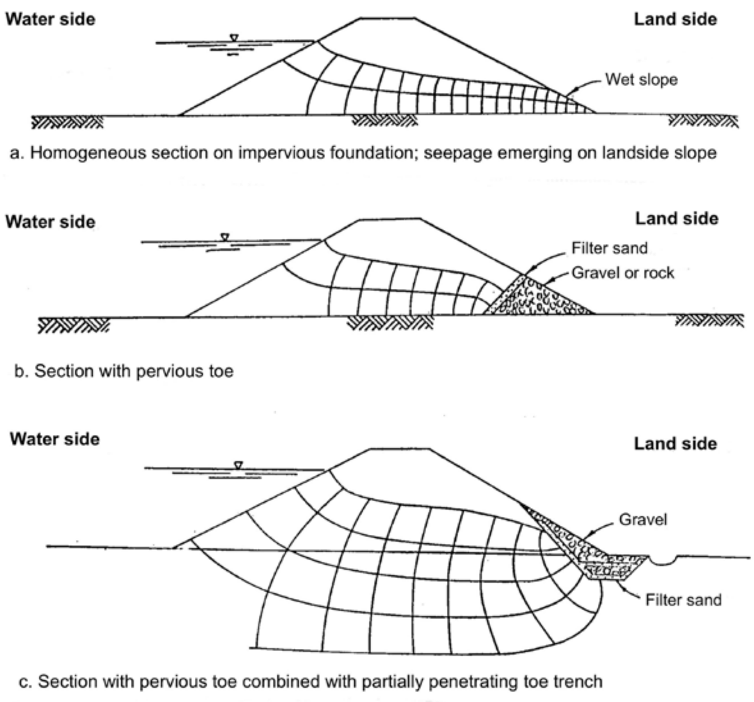


Figure 9.40 Control of seepage through a levee by toe drains (from USACE, 2000)

In cases where erodibility of the landward face is important, the simple granular stabilising berm shown in Figure 9.41 can be modified by the incorporation of surface protection measures (as described in Section 9.6), or the design of the berm can be modified to incorporate less erodible material. One solution in this case is to form the bulk of the berm out of cohesive materials, but also to install a drainage blanket or a chimney drain between the original levee and the berm. These berms have the benefit of increasing the stability of the landward slope during a flood situation and can be used to manage issues such as internal erosion and piping. Such berms were constructed on the landside levee slopes on the Sacramento River levees protecting the City of Sacramento and the area of Natomas (Figure 9.41).

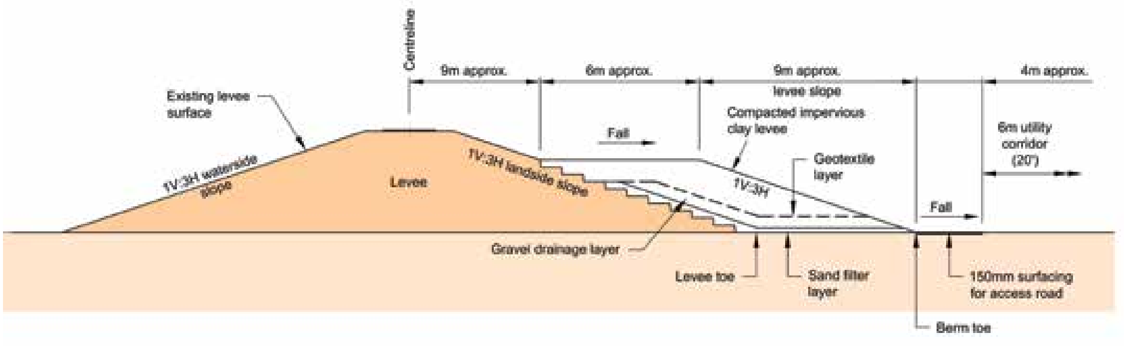


Figure 9.41 Stability berm with a chimney drain, Sacramento, California, USA (courtesy Mary Perlea, USACE)

In this case, the selection of the grading of the drainage material in relation to the materials used in the levee and the berm is of great importance. In particular, the stability of the materials against internal erosion (Sections 9.8 and 8.5) must be checked to avoid long-term deterioration. As the toe drains act as a focus for the collection of seepage flows, they can be subject to high seepage gradients and seepage velocities, which can cause rapid internal erosion and sudden failure if they are poorly designed, poorly constructed or if they are subject to more severe conditions than those for which they were designed. For example, a toe drain on the Feather River in California was subject to high flow rates and eventually blew out in 1996. This triggered a breach of the levee at that location, which resulted in three fatalities and extensive economic damage. Toe drain systems should be therefore be designed with an appropriate level of resilience (Section 9.7.3.7).

9.7.3.6 Relief wells

Relief wells can be installed at or near to the toe of the landward slope of the levee to reduce groundwater pressures in the foundation materials, thereby reducing the risk of uplift.

Relief wells can be passive (venting to the ground surface) or they can be pumped. Passive systems are preferred for flood control systems, unless there is a high degree of confidence that the pumped well system will always work during a flood (this may require a high level of redundancy and at least two different power sources). The water collected out of the relief wells can either be collected in a sump and pumped back into the river (as it is along the Missouri and Kansas Rivers) or it may be allowed to flow freely onto agricultural fields adjacent to the levee as is the case along the Mississippi River.

A typical detail of a relief well is shown in Figure 9.42. Where water can flow across an interface between different material types (such as between the natural ground and the filter material in the relief well), calculations should be carried out to check the internal stability of the interfaces between the materials (ie to ensure that suffusion and internal erosion are not likely to occur). Further design guidance for the design of relief wells is available in USACE (1996). An example relief well installation is described in Box 9.31.

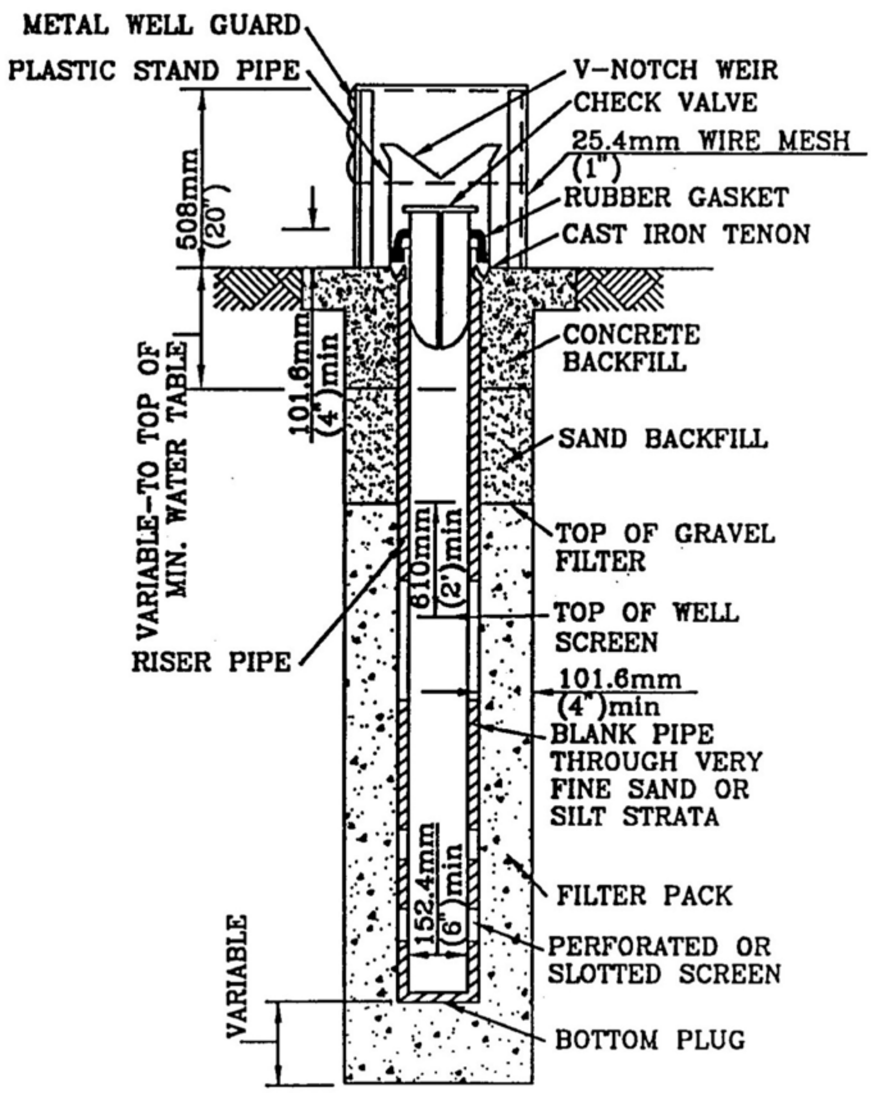


Figure 9.42 Relief well system (from USACE, 2000)

Box 9.31 ***Controlling pore pressures and seepage in levees near Lake Pontchartrain, New Orleans***

The Inner Harbour Navigation Canal (IHNC) Reach II, Emergency Interim Repairs, LPV–117, is an example of a project where measures were taken to control pore pressures and seepage beneath a levee.

The flood protection system at this location consisted of earthen levees with PZ-27 hot-rolled steel sheet piles installed from the crest. The sheet piles were capped with reinforced concrete to form I-walls. The top level of the I-walls was set at El. 12.5 ft (3.8 m), NAVD88. On plan, the I-wall extended a distance of about 1 mile (1.6 km). Pressure relief wells were installed at the landside toe of the levee throughout this reach as part of the original construction in the 1970s.

The wells discharged to the ground surface (about El. 0 to −2 ft, 0.0 to −0.6 m). Analyses of the flood protection system (and confirmed by piezometric data collected during storm surges in Lake Pontchartrain) indicated that the I-walls in this reach did not meet the design criteria for hydrostatic heave as the confining blanket on the landward side of the levee was found to be at a low level and was thin and of low density. The elevation of the ground surface at the levee toe was 14–18 ft (4.3 m to 5.5 m) below the top of the I-wall. Ground conditions at the site consisted of organic clay which was underlain by a layer of fine beach sand.

Part of the solution implemented along the east side of the IHNC in this reach included lowering the outlets of 21 existing relief wells by about 8 ft (2.4 m), installing 19 supplemental 40 ft (12.2 m) deep stainless steel relief wells discharging at the same elevation midway between the existing wells, and connecting all of the wells to a buried collector pipe that discharged into a drainage canal (whose water surface was maintained at about El. −10 ft (−3 m) by the operation of the nearby St Charles drainage pumping station. An 18 in (450 mm) diameter gate valve was installed in the new collector line. This will normally be closed to prevent the wells from working when they are not required (ie between hurricane events).

Construction in LPV–117 was completed in 2009. A similar solution was implemented in 2011 for LPV–119 (opposite LPV–117, 50 new wells and lowering the outlets of nine existing wells) along the western side of the IHNC along France Road. Figure 9.43 shows the connection of a new well to the new buried collector pipe, and Figure 9.44 is a view of the completed line of wells along the east side of the IHNC.

Figure 9.43 ***Connecting new relief well to buried collector pipe (courtesy Richard Bird, URS)***

Figure 9.44 ***Completed relief wells at the landside toe connected to a buried collector pipe (looking north toward Lake Pontchartrain) (courtesy Richard Bird, URS)***

9.7.3.7 Resilience and system fragility of groundwater control measures

When designing any drainage system for groundwater pressure relief a level of robustness and redundancy should be introduced that reflects the risk of failure. In particular, it is important to consider:

- the stability of the filter system separating the drain from the body of the levee, particularly the hydraulic gradients, the seepage velocities and the stability against internal erosion at the interfaces between material types
- the consequences of a local failure of the drainage system, either by internal erosion or by the failure, for example, of a pumping system
- how any water collected by the drainage system is managed; for example, if the water is pumped away from the collection system, what is the consequence of a pump failure?
- how the performance of the system can be monitored and maintained throughout the design life of the levee; for example, can the drains be maintained so that they do not become clogged over a period of years?

9.8 CONTROL OF INTERNAL EROSION

The issue of internal erosion is of critical importance to the designers of large dams. Such dams retain water for decades, and steady-state seepage conditions are generally reached. In these cases, internal erosion over extended periods of time can be highly damaging, but can occur in a way that is not obvious from the surface. However, it is a process that can rapidly accelerate once a critical condition is reached. Internal erosion is therefore of constant concern to dam operators.

While some levees in low-lying areas or alongside canals do retain water on an almost continuous basis, many other levees are only subject to their design water levels on an occasional or periodic basis, and they are therefore not subject to constant high seepage pressures. While this removes one of the causes of deterioration, these levees can go for decades without being tested. Deterioration of the levee over that period may reduce the ability of the levee to resist seepage (eg desiccation cracking, animal burrowing or internal erosion from a previous flood that was not detected). The design of adaptations to existing levees should therefore include an assessment of seepage flows and the resilience of the existing fill and filter materials to internal erosion.

Recent research (Benahmed and Philippe, 2012) has helped to advance knowledge and understanding in the area of seepage and internal erosion related to levees, and provides clear descriptions of four identified processes of internal erosion and a consideration of how these mechanisms can contribute to levee deterioration and failure.

9.8.1 Internal erosion – the basic processes

Benahmed and Philippe (2012) consider that internal erosion is related to all processes that involve the detachment and transportation of soil particles as a result of seepage flow through an earthen embankment such as a dam or a levee, or through the soils that constitute its foundation. They state that four different basic processes and methods can be identified, which contribute to internal erosion (introduced in Section 3.5 and described further in Section 8.5). These can be summarised as:

- **backward erosion** (Figure 9.45): the detachment of soil particles when the seepage exits to an unfiltered surface, leading to retrogressively growing pipes and sand boils
- **concentrated leak erosion** (Figure 9.46): the detachment of soil particles through a pre-existing, animal-made or man-made or natural path in the levee or its foundation
- **suffusion** (Figure 9.47): the selective erosion of the fine particles from the matrix of coarse particles under the action of a hydraulic gradient
- **contact erosion** (Figure 9.48): the selective erosion of the fine particles from a soil at the location through contact with a coarser layer.

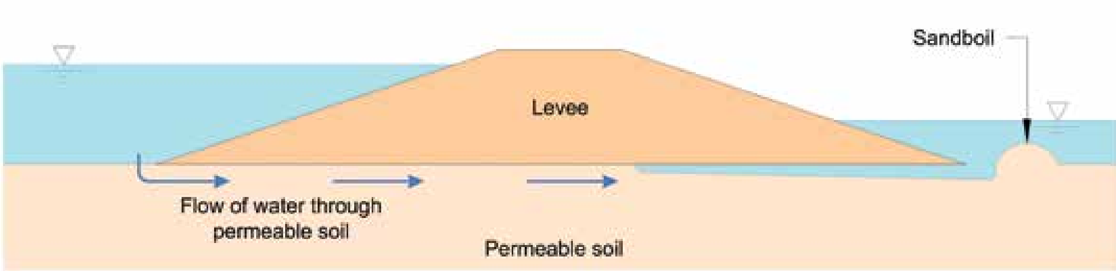


Figure 9.45 Typical example of backward erosion in a sandy layer (after Koenders and Sellmeijer, 1992)

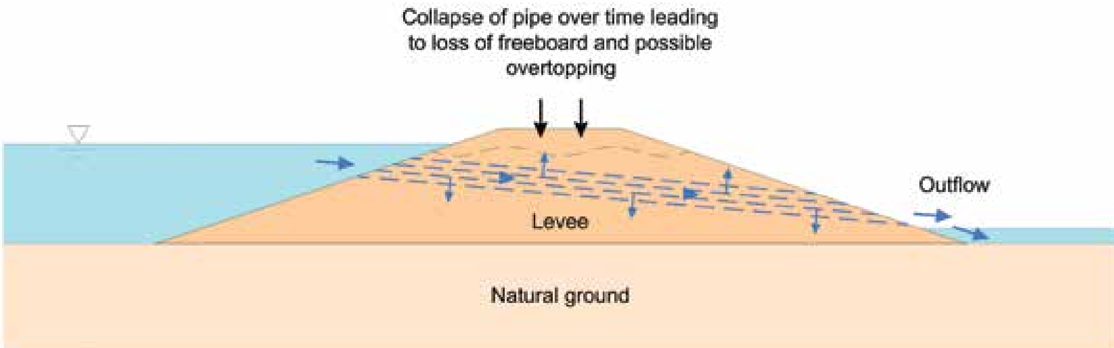


Figure 9.46 Typical example of concentrated leak erosion (after Fell and Fry, 2007)

Figure 9.47 Typical example of suffusion (after Fell and Fry, 2007)

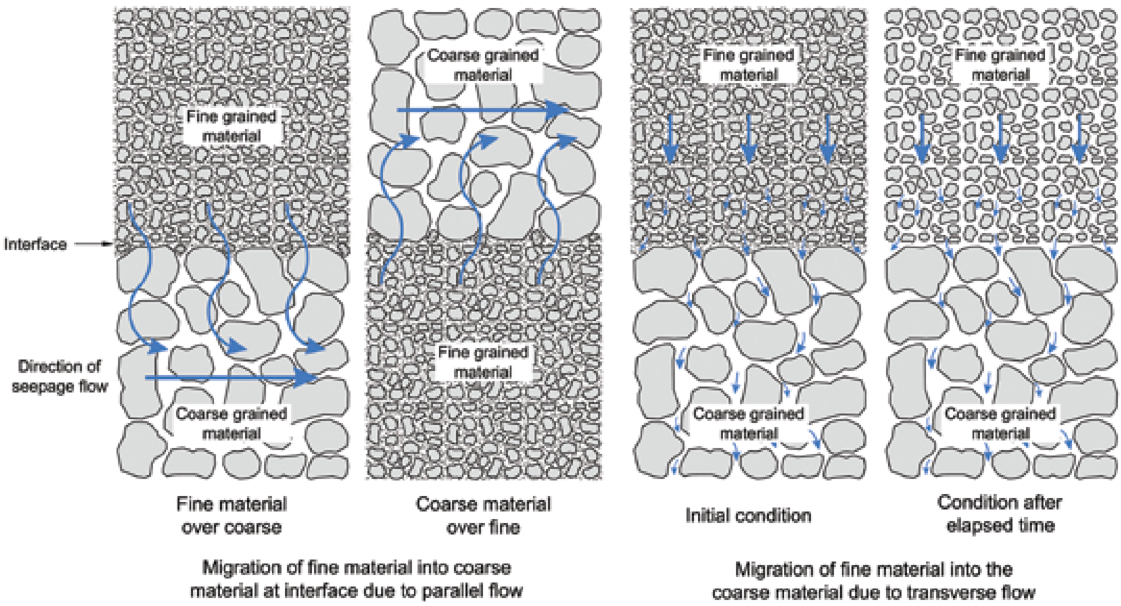


Figure 9.48 Sketch of contact erosion with parallel flow (a), and with transverse flow (b) (after Fell and Fry, 2007, and Ziems, 1969)

Assessing and mitigating the potential for internal erosion and seepage

When designing new levees or adaptations to existing levees, the vulnerability to internal erosion should be checked, and this may involve the consideration of some or all of the following steps:

1. Assess the permeability of the levee fill, as well as of the foundation soil layers (discussion in Section 9.7.2 and methods described in Chapter 7), remembering that the permeability of fill materials placed in the levee will depend on both the soil type and the level of compaction of the fill material.
2. Carefully assess the anisotropy (ratios between vertical and horizontal permeability) of the foundation soils, particularly the ratios between the horizontal permeability of any underlying aquifer and the vertical permeability of superficial, less permeable, blanket layers. Geomorphic conditions of the natural soils on which the levee is built may need to be studied to understand the formation of the soil strata, as this can affect the permeability of the individual soil layers.
3. Define the design flood conditions in terms of hydrographs showing the rise and fall of the flood level, the peak flows and the duration of each of the components of the flood.
4. Identify and anticipate seepage issues, based on the duration of flood and the nature of the soils in the levee and in its foundation. For low-risk levees subject to infrequent and short duration flooding, an assessment of the condition of similar local levees may determine whether an (occasional) process of internal erosion will threaten the levee's stability. Where the risk is low, an observational approach may be adopted to monitor performance and only make levee modifications if considered necessary.
5. If appropriate, carry out an appropriate seepage analysis, identifying characteristic phreatic surfaces, hydraulic gradients and internal flow velocities (Sections 9.7 and 8.3.1).
6. Consider the likelihood of suffusion of the levee soils, using the assessment methods set out in Chapter 8 and bearing in mind that some soil types are more susceptible than others. An example of a susceptible soil type would be a bimodal sandy gravel with less than about 25 per cent of sand, where overburden pressures may be carried by the primary framework of the gravel, leaving the sand relatively unloaded and hence free to move (Skempton and Brogan, 1994).
7. Assess the possibility of one or more of the four modes of internal erosion using the calculation techniques provided in Chapter 8.
8. Determine or estimate the exit velocity of any seepage to evaluate the possibility of hydraulic heave, boiling or internal erosion. This should consider the output from any seepage analyses and the nature of the various materials involved, particularly the type of material and the grain size distribution. Both the levee fill and the foundation soils should be considered.
9. If necessary, take steps to control or prevent internal erosion:
   - i check the stability of any boundaries between materials using methods provided in Section 8.5
   - ii assess the filter characteristics of the levee material using the techniques described in Section 8.5.5, considering the potential for the fill material to be self-filtering
10. If the potential seepage through the levee is considered to be significant and a threat to stability or a potential cause of deterioration, then the designer should take steps to mitigate the problem. Three main groups of measures are possible:
    - i lowering the seepage pressures in the levee through the use of berms or various types of drains (Sections 9.7.3)
    - ii reducing the hydraulic gradients by lengthening the drainage path by using impermeable layers and seepage cut-off walls (Section 9.7.3).
    - iii managing the interface between material types by appropriate filter design to prevent internal – this final option is now described (Section 9.8.2).

9.8.2 Filter design

Filters should be designed using the tools described in Section 8.5.5. The specified filter gradation should filter the levee's core material and the natural ground, if appropriate. It should also be permeable enough to avoid excess water pressure build-up. In some cases where the core and surface zones of the levee are formed of very different materials, a two-layer filter may be required.

The granular filter materials themselves should be hard, durable and stable:

- the materials should not break down during transportation, placement and compaction. Over-compaction can reduce permeability and increase fines
- the filter gradation should be designed to avoid segregation during placement or under the application of an adverse hydraulic gradient during a flood
- widely graded filters can be internally unstable (Skempton and Brogan, 1994). Internal stability can be checked by dividing the grain–size curve into two gradations and checking whether the coarse gradation meets the filter requirements for the fine gradation (ASDSO, 2003).

Sands used to construct filter diaphragms, filter collars, or chimney filters should be filter compatible with the levee zones being protected and should be able to deform and fill any cracks that may be propagated to the filter. An important property required for such sand filters, particularly those subject to continuous high water pressures, is referred to as *self-healing*. Without this property, cracks could propagate through the filter, and the filter would not satisfy its intended function.

- Vaughan and Soares (1982) and USACE (1993) have described a simple test for evaluating the self-healing ability of a filter, based on the percentage of fines and the plasticity of the fines. Filter designs usually require a small percentage of non-plastic fines (usually less than five per cent after placement in the embankment and compaction of the filter) so that they have appropriate permeability and self-healing characteristics.
- Fine, narrowly graded filters are less self-healing than widely graded, coarse filter materials, although the latter may need to be checked for internal stability (see above).

Filter materials are rarely directly available from an on-site source, and are generally obtained in one of two ways:

- by processing from materials near the site – to increase construction flexibility, designers should always determine the range of compatible filter gradations from those present in the natural foundation materials
- by purchasing from aggregate suppliers, ideally selecting standard aggregate gradings from prevailing specifications, such as ASTM C33/C33M–13 in the USA or BS EN 13242:2013 in Europe. This option may bring benefits, in that the standard materials are readily available and economic to use. However, the gradings may not always suit preferred levee fill or the natural foundation soil.

During construction:

- filter materials should be controlled to avoid them becoming contaminated or segregated during construction – the use of narrowly graded materials helps to prevent this problem
- placement of filter materials should be avoided during freezing temperatures, as frozen filter material cannot be properly compacted.

In selecting the final design for the filter, there will be a need to balance the fact that fine, narrowly graded filters are less self-healing, and the fact that such filters offer a way to overcome contamination and segregation problems during construction.

In some situations, geotextiles may be used as an alternative to graded filters, using the design tools provided in Section 8.5.5.2 and following the guidance provided in Section 9.12.6, as needed.

A useful summary of filter design methods is included in the technical manual by FEMA (2009 and 2011), including the Natural Resources Conservation Service (NRCS) design procedure (Natural Resources Conservation Service, 2007), USACE (2004), and the US Bureau of Reclamation (1999).

9.9 MASS STABILITY THROUGHOUT LEVEE LIFE

The general steps normally followed to verify stability as part of the design process are as follows:

- **Characterise the levee**, considering both the size and complexity of the structure and the level of risk (reflecting particularly on the consequences of failure).
- **Evaluate the design loads** (actions) including plausible design load combinations.
- Establish a ground model and a **determination of characteristic parameters** for that design (unit weights, strengths, stiffnesses, permeabilities, compressibilities etc).
- **Identify all potential failure mechanisms** and use experience or simple calculations to discount the non-critical mechanisms.
- **Establish acceptance criteria** (minimum factors of safety or partial factors, acceptable seepage conditions and maximum acceptable displacements under particular situations).
- **Use seepage calculations** or experience to establish phreatic surfaces for each material, within or beneath the levee, for the design situations.
- Perform calculations or use experienced-based judgement to **verify ultimate limit state stability** (Section 9.10). These assessments should include slope stability calculations for all potentially critical mechanisms, uplift checks (eg toe heave) and hydraulic calculations (eg internal erosion) and complex soil-structure analysis (if necessary).
- Consider ways to **counteract instability** due to slope failure and uplift, as appropriate, including milder slopes, berms and measures to control seepage and uplift, as discussed in this section.
- Carry out further calculations to **verify stability**.
- **Finalise and rationalise the design** into drawings, standard details and specifications.

Different types of methods can be used, depending on the complexity of the levee and the perceived level of risk:

- stability charts can be used to estimate the factor of safety of simple geometries and ground conditions
- hand calculations can be used to establish the stability of slopes against shallow slips
- limit-equilibrium slope stability programs can be used to determine the factor of safety of complex geometrical arrangements and extensive combinations of soil types, soil characteristics and groundwater conditions. A critical slip surface is identified by determining the factor of safety for a large number of potential failure mechanisms, establishing the lowest one. The methods are widely used
- finite element methods, which are based on a numerical continuum, can be used to determine detailed stress patterns. They have the benefit of finding their own failure mechanisms and can even determine likely displacements. They can be highly sophisticated models, and this can make them expensive to run and difficult to check.

The details of these methods are described in Section 8.6. The more complex methods involving software can only deliver realistic and accurate results if the input is correct and the bounds to the analyses, including the representation of any embedded walls, are correctly specified.

Establishing that a levee will not become unstable at any stage of its life typically involves consideration of three main stages in the life of a levee, which are:

1. At vulnerable stages during the construction process.
2. During flood events.
3. In post-flood situations, including rapid draw-down and reverse flood loading.

Where applicable, levees should also be assessed for their resistance to seismic activity.

9.9.1 Mass stability – designing to avoid failures during construction

Mass instability failure of a levee during construction will have a major impact on the construction programme, cost and viability. The foundations of a failed levee will almost certainly have been damaged by the failure, and further construction will be characterised by significant and unpredictable displacements. It is not uncommon to have to relocate the levee at this stage with significant planning cost and time implications for the construction programme.

If analysis indicates that the design configuration cannot be built safely in one stage of construction, then adaptations to the design will be required, such as:

- using flatter side-slopes or berms
- constructing the levee in stages so that the foundation soils can consolidate over time and consequently gain strength
- improving the strength characteristics of the foundation soils, if this can be achieved without introducing seepage paths.

Construction on weak soils and risk of undrained failure

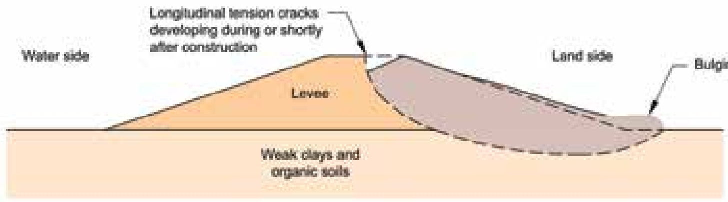
Levees are often constructed on alluvial clays, silts and peats. These soils are inherently weak and are generally of high compressibility. On these materials, the process of levee construction will generate excess pore pressures in the foundation soils and, as a result, the maximum shear strength that can be mobilised in the ground is low. In addition, consolidation settlement of the ground after construction can be large. The waterside and landward stability of levees built on soft alluvial or estuarine soils often becomes critical when the levee exceeds about 3 m in height (Jardine and Hight, 1987 and Tavenas and Leroueil, 1980). Behaviour of the first stage of construction up to the critical height is controlled by the initial undrained shear strength characteristics of the ground. Behaviour beyond this critical height is dominated by the generation of significant excess pore pressures in the foundation materials and pronounced lateral displacements of the levee. So the following must be balanced against each other:

- the benefits of building each lift of the levee as high as possible to dissipate excess pore pressures in the underlying soils and gain strength in them for further construction
- the imperative of avoiding undrained failure mechanisms (Boxes 9.32 and 9.33) arising from the over-rapid loading of the underlying ground.

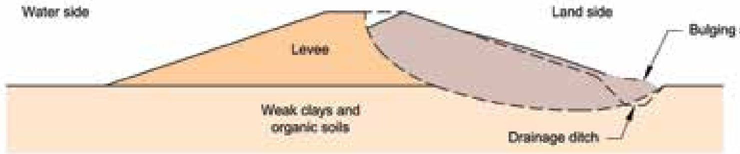
Box 9.32 *Managing construction stage failures – summary*

- Levee construction, raising or repair should be carried out safely and in such a way that will not impair future levee performance.
- All construction stages should be considered and analyses carried out if stability is uncertain.
- Particular care should be taken when raising levees to heights of more than 3 m above the floodplain.
- Stability analyses should be undertaken using strengths that are representative of those mobilised for each limit state considered. For example, the shear strength mobilised in a soft clay layer by translational shearing beneath a berm may be lower than the strength mobilised in the same material beneath the centre of the levee.
- The impact of excavating ditches in close proximity to a levee should be considered, regardless of whether the levee is being raised or not.

Deep rotational failure associated with weak soils during construction



Deep rotational failure associated with excavation of trench at levee toe



Translational failure of berm during construction

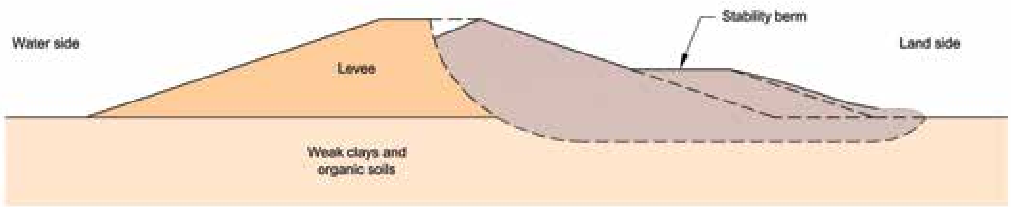


Figure 9.49 ***Construction-related failures***

Box 9.33 ***Levee construction failure***

Figure 9.50 shows an example of a levee in California that failed soon after construction on a foundation of soft alluvial soil (in this case organic clay and peat). Horizontal and vertical displacements of the levee were monitored during construction, but the contractor neglected to read the instrumentation and raised the levee rapidly. The levee failed immediately after construction.

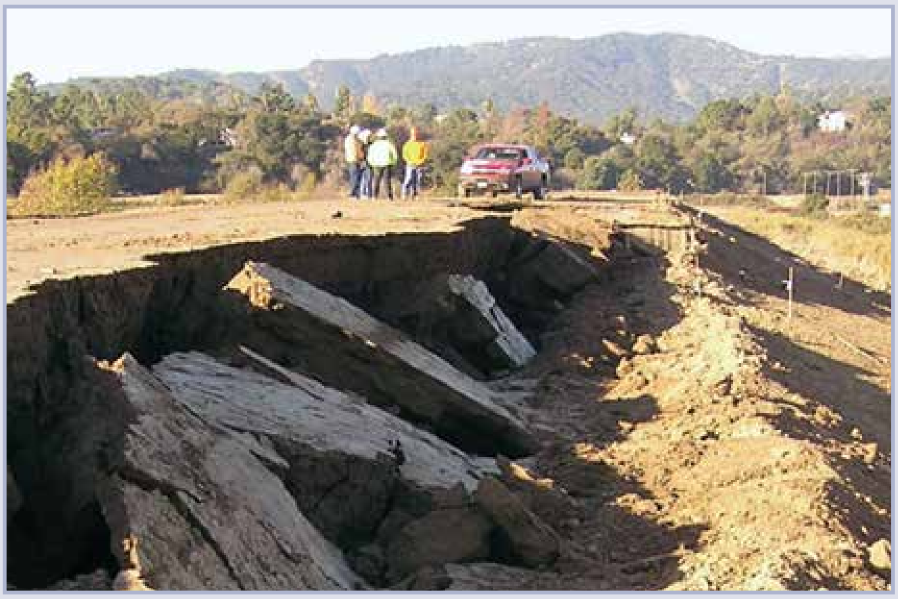


Figure 9.50 ***Levee failure at the end of construction, California, USA (courtesy Mary Perlea, USACE)***

In another example on the Missouri River, a levee was constructed on a very soft high plasticity clay foundation and built using by high plasticity clay material dredged from a nearby lake. It was placed without compaction. The slopes of the levees were 1V:3H on both the landside and waterside. The levee was constructed in one stage and, immediately after the end of construction, the levee slopes began to fail on both sides of the embankment. The material used for levee embankment had a liquid limit above 80 with a friction angle of less than 18°. The foundation soil was also a soft, high plasticity clay. In all, 11 slope failures occurred within the year after construction.

Stability analyses should therefore consider the strengths that can be mobilised in both the levee fill materials and the foundation soils during these situations. Critical factors are likely to include:

- the nature of the foundation soils – for fine-grained (cohesive) soils, it is likely that undrained shear strengths (either the initial shear strengths or those developed through consolidation during the construction of the levee) will be critical, but for granular soils, drained strengths should be adopted
- the levee geometry and strength anisotropy in the foundation soils (for example, the use of a large berm may result in a long sub-horizontal failure surface beneath the berm and, as a result, lower shear strengths may be mobilised within the foundation soils than would be the case for a near circular failure surface, Ladd, 1991)
- the nature (if any) of any ground improvement work
- the rate of levee construction.

Construction risks including health and safety issues should also be taken into account (see Chapter 10). One way to mitigate construction risks in building levees as high as possible at each stage without causing failure (Jardine and Hight, 1987, Leroueil *et al*, 1990) is to instrument and monitor the levee for geotechnical response (details of typical methods of instrumentation and monitoring can be found in Chapter 7). Measurements that can be useful to determine when the levee reaches its critical height include:

- the settlement of the crest of the levee
- the horizontal displacement of the levee toe
- the elevation of pore pressures within any soft, compressible soil layers beneath the levee and their subsequent dissipation after construction has paused.

To prevent undue delay in construction, preliminary analysis should be performed to predetermine the critical values of these measurements. The monitoring frequency should increase as these critical values approach. At a predetermined level of measurements, construction of the levee should cease until it has been established that construction can recommence safely (eg excess pore pressures within the founding materials have fallen to acceptable levels). Such an approach should only be adopted if remedial measures can easily be put in place should the measurements not validate the design assumptions.

Ground treatment methods that may accelerate the construction process, help prevent failures during construction or reduce the magnitude of long-term settlements should be considered. These methods may involve some form of drainage (Section 9.7), in order to control or reduce excess pore pressures. However, care is required to avoid compromising the effectiveness of the levee as a water retaining structure. (For example, prefabricated vertical drains that are often used to accelerate the construction of embankments on soft ground probably cannot be used, as these drains require a horizontal gravel collector drain, placed at or near to the original ground level, which would create a seepage path through the levee).

Once it has been established that the ground has improved adequately for construction to proceed, subsequent lifts should be placed in a controlled manner (Tavenas *et al*, 1978, Hight and Jardine, 1987, Jardine and Hight, 1987, Ladd, 1991 and Smith, 1992). The incremental heights of the subsequent lifts are likely to be much smaller than the first lift, and this fact has a great influence on the timing and feasibility of levee raising. This practical restriction on soft alluvial soils applies to both new levees and existing levees built to critical height and left to consolidate naturally.

Analysis tool selection

It is the designer's responsibility to identify the potential failure mechanisms and to use appropriate tools to check stability for each of the critical mechanisms. Slope stability analyses (Section 8.6) are usually carried out using limit-equilibrium software packages. In this case, it is the designer's responsibility to check that the software is capable of replicating the critical failure surfaces and of applying the appropriate shear strengths for any design situation, including:

- limit-equilibrium formulations that allow for non-circular slip surfaces, particularly if large berms are adopted
- shear strengths that are appropriate for the direction and nature of the applied shear stresses (eg reduced undrained shear strengths for direct simple shear conditions beneath wide berms or along long failure surfaces)
- increased shear strengths over time, resulting from the consolidation of cohesive soils beneath existing levees or the first stage of construction of new levees (taking account of the rotations of principal stresses beneath the levees after the first stage of levee construction).

9.9.2 Mass stability – designing to avoid failures during floods

Mass instability during a flood, leading to sudden failure or loss of levee crest, may induce overtopping which could lead to breach, especially where erodible fresh soil is exposed around the failure. Some of the resulting failure mechanisms will only be evident when the levee is tested by high river or coastal water levels.

Evaluation of key issues arising during flooding

A levee's margin against mass instability towards the landward side (as measured by the factor of safety or controlled by the partial factors) will always be reduced as the flood level increases. The presence of the floodwater can affect mass stability in a number of ways including the following:

- elevated pore pressures in the levee and the underlying natural soils can trigger slope instability as shown in Figure 9.51

- uplift of the soils beneath the toe on the landward side of the levee, or sand boils at this location, causing a loss of resistance and support, thereby triggering instability.

The likelihood of these failure mechanisms depends on the composition of the levee and foundation soils typically found on floodplains, including natural interlayering of soft clays, low density peats, highly plastic soils and permeable granular materials.

Figure 9.51 Slope stability failure due to elevated pore pressures during high water in the Sacramento River on the Natomas Levee in California, USA (courtesy Mary Perlea, USACE)

Before undertaking any calculations, a conceptual model of the completed levee and the underlying ground conditions should be developed to identify the potential failure mechanisms, including both circular and non-circular slips. Typical issues that may need consideration include the following:

- For levees located near a bank of a river or the active coastal zone, potential erosion or instability of the riverbank or beach itself must be assessed when considering the stability of the levee.
- The susceptibility of the levee construction to cracking as a result of desiccation (eg where high plasticity clays have been used) needs to be assessed, leading to a reduced factor of safety, particularly if they fill with water as a result of heavy rainfall or inundation during a flood.
- Excess under-seepage and the resulting uplift effect, often identified by the initiation of sand boils on the landward side of the levees (Section 9.7) has to be considered. Where low-density soil such as peat overlies an aquifer connected to the floodwater, uplift can lead to a rupture of an impermeable or lightweight soil near the landward toe of the levee (Figure 9.51). This was a common cause of levee failure during the 1993 Mississippi River flood and the 2011 Missouri River and Lower Mississippi River floods. Models which address the resulting loss of passive support on the landward side of the levee are described in Section 8.6.3 and include that by Van (2001) described in Box 8.19. Typical solutions to overcoming these failure mechanisms discussed in Section 9.7.3 include:
  - the use of berms on the landward side of the levee to introduce toe-weighting
  - cut-off walls, impermeable barriers to inhibit seepage through or beneath the levees
  - drainage systems such as toe drains or pressure relief wells on the landward side of the levee to control seepage or uplift pressures.

While solutions can be designed to overcome problems of seepage and uplift, as described in Section 9.7.3, the incorporation of relatively stiff structural elements such as steel or concrete cut-off walls or drainage pipes may adversely influence the behaviour of the levee during a flood.

9.9.3 Mass stability – designing to avoid failures after floods

After long durations of exposure to water, levee slopes can become saturated and softened. This is particularly the case for large rivers such as the Mississippi River in the USA or the Murray River in Australia, where flood durations can be measured in weeks rather than in days, but may be followed by eventual relatively rapid lowering of flood levels. If the levee is constructed of less permeable soils such as clays or silts, the groundwater levels in the levee cannot drop at the same rate as the fall in river level (Figures 9.52 and 9.53). The resulting over-elevated pore pressures can trigger surface slumping of the levee's waterside slope and, in extreme cases, deep-seated failures, particularly if the waterside face of the levee is over-steep. Such failures can be of significant size (Figure 9.54) and can leave the levees vulnerable to subsequent flood events if there is insufficient time for repairs before the next period of high water.

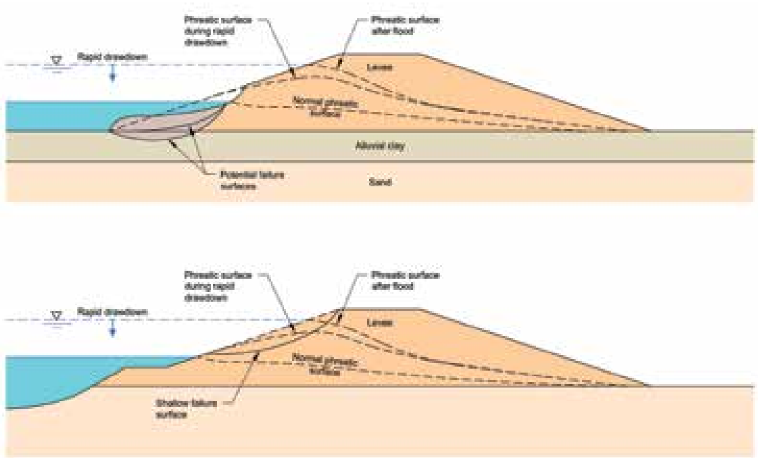


Figure 9.52 Rapid draw-down failure mechanisms after a flood

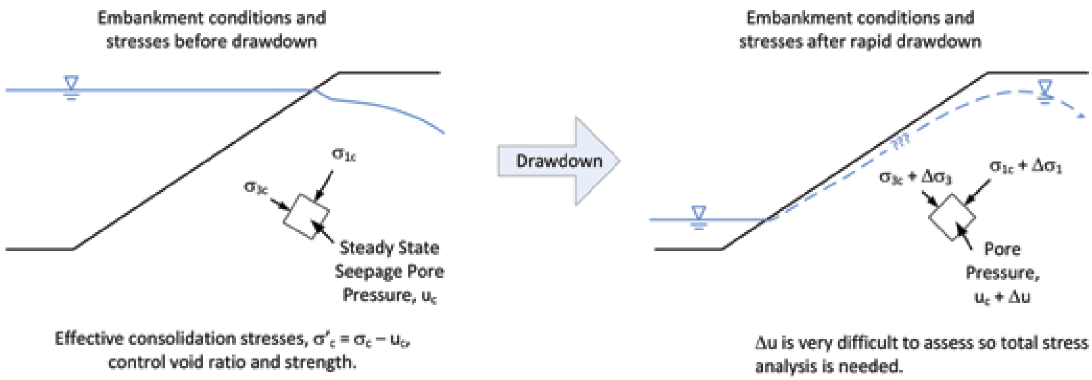


Figure 9.53 Stress conditions resulting from rapid draw-down (from VandenBerg, 2011)

Figure 9.54 Mississippi River failure attributed to rapid draw-down (USACE, 2003)

El Mountassir *et al* (2011) describe the behaviour of a newly constructed levee in Indonesia, and note that rapid draw-down failures were a common feature after periods of high water level. A review of such historical performances of similar levees in the location of the new or modified levee may help to determine how it is likely to behave under rapid draw-down conditions.

The design analysis should also check for stability in situations where floodwater could become trapped on the landward side of a levee, while the water level on the waterside falls rapidly.

Where potentially deep failure mechanisms are identified, these are likely to be costly to repair and may initiate a series of events leading to breach. On the other hand, shallow rapid draw-down failures identified that are no deeper than, say, 0.3 m may not always occur in practice. This is because of increases in surface stability due to unquantifiable phenomena such as the reinforcing effect of grass turf and the apparent cohesion of the soil. Further, occasional surface slumping type failures can usually be tolerated (Box 9.34), as the cost of occasional repair will normally be far smaller than the cost of having to reinforce or flatten the slopes of the levee along its entire length by design. Design should therefore concentrate on providing resilience to surface slumping through the selection of suitable surface reinforcement or modification, and focus the fully engineered solutions on avoiding the larger rapid draw-down failures which are deeper than, say, 0.3 m.

As well as conventional surface reinforcements, other forms of surface treatment can be considered to provide resistance against of rapid draw-down failure.

One option is to incorporate layers of geosynthetics into the waterside slope of the levee as it is constructed. As Han *et al* (2008) identify, these layers of reinforcement can increase the factor of safety of the levee against rapid draw-down failure to an acceptable level (Figure 9.55).

A different solution is to use a layer of coarse free-draining granular fill on the waterside levee surface. The free-draining characteristics and frictional strength characteristics have the effect of lowering the phreatic surface and increasing the factor of safety against rapid draw-down. However, the use of these materials will also impact the erosional resistance of the levee surface and may increase the levee's permeability. A sacrificial layer of topsoil (or any material allowing vegetation to grow) and grass protection placed on top of the granular soil may be helpful. In any event, if granular material is placed on the waterside slope, an impermeable core or barrier should be constructed behind it to prevent excessive seepage through the rest of the levee and the possibility of internal erosion.

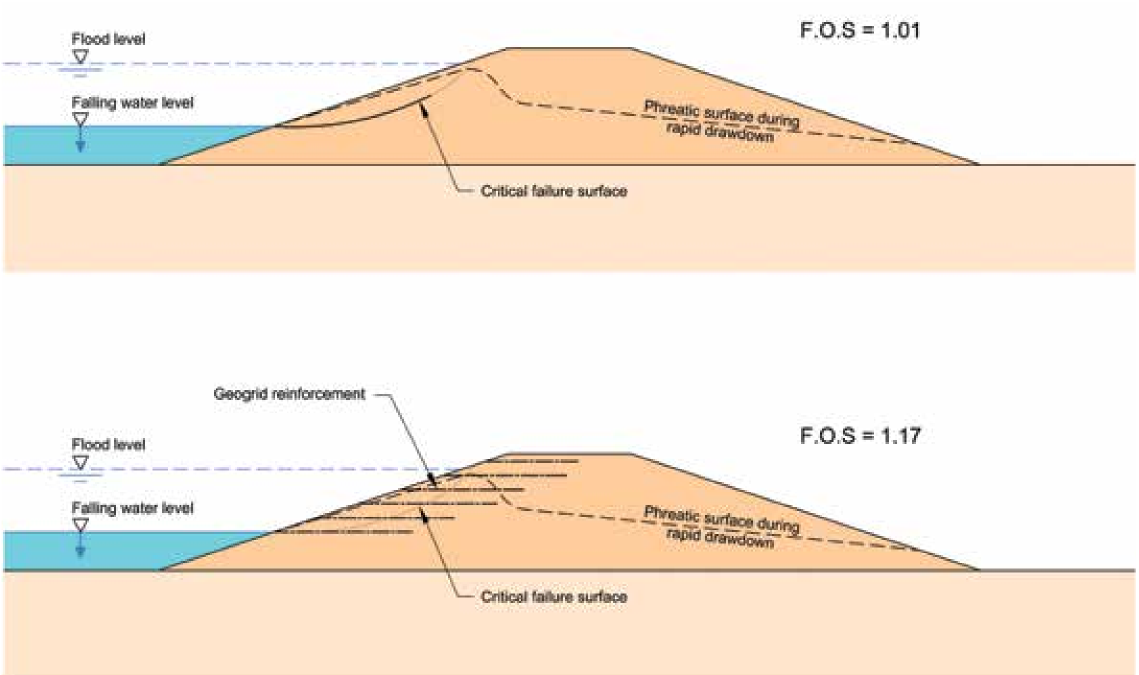


Figure 9.55 ***The use of geotextile reinforcement to overcome rapid draw-down (after Han et al, 2008)***

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Box 9.34 ***Rapid draw-down failure***

Levees constructed from high-plasticity and low-permeability clays along the Sacramento River have been subject to numerous waterside slope failures after flood events (Figure 9.56). However, while the failures have had the effect of damaging and steepening the waterside slope and, in a few instances, reducing the crest width, they do not generally create a situation where there is an imminent danger of a major failure of the levee, or a breach.

Figure 9.56 ***Waterside slope failure after rapid draw-down, Sacramento River levee, California (courtesy Mary Perlea, USACE)***

A summary of steps to design remedial measures to deal with rapid draw-down problems is given in Box 9.35. Similar measures may need to be incorporated into any remedial works that are considered necessary, to repair rapid draw-down failures that have occurred after a flood.

Box 9.35 ***Design steps to deal with rapid draw-down problems – summary***

1. Assign appropriate strengths to the levee materials for the rapid draw-down situation.
2. Using historic river hydrographs and extrapolating for the design water surface elevation, understand the action of the water on and within the waterside slope as the floodwater level drops rapidly but the levee remains saturated, and carry out steady-state or transient seepage analysis as appropriate (Section 9.7) to determine the phreatic surface and pore pressures.
3. Consider the reinforcing effect of grass turf and other beneficial elements such as geotextiles or surface protection systems.
4. Carry out slope stability calculations using the stability tools provided in Section 8.6, but using the elevated pore pressures determined in Step 2.
5. For shallow rapid draw-down failures estimate the time necessary to repair the levee to its pre-flood condition and use this information to decide whether such failures need to be prevented by design or can be tolerated.
6. Discuss the above issues with the levee owner and decide if a robust and conservative solution is required or if a wait-and-see approach can be adopted.
7. If it is concluded that there is a risk of deeper-seated rapid draw-down failures that may involve damage to the crest, or if it is decided that measures are required to tackle shallow rapid draw-down failures, then it is necessary to design appropriate measures such as:
    - i flattening the waterside levee slope
    - ii incorporating a berm on the waterside slope
    - iii protecting the waterside slope with surface protection measures that assist the drainage of the slope face and have the effect of driving the critical surface deeper into the levee reducing the likelihood of failure
    - iv reinforcing the waterside slope with geotextiles or geosynthetic reinforcement
    - v buttressing the waterside slope by constructing a rockfill or gabion support berm.

9.9.4 Seismic design for levees

While the concurrence of severe floods with significant seismic events is unlikely, a seismic event during a non-flood situation can cause extensive damage to a levee, which would leave it vulnerable if it were not repaired before the next major flood.

Earthquake design for levees is often dominated by the risk of liquefaction of the structures themselves or of their foundations, which could cause uncontrolled settlement or lateral displacement. USACE (2000) notes that levees constructed of, or founded on, loose cohesionless materials are particularly susceptible to failure due to liquefaction during earthquakes. Indeed, it is possible that only levees constructed of, or founded on, potentially liquefiable materials are vulnerable to earthquakes (USACE, 2012b).

Two basic modes of distress can be induced in levees as a result of liquefaction during an earthquake (USACE, 2012b):

1. Flow slides or post-earthquake slope instabilities which occur when the static driving shear stress is larger than the resisting strength that can be mobilised. In general, flow slides are associated with ground deformations that are of sufficient magnitude to constitute a structural failure. Even if failure does not occur, the softening of the levee or its foundations may leave it vulnerable to further deformation or failure in the event of a flood.
2. Lateral spreading or earthquake induced deformation involves the inertia-driven accumulation of deformation in cases where collapse type failure mechanisms do not occur. Although lateral spreading or earthquake induced deformation generates less displacement than a flow slide, large areas (or long lengths of levee) may be affected by significant drops in crest level.

The possible effects of these distress modes on a levee are summarised in Table 9.14.

Other major concerns include:

- potential transverse and longitudinal cracking, which are important for frequently hydraulically loaded levees. Transverse cracks can develop between liquefied levee reaches and non-liquefied levee reaches or at locations where liquefied levee reaches contain or abut appurtenant structures with rigid or deep foundations
- potential crest loss or lowering, which can lead to overtopping and damage to brittle structural elements (soil-cement slurry walls etc), and can compromise the ability of the levee to safely retain high water.

Table 9.14 Possible modes of damage induced by liquefaction on levees (USACE, 2012b)

Case	Stress condition along critical slip surface	Possible effect of liquefaction on a levee
Case 1: flow slide or post-earthquake Instability	Static driving shear greater than available strength resisting collapse	• loss of freeboard due to flow slide
		• major longitudinal cracking
		• transverse cracking at the ends of sliding zones
		• loss of freeboard-induced overtopping
		• transverse crack-induced piping
Case 2: lateral spreading or earthquake induced deformation	Static driving shear less than available strength, but static shear stresses plus inertial shear stress during shaking periodically exceed available shear strength causing displacement but not collapse	• loss of freeboard due to settlement
		• longitudinal cracking due to lateral spreading and associated settlement
		• transverse cracking due to sharp changes in foundation conditions
		• a two-way failure, possibly leading to additional settlement and more embankment distress

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Practice for the evaluation of seismic risk to levees (as opposed to dams) and the resulting design procedures is well developed in the USA and Canada, particularly on the western seaboard, (USACE, 2000, USACE, 2012b, Golder Associates, 2011 and DWR, 2012). The evaluation guidance which follows is largely based on these documents.

Levees that frequently retain water or protect significant landside physical assets or populations should have a seismic evaluation as part of the design process if they lie in an area at high risk of earthquakes. The issues that should be considered for evaluation of levees in seismically active areas are:

- the likelihood of any given design event, along with the window of potential exposure associated with the time required to implement post-seismic repairs before a potential flood event
- the need for levees to be able to resist the design earthquakes in a non-flood situation (USACE, 2000), unless there are specific local regulations giving other requirements.

In some seismically active areas, where the retained water can be at a high level for long periods of time or where high-risk conditions exist, the seismic design may need to consider the following interrelated issues (USACE, 2012b):

- probability of strong earthquake occurrence
- proximity of the levee to major faults
- whether the levees or the foundation soils are potentially liquefiable
- whether water retention may be continuous or frequent
- condition of the ground and condition of any existing levee
- consequences of levee damage (risk to life and/or critical infrastructure etc)
- potential consequences of levee breach.

Different levels of protection against seismic loadings should be determined using a risk-based approach (USACE, 2012b) and, therefore, be applied to levees based on their importance. Different evaluation and design approaches should be adopted depending on:

- the levee's probable water retention conditions (eg intermittent or frequent hydraulic loading) and the probability of flooding
- the height of the levee.

In particular, three height categories for levees are recommended for the purpose of seismic assessment and design:

1 Less than 3 m.
2 3 m to 6 m.
3 More than 6 m.

The mitigation of seismic vulnerability USACE (2012b) can take the form of modifications to the levee to reduce liquefaction and/or seismic deformations or the development of a plan for dealing with seismic damage, following an earthquake (see below). Figure 9.57 sets out a procedure based on USACE (2012b) for determining if it is necessary to carry out a detailed seismic behaviour evaluation for a levee in a seismic area. If the assessment process indicates that calculations are required, then a suggested methodology for the seismic design analysis is given in Figure 9.58. The procedure may be used for both for the design of new levees and the assessment of new works for existing levees. Both Figures 9.57 and 9.58 include references to the relevant tools in Section 8.8.

The basic procedure identified in Figure 9.58 is first to determine if the levee is stable when subject to the design seismic conditions and secondly, if flow slides are considered unlikely, to assess the magnitude of any displacement induced by the seismic event. Acceptance criteria adopted by DWR (2012) are presented in Table 9.15.

Table 9.15 Levee seismic vulnerability classification system (after DWR, 2012)

Amount of displacement relative to levee height	Significant damage to internal structures (eg cut-off walls)	Remaining freeboard for post-seismic evaluation (design floodwater surface elevation)	Post-seismic flood protection ability
< 5%	No	> 0.3 m	Probably uncompromised
< 10%	Possibly	> 0.3 m	Probably compromised
< 20%	Likely if existing	< 0.3 m, but > 0	Likely compromised
Unlimited (flow slide condition)	Yes	N/A	Very likely compromised

Note

The results of the analyses must pass each of the criteria in the first three columns of the table at a given vulnerability class (final column), or the corresponding segment should be rated at the next higher vulnerability class.

Any post-earthquake remediation of critical levees should be performed in accordance with a post-earthquake remediation plan (USACE, 2012b) and should include:

1. Plans to repair, within six to eight weeks, reaches of levees that no longer provide at least a 10-year level of flood protection with a freeboard equal to that of the original design (if it is not feasible to restore a 1 in 10-year level of protection within eight weeks then a risk management plan must be put in place and a plan for the staging of any works prepared).
2. Plans to restore the levee to its original design level of flood protection (with the design height of freeboard).
3. Borrow areas that could easily provide the materials needed for interim repairs need to be identified in advance.
4. Slope protection for any newly placed fill material.
5. Repair procedures for the interim remediation of cracked and slumped levee sections, including general procedures for:
   - i excavating and filling cracks (priority action), removing disturbed or slumped ground, and keying in new fill
   - ii addressing damage to other key elements of the levee system (foundation cut-offs, drains etc).
6. A general set of provisions for emergency preparations, mobilisation, data gathering, actions, interim repairs, long-term repairs and public notifications.

The post-earthquake remediation plan should be reviewed and updated after periodic inspections and condition assessment.

Levees that are frequently subject to hydraulic loads may need to be treated more like dams (USACE, 2000) and have a sufficient level of seismic stability to maintain the integrity of the levee and its internal structures without significant deformation in the event of an earthquake. Such frequently loaded levees should have:

- adequate post-earthquake freeboard for the design flood event
- rigid penetrations or appurtenances (eg cut-off walls) or other features (eg toe drains) designed such that seismic-induced deformations are small (to avoid damage).

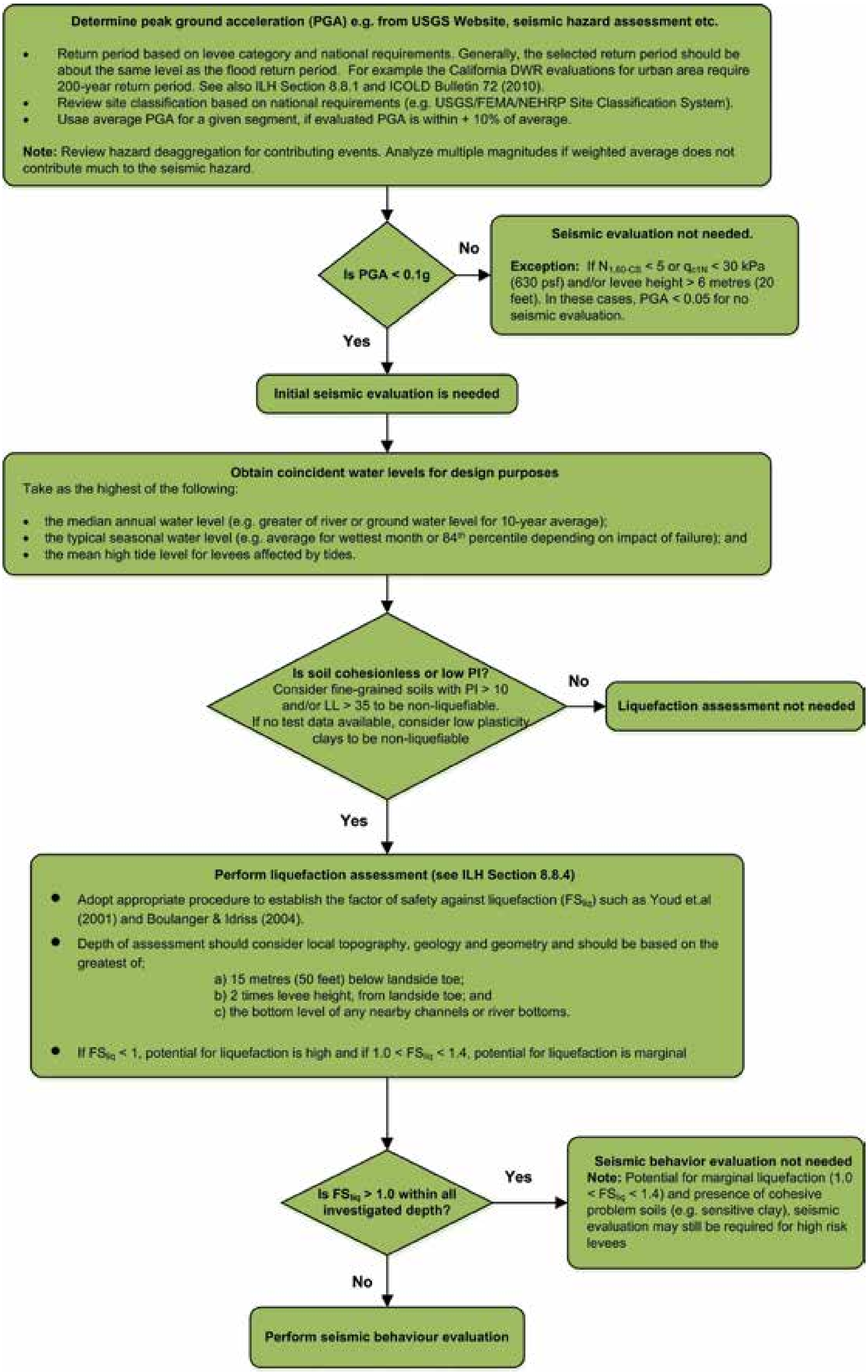

Figure 9.57 Procedure for seismic evaluation of levees (after USACE, 2012b)

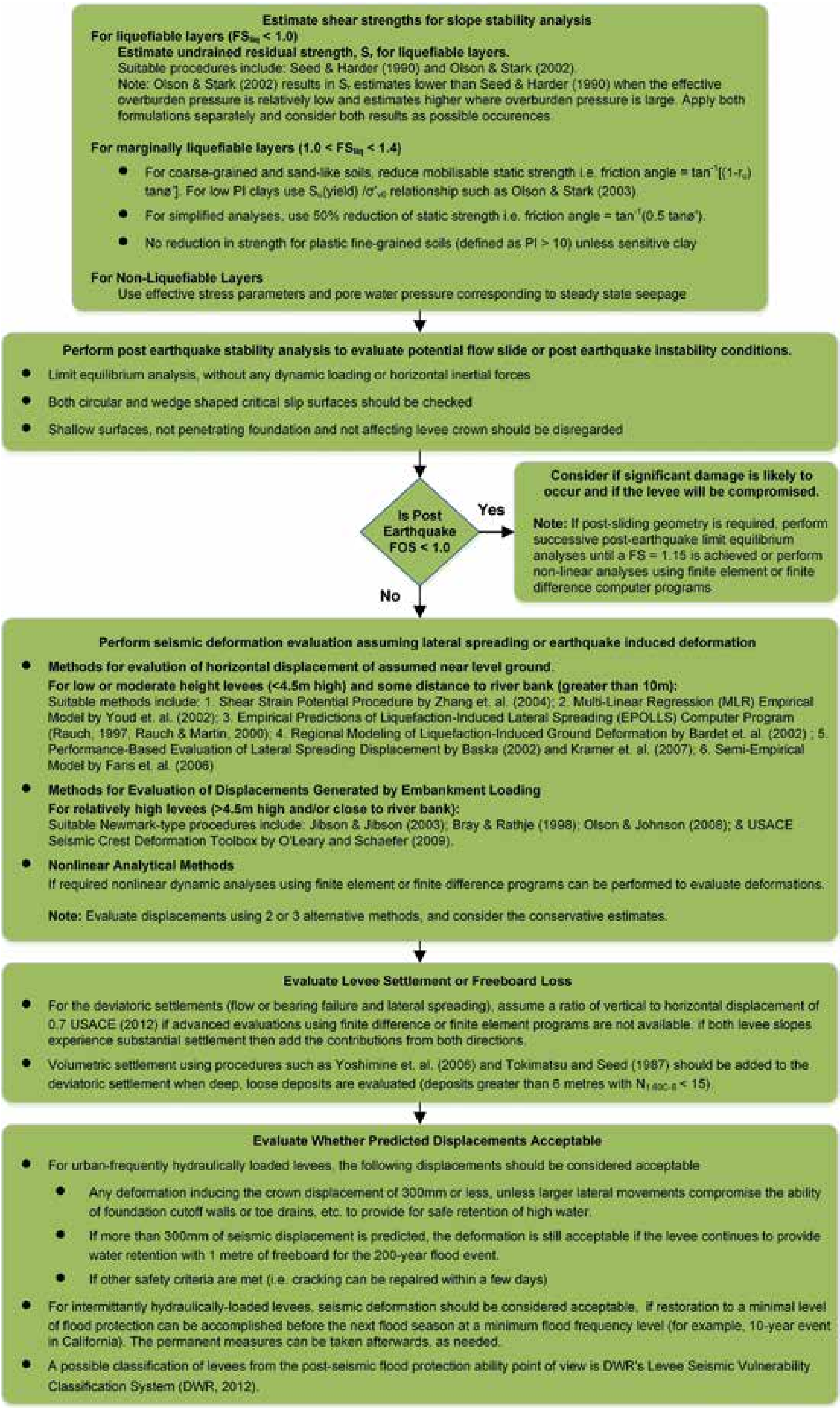

Figure 9.58 Seismic behaviour evaluation for levees (after USACE, 2012b)

9.10 ANALYSING FAILURE MECHANISMS

The analysis of failure mechanisms generally includes the following steps:

- characterisation of the site, including establishing ground and geomorphological models
- definition of design situations in terms of applied hydraulic loads, external forces (actions) and load combinations
- identification of potential failure mechanisms (or limit states)
- establishment of geotechnical parameters, particularly for unit weight and strengths (resistances) that are appropriate to each of the identified limit states
- determination of the groundwater conditions to be applied to each limit state
- designing for resistance to failure mechanisms through an iterative process of calculation (or estimation) and modification (of geometry, groundwater control measures, structural resistances etc).

For the flood situation, it is common to carry out analyses to verify that failure will not occur for a range of floodwater levels but these normally include two important situations:

- 'normal' design conditions – based on the design hydraulic conditions (water levels and waves) including all situations involving transient and persistent loading situations up to the magnitude of those design conditions
- 'extreme' conditions including seismic and accidental loading (such as ship impact) and the case where the hydraulic conditions exceed the 'normal' design case but during which the levee itself must not fail suddenly even if overtopped.

Loads (actions) are typically classified into three categories:

1 Permanent loads: mainly the dead load of the levee itself.
2 Variable loads: for levees, this is mainly the action of floodwater (as opposed to the normal water level) but it may include live loads such as maintenance or emergency vehicles; water loads are directly assessed in the design situation by means of a hydraulic model of the levee.
3 Seismic or accidental load (such as ship impact) but these loads are generally not combined with a flood.

Stability calculations generally use 'characteristic' values of actions (loads) and resistances (strengths). Characteristic values are also commonly termed 'representative' or 'moderately conservative' values. In the Eurocodes, characteristic values are given the subscript 'k'. The characteristic value of a geotechnical parameter is usually selected as a cautious estimate of the value affecting the occurrence of the relevant limit state. If an adequate data set is available for statistical analysis then the characteristic values are established on the basis of the 95 per cent fractile (or five per cent, depending on the positive or adverse nature). However, while this is a useful conceptual measure, it is rare that sufficient data is available to permit an appropriate statistical analysis. Further information on establishing loads, actions, resistances and strengths is given in Chapter 7.

The procedure for determination of the ground model for limit state calculations is described in Chapter 7 and includes developing or confirming the following:

- **geological model for the foundation**, including information on stratification and the characteristic properties of each identifiable layer such as unit weight, undrained shear strength, drained strength characteristics, permeability, compressibility, coefficient of consolidation etc
- **geometrical model** which provides a streamlined representation of the levee and its foundation geometry, including wisely selecting specific cross-sections of the works for analysis so that potentially critical failure mechanisms can be modelled appropriately
- **geotechnical model** for the levee and its foundation, in other words the characteristic values of geotechnical parameters (Chapter 7).

The range of potential failure mechanisms (see Table 9.16 for a short list and Chapter 3 for further details) must then be considered for each identifiable stage in the life of the levee, and the design adjusted until an acceptable level of stability has been demonstrated for each mechanism.

Table 9.16 Main ultimate limit states (ULS)

Scale	Type	Typical ULS failure mechanisms
Intergranular, phreatic and hydrostatic forces at global scale	Shear failure	Overall stability during construction or levee raising[1]
		Overall stability during a flood – rotational, non-circular, translational, sliding, uplift (see below) etc
		Stability after a flood (rapid draw-down)
		Seismic stability without pressure rise
	Hydraulic heave	Hydraulic uplift of soil at landward toe (may contribute to loss of overall stability)
		Hydraulic separation between the levee and rigid embedded structure caused by the seepage pressure exceeding the total contact pressure
Hydrodynamic forces at global scale	Static or dynamic liquefaction	Sand boiling in the vicinity of the landward toe
		Liquefaction (primarily due to seismic event)
Hydrodynamic forces at local scale	Erosion (deterioration that may trigger instability)	Internal erosion
		External erosion by overflowing
		External erosion by scouring

Note

1 In France, assessment of potential failure of levees under undrained conditions in which the strength of the levee is conservatively ignored is termed a bearing capacity failure.

A range of techniques is provided in Chapter 8 to analyse each of the potential failure mechanisms in order to verify the design. Lower levels of stability are normally permitted for the extreme cases than for the normal case. The design requirements for the extreme cases are usually established on the basis of risk of failure.

9.10.1 Factors of safety/partial factors for levee stability analyses

The basic objective of calculations used to analyse potential failure mechanisms is to check that the sum of the resistances to failure (in terms of forces or moments) exceeds the destabilising forces (or moments). Traditionally, this margin is expressed as a 'factor of safety'. However, more recently, the introduction of the Eurocodes in Europe has established the use of limit state calculation methodologies to geotechnical design. This approach incorporates the application of partial factors to actions (loads) or resistances (strengths) or both as an alternative to the 'global factor' approach.

Factors of safety in design calculations have two main purposes:

1 To provide a margin against uncertainty and unquantifiable factors, which are often encountered in geotechnical and hydraulic engineering.
2 To control displacements under normal 'working' conditions – soils exhibit marked non-linear stress–strain characteristics with small changes in applied loading producing a stiff response (small deflections) but significantly greater displacements being generated closer to the onset of a failure.

Efficient modern levee design processes try to recognise that:

- different factors need to be applied to different soil types and to different loading conditions, taking account of the fact that resistances mobilised under different loading situations can be predicted with varying degrees of confidence

1 2 3 4 5 6 7 8 9 10

- some types of failure (such as translational or deep rotational failures which affect the main body of a levee) pose a greater risk of breach than do others (such as surface slumping or post-flood rapid draw-down).

This approach is generally embodied both in the Eurocodes and in design approaches commonly used in the USA, such as DWR (2012).

The two approaches are described below:

1 In the traditional global factor ('lumped factor') approach, which is still used in the USA, the margin of the resistances to failure (in terms of forces or moments) to the destabilising forces (or moments) is expressed as the 'factor of safety'. In the case of mass instability, it can be described as the ratio of the actual shear strength available to the mobilised shear strength (Bromhead, 1992). In this approach, the calculation uses characteristic, representative or moderately conservative values of geotechnical parameters.

2 The partial factor approach now adopted in the Eurocodes in which independent 'partial' factors are applied to different actions (loads) and resistances (strengths) so that ultimate limit state (ULS) or serviceability limit state (SLS) calculations can be performed. Partial factors are applied to either the actions (applied loads, forces etc) or the resistances (material strengths etc) or both.

When producing a design or carrying out an assessment of stability, the designer should ideally be able to use a level of discretion in selecting the global or partial factors to reflect the associated risk. For example, where the risk is high, the designer could use additional partial factors or set higher global factor of safety targets. Such an approach requires that a consistent methodology is used for categorising levees in terms of complexity and risk. The suggested categories set out in Box 9.36 are based on the approach set out in Eurocode 7 but are adapted for use with levees. It is intended that these categories can be used with either the Eurocode (partial factor) type approach or with the 'lumped factor' approach.

Box 9.36 ***Possible geotechnical categories for levees***

Geotechnical category 1 (applicable to small, simple levee structures)

- structures involving negligible risk
- fundamental requirements satisfied on the basis of experience and qualitative geotechnical investigations – design calculations are probably unnecessary
- no design calculations are usually required – levees falling into this category can usually be designed by experience or through a knowledge of what has worked adequately in the past for similar ground conditions and hydraulic conditions.

Geotechnical category 2 (applicable to most conventionally engineered levees)

- no exceptional risk, no difficult ground conditions or no unusual loadings
- routine procedures for field and laboratory testing
- quantitative geotechnical data used for design
- geotechnical analysis and design carried out to satisfy fundamental requirements
- for levees falling into geotechnical category 2, geotechnical designs would normally be carried out using the default partial factors prescribed by Eurocode 7 or to the lower end of the factors of safety set out in Section 9.10.3.

Geotechnical category 3 (applicable to significant, complex or large levees)

- large or unusual structures
- abnormal or extreme risk (particularly relating to the consequences of failure), unusual or exceptionally difficult ground conditions or extreme loadings
- levees constructed in highly seismic or naturally unstable areas
- for category 3 structures, the increased risks posed by the consequences of failure or the size or complexity of the levees can be managed through the application of higher partial factors or the use of additional model factors if Eurocodes are being used or through the adoption of the higher end of the factors of safety set out in Tables 9.17 and 9.18. Depending on the level of risk, it may also be appropriate to apply other risk management strategies, such as increased resilience to breach or well-defined evacuation strategies.

9.10.2 Global factor of safety approach (US)

For global stability calculations, the following design conditions would normally be modelled in the design calculations:

- worst foreseeable combination of ground conditions and levee geometry (also at key construction stages, if appropriate)
- moderately conservative geotechnical parameters (also termed characteristic or representative) – these should be appropriate to each failure mechanism and may vary from one mechanism to another. For example, the shear strength mobilised in the ground along long translational slides may be different from the strengths mobilised in the same material by a short rotational slip mechanism. Further guidance on parameter selection is given in Section 7.8
- critical hydraulic conditions (sea level, storm surge level, river level, wave height etc)
- critical groundwater conditions (often resulting from the critical hydraulic conditions), including the phreatic surface – this is often one of the most difficult characteristics to assess (Section 9.7) and will depend on many factors such as:
  - the permeability of each relevant layer in the ground beneath the levee
  - the permeability of the levee fill
  - the uniformity of the levee fill and the natural ground, particularly if the ground is stratified and contains highly permeable soils that have a direct hydraulic connection to the source of the floodwater
  - the duration and shape of the flood hydrograph
  - the contribution of groundwater control systems used in the design
- critical coincident applied loads (construction loads for construction case, emergency plant, impact loads etc).

An example of the approach to determination of a global factor of safety is given in Box 9.37 for the case of Bishop's method for slope stability analysis (Section 8.6).

Box 9.37 Application of global factors of safety approach to Bishop's method

This method allows the determination of a global factor of safety using Equation 9.1, in which the soil above a hypothesised failure surface is divided into a number vertical slices of width *b* and self-weight *W* and with an imposed vertical surface load of *Q*. The pore pressure acting at the centre of the base of the slice is *u*, *α* is the inclination of the base of the slice to the horizontal, *c'* is the effective cohesion and *ϕ'i* is the effective angle of shearing resistance. For undrained conditions, the undrained shear strength, C_u, replaces *c'* and *ϕ* is taken to be zero.

$$F = \frac{\sum_i \left\{ \dfrac{(c_i' b_i + (W_i + Q_i - u_i b_i) \tan \phi_i) \sec \alpha_i}{1 + \tan \alpha_i \left[\dfrac{\tan \phi_i}{F}\right]} \right\}}{\sum_i \{(W_i + Q_i) \sin \alpha_i\}} \quad (9.1)$$

Bishop's routine method makes some simplifying assumptions about inter-slice forces but it does satisfy horizontal and moment equilibrium. Its solution requires a process of iteration as *F*, the global factor of safety, appears on both sides of the equation. Other methods of analysis adopt similar approaches but make different simplifying assumptions (Chapter 8). Similar algorithms to Bishop's routine method have also been developed for non-circular slips (Chapter 8).

Typical global factors of safety adopted for mass instability calculations on levees are provided by USACE (2000) in Table 9.17 and DWR (2012) are given in Table 9.18. Where a range of factors of safety is quoted, the designer should use judgement to balance risk level with the target factor of safety. In general, the lower factors quoted should be taken as the lowest values that should be adopted for the lowest risk design, and the higher end should correlate with higher risk levees. Both references provide guidance for specific design situations, seeking to distinguish between different design scenarios and applying appropriate target factors of safety to each.

Table 9.17 Minimum factors of safety – levee slope stability (USACE, 2000)

	Applicable stability conditions and required factors of safety (FS)			
Type of slope	*End-of- construction*	*Long-term (steady seepage)*	*Rapid draw-down[a]*	*Earthquake[b]*
New levees	1.3	1.4	1.0 to 1.2	(see notes)
Existing levees	–	1.4[c]	1.0 to 1.2	(see notes)
Other dikes and embankments[d]	1.3[e,f]	1.4[c,f]	1.0 to 1.2[f]	(see notes)

Notes

a Sudden draw-down analyses. FS = 1.0 applies to pool levels prior to draw-down for conditions where these water levels are unlikely to persist for long periods preceding draw-down. FS = 1.2 applies to pool level, likely to persist for long periods prior to draw-down.

b See USACE (1995b) for guidance. An ER for seismic stability analysis is under preparation.

c For existing slopes where either sliding or large deformation has occurred previously, and back analyses have been performed to establish design shear strengths, lower factors of safety may be used. In such cases probabilistic analyses may be useful in supporting the use of lower factors of safety for design.

d Includes slopes which are part of cofferdams, retention dikes, stockpiles, navigation channels, breakwater, riverbanks and excavation slopes.

e Temporary excavated slopes are sometimes designed for only short-term stability with the knowledge that long-term stability is not adequate. In such cases, higher factors of safety may be required for end-of-construction to ensure stability during the time that the excavation is to remain open. Special care is required in design of temporary slopes, which do not have adequate stability for the long-term (steady seepage) condition.

f Lower factors of safety may be appropriate when the consequences of failure in terms of safety, environmental damage and economic losses are small.

Table 9.18 Global factors of safety (FS) and allowable hydraulic gradients (i) (DWR, 2012)

Parameter	**Criteria**			
Seepage-exit gradient at levee toe	For DWSE		For HTOL	
	$\gamma \geq 17.6$ kN/m^3	$\gamma < 17.6$ kN/m^3	$\gamma \geq 17.6$ kN/m^3	$\gamma < 17.6$ kN/m^3
	$i \leq 0.5$	FS $\geq$ 1.6	$i \leq 0.6$	FS $\geq$ 1.3
Seepage-exit gradient at seepage berm toe	$i \leq 0.8$	FS $\geq$ 1.0	<20% FS degradation for berms less than 100 ft (30.5 m)	<10% FS degradation for berms less than 100 ft (30.5 m)
Steady-state slope stability (landside)	FS $\geq$ 1.5		FS $\geq$ 1.3	
Steady-state slope stability (landside)	FS $\geq$ 1.4		FS $\geq$ 1.2	
Rapid draw-down slope stability (waterside)	FS $\geq$ 1.2 (prolonged high stage) FS $\geq$ 1.0 (short lasting high stage)			
Minimum allowable rapid draw-down slope stability (waterside)	FS $\geq$ 1.2*			
Frequent, large, tidal fluctuations rapid draw-down slope stability (waterside)	FS $\geq$ 1.4**			
Seismic vulnerability	No significant deformation, usually limited to 3 ft (0.91 m) maximum with 1 ft (0.3 m) of vertical settlement			

Notes

These criteria are additions or exceptions to the criteria presented for intermittently loaded levees.

* Applies for the DWSE.

** Additional criterion that applies for the range of tidal fluctuation, not the DWSE.

Key

DWSE	Design water surface elevation	i	Exit gradient
FS	Factor of safety	γ	Saturated unit weight of soil (blanket layer)
HTOL	Hydraulic top of levee		

9.10.3 Partial factor of safety approach (Eurocodes)

Eurocode 7 (2004) on geotechnical design requires separate consideration of ultimate limit states (ULS) and the serviceability limit states (SLS). For the global factor approach, the need to consider serviceability issues such as settlement and lateral deformation is not usually tied directly to the stability verification. However, it is important that designers do consider such issues to an appropriate level.

The overriding principle applied in limit state approaches to design such as the Eurocodes is that the design (factored) effect of the actions (E_d) does not exceed the corresponding design (factored) resistances (R_d) as set out in Equation 9.2.

$$E_d \leq R_d \tag{9.2}$$

where:

E_d = design value of the effect of actions

R_d = design value of the resistance to an action

Under Eurocode 7, five ultimate limit states must be considered and these are now discussed in turn.

9.10.3.1 EQU, loss of equilibrium of the structure or the ground

This case is only relevant if the structure acts as a rigid body, where the strengths of structural materials and the ground are insignificant in providing resistance. An example of the EQU limit state would be a rigid foundation bearing on rock and tilting about an edge. The EQU limit state is a rare situation in geotechnical engineering and is not normally relevant to the stability of a levee.

9.10.3.2 STR, failure or excessive deformation of structure or structural elements, and GEO, failure or excessive deformation of the ground

For levees, geotechnical stability is normally dominated by the GEO verification calculations. However, failure mechanisms involving any structural elements may be governed by the STR calculations. Note that limit state GEO is often critical to the sizing of structural elements involved in foundations or retaining structures, and sometimes to the strength of the structural elements themselves.

Eurocode 7 defines three different loading situations that need to be considered for the GEO and STR ULS assessments for levees:

- **persistent (ULS) design situation:** "a design situation that is relevant during a period of the same order as the design working life of the structure." This generally refers to conditions of normal use and, for the case of levees, would usually include normal river levels or common tidal situations
- **transient (ULS) design situation:** "a design situation that is relevant during a period much shorter than the design working life of the structure and which has a high probability of occurrence." A transient design situation refers to temporary conditions of the structure, of use or of exposure. So, this situation would include the flood level corresponding to the design return period or to conditions of construction or repair
- **accidental (ULS) design situation:** "a design situation involving exceptional conditions of the structure or its exposure, including fire, explosion, impact or local failure." For levees, in addition to accidental events such as ship impact, this case may include floodwater levels that exceed those established for the design return period.

In seismic areas, the verification of stability under seismic loads is also required (but for levees, it is not normal to apply coincident flood and seismic loads). In the Eurocodes, the verification of stability under seismic actions is covered by Eurocode 8 (2004b and 2004c).

For limit equilibrium slope stability calculations of the type required for levee stability design, the overturning moment M_E is the action effect and the restoring moment M_R is the resistance to that effect. For slope stability analysis, Eurocode 7 therefore requires that designers to demonstrate that the restoring moment exceeds the overturning moment for each of the potential failure mechanisms.

$$\frac{E_d}{R_d} = \frac{M_{Ed}}{M_{Rd}} = \Lambda_{GEO} \leq 1 \tag{9.3}$$

where Λ_{GEO} is a degree of utilisation of the available design resistances by the design actions or the effects of the design actions. Note that Λ_{GEO} is the inverse of a factor of safety. The application of this approach to Bishop's routine method for this partial factor of safety approach is given in Box 9.38 as an example.

Box 9.38 ***Application of partial factor of safety approach to Bishop's method***

Converting Equation 9.1 into a form suitable for use with a partial factor approach such as Eurocode 7 gives the following equation for Bishop's routine method:

$$\Lambda_{GEO} = \frac{\Sigma_i\{(W_{d,i} + Q_{d,i})\sin\alpha_i\}}{\Sigma_i\left\{\dfrac{(c'_{d,i}\,b_i + (W_{d,i} + Q_{d,i} - u_{d,i}b_i)\tan\phi_{d,i})\sec\alpha_i}{1 + \tan\alpha_i\tan\phi_{d,i}\,(\Lambda_{GEO})}\right\}} \leq 1.0 \tag{9.4}$$

In this case, the subscript 'd' denotes 'design values' (values to which appropriate partial factors have already been applied). In Equation 9.4, the design values are replaced by the characteristic values and the appropriate partial factors are therefore included.

It should be noted that the terms $W_{d,i}$ and $Q_{d,i}$ contribute to both the value of the actions and the value of the resistances. This complicates the application of the slope stability equations in situations where different partial factors can be applied to the same characteristic parameters for favourable and unfavourable situations.

$$\Lambda_{GEO} = \frac{(\gamma_G\,\gamma_c\,\gamma_{Re})\,\Sigma_i\{(W_{k,i} + (\gamma_Q/\gamma_G)\,Q_{k,i})\sin\alpha_i\}}{\Sigma_i\left\{\dfrac{\left(c'_{k,i}\,b_i + \left(\dfrac{\gamma_G\gamma_c}{\gamma_\phi}\right)(W_{k,i} + Q_{k,i} - u_{k,i}b_i)\tan\phi_{k,i}\right)\sec\alpha_i}{1 + \tan\alpha_i\left[\dfrac{\tan\phi_{k,i}}{\gamma_\phi}\right](\Lambda_{GEO})}\right\}} \leq 1.0 \tag{9.5}$$

where:

γ_G = partial factor applied to permanent actions (including self-weight)

γ_Q = partial factor applied to variable actions (including applied surface loads)

γ_{Re} = partial factor applied to earth resistances

$\gamma\phi$ = partial factor applied to φ

γ_c = partial factor applied to effective cohesion

other symbols are as given in Box 9.37.

As with the global factor approach, as the term Λ_{GEO} appears on both sides of the equation, an iterative process is required for its solution.

As illustrated in Box 9.38, the self-weight and imposed load components may contribute to both the value of the actions and the value of the resistances. If partial factors are applied to these individual terms in the equation, different factors would have to be applied to the same terms depending on whether the actions are favourable or unfavourable. To avoid this problem, Eurocode 7 introduces the concept of a single source in a note to Clause 2.4.2 9(P) of the code:

"Unfavourable (or destabilising) and favourable (or stabilising) permanent actions may in some situations be considered as coming from a single source. If they are considered so, a single partial factor may be applied to the sum of these actions or to the sum of their effects."

Further, under Table A1.2(B) and A2.4(B), EN 1990 notes that:

"The characteristic values of all permanent actions from one source are multiplied by $\gamma_{G,sup}$ if the total resulting action effect is unfavourable and $\gamma_{G,inf}$ if the total resulting action effect is favourable. For example, all actions originating from the self-weight of the structure may be considered as coming from one source; this also applies if different materials are involved."

Bond and Harris (2008) highlight the problems of using factored parameters in slope stability analyses. In particular, they note that the use of factored parameters may induce the slope stability programs to identify 'critical' slip surfaces that would be different to the critical surfaces identified if characteristic parameters were used (and hence they may be different from reality). To counter this problem, they suggest that slope stability analyses are first carried out using 'characteristic' (unfactored) parameters, actions and resistances to identify the critical failure mechanisms and then that a further set of analyses (using the partial factors from Eurocode 7) is carried out on the identified critical failure mechanisms to validate stability for these cases.

Similarly, the application of partial factors to resistances for use in finite element analyses can be problematic and can influence the behaviour of the model, particularly as mobilisation factors approach unity (which they should do in an efficient design).

Undrained failures (often linked to end of construction – see Section 9.7)

As stated in Box 9.38, slope stability equations can be applied either to drained strength parameters (c' and ϕ') or to undrained parameters (C_u and $\phi_u = 0$). When constructing levees on soft clay foundations, it is usual to use undrained strength parameters for determining stability. This is because of the problems associated with predicting pore pressure changes as the levee is constructed, and incorporating these values into limit equilibrium effective stress analyses. In particular, the use of effective stress analyses for predicting stability during construction can be non-conservative. The use of undrained parameters in stability analyses for the determination of the stability of levees built on soft clays is well established (Ladd and Foott, 1974, Tavenas and Leroueil, 1980, Jardine and Hight, 1987 and Ladd, 1991).

In the UK and the US, it is usual to use undrained shear strengths with limit equilibrium slope stability calculations to determine the stability of levees built on soft clays. One particular benefit of this approach is that the search routines incorporated into modern computer programs can handle irregular geometries and can allow non-circular slip analyses to be carried out. The strength of the soils in the foundations of historical levees can also be 'zoned' to take account of strength anisotropy within the levee's foundations.

As can be seen in Box 9.40, in France the slope stability methods used for levees do not allow for the use of undrained shear strengths in the analyses. The issue of potential failures during construction is therefore handled as a bearing capacity problem. The concept of bearing capacity of the levee foundation refers to shear failure of the foundation by punching into the soft clay. The failure of the foundation is considered to affect the entire width of the levee.

Under the French guidelines, the stability check normally assumes that the levee is built instantly, without dissipation of pore pressures in the foundation. It therefore relies on the use of undrained shear strengths and therefore applied to new levees or significant heightening of existing levees.

In the French bearing capacity method, the vertical stress, q, under an levee of height, H and unit weight γ is approximated by:

$$q = \gamma H \tag{9.6}$$

The critical value of q (q_{max}) can be estimated using the relationships given in Chapter 8.

9.10.3.3 UPL, loss of equilibrium of the structure or ground due to uplift by water pressure (buoyancy) or other vertical actions

The UPL limit state considers the loss of equilibrium of the structure or the ground due to uplift by water pressure (buoyancy) or other vertical actions. This may be relevant to levees, for example in connection to the design of structures buried inside the levee such as pipes or culverts.

9.10.3.4 HYD, hydraulic heave, internal erosion and piping in the ground, caused by hydraulic gradients

The HYD limit state is directly applicable to levees because it considers the impacts of hydraulic heave, internal erosion and piping in the ground, caused by hydraulic gradients. These are issues that must be considered when designing a levee or parts of a levee. As explained in Section 9.9.2, hydraulic heave may become important in levee design where uplift of the levee toe is possible (for example, the situation illustrated in Figure 9.59 and Figure 9.60 or when sand boils are possible). In these situations, stability against hydraulic heave should be verified in addition to any geotechnical stability calculations carried out in a manner that incorporates uplift under the landward toe (see methods such as Van, 2001, described in Section 8.6.3).

For hydraulic heave, Clause 2.4.7.5.(1)P of Eurocode 7 states that "when considering a limit state of failure due to heave by seepage of water in the ground, it shall be verified, for every relevant soil column, that the design value of the destabilising total pore water pressure ($u_{dst;d}$) at the bottom of the column, or the design value of the seepage force ($S_{dst;d}$) in the column is less than or equal to the stabilising total vertical stress ($\sigma_{stb;d}$) at the bottom of the column, or the submerged weight ($G'_{stb;d}$) of the same column:

$$u_{dst;d} \leq \sigma_{stb;d} \tag{9.7}$$

and

$$S_{dst;d} \leq G'_{stb;d} \tag{9.8}$$

The hydraulic heave limit state at the downstream toe must be considered when the levee has been built on a foundation with a low permeability soil layer overlying a more permeable layer of soil (Figure 9.59). Failure of the toe of the levee caused by hydraulic heave could destabilise the complete levee and trigger a breach.

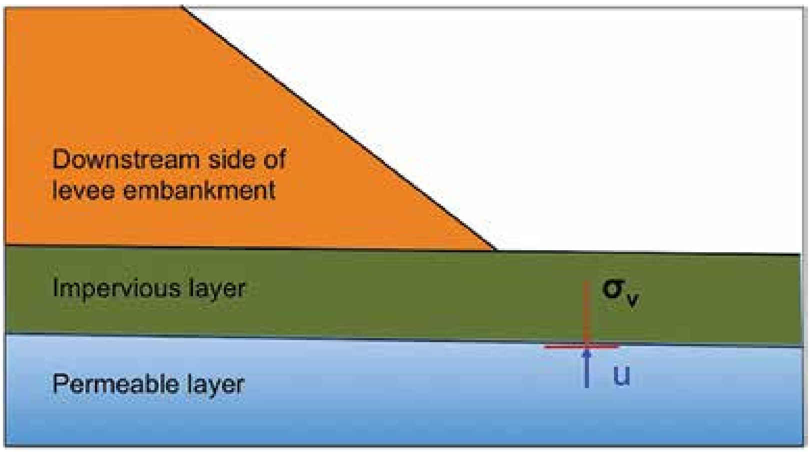

Figure 9.59 Physical model for hydraulic heave limit state (after Royet and Peyras, 2010)

Clause 2.4.7.5. (2)P states that the partial factors for $u_{dst;d}$, $\sigma_{stb;d}$, $S_{dst;d}$ and $G'_{stb;d}$ for persistent and transient situations are defined in Annex A.5(1)P of Eurocode 7 and the National Annexes. These are reproduced in Table 9.19.

Table 9.19 Partial factors on actions for hydraulic heave (after Eurocode 7)

Action	Symbol	Value
Permanent		
Unfavourable[a]	$\gamma_{G;dst}$	1.35
Favourable[b]	$\gamma_{G;stb}$	0.90
Variable		
Unfavourable[a]	$\gamma_{Q;dst}$	1.50

Notes

a Destabilising

b Stabilising

Bond and Harris (2008) suggest that the effective stress 'seepage force' equation (Equation 9.8) is less conservative than the total stress approach set out in Equation 9.7, which is more consistent with previous practice. As a result, they recommend that Equation 9.7 is used for design, wherever possible. However, it should be noted that whilst the total stress approach is generally more conservative, there are certain conditions where this is not the case (for example, when an impervious layer is submerged). The designer should consider such possibilities carefully and adapt the design if necessary.

The hydraulic failure mechanisms detailed in Eurocode 7 also include internal erosion and piping. Figure 9.60, taken from Eurocode 7, indicates conditions in which seepage and piping may occur beneath a levee. In this case, Eurocode 7 requires that the design of a filter system is carried out and appropriate measures are taken to prevent internal erosion.

Note

Piping and internal erosion can occur when the impervious layer has localised defects that allow concentrated seepage to exit or the impervious layer is discontinous allowing the pervious layer to exit at the ground surface. Designers should consider such possibilities and should draw on local knowledge and an understanding of the characteristics of the impervious layer when making such assessments.

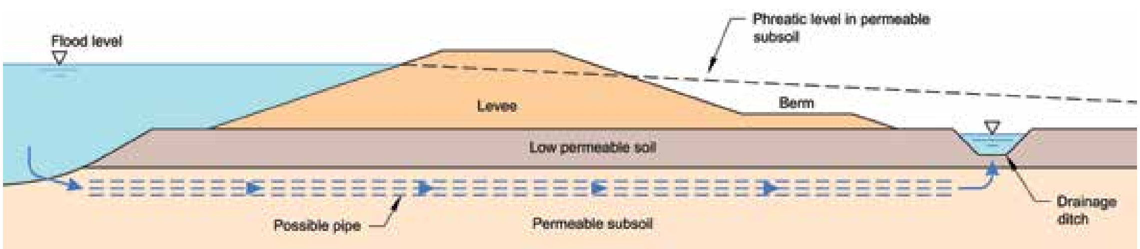

Figure 9.60 Hydraulic failure (ULS HYD): conditions that might cause piping (after BS EN1997-1:2004)

Eurocode 7 does not provide detailed rules for filter design for such situations. However, it does state that if the filter criteria are not satisfied then it shall be verified that the hydraulic gradient is well below the critical hydraulic gradient, which shall be established on the basis of:

- direction of flow
- grain size distribution and grain shape
- stratification of the soil.

9.10.3.5 Water pressures in marine or fluvial environments

In the case of a levee, actions caused by water pressures (and groundwater pressures) may be considered as permanent and/or variable, depending on the variation of their magnitude with time. Depending on the selected design approach (Section 9.10.3.6), different factors will be applied to variable and permanent actions (as well as favourable and unfavourable actions).

Structures in coastal or estuarine environments differ from terrestrial structures in that the tidally driven cycle of water levels occurs once or twice daily. Both the tidal variation and the occasional extreme design flood events should therefore be considered as variable actions. However, given that variable actions attract higher partial factors than permanent actions, it would be too conservative to apply the variable action partial factors to all of the hydrostatic and piezometric pressures. Part of the water pressure (ie the zone below the variable element of the water level or the groundwater level) can therefore be considered to be permanent.

Under the Eurocodes, levee designers have the choice of:

- assuming a characteristic phreatic surface (which should be the most unfavourable surface foreseeable)
- carrying out a seepage analysis (see discussion in Section 9.7 for options).

The following approach to selecting the design groundwater level is recommended:

1 Establish a characteristic groundwater level for the normal (non-flood situation), making use of ground investigation and groundwater level monitoring, coupled with experience and judgement. This is the characteristic permanent situation.

2 Establish the worst foreseeable characteristic groundwater level during the design flood, using a combination of seepage analysis and judgement. The difference between the characteristic groundwater level for the non-flood situation and the characteristic groundwater level for the flood situation is then the variable groundwater action.

3 Determine the design groundwater profile. Eurocode 7 Clause 2.4.6.1(8) states: "Design values of groundwater pressures may be derived by applying partial factors to characteristic water pressures or by applying a safety margin to the characteristic water level…". In the case of design approaches using unfactored water loadings (such as Design Approach 3 or Design Approach 1 Combination 2 – see Section 9.6.3.6), this is straightforward. Where the loadings are factored, however, a normalisation procedure can be adopted, whereby the variable element of the flood groundwater level is factored upwards by the ratio of the partial factors γ_Q/γ_G before being added to the characteristic permanent groundwater level. For example, For Design Approach 1, combination 1, the variable part of the groundwater level is multiplied by the ratio of the partial factors γ_Q/γ_G = 1.5/1.35 = 1.11 and then added to the permanent part of the groundwater profile. The resulting total (normalised characteristic) groundwater level is then subject to the permanent action partial factor, γ_G.

4 The resulting design groundwater pressures should be checked for reasonableness (so that physically impossible or physically unreasonable values are not used in the calculations).

9.10.3.6 Variation in design approaches across Europe

The way in which the partial factors are applied to any equation that uses partial factors in accordance with the Eurocodes is not the same across Europe. Eurocode 7 allows three different design approaches, depending on the country of application. The selection of different design approaches by different countries is shown in Figure 9.61. Table 9.20 then gives the equivalent partial factors in Eurocode 7 for slope stability analysis and the values of the grouped factors included in Equation 9.4 in Box 9.38.

Figure 9.61 Design approaches adopted by different European countries for design of slopes and embankments (Bond, 2013)

Table 9.20 *Partial factors for slope stability analysis (after Bond and Harris, 2008)*

Individual partial factor or partial factor 'grouping'	Design approach			
	1		2	3
	Combination 1	*Combination 2*		
γ_G	1.35	1.0	1.35	1.0*
$\gamma_{G,fav}$	1.0	1.0	1.0	1.0
γ_Q	1.5	1.3	1.5	1.3*
$\gamma_\phi = \gamma_c$	1.0	1.25	1.0	1.25
γ_{cu}	1.0	1.4	1.0	1.4
γ_{Re}	1.0	1.0	1.1	1.0
$\gamma_G \times \gamma_c \times \gamma_{Re}$	1.35	1.25	1.485	1.25
$\gamma_G \times \gamma_{cu} \times \gamma_{Re}$	1.35	1.4	1.485	1.4
$\gamma_G \times \gamma_c / \gamma_\varphi$	1.35	1.0	1.35	1.0
$\gamma_{G,fav} \times \gamma_c / \gamma_\varphi$	1.0	1.0	1.0	1.0
γ_Q / γ_G	1.11	1.3	1.11	1.3

Notes

* = Factor taken from set A2 for geotechnical actions.

A permanent action (G) is "an action that is likely to act throughout a given reference period and for which the variation in magnitude with time is negligible, or for which the variation is always in the same direction (monotonic) until the action attains a certain limit value".

A variable action (Q) is "an action for which the variation in magnitude with time is neither negligible nor monotonic".

A geotechnical action as "an action transmitted to the structure by the ground, fill, standing water or ground water".

These partial factors may be subject to change in future publications of the Eurocode.

Different partial factors are applied to favourable and unfavourable actions.

Boxes 9.39 to 9.41 describe the different approaches to the analysis of structural safety for levee design in Germany, France and the Netherlands.

Box 9.39 *Application of Eurocode standards for levee design in Germany*

While the German standard for dams DIN 19700 (2004) still uses the method of global safety factors related to limit state equations, the new standard DIN 19712 for flood protection works including levees, flood walls and regular temporary elements (2013) as well as DWA-M 507-1 (2011) recommend the application of the Eurocode, including the application of the partial safety factors. It is suggested to apply both methods for a certain transitional period to compare both results, and to get more experience in the application of the new method, especially in finding appropriate safety factors which provide at least the same level of safety as before.

In 1997, the previous standard DIN 19712, had already introduced a geotechnical verification concept with modular safety requirements (also referred to as partial safety concept). With the implementation of the European standards, in particular of Eurocode 7 with its Part 1 as DIN EN 1997-1, and of those standards relating to the required verification of safety of earthworks and foundations (here in particular DIN 1054:2010-12), the application of the above concept will also become mandatory under the updated DIN 19712, expected for publication in early 2013.

For the verification of stability, limit states on load-bearing capacity and serviceability must be evaluated (DIN 19712). Both the load-bearing capacity and serviceability will have to be assured with an adequate degree of probability. In the case of levees, the safety against material transport/migration inside the levee and in the subsoil stratum will also have to be considered. Levees will also be classified into various categories according to their height and damage potential which in turn influences the extent of the required site investigation.

Box 9.40 *French guidelines for structural safety of levees in a semi-probabilistic format*

The following information is based on the conclusions (Royet and Peyras, 2011, and Degoutte, 1997) of a working group representing French engineering skills and national bodies involved in hydraulic engineering (also summarised by Royet and Peyras, 2010).

Design situations

Design situations are classified into the following categories, considering how important flood-related specific situations are:

- **normal operating situation:** mean river level (out of floods)
- **transient or unusual situations:** end of construction, rapid draw-down, unusual low river level
- **accidental situation:** maximum credible earthquake (MCE).

A specific focus has been put on flood situations. Those include:

- **unusual flood situation:** protection level of the levee (ie spillway level if any) – it relates to protection goals assigned to the levee and typically has a 10 to 100-years return period

1
2
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Box 9.40 French guidelines for structural safety of levees in a semi-probabilistic format (contd)

- **exceptional flood situation:** relates to floods that increase the river level up to the maximum headwater level (MHL), with freeboard remaining until the crest
- **extreme flood situation:** relates to reaching a level above which the levee might suffer from major damage that could quickly lead to breach.

Return periods for exceptional and extreme floods will be specified in a future French regulation, in relation to the class of the levee (A to D, depending on population in the leveed area).

The final category of design situations corresponds to **failures of elements or components** directly involved in the safety of the levee, such as failure of a spillway by partial or complete obstruction, disruption of the drainage system, failure of a conduit, failure of waterproofing etc. These failures can lead to water levels potentially worse than the previous situations. The determination of rare or accidental situations related to failures of the security features derives from the risk analysis study, which is in the French regulation and is mandatory for levees of classes A, B and C. Risk analysis studies will be used to estimate the probabilities of failure of the security features combined with the water level in the river. They evaluate overall probability of occurrence attached to a scenario combining the simultaneous failure of a component and a water level. Depending on the likelihood of the situation examined, it may be considered as a rare situation or as accidental.

At the moment, the FrCOLD guidelines for embankment dams and levees (FRCOLD, 2012) only provide detailed recommendations for the following ULS:

- overall stability – sliding
- lack of bearing capacity – punching
- hydraulic uplift
- scouring.

The selection of partial factors was based on Approach 3 of the Eurocode 7 and adjusted by design practice for hydraulic works adopting security criteria differentiation depending on the design situation. Such partial factors are applied to characteristic values specific to material strength properties. The limit state condition is written as an inequality which compares (see Equation 9.9) the ratio of resisting forces (or their moment) to driving forces (or their moment) with the model coefficient. The mathematical expression of the limit state condition depends on the model adopted. It involves the characteristic values of strength properties weighted by their partial factor, and representative values of actions corresponding to the reviewed design situation.

Royet and Peyras (2010) have considered the application of the Eurocodes (particularly Eurocode 7) to the design of embankment dams and large levees and have concluded that additional model factors (γd) should be applied to the results of the stability calculations using Bishop's routine method. So the term, ΛGEO, which describes the utilisation ratio in Equation 9.4, should be replaced by $1/\gamma_d$ to give Equation 9.9.

$$\frac{1}{\gamma_d} \geq \frac{(\gamma_G\, \gamma_C\, \gamma_{Re})\, \Sigma_i\{(W_{k,i} + (\gamma_Q/\gamma_G)\, Q_{k,i})\sin\alpha_i\}}{\Sigma_i\left\{\dfrac{\left(\left(c'_{k,i}\, b_i + \left(\dfrac{\gamma_G\gamma_C}{\gamma_\phi}\right)(W_{k,i} + Q_{k,i} - u_{k,i}b_i)\tan\phi_{k,i}\right)\sec\alpha_i\right)}{1+\tan\alpha_i\left[\dfrac{\tan\phi_{k,i}}{\gamma_\phi\gamma_d}\right]}\right\}} \tag{9.9}$$

The γ_d model factor was calibrated with reference to control standard practices so as to remain as close as possible to current sizing. The final selection of suggested model factors included the required consensus to be reached within the industry and specific to each standardisation process. The set of partial and model factors agreed for the FrCOLD semi-probabilistic method is outlined in Table 9.21.

Table 9.21 Set of partial and model factors for overall stability limit states

Design situations	Partial factor applied to cohesion (γ_c') and to the tangent of angle of friction ($\gamma_{\tan}\phi'$)	Partial factor applied to unit weight (γ_γ) and to soil strength ($\gamma_{R;e}$)	Model factor (γ_d)
Normal operating	1.25	1.0	1.2
Transient or unusual	1.1	1.0	1.2
Exceptional flood (MHL)	1.1	1.0	1.2
Extreme flood	1.0	1.0	1.1
Accidental	1.0	1.0	1.1

The above rules are modified in some cases:

1 It is sometimes useful or necessary to conduct a more comprehensive model – for example, using the finite element method – to complement the usual limit equilibrium models. The stability criteria may remain similar to those for the limit equilibrium calculations. The calculations can be performed with the characteristic values of the criterion of plasticity and stiffness module, bearing the same partial factors applied to soil properties and presented in Table 9.21.

2 For the situation at the end of construction, in addition to calculating effective stresses, it is possible to conduct a calculation in terms of total stresses.

3 A specific section of the rules is devoted to levees with plastic clay materials.

Box 9.41 ***Safety criteria for slope instability in the Netherlands***

The safety approach for slope instability is a semi-probabilistic approach based on partial safety factors for the strength parameters of the soil and deterministic values for the loads. For the design of levees along the coast and the upper rivers a levee section approach is applied with a default consequence factor per section, related to the safety standard of the relevant levee ring area. For the design of river levees near the sea, where the loads consist of a combination of sea level and river discharge, a levee ring approach is applied, which takes into account the length of the considered levee ring. The required probability of slope instability for all the sections of the levee ring together is 10 per cent of the safety standard.

Bishop's method is used for stability calculations, but advanced calculations can also be made with a probabilistic versions or on the basis of the finite element method. The design water level is interpreted as a deterministic value, with an additional robust supplement of 0.3 m for river levees and 0.1 m for lake and sea levees.

The partial safety factors (ENW, 2007 and Deltares, 2011) are related to uncertainties in the schematisation of the subsoil (γ_b), the calculation model (γ_d), the consequences of instability (γ_n) and the strength of the soil (γ_m). The default value for the schematisation factor is γ_b = 1.3. Reduction to 1.2 or 1.1 is possible on the basis of a stepping stone plan, which takes into account the added value of more soil investigation. The model factor depends on the adopted calculation method: Bishop: γ_d = 1.0, finite elements method: γ_d = 1.0. The consequence factor for the levee section approach is based on a required local probability of macro-instability ($P_{f,loc,req}$) of 0.26 per cent of the safety standard. This value is expressed as a required reliability index (β_{req}) and to a consequence factor (γ_n) with the formula $\gamma_n = 1 + \{0.13 \times (\beta_{req} - 4.0)\}$. The consequence factors for landward sliding in zone 1, coinciding with extreme high water are presented in Table 9.22.

Table 9.22 ***Required stability factors for landward slopes in zone 1 (dike section approach for category A dikes)***

Safety standard (1/year)	$P_{f,loc,req}$ (1/year)	β_{req} (1/year)	γ_n (1/year)
1/250 (4.0E–3)	5.20E–6	4.41	1.05
1/1250 (8.0E–4)	2.08E–6	4.60	1.08
1/2000 (5.0E–4)	1.30E–6	4.70	1.09
1/4000 (2.5E–4)	6.50E–7	4.84	1.11
1/10000 (1.0E–4)	2.60E–7	5.02	1.13

The position of a sliding plane in the cross-section of the levee is taken into account by means of zones with less or more severe criteria. The zoning principle is presented in Figure 9.62.

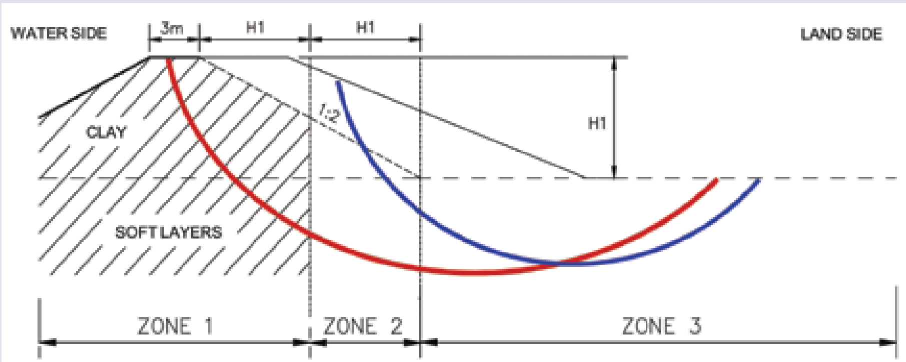


Figure 9.62 ***Principle of levee zoning (courtesy Harry Schelfhout, Deltares)***

If slope instability does not occur at the same as the extreme high water level then the required local probability of slope instability ($P_{f,loc,req}$) in a cross-section is 2.6 per cent of the safety standard. The required stability factors for landward sliding (in zone 2, not coinciding with extreme high water but for instance with extreme rainfall) are presented in Table 9.23.

Table 9.23 ***Required stability factors for landward slopes in zone 1 (not coinciding with extreme high water), zone 2 (coinciding with high water) and waterside slopes (levee section approach for category A levees)***

Safety standard (1/year)	$P_{f,loc,req}$ (1/year)	β_{req} (1/year)	γ_n (1/year)
1/250 (4.0E–3)	5.20E–5	3.88	0.98
1/1250 (8.0E–4)	2.08E–5	4.10	1.01
1/2000 (5.0E–4)	1.30E–5	4.21	1.03
1/4000 (2.5E–4)	6.50E–6	4.36	1.05
1/10000 (1.0E–4)	2.60E–6	4.56	1.07

Box 9.41 ***Safety criteria for slope instability in the Netherlands (contd)***

The required slope stability factor for the levee ring approach is based on a required local probability of slope instability of 10 per cent (ξ) of the safety standard for all the sections in the levee ring together. To determine the length-effect also the length of the levee ring (L), the sliding length (ℓ) and the correlation (ξ) with the strength parameters of the soil are taken into account. In formula form:

$$P_{f,loc,req} = (\xi \times \text{safety standard}) / [\{1 + (\alpha \times L/\ell)\} \times P_{f|inst}] \quad (9.10)$$

$$\beta_{req} = -\Phi^{-1} (P_{f,loc,req}) \quad (9.11)$$

in which 'safety standard' is the standard for the exceedence frequency of the high water level for the levee ring [1/year], β_{req} is the required reliability index for a dike section and Φ^{-1} = inverse Gauss probability function.

in which the 'safety standard' is the exceedance frequency of the extreme high water level for the levee ring (1/year).

The probability of breaching, given the occurrence of slope instability of the inner slope, $P_{f|inst}$ reflects the fact that a slope instability does not necessarily lead to breaching of a levee. Therefore a difference is made between the occurrence of a slope instability in two conditions: (a) the slope instability is a direct effect of a high water condition and (b) the slope instability is an effect of, for example, extreme rainfall. Condition (a) represents a more dangerous condition than condition (b). For that reason for condition (a) $P_{f|inst}$ = 1 and for condition (b) $P_{f|inst}$ = 0.1 is applied. Furthermore specific zones are used for the assessment of the stability of levee. These are a critical zone 1, where $P_{f|inst}$ = 1 is applied and a less critical zone 2, where $P_{f|inst}$ = 0.1 is applied.

The required slope instability factor (γ_n) can be calculated with the formula $\gamma_n = 1 + \{0.3 \times (\beta_{req} - 4.0)\}$. For different safety standards and levee ring lengths, the required slope stability factors for landward sliding are presented in Figures 9.63 and 9.64.

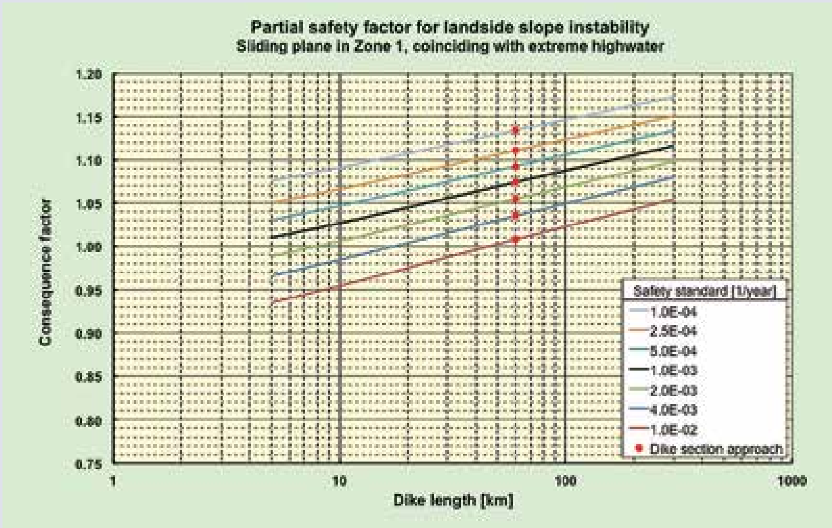


Figure 9.63 ***Required stability factors for zone 1 and $P_{f|inst}$ = 1 (dike ring approach) (courtesy Harry Schelfhout, Deltares)***

Figure 9.64 ***Required stability factors for zone 2 and $P_{f|inst}$ = 0.1 (dike ring approach) (courtesy Harry Schelfhout, Deltares)***

The previous discussion on the application of the Eurocode 7 partial factors has only considered the standard (geotechnical category 2) situation. In reality, some levees, like many large dams will be considered to be geotechnical category 3 structures, in that they may be large or unusual or that the risk of failure is considerable. For geotechnical category 3 structures, it is appropriate to consider whether additional partial factors should be applied to provide a greater margin against failure to balance the greater risk. Note that the global factors of safety suggested by DWR (2012) and summarised in Table 9.18 allow a range of global factors for exactly the same reason.

9.10.4 Probabilistic stability analyses

Levee stability has traditionally been assessed using the deterministic methods of geotechnical analyses discussed above (in which carefully assessed geotechnical parameters are used in conjunction with postulated failure mechanisms). For such analyses, the risk of failure is managed through the selection of conservative parameters and the use of appropriate factors of safety (or partial factors for the Eurocodes). In general, the higher the factor of safety (or partial factor), the lower the likelihood of failure. While the deterministic approach is generally a robust and well-used tool for the purpose of design, it can produce conservative designs (as each parameter is usually assessed individually on a conservative basis). The approach is also of less use for quantifying the risk of failure of an existing levee under different flood scenarios.

The development and application of probabilistic risk analysis techniques holds great promise for addressing the uncertainties in hydrology, hydraulics and geotechnical engineering, as these techniques aim to quantify and explicitly incorporate uncertainties arising out of historical construction and the natural world into levee design. Risk-based analysis can capture and quantify the magnitude of the risk and uncertainty associated with the various planning, economic and engineering components of a levee project. Such an approach can therefore be very useful for selecting the best scheme or the most beneficial components of a scheme. It may be particularly useful for the initial planning phase of a levee design to establish the impact of different factors on the risk of failure, considering different failure mechanisms.

The effect of risk and uncertainty on a levee project's engineering and economic viability means that conscious decisions have to be made to manage the explicit trade-off between risk and cost. Risk-based analyses can be used in this context to compare plans in terms of likelihood and variability of their physical performance, economic success and residual risk.

However, the use of probabilistic analyses for levee assessment or design requires many sources of uncertainty and variability to be addressed and an appropriate probabilistic method identified for dealing with each. The main categories and their challenges are:

- determination of the magnitude and frequency of loadings such as floods, earthquakes and accidental impacts – analysis of actual or synthetic historical data sets can be helpful here
- selection of variables and their statistical distributions – soil strengths, compressibilities and permeabilities have many components of uncertainty, including the soil's natural spatial variability, the quality and methodology of sampling and testing (including for example rate and direction of loading), obtaining a sufficient number of test results for the outcome to be statistically reliable. In addition, assessment of extreme values of the loadings that a levee may experience may need an extreme value analysis (Chapter 7)
- appropriate selection of the analytical models (eg slope stability, seepage analyses) used to represent failure mechanisms or modes of performance, bearing in mind that the models are mathematical simplifications of more complex problems:
  - for some performance modes with well-defined models and parameters such as slope stability, seepage, settlement, the probability of satisfactory or unsatisfactory performance may be calculated using a variety of approaches including Monte-Carlo methods and first order second moment (FOSM) methods, such as Taylor series methods. Both yield a reliability index, b, or probability of unsatisfactory performance $Pr(U)$.
  - for other performance modes without defined limit states models, estimates of performance

probabilities may need to rely upon experience-based practice. Uncertainty in performance caused by unseen, unforeseen (or unforeseeable) features such as unidentified cracks, burrows or other defects, or inadequacy of the grout curtain. Such factors may best be taken into account by quantifying the experience and judgement of experts rather than attempting to estimate uncertainty in parameters or to fit distributions to historical data.

- magnitude and extent of physical changes to both the loadings and the state of the structure (deterioration) within the lifetime of the model in a simulation (eg scour around a foundation, clogging of relief well screens, or the development of seepage windows through seepage cut-off walls). The development of these changes may need to be assessed based on experience or on models where available.

For probabilistic analyses to be used effectively for levee assessment or design, these uncertainties must be understood and managed appropriately. Further, the interrelationship of these uncertainties, variables and parameters must also be understood and managed.

On the basis of the above discussion, it should be appreciated that the application of probabilistic techniques to the assessment of levee reliability requires considerable knowledge and judgement. For this reason, it is suggested that if the technique is used for design, it should be validated by careful calibration against known historical performance of similar structures.

9.11 TRANSITIONS

Most levee failures during flooding that are not caused by overtopping or overflowing are related to internal erosion in one form or another. Tourment *et al* (2012) suggested that more than half of these internal erosion problems are linked to some form of transition in the levee. This finding demonstrates that transitions are commonly the weakest link in a levee system, and it highlights the importance of careful design and detailing of any transition.

Section 3.5.2.2 discusses the contributions of transitions to levee failures. A levee failure is commonly caused by a series of different (but often interrelated) physical mechanisms, causing degradation or damage to one or more components. Finally, the levee itself can fail, by breaching or by letting uncontrolled water into the leveed area. While failure modes are commonly named from the leading or originating mechanism (eg overtopping, external erosion, sliding), they may have been initiated by a number of contributory factors connected with the detailing of transitions.

Transitions can be broadly divided into the following subgroups:

1 Change in levee type or structural form:
   - i contact between a structure that is predominantly an earthen levee and a structural flood wall (in plan or cross-section)
   - ii contact between different levee reaches
   - iii contact between the levee and levee structures, such as crest walls, internal drainage systems, spillways and flood gates
   - iv interfaces between different types of external erosion protection
   - v contact between different foundation conditions (natural or manmade)
   - vi contact between the levee and natural high ground.

2 Part buried, but visible structures and encroachments:
   - i culverts
   - ii houses
   - iii stairs
   - iv bridge abutments
   - v manholes (probably associated with a buried structure or network).

3 Buried structures:

   i pipes (metal, plastic, concrete, masonry etc)

   ii cables.

4 External structures:

   i roads or railways

   ii drainage ditches

   iii boundary walls.

Examples of common transition types, and descriptions of their potential contribution towards levee failure are provided in Section 3.4.3.

9.11.1 Principles of transition design

Given the historical external and internal erosion problems associated with transitions which have led to levee failures, great care must be taken when carrying out the design of transition details. In general:

- the detail must be considered in three dimensions (not just in plan or in cross-section)
- the magnitude and characteristics of the hydraulic loads and external actions should be considered
- potential failure mechanisms should be considered and particular consideration given to issues such as uplift, internal erosion and hydraulic separation
- erosion mechanisms should be identified and their potential impact on the design considered
- appropriately robust design solutions should be selected.

A range of foreseeable loading scenarios should be considered. These may include the normal operating conditions, the design flood events and extreme events which are more onerous than the design flood situation but should not cause the levee to fail.

Where a robust transition cannot be designed with an adequate degree of confidence then one of two approaches should be adopted:

- a fail-safe solution incorporating two or more controlling features
- (in low risk situations) using an observational approach wherein careful condition assessments are made after each flood event.

The following hydraulic parameters are important for determining the possibility and intensity of erosion that needs to be considered as part of a design process:

- velocity and direction of the water flow (relative to the transition direction) and including possible sediment transport
- water level and its dynamic change, including wave characteristics
- the resulting hydraulic head along and across the transition zone as well as the resulting potential for uplift.

The **water energy** (water velocities and waves) in the river or coastal water, specifically along the levee face or toe, should be considered when surface erosion is a possible failure mechanism. Associated with water velocity and wave characteristics, local **turbulence** is often the initiating cause of erosion, and this commonly occurs at the interface between two different revetments on the waterside of the levee.

The **hydraulic gradient** and/or the hydraulic head has to be considered in the case of seepage or flow through the levee. It is strongly linked to the risk of various forms of internal erosion.

The **uplift-pressure** (or interstitial water pressure) is involved in all mechanisms regarding overall stability. The possibility of uplift occurring should be identified on the basis of a geological and geotechnical assessment of the levee and of the soils beneath the levee. It should be calculated by the

1 2 3 4 5 6 7 8 9 10

execution of seepage analyses in transient or steady conditions, depending upon the duration of the hydraulic head.

Many possible combinations of the above processes in conjunction with different types of transition structure and hydraulic load may need to be considered in the design.

Designs at transitions should also allow for the possibility of deterioration with time that could compromise the serviceability requirements set out in Section 9.12. This is illustrated by the differential settlement example in Box 9.42, in which a risk management approach is suggested.

Box 9.42 ***Risk management approach to mitigation of differential settlement at transitions***

Differential settlement between a hard structure and a levee can create gaps, voids or zones of softened material which would then have a reduced resilience to hydraulic or internal erosion. Such differential settlement and its consequences can be foreseeable and predictable, but difficult to prevent. Given the uncertainties, a risk management approach could be adopted to the problem considering, in order of preference:

- **avoid** – relocate the levee to a location that avoids an area where differential settlement is likely
- **mitigate** – incorporate (structural) measures which spread the differential settlement over a larger area to reduce the likelihood of concentrated zones of damage
- **transfer** – pass the risk to another party better positioned to manage the risk
- **accept** – accept that problems caused by differential settlement will occur and anticipate that repairs may need to be carried out regularly at known locations.

9.11.2 External erosion at transitions

Transitions in flood defences between an earthen levee and a rigid flood wall (or other structure) are vulnerable to external erosion processes due to locally increased overtopping velocities, irregular geometries and the localised effects of highly turbulent flow (Figure 9.65). Typical external erosion processes related to transition structures include:

- surface erosion and gullying as a result of surface water runoff being focused at the interface between the structure and the levee
- surface erosion at the transition due to local flow turbulence caused by structure geometry or roughness variations
- scour may initiate close to the base of the transition flood wall but it can lead to rapid progression of erosion away from the wall.

Simple analytical methods for estimating the increased flow velocities that occur at transitions are lacking and present guidance for sizing armour requirements at transitions is limited to past experience and field observations. However, results from two-dimensional inviscid jet theory suggest that the flow velocity along the outer edge of the jet is about 1.7 times the flow velocity through the middle of the gap. Therefore, it is easy to see that the region immediately adjacent to a vertical crest wall experiences the largest flow velocities and that sizes of protection systems may need to be increased, perhaps by an order of magnitude. The addition of waves propagating on top of the overtopping surge compounds the complexity of the flow situation, and no simple procedures are available to address this case.

Due to these complexities, it is probable that physical modelling or sophisticated numerical simulations will be required to establish allowable flow velocities and the stability of protection measures.

Box 9.43 Design of armour at transitions for Greater New Orleans Hurricane Protection System post Hurricane Katrina

Figure 9.65 New Orleans levee scour at transition with a flood wall (courtesy USACE)

The standard practice based on observed scour following Hurricane Katrina (Figure 9.65) is to use concrete slope paving at all transitions. Slope protection should extend along the wall for the full height of the embankment where the flood wall height is at full section, and slope paving should extend a distance of 30 feet from the end of the rigid wall. Adequate armour should be provided along the entire length of flood wall where overtopping may occur.

9.11.3 Internal erosion at transitions

The impact of internal erosion processes at transitions, on the performance of levees is less well understood than that of external erosion, primarily because the problem is commonly hidden from view until the erosion reaches such an extent that sudden partial or complete failure of the levee occurs. In this case, the damage caused by any resulting breach may well obscure the extent of any internal erosion up to the point of sudden failure. Some indicators of internal erosion, such as seepage flow, may be visible before sudden failure, but this is not always the case.

Transitions between earthfill materials and rigid structures such as concrete wall, culverts and pipes must be detailed carefully because the interface will potentially be the focus for differential settlement and preferential seepage and this may lead to the development of internal erosion.

To prevent the development of internal erosion along such a transition, four complementary methodologies are suggested, which can be used singly or in combination, depending on the dimensions of the levee, the nature of the levee fill materials and of the natural ground beneath the levee, and the magnitude of the hydraulic load.

These control measures are:

1 **Lengthen the seepage path, to reduce the hydraulic gradient:** the following construction devices can be used to lengthen the flow path:

- i concrete collars or flanges can be fitted around pipes
- ii concrete beams or cut-offs, constructed beneath the foundations to drive the seepage water deeper into the ground
- iii transverse cut-off walls, installed beneath the levee.

Section 8.5.1 presents methods that can be used to determine the minimum length of the drainage path as a function of the head difference and the nature of the relevant soil types.

Note that if collars or flanges are used, it is important to ensure compaction of the levee around the pipe in the area of the collars. One suitable approach is:

- i lay the main pipe and compact the levee material around it
- ii make a small excavation for the collar and fit it
- iii backfill the small excavation with concrete.

1 2 3 4 5 6 7 8 9 10

2 **Decrease the hydraulic gradient, to reduce the possibility of suffusion, liquefaction and sand boils:** in addition to lengthening the seepage path, the hydraulic gradient can be controlled through the careful selection and use of impermeable materials in appropriate locations.

3 **Increase the quality of contact between earthfill and structure, by roughening the structural surface:** the quality of the contact between earthfill and concrete can be improved by:

i roughening the surface of the concrete

ii placing concrete in excavations adjacent to the hard structure

iii coating the concrete with an adhesive compound such as bituminous paint before compaction of the soil adjacent to the hard structure.

4 **Install filters and drains in or beneath the landward slope of the levee:** if the seepage cannot be restricted to minimal quantities then the flow of water must be managed through the design of appropriate filter systems to inhibit suffusion, to control phreatic surfaces to acceptable levels and to collect and disperse any water that passes through the levee.

Examples of managed transitions between earthen levees and rigid structures are provided in Boxes 9.44 and 9.45.

Box 9.44 ***Example transition between earthern levee and a flood wall***

An example of a managed transition between a concrete flood wall and a levee is provided at interface at Comps in Languedoc-Rousillon, close to the confluence of the Gard and the Rhône rivers. A levee was to be constructed to abut a return flood wall which was built perpendicular to the main flood wall. There was a concern that during a flood, water would seep along the interface between the wall and the levee. The following work was therefore carried out.

Figure 9.66 ***Preparation of the flood wall transition at Comps (courtesy Thibaut Mallet)***

Stage 1

The concrete surface was roughened using a jackhammer to increase contact between earth and concrete (Figure 9.66). The wall foundations were roughened in a similar manner before the deposition and compaction of the levee fill material.

Figure 9.67 ***Compaction of levee fill against flood wall transition (courtesy Thibaut Mallet)***

Stage 2

The levee fill material was deposited and compacted against the flood wall (Figure 9.67).

For details of construction methods and equipment see Section 10.5.

Box 9.44 Example transition between earthern levee and a flood wall (contd)

Figure 9.68 Excavation of trench perpendicular to seepage path (courtesy Thibaut Mallet)

Stage 3

A large trench was excavated perpendicular to the potential seepage path between the levee and the return flood wall to expose the roughened surface of the wall and the wall's foundations (Figure 9.68).

Figure 9.69 Concrete plug installed to lengthen potential seepage path (courtesy Thibaut Mallet)

Stage 4

Concrete was poured into the trench to produce a good connection between the earthfill and concrete, lengthening the seepage path (Figure 9.69).

Box 9.45 Treatment of transition between earthen levee and closure gate

Here the transition between both of the structures has been treated by lengthening the preferred path of water seepage and by putting bituminous paint on the wall to increase the efficiency of the contact between earth and concrete. The lengthened seepage path was created by:

- adding concrete beams into the underside of the foundations by placing concrete directly into a trench (Figures 9.70 and 9.71)
- using sloped side walls and bituminous paint to improve compaction of soil against the interface with the structure (Figures 9.72 and 9.73).

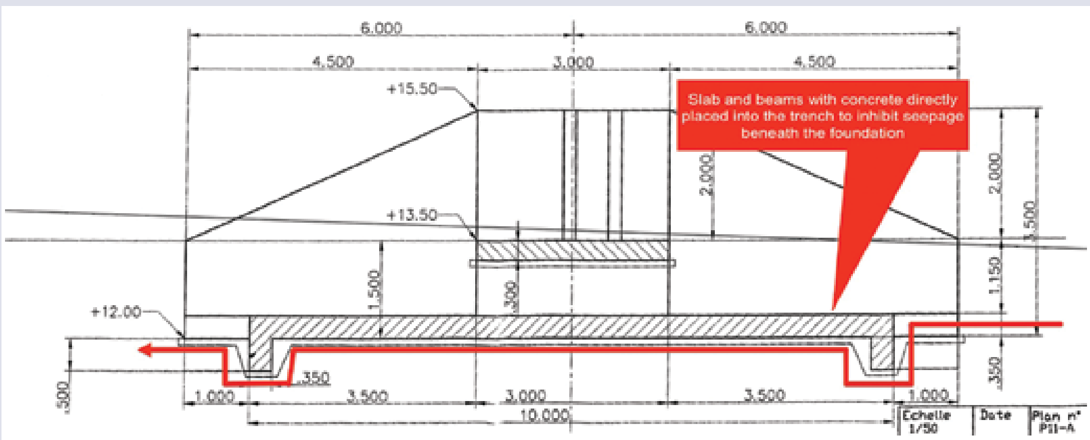


Figure 9.70 Use of concrete beams to act as cut-offs to reduce seepage (courtesy Thibaut Mallet)

Box 9.45 *Treatment of transition between earthen levee and closure gate (contd)*

Figure 9.71 Concrete slab and beams cast directly into excavations, Aramon Levee, South of France (courtesy Thibaut Mallet)

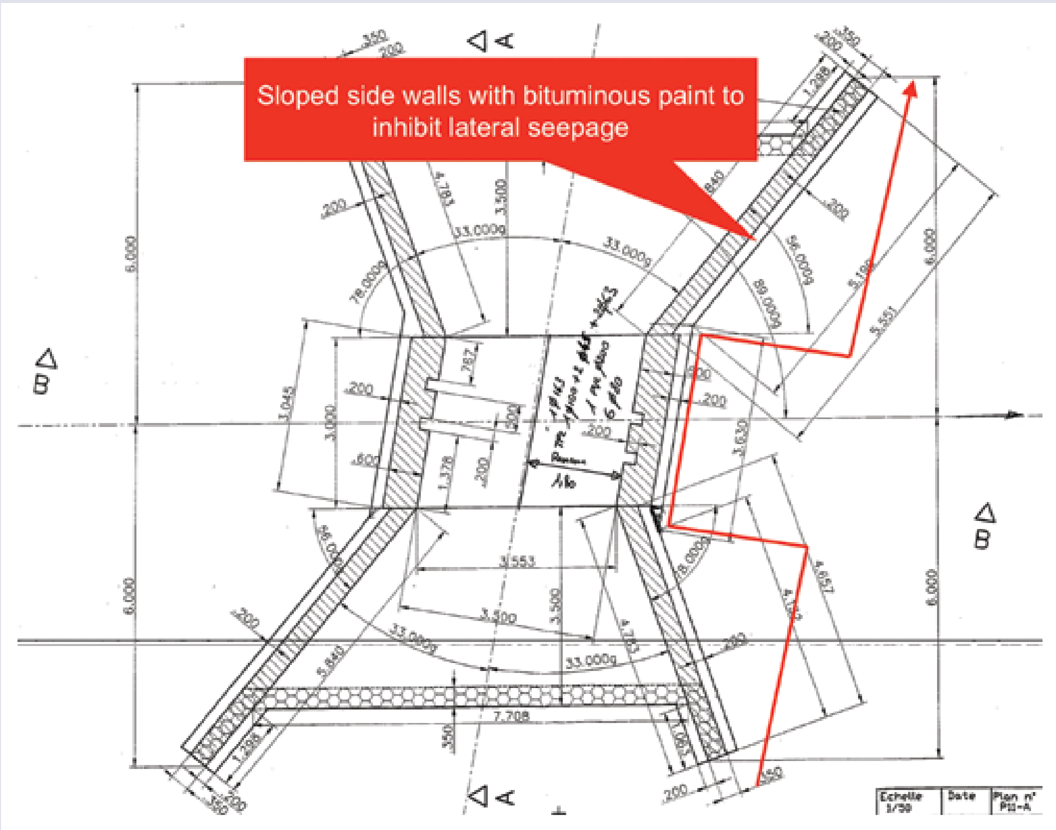


Figure 9.72 Plan view showing closure gate interface details, Aramon Levee, South of France (courtesy Thibaut Mallet)

Box 9.45 Treatment of transition between earthen levee and closure gate (contd)

Figure 9.73 Completed closure gate transitions, Aramon Levee, South of France (courtesy Thibaut Mallet)

9.12 DESIGN FOR SERVICEABILITY

There are a number of key serviceability design requirements for good levee performance. These include some aspects already discussed elsewhere, including seepage control (Section 9.7). This section focuses on three particularly important aspects of serviceability design:

- settlement and rutting, which can reduce the effective crest level of the levee
- desiccation cracking which can increase soil permeability, allow the ingress of water and generate internal loads
- control of animal burrowing which can increase seepage and induce internal erosion.

For a discussion of these issues from an operations and maintenance perspective see Chapter 4.

9.12.1 Designing to manage settlement and rutting

The construction or raising of a levee imposes external loads on the surface of the supporting soil masses, and this induces increased stress levels within the foundation soils. These increased shear stresses in turn cause the foundation soil to deform both vertically and laterally, and this process can happen quickly or slowly over an extended period of time, depending on the permeability and drainage characteristics of the foundation soils. This displacement is observed as settlement, horizontal movement and distortion of the levee, and in the case of foundation soils that comprise soft clays and/or peats, the magnitude of settlement can be significant. Settlements in excess of 1 m are not uncommon for large levees constructed on alluvial, estuarine or deltaic floodplains.

The causes of levee settlement include:

- plastic deformation during construction

- post-construction consolidation of soft compressible layers in the levee's foundation soils (Figures 9.74 and 9.75) which can occur over extended periods of time (depending on the thickness and the permeability of the consolidating layers and their drainage characteristics)
- instability of the levee such as rotational failure affecting all or part of the crest
- collapse settlement or creep of poorly compacted fill, caused by submersion or inundation of fill materials or vibration (from traffic, construction equipment etc)
- internal erosion of the levee of the underlying soils creating voids which collapse over time
- burrowing animals creating voids which collapse over time
- surface erosion due to laminar flow, overflow and overtopping
- surface erosion or rutting caused by human or animal tracking or by vehicular trafficking.

Differential settlement is of particular importance for the behaviour of a levee (Figure 9.75). Locally, excessive settlement will create a low spot on the levee crest which could be the point where overflow commences during a flood. Similarly, a hard spot such as a culvert could initiate deformation cracking of a levee, which will weaken it. An understanding of local geology is an important element in being able to predict the location and nature of any differential settlement.

The management or control of total and differential settlement is an important part of the design process to ensure that levees are operating as required to retain water. The approaches adopted to manage settlement through the design process can be described as either proactive or reactive:

- **proactive settlement assessment** is carried out where new construction or levee raising will increase total stresses within the foundation soils and will also increase effective stresses as the foundation soil consolidates. Settlement of this nature can be predicted using the techniques described in Section 8.7 and can include the prediction of differential settlement between different foundation conditions or between stiff structures and earthen levees. Assessments should be supported by appropriate field investigations
- **reactive settlement assessment** is required where unanticipated settlements have occurred, possibly following new construction works, as a result of ongoing internal or external erosion, or possibly as a result of collapse settlements. In this case, the cause of the settlements must be determined before a remedial works solution can be designed (Chapters 4 and 5).

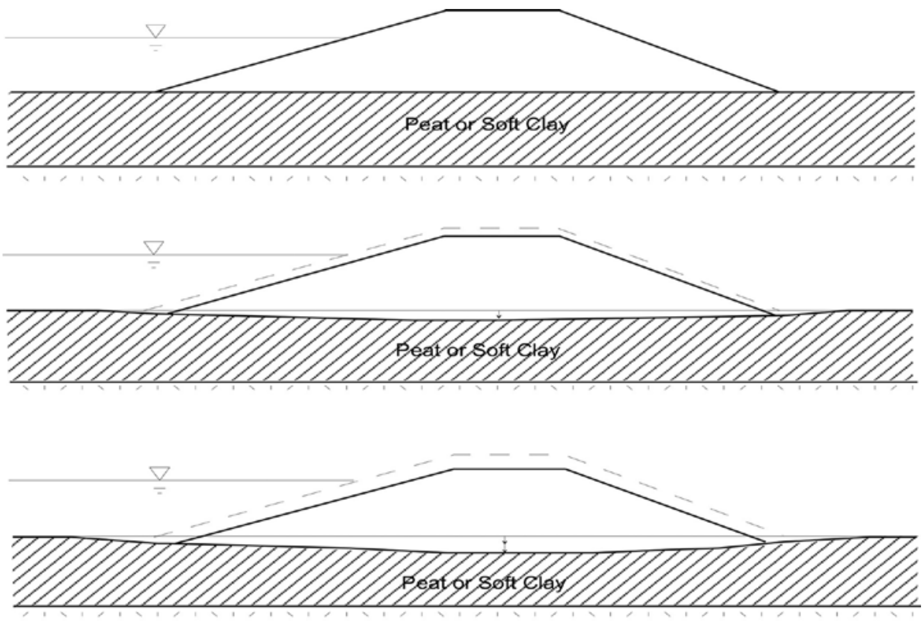


Figure 9.74 Settlement of levee on compressible foundation

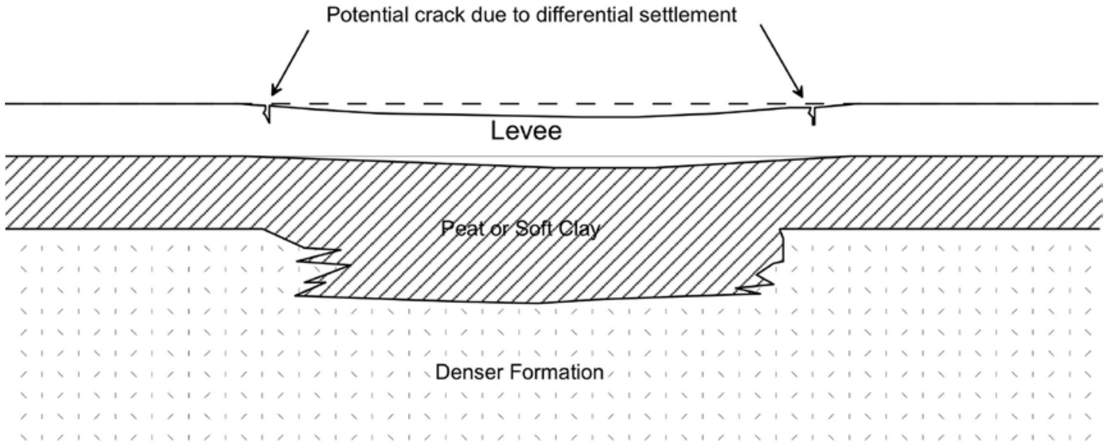


Figure 9.75 Typical levee crest profile showing differential settlement

9.12.1.1 Proactive settlement assessment

Realistic determination of the magnitude and rate of settlement requires a good understanding of the ground conditions along the length of the levee, of the engineering characteristics of the levee fill materials (Chapter 7) and of the method of construction (Section 9.13 and Chapter 10). Guidance on appropriate settlement calculation techniques is provided in Section 8.7 and procedures for calculation in the USA are given in Box 9.46. Calculated magnitudes and rates of settlement should be treated as estimates rather than precise predictions.

Accommodation of consolidation settlement is achieved by overbuilding the levee section by the anticipated amount of settlement. In this case, checks on mass instability will be required, using the maximum height of the levee (not the height after the consolidation process) because the additional height may be sufficient to cause an undrained failure of the levee during construction (Section 9.9.1).

Collapse settlement is linked to the bulk permeability of the clay fill. If air voids are reduced to 5 per cent or less, the resulting permeability will commonly be low (typically 10^{-9} m/s to 10^{-10} m/s) and the vulnerability to collapse compression will be substantially reduced (Charles and Watt, 2001). Means of achieving this reduction during construction by compactive effort are discussed in Section 9.13.3.

Box 9.46 Procedures for estimating settlement in the USA

In the USA, FEMA requires that engineering analyses are carried out to assess the potential for, and magnitude of, future losses of freeboard as a result of levee settlement, and to demonstrate that crest levels will be maintained within the minimum requirements for the duration of the levee service period. Detailed analysis procedures, such as those specified by USACE (1990) are usually followed. The required factors for evaluation include: levee loads, compressibility of levee soils, compressibility of foundation soils, age of the levee system and construction compaction methods. The FEMA guidelines do not provide guidance on acceptable performance criteria/standards of the identified stability factors to be evaluated. Final top of levee elevations should account for future settlements (USACE, 1990).

9.12.1.2 Reactive settlement assessment and design

Where ongoing and unexplained settlement takes place, determining which of the possible causes of the settlement may be responsible is essential before setting out on a course of remedial works. This will avoid unnecessary remedial works, bearing in mind that remedial action could worsen the situation. (For example, if the settlement of the crest is being caused by a rotational failure of the levee, the placement of additional fill on the crest could accelerate the movement.) On the other hand, where the problem has a clear cause, local repairs can be carried out quickly and effectively.

An example of a more obvious cause of settlement is surface rutting resulting from trafficking or animal tracking (Section 4.8). This can be addressed by:

- reinstating the original arrangement

- using similar materials to the original levee (so as not to increase the permeability of the material in the levee crest)
- taking care to bind the new repair into the existing structure (Section 9.13.3).

If the problem persists, the cause of the rutting should be investigated further and, if different details from the originals are required, they should be designed with an understanding of the required material characteristics and a knowledge of the principles of levee design.

9.12.2 Controlling or remediating desiccation

Levees may be constructed out of low permeability clays and are prone to volume change as a result of variations in moisture content caused by seasonal wetting, drying and wetting or by the growth of vegetation. Cracking can result from during the drying (desiccation) phases and the process of desiccation may have the following adverse effects on levee performance:

- increases in soil permeability near the levee surface can leave the levee vulnerable to seepage flows, particularly during the transition between hot, dry periods and cooler wetter ones
- desiccation cracks can facilitate the ingress of rain or floodwater, which can lead to softening of the levee crest and side slopes
- water-filled cracks will generate internal loads that can have an adverse effect on levee stability (Section 9.9).

The desiccation potential of a soil can be related to the soil's plasticity index (Frith *et al*, 1997). Very high plasticity clays (with a plasticity index of more than 40 per cent) are known to have a very high shrinkage potential, whereas low plasticity clays (with a plasticity index of less than 10 per cent) are known to be of low shrinkage potential.

Desiccation cracks can form to significant depths. In the UK (Figure 9.76), these can reach a depth of up to one metre (Dyer *et al*, 2009) and in more arid countries up to 3 m. Cracks can develop in both longitudinal and transverse directions forming an 'alligator skin' type appearance on the levee surface (Figure 9.77).

If left untreated, desiccation cracks will weaken the levee and make it more vulnerable to failure during a flood event, such as the failures described by Cooling and Marsland (1954). The tension cracks may also prove to be a 'final straw' for a levee of marginal stability as they have the dual effect of reducing the resistance to failure and increasing the disturbing forces, leading to a failure such as the one shown in Figure 9.78.

Figure 9.76 Desiccation cracking in levee at Thorgumbald (from Dyer et al, 2009)

Figure 9.77 Formation of 'alligator skin' type appearance (courtesy Said Salah-Mars, URS Oakland)

Figure 9.78 Head scarp of a slide on the levee crest (courtesy Said Salah-Mars, URS Oakland)

The onset of desiccation can be delayed (Frith *et al*, 1997) by compacting the clay fill at a moisture content close to the plastic limit (no more than two per cent above the plastic limit and no more than five per cent below). However, it is considered that the natural process of weathering (particularly the seasonal wetting and drying of the near-surface materials) will inevitably lead to the cracking of even the most well-compacted materials. Alternative methods of controlling desiccation cracking should therefore be adopted if it is considered that the fill material will be prone to cracking. Methods (Figure 9.79) given by Frith *et al* (1997) are therefore now summarised for both remediating existing desiccation problems and for avoiding them in new designs.

9.12.2.1 Remedial actions for desiccation

Cracking can be minimised by adopting suitable topsoil and vegetation and following suitable maintenance methodologies (Section 4.12). However, where full remedial measures are required the following may be considered:

Digging in (Figure 9.79c): consists of removing the topsoil, excavating a trench along the centreline to a depth of 1.2 m to 1.5 m below crest level, breaking and mixing the excavated material and then recompacting it back into the excavation as a dry cohesive material and then replacing the topsoil. While this solution is quick and relatively cheap, it involves some risk as it requires excavation into the core of the levee. In addition, it may not be effective over a long period of time as the replaced material will still be of high plasticity and will therefore still be prone to desiccation.

Berm construction (Figure 9.79d): on the landward side of the levee does not treat the desiccation cracks directly but increases the stability of the levee overall so as to compensate for the deleterious effects of the cracking. A berm also has the beneficial effect of increasing the levee's resilience to overtopping flow.

Replacement of surface materials with well-graded granular material: called hoggin in the UK (Figure 9.79e), with a gravel content of about 50 per cent as well as avoiding desiccation cracking also provides increases in shear strength. Excessive permeability can be avoided by a fines content (material finer than 0.063 mm) of 5 to 25 per cent. A grading envelope (Figure 9.79) suggested by Frith *et al* (1997) for such material is indicative, but should not be viewed as the only solution. Materials meeting the specification of Frith *et al* (1997) can be created by mixing fines and medium to coarse gravel. However, such materials should be checked for their internal stability to suffusion (Section 9.8) during flood flow induced seepage, by checks on their uniformity coefficient (Section 8.5.3) and for the appropriateness of their permeability.

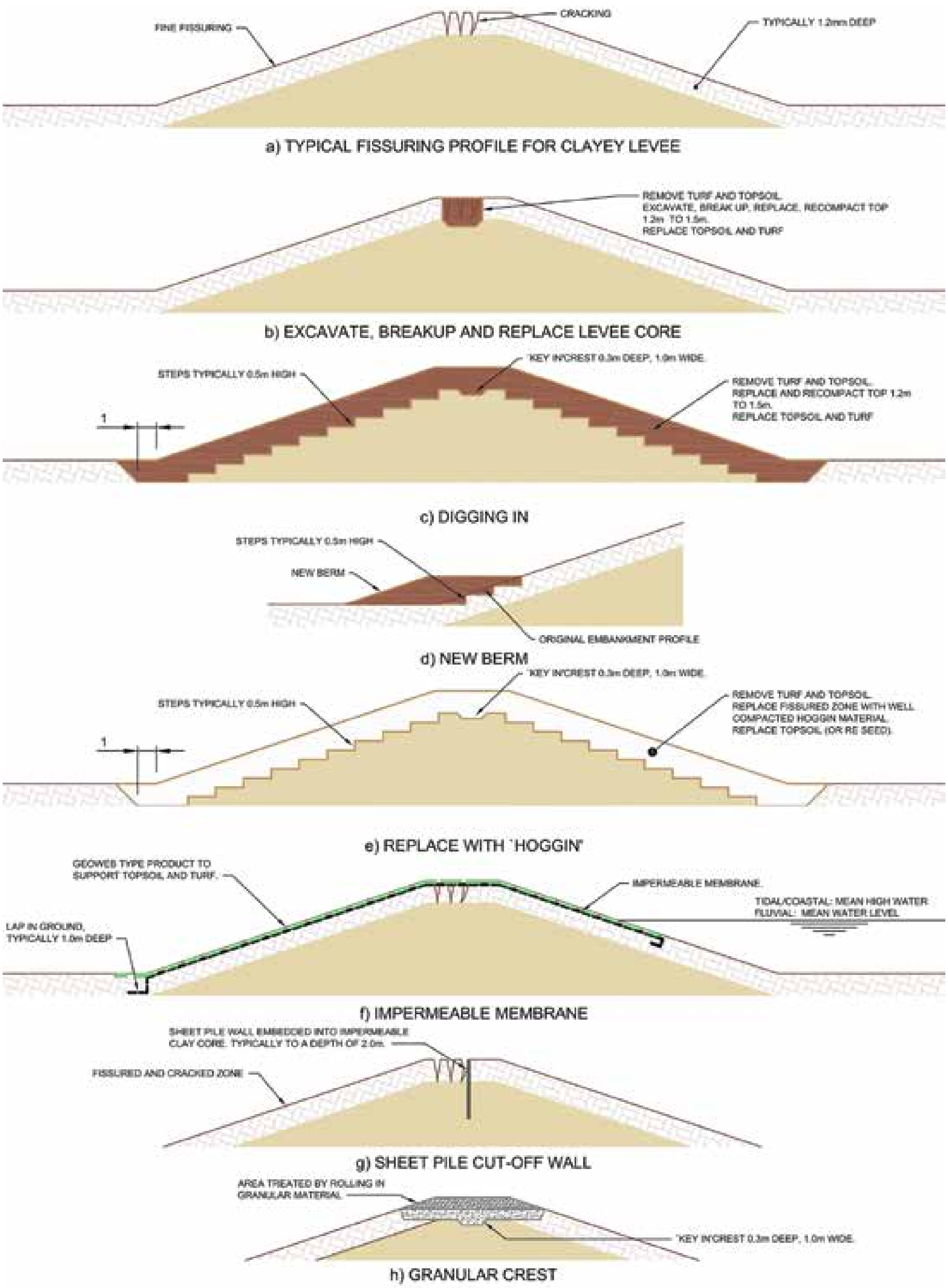


Figure 9.79 Measures to avoid or remediate desiccation cracking (after Frith et al, 1997)

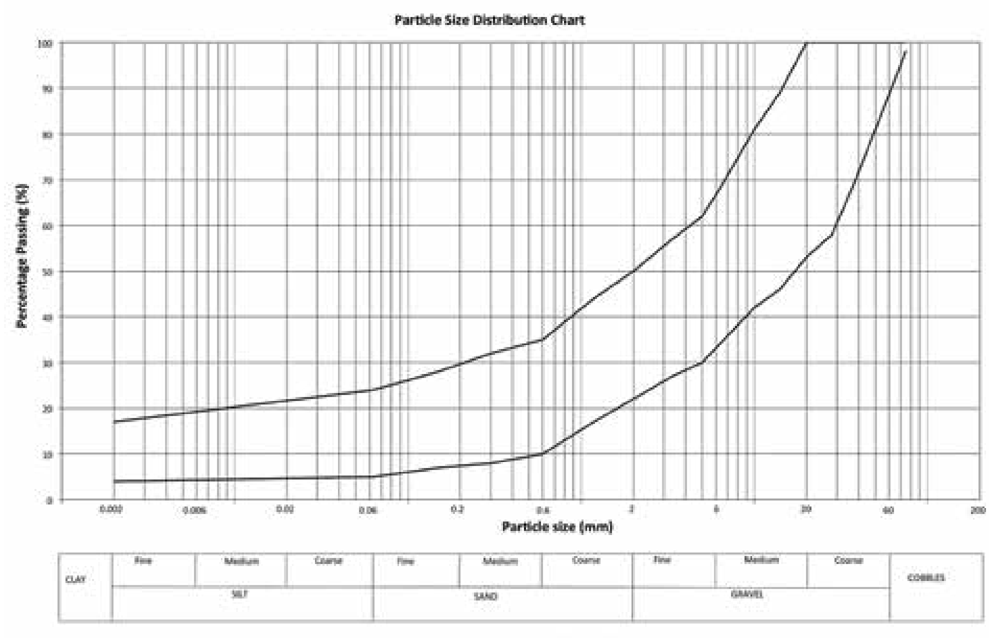


Figure 9.80 ***Grading envelope for hoggin (after Frith et al, 1997)***

The replacement depth of well-graded granular material (hoggin – see Figure 9.80) should be selected according to the circumstances, but 0.5 m to 1 m may suffice in most circumstances. It will also need to be decided whether to replace the complete surface of the levee (crest and side slopes) or just the crest. If just the crest is to be replaced, then the fissured material should be removed and the surface scarified (and ideally a 'key' excavation created), before the deposition and compaction of the well-graded granular material. If the replacement of the existing fill on the side slopes is required then the hoggin should be placed and compacted on a series of horizontal steps to aid compaction and to avoid the possibility of the replacement fill slipping down the top of the old material.

Impermeable (HDPE or PVC) membranes (Figure 9.79f): can be used (Frith *et al*, 1997) to limit the impact of desiccation by preventing the permeation of water into any fissures. Such an impermeable membrane should cover the entire surface of the levee from the landward toe to the normal water line on the water-facing slope and then be covered by a layer of topsoil. However, the solution is often expensive and checks are necessary:

- to ensure sufficient friction between the membrane and the topsoil to avoid the topsoil sliding off the membrane during a flood
- on the likely adverse effect of the membrane on any surface vegetation and the possibility that seepage through the levee might lift the membrane off the landward slope of the levee.

Cut-off walls of sheet piling or concrete (Figure 9.79g): would prevent flow through the fissured zone (Frith *et al*, 1997). Such walls would need to be installed into an underlying impermeable layer by about 0.5 mm and would need to be extended to penetrate below underlying layers of permeable soil. Such solutions have some disadvantages:

- they do not improve the resilience of the landward face to overtopping flow induced erosion. Alternative methods such as a berm or flatter side slopes would be required to reinforce the slope if the soil on the slope is fissured
- cut-off walls carry the full hydrostatic load in the water-facing side and are vulnerable to movement and failure, particularly if passive resistance is lost due to overtopping flow erosion (Section 9.15).

Provision of a granular crest (Figure 9.79h): was devised by Frith *et al* (1997) as a simple low-cost way of overcoming desiccation cracking and would need to be carried out during a season when the surface cracks had largely closed. A thin layer (say, 100 mm) of well-graded granular fill material is spread over the surface of the levee crest and then harrowed and compacted into place using a roller. The process is continued until the granular material is no longer pushed into the body of the levee and a dense, mainly granular surface is produced. The method reduces the impact of the fissuring but does not eliminate it, and regular inspections and occasional excavations of the treated surface will be necessary to check that the method is working as intended. As with cut-off walls, this method also does not improve the condition of the landward face or its resilience to overtopping. Furthermore, checks on seepage rates and internal erosion are required for the situation where the retained water level exceeds the top level of the clayey fill and flows through the granular overburden.

9.12.2.2 Designing to avoid desiccation cracking

Where desiccation-prone material has to be used for construction, the desiccation prone material could be used in the core of the levee and a layer of lower-plasticity soil, of at least 1 m thickness, be used on the surface. Note that if silts or sandy silts are used on the surface to cover the high plasticity clay, their resistance to surface erosion as a result of overtopping, overflow and run-off should be checked (see Section 9.6).

When there is no economic alternative to the use of a highly plastic clay for levee construction, often sourced from ditches or trenches adjacent to the levee, forms of mitigation may include (Frith *et al*, 1997) incorporation of features such as those described in the previous section or the following measures:

An additional 'freeboard' allowance can be provided, the height of which should be related to the susceptibility of the fill material to fissuring and the consequences of levee failure (Table 9.24).

Table 9.24 Suggested freeboard allowance to account for fissuring (after Frith et al, 1997)

	Site sensitivity		
Tendency of soil to fissure	*High*	*Medium*	*Low*
High	900 mm	900 mm	600 mm
Medium	900 mm	600 mm	600 mm
Low	600 mm	600 mm	300 mm

Capping with well-graded granular material (hoggin) at least 1 m thick over a clay core. The compatibility of the two layers would need to be considered from the point of view of internal erosion and the magnitude of seepage through the capping layer during the design flood event assessed.

Cementitious materials such as lime could be added into the top 1 m or so of levee fill material before deposition and compaction to cement the soil and reduce its plasticity index, reducing its susceptibility to desiccation cracking. For organic soils, cement could be added to achieve the same effect. However, the use of admixtures may have an adverse environmental impact, particularly on the growth of vegetation, and it may also behave in a brittle manner (being prone to cracking due to differential settlement).

Construction of a berm or increases in crest width can be provided. These increase the factor of safety against geotechnical instability rather than directly dealing with the problem of using a high plasticity clay as a fill material for levee construction. Details of the method are similar to those suggested for remediation in the previous subsection.

Careful management of material deposition and compaction will limit susceptibility to desiccation cracking, at least in the medium term. Frith *et al* (1997) suggest that clays should be compacted at a moisture content of between two per cent above the plastic limit and five per cent below the plastic limit. Earthworks materials should be protected from the weather and the levee should be constructed or raised in short lengths, to reduce the risk of deterioration of the fill material. This method will only reduce and delay the impact of desiccation cracking. As the levee weathers naturally from the surface down over time, the likelihood of desiccation cracks occurring will increase.

Provision of good quality grass cover (Section 4.5) will reduce the impact of desiccation cracks as these crack are always more prevalent in bare areas than they are in areas where there is a good coverage of grass. Ideally, to avoid environmental issues, the grass species selected should be native to the area and should be selected (Hewlett *et al*, 1987 and Coppin and Richards, 2007) taking account of the following issues:

- the match between the topsoil and the grass species
- the balance between the topsoil and the growing characteristics of the grass
- the ability of the grass's root system to bind the topsoil together and inhibit or reduce the formation of the desiccation fissures
- the ability of the grass species to cope with the variety of atmospheric conditions anticipated and other threats such as disease and infestation with insects
- the maintenance required to ensure a good quality short and dense turf.

9.12.3 Controlling or remediating animal burrowing

Burrows created by animals such as rabbits, gophers, muskrats, opossums, badgers, foxes, wombats and other animals can lead to rapid levee failures during floods. Burrowing animals (Section 4.6) can present a significant threat to levee integrity and, although not always obvious from the surface, individual or networked animal burrows may completely traverse a levee section. While burrowing animal control techniques such as fumigation, bait stations, bait broadcasting or trapping and removal can be successful remediation techniques, the potential for burrowing animal damage and associated remediation should also be considered during design stages. This is particularly the case in areas where burrowing animals are known to inhabit levees.

9.12.3.1 Repairing animal burrows

While there is no effective method of completely excluding burrowing animals from occupying grass-covered levees, more effective methods for repairing burrows include excavating and backfilling and grouting (DWR, 2012).

If the 'excavation and replacement' method is to be used, the process should be carried out immediately after the levee has been mowed. The excavation should be backfilled with compacted soil that is similar to levee fill material and reseeded. Burrows may turn out to be extensive, and the possible temporary flood risk resulting from large excavations into a levee should be considered before adopting this solution.

Grouting with a cementitious flowable material is generally found to be more cost effective. Commonly adopted and effective grout mixes include:

- nine parts cement, one part bentonite and water added to achieve 8 to 10 inches of slump (USACE, 2006)
- per cubic metre (Pennsylvania Department of Environmental Protection, 2011): Type I Portland cement – 430 kg, fine masonry sand – 1250 kg, clean water – 230 litres.

After the mixture is thoroughly mixed, it should be pumped into the burrow at low pressures (to avoid damaging the embankment) entirely filling the void. The process should start at the base of the levee and proceed up the levee slope. For a large levee, it may be necessary to carry out the grouting in stages, allowing the grout to set before subsequent lifts. The following practical issues should be considered:

- levee dragging should only occur after burrows are repaired
- burrows temporarily covered for fumigation should be marked for later excavation and repair
- extra vigilance in monitoring and repair of burrows is needed for frequently loaded levees
- for certain situations, such as short levee reaches or areas where burrowing animals are known to be particularly damaging, permanent burrowing-animal barriers should be considered in designs.

9.12.3.2 Barriers against animal burrowing

Where burrowing animals are known to inhabit and damage levees, barriers may need to be incorporated within the levee structure, taking account of the following issues:

1. The **barrier location** should be close to the levee surfaces to discourage the burrowing animals at the potential points of entry.
2. The **barrier strength** should be sufficient to discourage the type of burrowing animal likely to be encountered.
3. The **barrier resistance to erosion and material deterioration** should be sufficient for the full life of the levee structure, or it should be installed so that its condition can be monitored and so that it can be easily repaired or replaced if damaged.
4. The barrier should not adversely affect the performance characteristics of the levee including, erosion resistance, stability and permeability.

Solutions that have been adopted include the introduction of surface grid meshes (Box 9.47) and non-cohesive cover layers (Figures 9.83 and 9.84) as recommended by Heerten and Werth (2006).

Box 9.47 Animal burrowing solution adopted at a levee project in Arles, France

Figure 9.81 shows a construction detail from a levee remediation project at Arles in France where a wildlife protection wire mesh was incorporated in the levee construction to discourage borrowing animals. The mesh was pinned down onto the compacted levee fill before the placement of the seeded topsoil. A photograph showing the mesh during construction is shown in Figure 9.82.

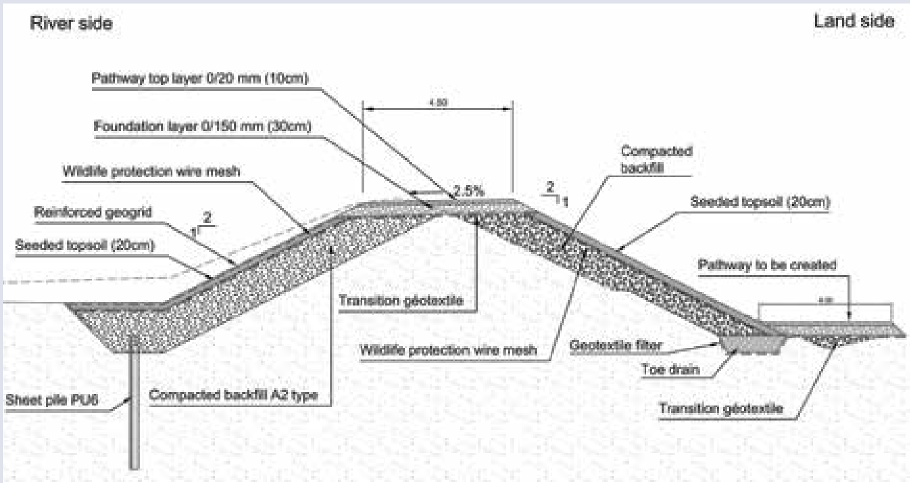


Figure 9.81 Levee improvement works at Arles, France (courtesy SYMADREM and EGISeau)

Figure 9.82 Wildlife protection mesh incorporated into levee improvement works at Arles, France (courtesy SYMADREM and EGISeau)

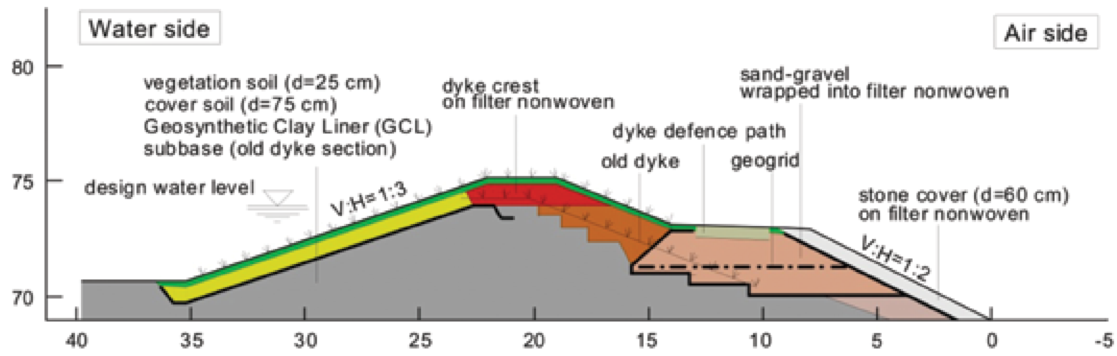


Figure 9.83 Standard cross-section of a reconstructed Elbe levee near Bösewig/Sachsen-Anhalt (Heerten and Werth, 2006)

Figure 9.84 Covering geosynthetic clay liners with locally available gravel to ward off burrowing animals (Heerten and Werth, 2006)

9.13 LEVEE EARTHWORKS

This section of the handbook first considers how the performance characteristics of levees can be achieved through the selection of appropriate earthworks materials. It then focuses on the management and control of earthworks materials for levee construction. The use of earthworks for levee raising and repair is then discussed. This section concludes with guidance on the use of geofabrics (Section 13.4) and some innovative methods of ground improvement (Section 13.5) which can be used in association with levees.

9.13.1 Selection of earthworks materials

This section considers the design criteria necessary for controlling the selection of the fill materials and the methods of deposition and compaction needed to fulfil the design requirements. In most cases, earthworks materials for levee construction are selected through the end product testing of on-site materials and those from potential borrow sources. While use of selected imported materials for levee construction may offer the best engineering performance, cost and environmental drivers may require the use of locally available soils with lower levels of performance (Box 9.48).

Box 9.48 Use of poor-quality fill materials for low-risk levees

The Broadland Project on the Norfolk Broads in the UK was organised on the basis that locally sourced material was used for levee raising works because the general inaccessibility of the Broadland sites make importation too expensive, and in many cases impractical. The land protected by the local levees is predominately grazing marsh and low-risk pasture.

Due to the lack of availability of suitable fill material in some areas, artificial fill material was produced by the mixing of peat and gravel on site, as no clay was locally available. Tests were carried out to determine the best mix ratio and 15 per cent peat by weight gave an acceptable grading. By volume, this amounted to equal amounts of peat and gravel (dry densities of peat and gravel were 0.36 and 2.0 mg/m^3 respectively). This mixing worked very well when undertaken by a skilled machine driver.

Standard bank slopes (for the Broadlands project) were 1 in 2 on the waterside face of the levee and 1 in 3 on the landward face. The levee was overtopped by about 50 mm during a 1 in 10 year flood event, but suffered very little damage despite the vegetation only just having been established. The levee also leaked very little, although the existing clay core of the original bank was still in place to help maintain watertightness.

Construction reflected the low-risk nature of the levee and involved experienced machine operatives as much as engineers. The general process of construction was as follows:

- The materials were excavated on site simultaneously by two excavators and placed in layers on the new bank area.
- These soils were immediately mixed with the bucket of a third excavator.
- This method was weather and moisture dependent – time to dry out (and drain) was necessary, but it provided a very firm finished embankment.

Figure 9.85 Levee constructed of peat and gravel, Broadlands, UK (courtesy BAM Nuttall, and CH2MHill)

The following soil properties should be considered during testing (Sections 7.8 and 7.9), selection and specification.

9.13.1.1 Resistance to external and internal erosion

Fine-grained silts and granular sands are more susceptible to erosion than clay soils. However, clay soils may be susceptible to desiccation cracking unless protected, and so a design balance will be needed.

Erodible soils may be treated to improve erosion resistance, using varying dosages of materials such as lime, fly ash, cement etc. However, it should be noted that dosing materials may have an environmental impact, with alterations of the geo-chemistry of the ground surface inhibiting the growth of suitable grassy vegetation. A laboratory study will normally be necessary to determine dosage rates for treatment and a test section may be required in order to verify performance under erosive conditions. Further information is given in Sections 9.6 and 9.8.

It is well established that sandy soils are far less resilient to overtopping erosion than clayey soils. So, a plastic clay, free from organic materials, with a limited or zero sand content, and prepared, deposited and compacted to an appropriate water content and density will provide a durable soil when subjected to overtopping. This is the case as long as it is not prone to pronounced desiccation cracking.

Erodibility testing (Box 9.49) can be performed on material from potential borrow areas, on blends of materials or treated materials before production of the earthworks specification so as to identify more desirable erodibility resistance characteristics. The extent of testing depends on the degree of variation encountered, including in the quantities of organics, sand, shell or other materials. From such preliminary testing, acceptance criteria for levee construction can be developed, and appropriate uses identified for each given soil or soil blend.

Box 9.49 ***Erodibility of organic clay soil***

Figure 9.86 ***Determining erodibility of organic clay; photographs taken before and after application of a water jet (courtesy Mike Wielputz, USACE)***

Figure 9.86 shows images of New Orleans Clay – which can have an organic content of up to 18 per cent by weight – before and after water jet testing, The tests were used to determine limits for allowable organic contents, sand contents, unit weight, unconfined compressive strength, at various moisture contents and compactive efforts.

9.13.1.2 Permeability

Selected fill materials must have a sufficiently low permeability to adequately inhibit the passage of water (Section 9.7) and to control seepage pressures and velocities. Section 7.8.3.5 gives full details of testing methods to determine permeability.

Fine-grained soils such as clays or silts will normally provide a sufficiently low permeability (less than 1×10^{-6} m/s) as long as they are well compacted and the volume of air voids is low. Testing for permeability of such soils normally involves a mixture of laboratory tests in (flexible wall) permeameters (on compacted samples) and *in situ* tests in boreholes or in trial excavations.

For granular soils or clays with significant granular content (with hydraulic conductivities in excess of 1×10^{-6} m/s), careful sampling is required to avoid loss of fines, and testing in a constant head permeameter may require the mixing of different samples followed by a process of remolding and compaction. Where sampling is impractical for granular soils, design values of permeability may be determined by established correlations with parameters such as effective grain size D_{10} (Figure 9.87).

The relationship between the level of compaction and a soil's permeability is important. Specifications typically require the achievement of a minimum density, and the greater the density achieved by compaction, the more permeability is reduced. However, if drainage features are an important element in a design (eg granular drains), over-compaction will need to be avoided in order to maintain the effectiveness of the drain.

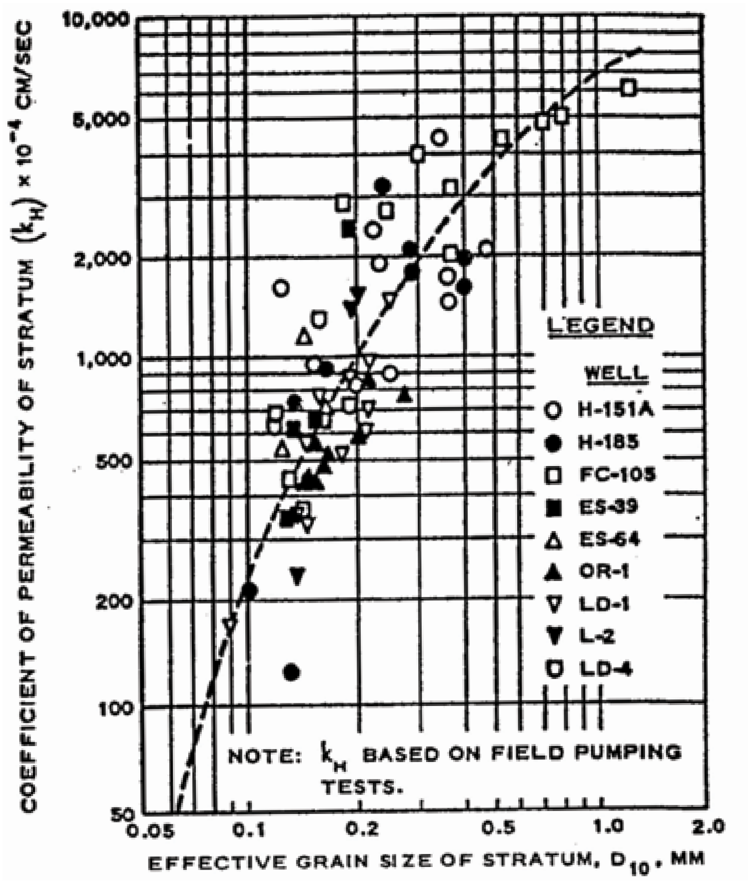


Figure 9.87 *Effective grain size, D_{10} versus coefficient of permeability, k_h (from USACE, 2000)*

Where foundation soils are highly permeable and seepage through the foundation may create uplift, sand boils, piping and heave, an alternative to structural barriers such as sheet piles, may be mixing soils and other materials, such as bentonite and cement, to create a seepage barrier (Box 9.50).

Box 9.50 *Use of soil mixing techniques to control seepage*

Soil–bentonite mixtures can be used in seepage barriers. Cement may also be added to the soil–bentonite mix if there is a high organic content to the soil. However, in seismically sensitive areas it is preferable to use only soil–bentonite since this type of cut-off wall is less vulnerable to cracking and permanent damage from seismic vibrations.

Construction of seepage barriers using such mixtures is usually accomplished using specialised equipment such as long reach-back hoes to excavate a 1 m wide trench to depths of up to 7.3 m (Figure 9.88a). For deeper cut-offs (Figure 9.88b), a deep soil mixing rig with multiple overlapping augers can be used to create a wall 1 m wide by up to 10.3 m depth. Pumping of the fluidised bentonite or cement-bentonite is conducted with blending operations of the existing soil.

Figure 9.88 *Seepage cut-off wall construction along levee centrelines, Sacramento, CA (courtesy Mike Wielputz, USACE)*

9.13.1.3 Shear strength after placing and compacting

Strength characteristics of fill materials (Section 9.9) will vary over the levee's design life. The shear strength that can be mobilised along any potential failure surface within a levee will depend on many factors (such as rate of loading, direction of loading, positions within the levee or the underlying ground and elapsed time since construction). This issue is discussed in detail in Chapter 8, and testing methods in Chapter 7.

It is usual for both drained and undrained shear strength characteristics to be specified, bearing in mind that the measured strengths are dependent on the methodology used for testing:

1 For fine-grained clay fill materials, the **undrained** shear strength of the compacted fill will control stability during and shortly after the construction phase of the work. Thereafter the fill will soften to the point where the long-term **drained** shear strengths govern the stability, particularly in locations close to the surface of the levee where the normal effective stresses are low. Section 9.9 discusses the different situations in which analyses using drained and undrained strength characteristics should be used.
2 For granular soils it is usual to adopt **drained** shear strength parameters for most design situations (except, possibly, for dynamic loading).

The following steps are suggested when specifying shear strength of materials:

1 Identify the fill materials required for levee construction.
2 Identify the characteristic strengths (both drained and undrained) that these materials can mobilise under appropriate loading conditions (ie representative of the loading conditions to which the materials will be subjected in the levees).
3 Establish a method of testing for verification of strength parameters that can be carried out readily during construction.
4 Establish any correlation factors required to convert the measured shear strengths into characteristic shear strengths for design purposes (Section 7.8.3.3).
5 Identify an acceptable range of realistic, non-contradictory and achievable material conditions after compaction, based on the acceptable bounds of density and moisture content and then specify related requirements for relationship to:
   i maximum dry density (MDD), eg greater than 95 per cent of MDD
   ii optimum moisture content (OMC), eg within ±2 per cent of the OMC.

   Guidance on equipment and methods of compaction are given in Sections 9.13.4 and 10.4.3.4.

The preliminary exploratory phase of sampling and testing, coupled with the design analysis, should provide the necessary parameters and criteria for the constructed levee fill. However, a post-construction phase of exploratory analysis may be deemed necessary to validate the design.

9.13.1.4 Selection and specification for mass density

Mass density will affect:

- the balance between the disturbing and restoring forces (Sections 9.9 and 9.10)
- the ability of the levee to resist uplift (Section 9.7 and 9.9)
- the magnitude of the post-construction settlement of the levee (Section 9.12).

Characteristic densities of compacted materials can normally be established with a good degree of confidence. For preliminary design purposes, estimates will normally be established on the basis of experience. Compaction testing of samples to provide ranges of the actual densities can then be carried out during the investigation of the materials in the potential borrow pits (Section 7.7). The number of tests should be sufficient to represent the variability of the materials.

9.13.1.5 Resistance to liquefaction under seismic action

Studies of seismic activity in the area of the levee may suggest the possibility of damage due to liquefaction, with loose sands and silts generally being most at risk. Evaluation of materials' acceptability may be based on laboratory studies including cyclic triaxial testing of the available materials. Alternatively, established correlations between material type, density and liquefaction potential may be used (Section 7.8 and Seed and Idriss, 1971, Seed *et al*, 1983, Youd *et al*, 2001 and Seed, 2010).

9.13.1.6 Selection and specification for compressibility

Self-weight consolidation or compaction of the fill material itself will affect post-construction settlements (Section 9.12.2). However, consolidation testing is generally not performed on levee fill. Allowances for self-settlement of levees should instead be based on the state of the fill materials at the end of the compaction process. The correlation of Atterberg limits with the coefficient of consolidation developed for normally consolidated soils (Figure 9.89) may be used, noting that as compacted soils behave more like over-consolidated soils, this correlation will probably overestimate settlement of the levee itself. An alternative would be to use established correlations between *in situ* test data, such as the standard penetration test and compressibility (Stroud, 1989). More information is given in Sections 7.8.

In some parts of the world where the predominant soils are very peaty (such as the Netherlands and Ireland), levees can be constructed out of materials with high organic contents. If possible in such situations, highly compressible, low strength soils should be excluded from use, including:

- organic soils such as peat, moss or clays and silts with a significant organic content
- high liquid limit clays (above 90 per cent)
- high plastic index clays (above 65 per cent).

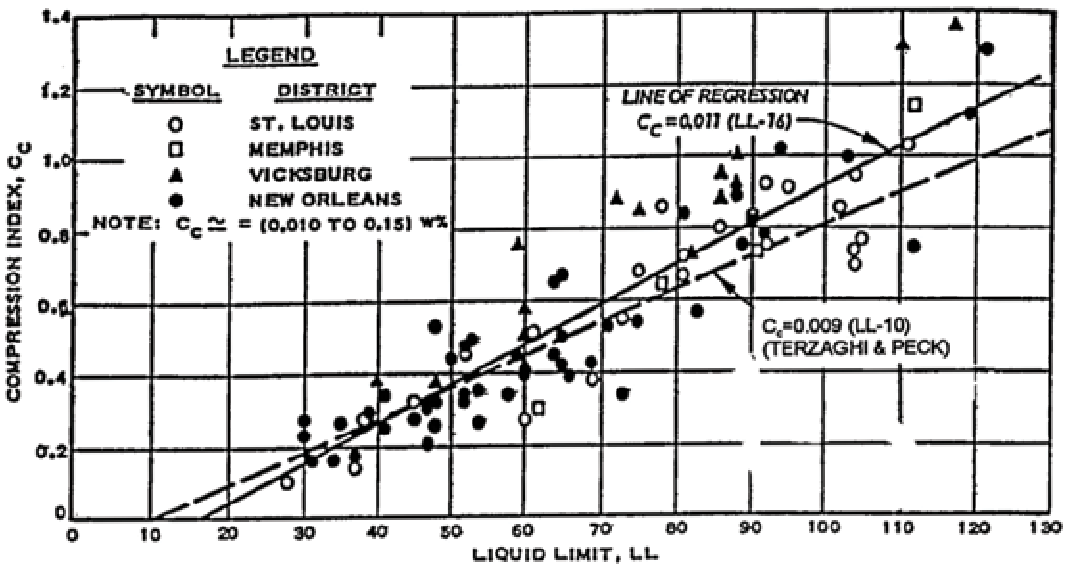


Figure 9.89 Compression index versus liquid limit for normally consolidated soils (from USACE, 2000)

9.13.1.7 Resistance to deterioration including desiccation cracking

Materials selected for resistance to deterioration are typically those that withstand long durations of time with exposure to wetting, drying, freezing, thawing and chemical degradation. Where necessary and/or where mitigation measures such as surface protection systems discussed in Section 9.6 are not feasible, laboratory studies of available materials may be necessary, to determine resistance to degradation under the long-term weathering exposure at the levee project site. It may be necessary to use typical water from the site including seawater, brackish water etc.

As mentioned in Section 9.13.1.1 above, highly plastic clays are typically good for erosion resistance, yet may exhibit extensive desiccation cracking and degradation on the exposed surfaces. One such remedy for this issue is to provide a sacrificial layer of granular soil which will create a freeboard against wave attack. This can be used to promote the growth of suitable vegetation and will protect the underlying clay from desiccation. External erosion could then peel away the sacrificial layer under the action of overflow or overtopping, while still allowing the underlying clay of the levee to perform as an erosion resistant.

9.13.2 Managing and controlling earthworks materials

The identification, management and operation of suitable borrow areas is an important part of the levee design process and requires consideration of impacts on the environment and on land values. In addition, the effect on levee performance (erosion, under-seepage, uplift pressures, overall levee stability etc) of any borrow pits located in the immediate vicinity of the levees should be evaluated. Section 7.7 sets out methods for identifying and investigating potential borrow sites. The results determined from preliminary investigations should be sufficient for design, but should be verified upon completion of construction through construction field data testing analysis and possibly verification borings.

When fill materials of marginal or poor quality are selected for economic or environmental reasons, the use of such materials will almost certainly require that a special regime be established of treatment, blending, extraction and/or modified compaction. The methodology and construction equipment needed to extract it, to separate the suitable material from the unsuitable material and to deliver it to the levee location will have an impact on:

- the cost of extracting the material, including the effect of accessibility and proximity of borrow areas to the location of the levee
- the environment (via issues such as working hours, the number of truck journeys, the depth of excavation and the methods of dewatering used)
- the rate of delivery of fill material, which may in turn affect the whole viability of the source of material
- the condition of the earthworks materials once they have been subject to excavation, selection, transportation, deposition and then compaction.

For these reasons, an experienced contractor should ideally be involved in the selection of a suitable borrow area at as early a stage as possible.

Based on the levee design, an earthworks specification will normally place controls on:

- material type to exclude unsuitable materials and to indicate the basic material characteristics required (cohesive or granular fills) for different features within the structure (Section 9.13.3)
- material grading (particle size distribution) to control the quality of the material, seepage and erodibility (Section 9.13.3)
- dry density of the compacted soil (Section 9.13.1), usually defined as a minimum acceptable relative compaction – the relative percentage of the maximum dry density obtained by standard compaction tests
- moisture content of the compacted soil (Section 9.13.2), usually defined as tolerance from the optimum moisture content.

9.13.2.1 Earthworks compaction criteria

Strength, compressibility and permeability all vary with the degree of compaction of the levee fill material, and appropriate compaction criteria should therefore be determined for the selected fill material on site. Compaction criteria are normally related to three interrelated parameters: dry density (relative compaction) moisture content and percentage air voids.

1 Dry density (relative compaction)

The maximum density that can be achieved for any given fill material is dependent on the compactive effort applied, and this, in turn, is dependent on the characteristics of the compaction plant. The efficiency of compaction of any given fill material is dependent on many factors including the thickness of the layers in which the soil is placed before compaction, the moisture content of the fill material as placed and the suitability of the compaction plant to the fill material used.

Determination and control of relative compaction is based on the ratio between the soil's actual (*in situ*) density and the maximum dry density achieved in a standard compaction test.

Density–moisture content curves can be prepared, based on the test results such as shown in Figure 9.90 and hence target densities and ranges of moisture content identified. Soils classifying and compacting in a similar manner can be grouped together to determine optimum moisture contents for that group.

It is usual to define acceptance of the compacted earthworks materials on the basis of achieving a minimum relative compaction in each layer placed. Depending on the nature of the earthworks materials, the relative importance of the levee and the position of the fill within the overall structure, the target relative compactions will often be selected as 90, 95 or 98 per cent of maximum dry density (MDD), obtained from standard (2.5 kg) or modified/heavy (4.5 kg) compaction (Proctor) tests.

Because of interrelations with the resulting density, controls are also normally applied to the moisture content of the fill (see point 2) at which this is attained. So, a compaction specification will normally consist of a range of relative compaction values (eg minimum relative compaction of 95 per cent), together with a range of acceptable moisture contents (eg optimum moisture content ±2 per cent).

More heavily compacted materials are generally less subject to self-weight compaction and *collapse settlement* arising from air voids (see point 3) when they are first subject to high water levels. However, against this, heavier compaction is usually more costly than lighter compaction.

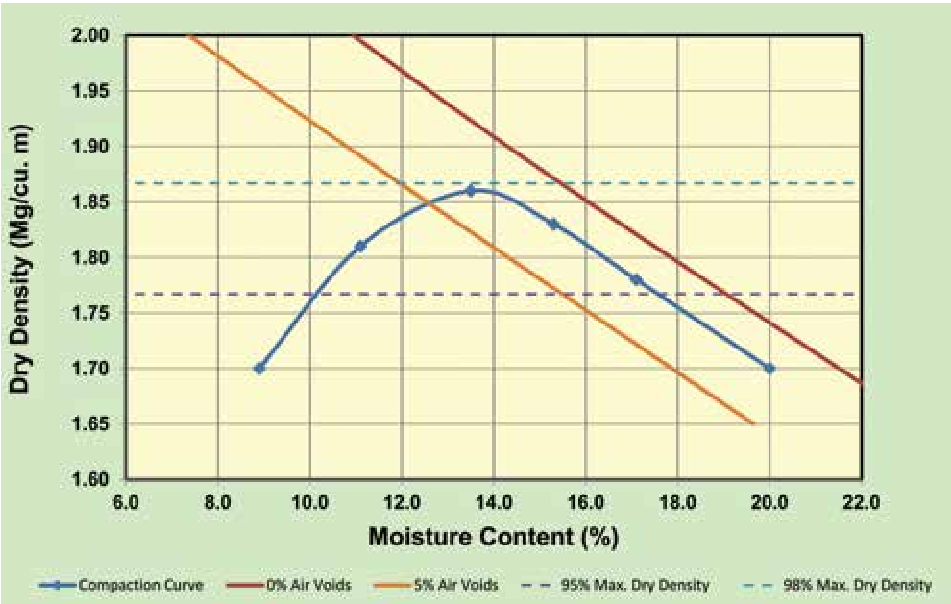


Figure 9.90 Compaction curve displaying relative compaction relationship

2 Moisture or water content

Acceptable bounds for moisture content variation are commonly specified as either ±3 per cent or ±2 per cent etc of the optimum moisture content (OMC).

Ideally, the moisture condition of the soil delivered from the borrow should already be close to the specified range. Where necessary to achieve this, any large exposed surface areas available in borrow areas should be used for processing and drying soil, prior to delivery to the levee site. Seasonal water content variations in the borrow pit should be measured or anticipated, and their likely impacts on construction methodology assessed. Methods for assessing moisture contents of fill materials in borrow areas are described in Section 7.9, and strategies for controlling the moisture contents of fill materials during construction are provided in Chapter 10.

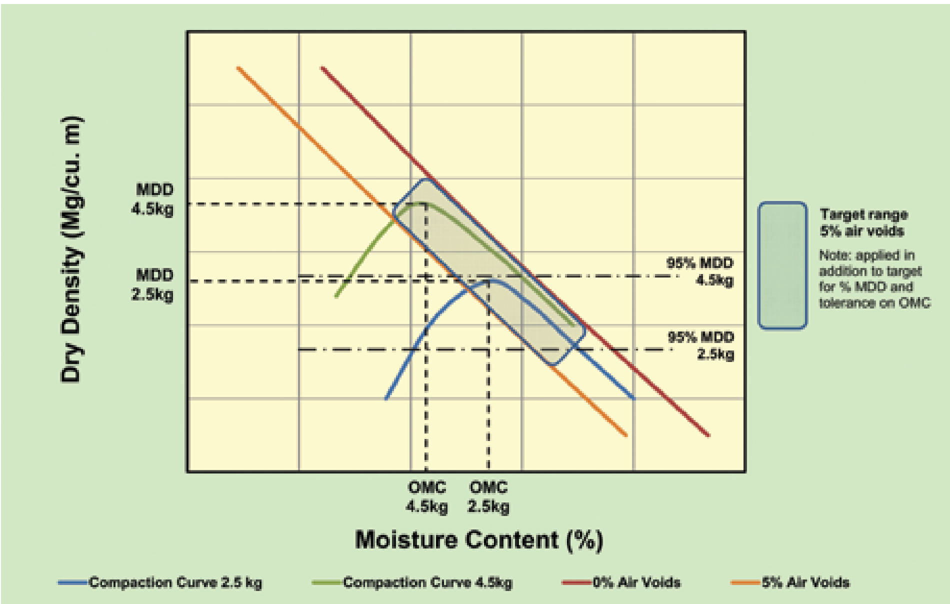
3 *Percentage air voids*

As explained in Section 9.12.1 excessive air voids left in levee soils can permit collapse settlement to occur when the soil is saturated. To mitigate this, the following approaches can be adopted:

1 For **clay soils**, the process of fill deposition and compaction should be controlled (Charles and Watt, 2001) to achieve an air voids ratio of less than five per cent. This should be sufficient to eliminate any large pores and voids between lumps of clay. For very stiff clays, it may not be possible to reduce air voids to below five per cent with normal compaction plant, and it may be necessary (Charles and Watt, 2001) to add some water during the fill deposition process. In this case, designers should check that the resulting undrained shear strength of the compacted material is sufficient for design purposes, as it may well not be the same as the original soil.

2 For **granular soils**, heavy compaction of granular fills during placement in layers should eliminate or greatly reduce subsequent vulnerability to collapse compression Charles and Watts (2001). Soils should be deposited slightly wet of optimum and, if found helpful during the trial compaction process, addition of a small quantity of water to the fill during placement may also be beneficial.

A specification based purely on a maximum allowable air voids ratio would not, in itself, be sufficient to adequately control the engineering characteristics of the compacted material (Charles and Watts, 2001). Figure 9.91 shows typical compaction curves and acceptance ranges (hatched grey) for a specification based on an air voids ratio of five per cent for both the standard Proctor test (2.5 kg) and the modified Proctor test (4.5 kg). Without restrictions on the percentage dry density and the percentage of optimum moisture content, the resulting compacted material could be too soft and could be potentially shrinkable. For this reason, when specifying a maximum air voids ratio, it is usual to also apply additional restrictions on moisture content (to be within a specified range of the optimum) and on dry density (to be within a defined tolerance of the optimum).



Notes

OMC (2.5 kg) = optimum moisture content obtained in the standard Proctor test.
OMC (4.5 kg) = optimum moisture content obtained in the modified Proctor test.
MDD (2.5 kg) = maximum dry density obtained in the standard Proctor test.
MDD (4.5 kg) = maximum dry density obtained in the modified Proctor test.

***Figure 9.91** Basis of design for engineering fill using five per cent air voids (after Trenter and Charles, 1996)*

9.13.2.2 Earthworks compaction regimes

There are three regimes commonly adopted for specification of compaction:

1. **Method specification:** this consists of density control through moisture conditioning of materials to a range of acceptable moisture contents, and compacting with an identified number of passes with a specified piece of equipment. This method is often favoured by constructors because it is predictable, is easily controlled and avoids the risk of delays due to the materials testing required for each layer. Generally, this method is more suited to situations where compaction control is of less importance, such as small levees or less important parts of large levees such as berms. Some standard method specifications are available (eg Highways Agency, 2009). Alternatively, site and material specific compactive efforts can be established on the basis of site trials.
2. **End-product specification:** this provides requirements for the density, moisture contents and voids ratio of the as-placed, as-compacted material. This is commonly done by defining an acceptable size of grading envelope, establishing a minimum acceptable relative compaction, establishing any controls on the percentage of air voids and setting the acceptable bounds for moisture content variation (as discussed in Section 9.13.2.1). Additional requirements, such as undrained shear strength, drained shear strength characteristics, unit weight and permeability (Section 9.13.2.3), may be specified if critical to the design, as values for these can vary within the specified tolerances on moisture content and density.
3. **Performance specification:** here the resulting performance of the levee is the basis of assessment. For example, the specification might require that the total settlement of the levee does not exceed a given depth at a set time interval after completion of construction.

Advantages and disadvantages of the method and end-product specifications are given in Table 9.25.

Table 9.25 Advantages and disadvantages of method vs end-product specification

Specification type	Advantages	Disadvantages

Method specification	• suitable where:
	  ◦ soils are generally homogeneous
	  ◦ soils can have a wide allowable variance in moisture condition and still comply with the performance objectives
	• can be validated through the use of a test fill or test levee section
	• easier to administer on site
	• reduces the risk of delays during construction (Section 10.4), which reduces the risk of softening or damage of compacted materials as a result of construction moving into a poor weather season
	• suitable for experienced constructors.
		• less control over end product
		• constructor may put less effort into controlling moisture condition of material, which affects resulting density and voids ratio of material.

End-product specification	• suitable where soils require tighter control of moisture content to achieve performance specification.	• risk of construction delays • more difficult to administer on site • more suited to inexperienced constructors.

The following process should be adopted for developing a compaction control regime.

1. **Identify performance objectives** including various levels of designed stability, seepage control and overtopping or erosion control.
2. **Decide on approach to specification methodology.**
3. **Set specification criteria** on the basis of data from laboratory tests from ground investigation.
4. **Finalise contractual specification criteria** following completion of field tests carried out at the early stage of construction works:
   i. preliminary testing results may be sufficient to guide the designer in preparing the final specifications, but may be conservative and costly.

ii a test fill or test levee section may be constructed to refine the specifications and to base criteria on actual performance of the materials when constructed by the contracted compaction plant. This is particularly important for levees on soft foundations where the unit weight of the as-constructed levee materials may be significantly greater than the minimum expected design values and can therefore potentially lead to unforeseen foundation failures or excess settlement.

5 **Monitor compaction during construction:** in the case of end-product specification, the work produced by the compaction plant should be checked by appropriate testing. Testing methods that can be used for such monitoring are given in Section 7.8 and Section 10.4.2. The basic required tests are those associated with classifying soil such as grain-size and Atterberg limits determinations, as well as moisture and density determinations for ensuring compliance with unit weight and strength requirements.

Where end-product specification is being used, a field laboratory may be necessary. Delays to construction can then be minimised by the used of rapid test methods for density and moisture content. For density determination, nuclear density meters are currently providing good correlations with standard density testing. For moisture content, there are a variety of possible methods, such as nuclear, microwave or hot-plate applications. Trials are advised to determine which method gives the best correlations with and standard (oven drying) methods and to understand accuracy limitations.

Where soil borrow materials change in classification significantly during production fill placement activities, field determinations for density control on a levee project can be difficult to accomplish accurately. Tools such as those described in Boxes 9.51 and 9.52 can assist in expediting the field density control through rapid field tests, although these particular techniques are more commonly used in the USA than in Europe.

Box 9.51 Use of one-point compaction for rapid field relative compaction determination

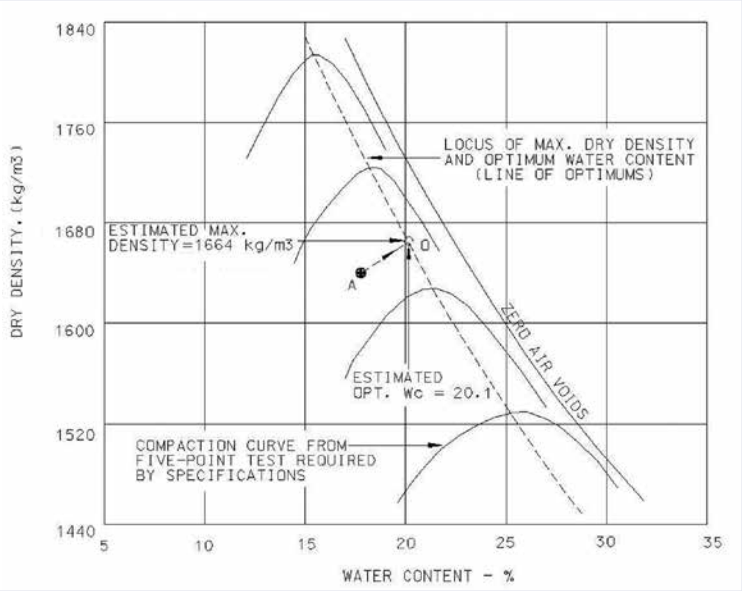


Figure 9.92 One-point compaction procedures (courtesy Mike Wielputz, USACE)

A number of compaction tests are carried out on samples of material to be tested, using the chosen compactive effort. A family of curves is then generated, which reflects the variability of the soil to be compacted (rather than the method of compaction). With a family of curves established, a locus of maximum dry density (MDD) and optimum moisture contents (OMC) can be established. Individual points established for individual samples (on the dry side of optimum) can then be extrapolated, as shown in Figure 9.92, to establish MDD and OMC for that particular sample with a reasonable degree of confidence.

Notes

1 Point A is the result of a one-point compaction test on material from density test. This point must be on the dry side of optimum water content.

2 Point O is the estimated optimum water content and maximum dry density of the fill material based on a projection of point A approximately parallel to the adjacent compaction curves.

3 Point A must plot within three per cent of the line of optimums.

Box 9.52 Use of two-point compaction for rapid field relative compaction determination

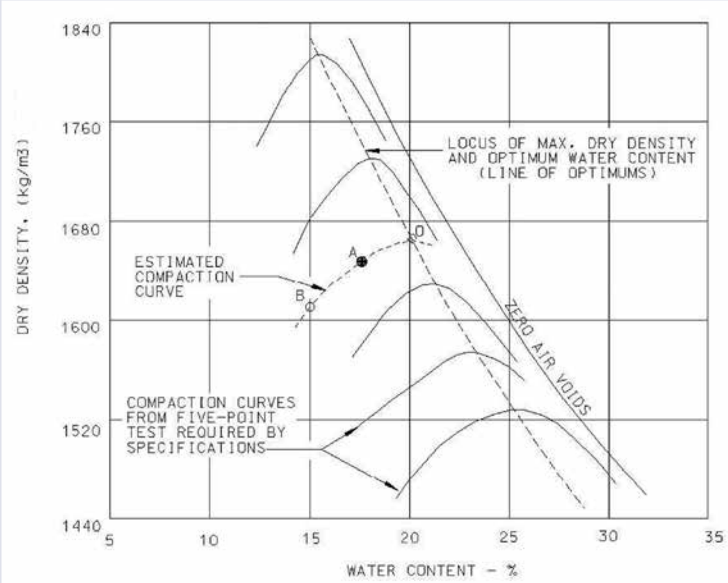


Figure 9.93 Two-point compaction procedure (courtesy Mike Wielputz, USACE)

A number of compaction tests are carried out on samples of material to be tested, using the chosen compactive effort. A family of curves is generated, which reflects the variability of the soil to be compacted (rather than the method of compaction). With a family of curves established, a locus of maximum dry density (MDD) and optimum moisture contents (OMC) can be established. Individual points established for individual samples (on the dry side of optimum) can then be extrapolated, as shown in Figure 9.93, to establish MDD and OMC for that particular sample with a reasonable degree of confidence.

Note

1. Points A and B are results of a two-point compaction test on the material from field density test. Points A and B must be on the dry side of optimum water content.
2. The estimated compaction curve based on Points A and B establishes point O on the locus, which is the estimated MDD and OMC of the fill material.
3. One point must plot within three per cent of the line of optimums.

9.13.2.3 Other controls on earthworks materials

Other typical controls applied to earthworks materials include:

1 *Exclusion of unsuitable materials*

Unsuitable materials which might have a deleterious effect on the performance of the earthworks should normally be excluded by specifications (eg Highways Agency, 2009), including:

- peat, materials from swamps, marshes and bogs. In the USA, this is defined as soil having an organic content in excess of four per cent by weight (although in some cases where decomposed marsh grasses are present, organic contents of up to nine per cent have been accepted, as long as it can be proven that this will not have an adverse effect on the performance of the clay)
- wood and debris such as logs, stumps, roots and perishable material (including refuse), metal, rubber, plastic or synthetic material, such material may either be excluded, or limits may be specified, such as a maximum of one per cent by volume
- materials in a frozen condition
- clay having a liquid limit exceeding 90 per cent or plasticity index exceeding 65 per cent (this limit may vary depending upon available sources of material)
- materials susceptible to spontaneous combustion (such as coal)
- swelling soils or collapsible soils
- materials contaminated with hazardous chemicals or containing natural chemical compounds which will have a deleterious effect on the permanent works, or material having hazardous

chemical or physical properties requiring special measures for its excavation, handling, storing, transportation, deposition and disposal.

For water-retaining structures such as levees, it may also be necessary to add further restrictions to disqualify the use of materials that may deteriorate over time in the presence of water. Such restrictions include the two following examples:

1. The salt content (NaCl) in the pore fluid should not exceed 4 g/l soil moisture.
2. The content of low or medium density chalk should not exceed 25 per cent by volume of the deposited fill material.

2 Controls on material grading

Simple limits on the gradings of the earthworks materials used in levee construction or adaptation works should normally be imposed, adopting standard specifications wherever possible to avoid excessive costs. Particle size distributions for materials to be used for levee construction should be consistent with the materials requirements set out in Section 9.13.3.

Many natural clay soils are not homogeneous and may contain sand layers or lenses (Figure 9.94) or may become segregated during deposition. If unregulated, the sand content of short sections of levee could be more permeable than surrounding areas and this could lead to seepage and a risk of internal erosion. Sand lenses may not necessarily be picked up during conventional borings, as the sand may be spread uniformly through the test sample, but properly supervised borrow pit investigations including trial pits should be able to identify this risk.

Figure 9.94 Sample of clay with sand lenses (courtesy Mike Wielputz, USACE)

3 Controls on undrained and drained shear strength

Control of the undrained and drained shear strength (Section 9.13.1.3) of placed levee materials and related dry densities moisture contents and air voids is achieved by controlling both the materials used and the method of placing and compaction.

4 Controls on permeability characteristics

Requirements for permeability are set out in Section 9.13.1.2 and, if critical, should be controlled by appropriate testing.

9.13.3 Earthworks for levee raising and repair

Use of earthwork materials to raise or repair existing levees can create a number of potential issues that need to be considered in design. These are examined in this section, along with suggested methods to deal with these issues. An example of a project including a number of these measures is given in Box 9.53.

9.13.3.1 Geometry

Raising the top level of the levee may not be as simple as adding material to the levee's crest, particularly if the existing levee has a crest of minimum width. In this case the usual approach is to add fill material to both the crest and the landward slope (Figure 9.95), but a good connection between the existing levee and the new fill material is necessary to ensure that:

- the new fill does not slide down the existing landward slope
- the interface between the old and the new fill does not create a path for seepage.

It is therefore usual to remove the existing topsoil and vegetation from the crest and the landward slope and then to create a series of steps (Figure 9.96) on which the new fill is deposited and compacted. It may be necessary to check that the grading of the new fill material is compatible with the old materials; if necessary, a filter layer should be provided between the two.

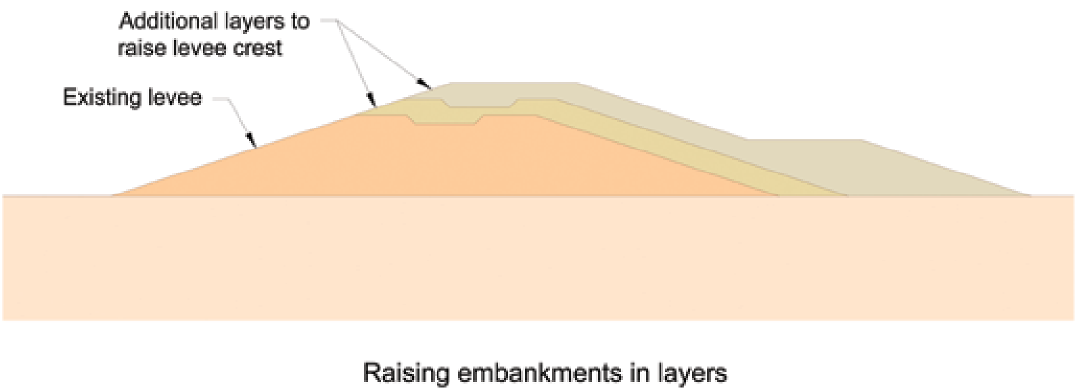


Figure 9.95 Use of earthworks material to raise levee crest

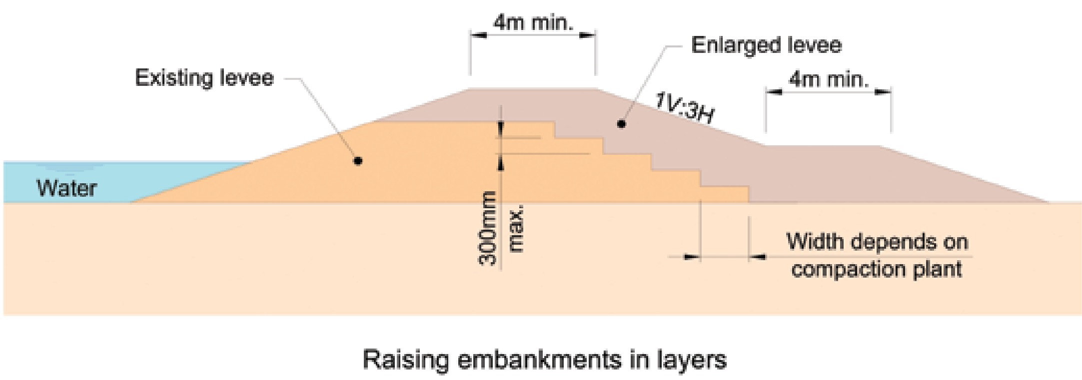


Figure 9.96 Interface between old and new fill materials for levee raising

9.13.3.2 Disturbance of existing levee

When considering removing vegetation and topsoil, the designer should be aware that some older levees (particularly rural levees) may be constructed almost entirely out of topsoil. In this case, a limit should be applied to the extent of any excavation, and the interaction assessed between the old levee (typically containing poorly compacted soils with some voiding) and the new levee (containing dense, well compacted, impermeable soil).

If necessary, a cut-off should be created through the crest of the existing levee to reduce the likelihood of seepage (Figures 9.97 and 9.98). Alternatively, a drainage layer should be provided between the old levee and the new material to prevent seepage through the old section of the levee causing uplift and pushing the new earthworks off face of the old levee (Figure 9.99).

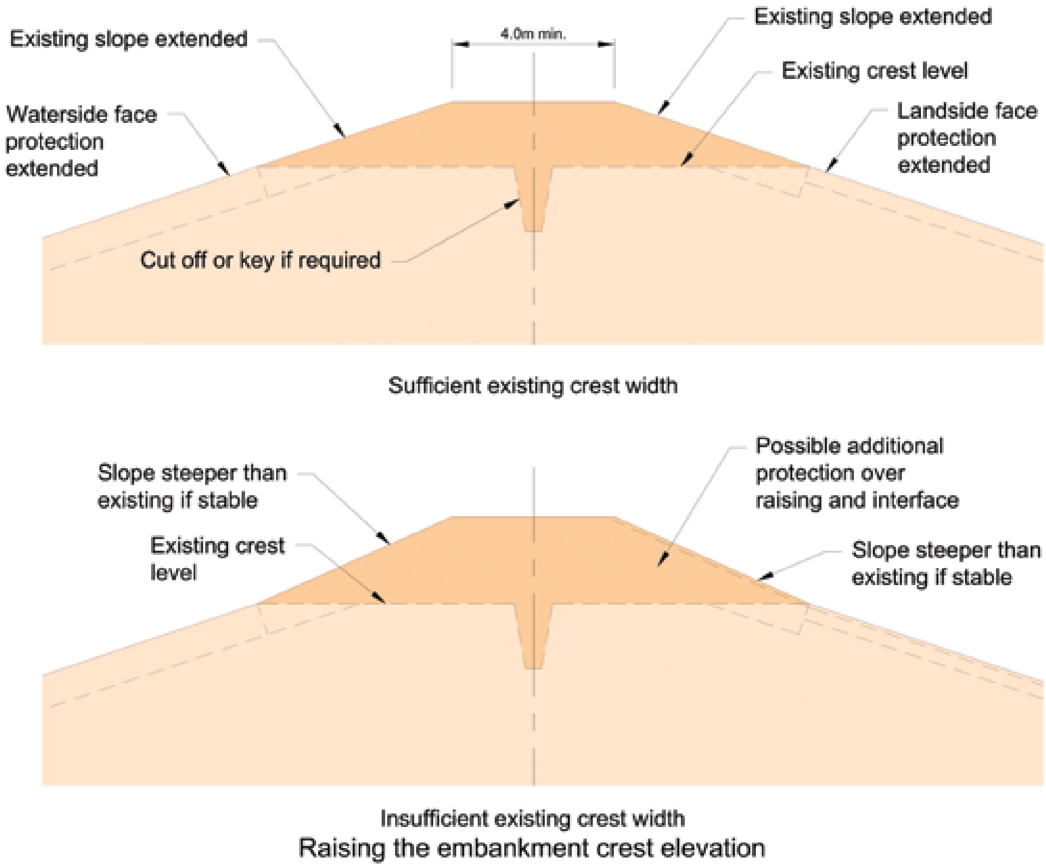


Figure 9.97 Use of cut-off or key to avoid creating a seepage path during levee raising

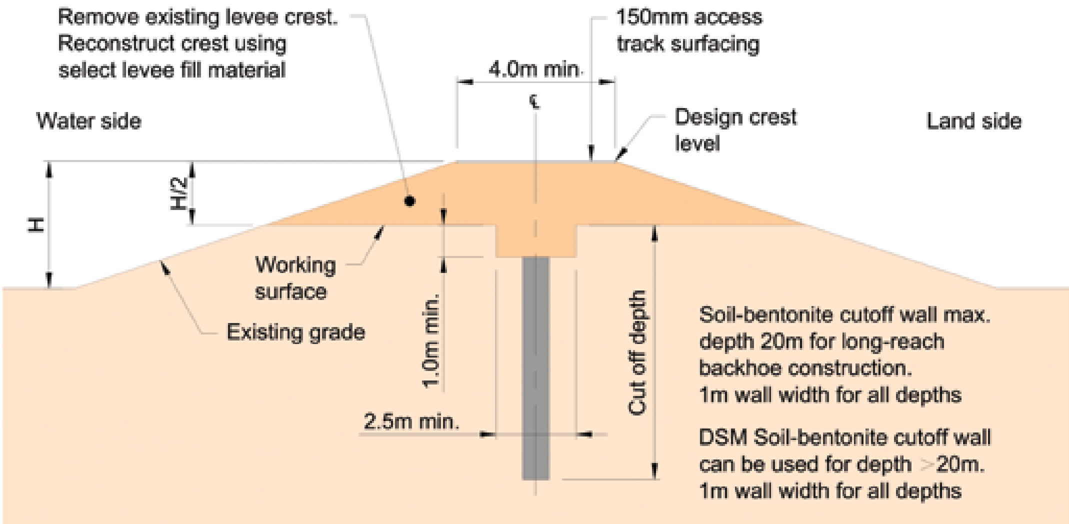


Figure 9.98 Use of cut-off wall to avoid creating a seepage path during levee raising

(For information on soil–bentonite walls to control permeability, see Box 9.50 in Section 9.13.1.2.)

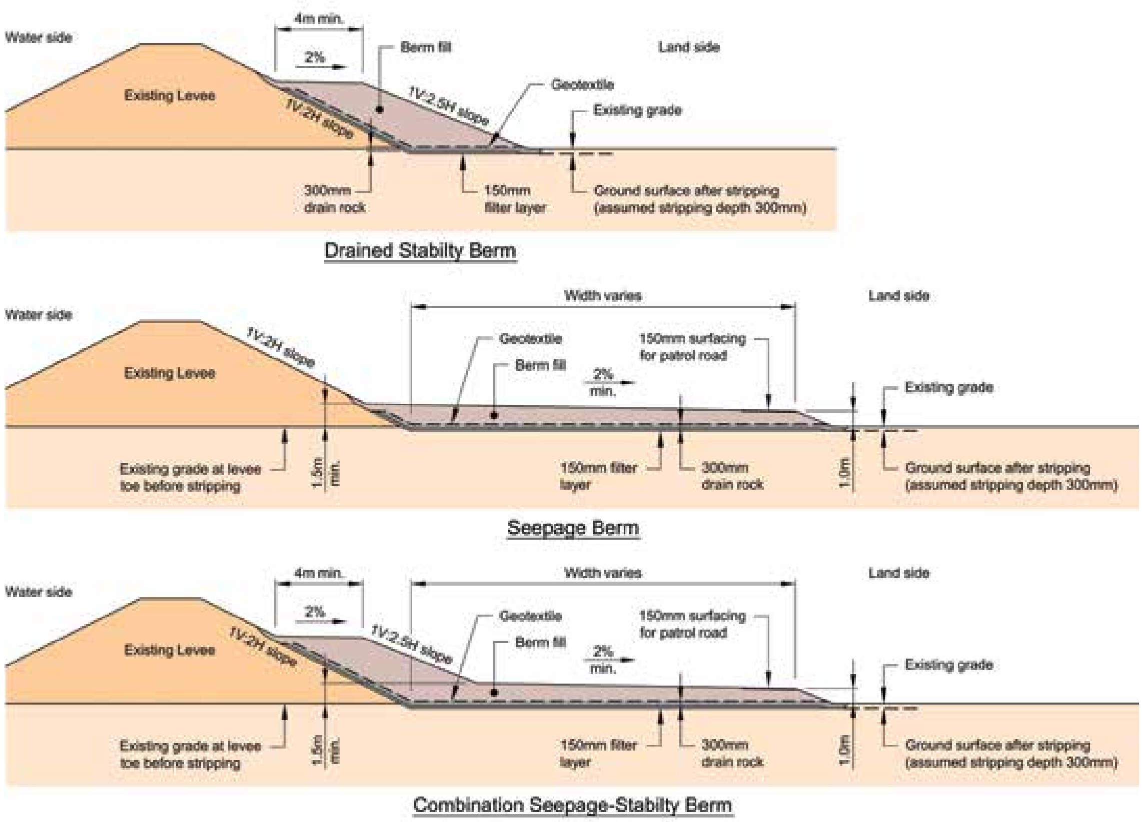


Figure 9.99 Control of seepage beneath new earthworks on landward side of levees (courtesy Mary Perlea, USACE)

9.13.3.3 Stability

The additional weight of the fill used for levee raising may have a destabilising effect on the existing levee. The effect of the additional earthworks material on the existing levee should be considered for all plausible loading conditions during its life, following the guidance in Section 9.9. In particular, it should be noted that the greatest thickness of fill material placed is often to the landward side of the original crest and this is the location where short-term undrained failure mechanisms may initiate.

9.13.3.4 Settlement

Settlement associated with levee raising is most likely to occur to the landward side of the new crest as this is where the greatest thickness of new fill material is placed. The magnitude of the settlement caused by the levee raising should be assessed and any anticipated differential settlement controlled by appropriate measures (Section 9.12.1). Commonly used approaches include careful control of the landward slope or the use of geofabrics (Section 9.13.4).

9.13.3.5 Use of crest structures and compaction of new fill material

Where crest structures are used to support new levee fill, the main issues are stability, impermeability and settlement. More specific issues include:

- construction of interfaces between the old and the new structures
- compaction of fill materials up against the new crest structures, as compaction may increase the lateral loads on the wall and also affect the wall's performance
- seepage and internal erosion, for example, are filter layers required between the fill material and the wall?

Section 9.14 gives more general information relating to crest structure design.

Typical examples of the use of crest structures to support new levee fill are given in Figure 9.100.

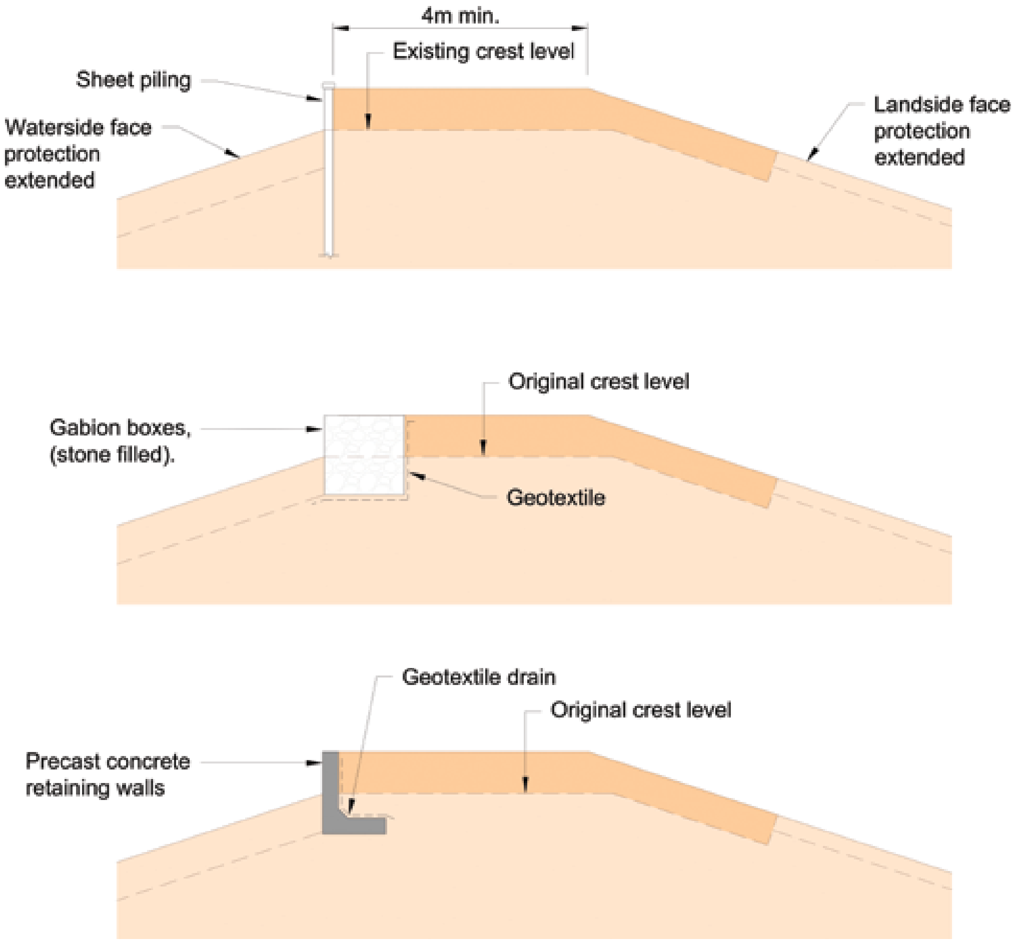


Figure 9.100 Use of crest structures to facilitate crest level raising

9.13.3.6 Resilience to external erosion and seepage

Following any works on or around a levee, the process of construction may leave the levee surface bare, and vulnerable to external erosion. Designers should consider this vulnerability and take steps to provide a suitable level of resilience in the short term (Section 9.6). Where it is desired to provide grass cover in the long term, suitable measures may include the use of turf impregnated geotextiles.

Box 9.53 *Levee improvement works, near Arles, France*

Figure 9.101 shows a section through the improvement works. A cut-off sheet pile wall was installed to reduce seepage, surface protection measures were incorporated into the structure and anti-burrowing meshing was installed. In some areas, the levee was located close to a road and so a vertical wall had to be installed on the landward side of the levee as the available space was limited.

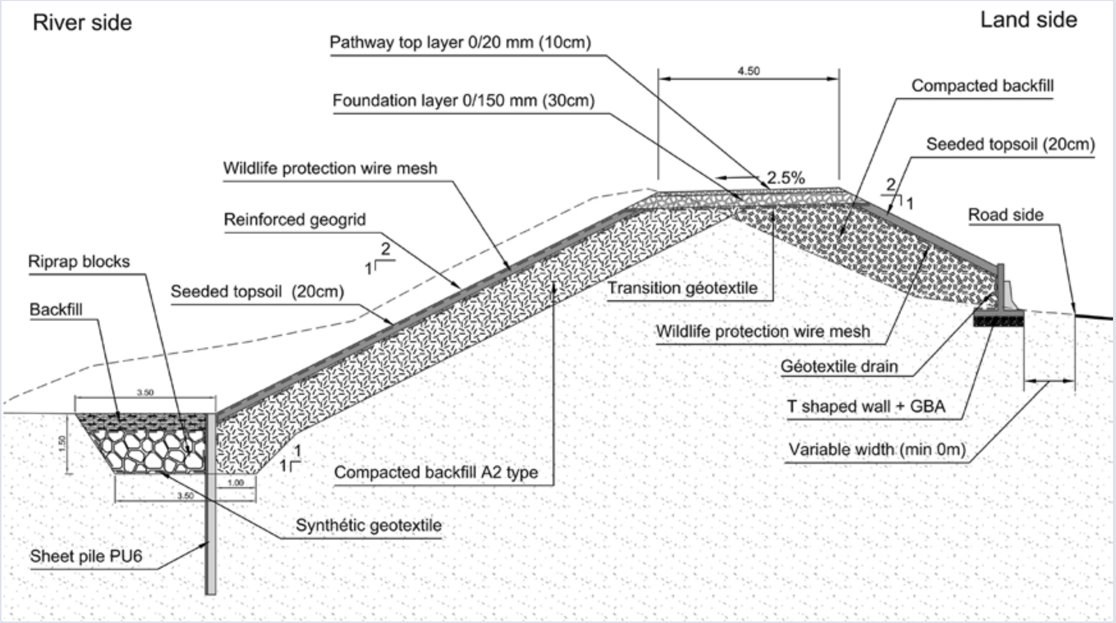


Figure 9.101 *Levee improvement works adjacent to a road, at Arles, France (courtesy SYMADREM and EGISeau)*

9.13.4 Use of geofabrics for levee construction

With the development of a growing range of tried and tested man-made materials, geofabrics are being used, with increasing frequency, to enhance various aspect of levee performance. Common examples include:

- woven and non-woven filter fabrics to form stable, erosion-resistant transitions between different material types and prevent suffusion
- erosion-resistant surface layers that can promote the growth of grass and can control scour of the crest, the landward slope and the toe of the levee, arising from overtopping and overflow
- geosynthetic clay liners (bentonite mats) used predominantly as a seal on the waterside of the levee to inhibit seepage through the levee
- geogrids (made out of materials such as PVC coated polyester yarns) to reinforce the base or the landward face of the levee.

Where geofabrics are used as filter layers, design calculations may use the tools provided in Section 8.5.5.2. Design issues that need to be considered include:

- material variability
- installation damage
- clogging or other deterioration of the filter, over time, leading to elevated pore pressures within the body of the levee and/or uplift of the geofabric
- damage to the integrity of the filter system, caused by burrowing animals or the growth of trees or shrubs.

Figures 9.102, 9.103 and 9.104 illustrate how geofabrics and geogrids can be used (Heerten and Werth, 2012) to enhance both resistance to external erosion and stability of levee. With regard to stability, they can:

- significantly reduce the potential for mass instability (Box 9.54)
- provide an enhanced resistance to seismic loading in earthquake-prone areas.

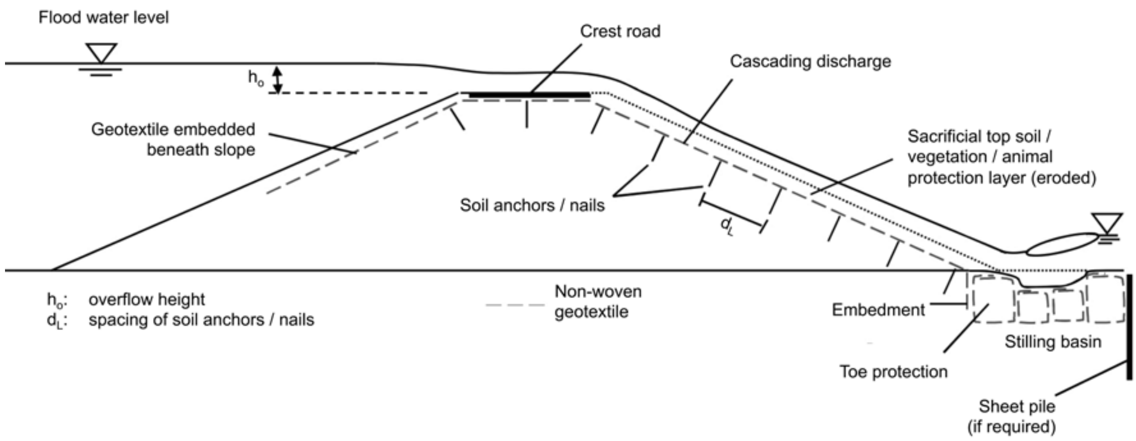


Figure 9.102 Crest overflow protection using geogrid/nonwoven combinations and ground pins (after Haselsteiner et al, 2007)

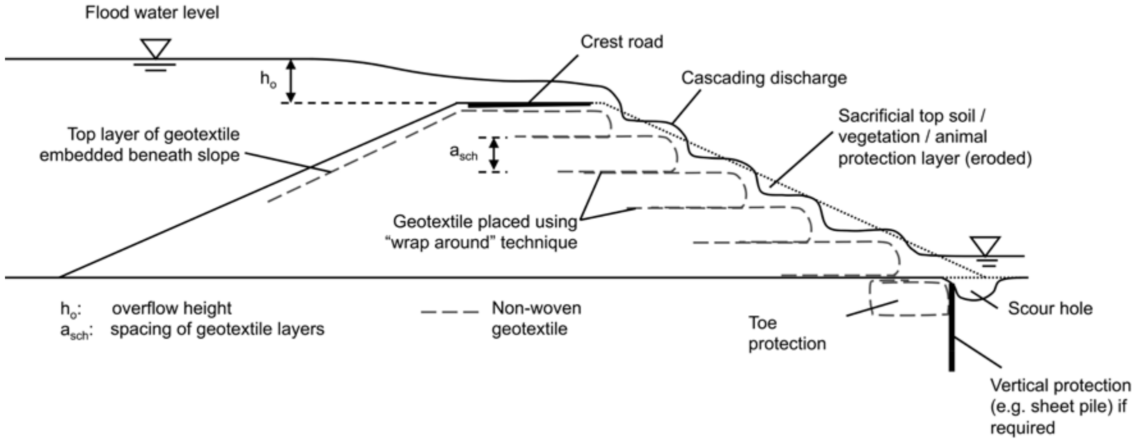


Figure 9.103 Integrated crest overflow protection using the envelope method (after Haselsteiner et al, 2007)

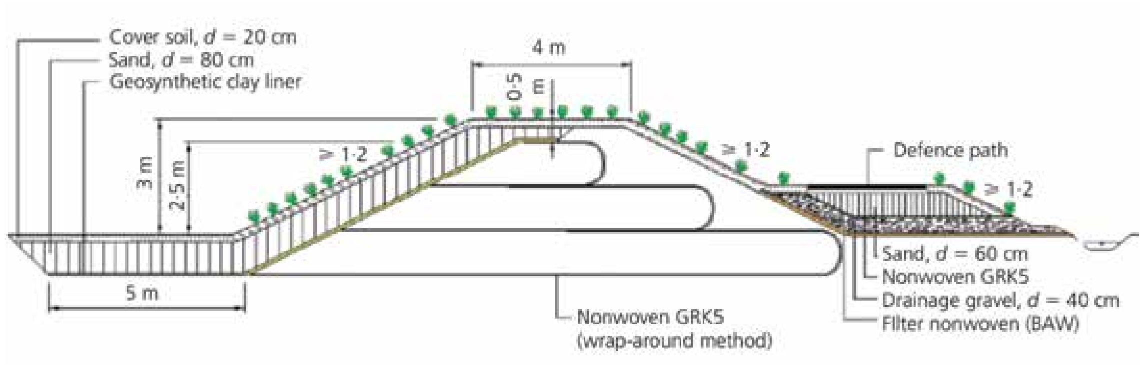


Figure 9.104 Cross-section of a levee after rehabilitation on the Oder River, Poland (from Heerten, 1999)

However, it is important to recognise that geofabrics may introduce flaws which might affect levee performance. For example:

- geogrids used for levee reinforcement may introduce a seepage path through or beneath the levee
- geotextile filters installed beneath the landward face of a levee might clog and then be lifted upwards by pore pressures acting within the levee during a flood.

Box 9.54 ***Use of geosynthetics to improve levee stability, New Orleans, LA, USA***

Figure 9.105 ***High strength geotextile used at the base of the New Orleans levee (courtesy Mike Wielputz, USACE)***

Geotextiles are used at the base of hurricane risk reduction levees in the New Orleans area to provide additional shear resistance in places where real estate constraints do not allow construction of wide stability berms. The geotextile provides the resistance needed to achieve the required factor of safety for global stability. In Figure 9.105, a high-strength geotextile has been placed and stretched at the base of a levee which is being enlarged. The old levee, a part of which has been excavated in time will become part of the landside berm of the new levee. The geotextile is being placed on the excavated surface of the old levee. Both waterside and landward side berm lengths are reduced by introduction of this reinforcement.

Bentonite mats (geosynthetic clay liners, GCLs) can be incorporated into a levee (Figure 9.106) that is built out of higher permeability fill than would normally be adopted for levee construction, in order to reduce seepage and internal erosion. GCLs have been used for many years as environmental liners for waste disposal dumps, contaminated spoil pits and such like (Heerten and Werth, 2012) and offer the advantages of:

- low permeability
- self-healing behaviour
- good durability
- good friction behaviour for embankment slopes
- good control of quality of a manufactured product.

However, the performance of GCLs can be affected adversely by root penetration and damage caused by burrowing animals. These impacts can be counteracted by:

- the design of the levee's cross-sectional geometry
- the use of non-cohesive cover layers unattractive to burrowing animals (Section 9.12.3.2)
- additional engineering measures.

An 800 mm cover layer is recommended over GCLs, in order to withstand climatic influences (DWA, 2005). Further information about planning and building with geosynthetic clay liners can be found in the literature (eg BAW 2006, DGGT 2002, Heerten, 2007, and Saathoff and Werth, 2003).

Figure 9.106 ***Geosynthetic clay liner being incorporated into a sand levee in Elgin, Scotland (courtesy Mark Donoghue, Royal HaskoningDHV and Moray Flood Alleviation)***

Some aspects of the performance of geofabrics are known to deteriorate with time. For geogrids, strength reduction factors are commonly considered for:

- material variability
- installation damage
- creep-related strength reduction
- corrosion of metallic reinforcement
- strength reduction due to temperature.

9.13.5 Innovative ground improvement methods

9.13.5.1 Tyre shred and bales

Lightweight tyre fill (in whichever grade) can replace other lightweight materials that can be used for reducing the weight of levees built on soft ground, such as lightweight expanded clay aggregate, lightweight concrete, pulverised fuel ash (PFA) and expanded polystyrene blocks. Lightweight fills from tyres can be engineered using tyre bales, whole tyres, tyre shreds (50 mm to 300 mm) and chips (10 mm to 50 mm). The choice of which type, will depend on the relative costs of treatment, transport, and locally sourced materials as well as the site or structure-specific requirements.

Tyre shreds and tyre bales have also been used in embankments and levees to increase shear strength against instability.

In the USA and the UK, standards have been developed for the manufacture and use of tyre shred (ASTM, 1998, PAS 108:2007 and PAS 107:2012) in civil engineering applications. The specific advantages for **tyre shred** used as levee fill are identified as:

- reduced settlement (after initial loading)
- increased stability due to low density
- improvements in strength and reduced deformation when mixed with moderately plastic clay soils
- improved angle of friction when mixed with silty plastic clay
- a cohesion intercept for tyre shreds of 8 kPa to 11 kPa
- high compressibility on initial loading, but increased stiffness and reduced compressibility on subsequent unloading and reloading.

In the UK, a standard has been developed for the manufacture and use of tyre bales (PAS 108:2007) in civil engineering applications. The inert, durable, free draining, low density and high shear strength properties of tyre bales have proved particularly attractive (HR Wallingford, 2005). The use of tyre bales in a levee on the River Witham near Lincoln, UK, is described in Box 9.55. Reductions in the overall ground pressures exerted by a levee partly formed of tyre bales will reduce both the risk of slope or foundation failure, and the magnitude of consequential settlement and hence tyre shred and tyre bales can be used to improve mass stability, so long as the reduced pressures do not increase the risk of uplift (Sections 9.9. and 9.10). As tyre bales are highly permeable, it is also important to recognise the need to provide adequate design in other parts of the levee for control of seepage and internal erosion (Section 9.8).

Tyres can also be used individually to reinforce levees (Sayao *et al*, 2002), but in Europe their use in this way is limited because of waste control restrictions on the disposal of whole tyres.

Box 9.55 Use of tyre bales for levee construction in the UK (HR Wallingford, 2005)

Figure 9.107 Use of tyre bales for levee rehabilitation (courtesy Environment Agency)

Figure 9.108 Tyre bale placement completed on one levee segment (courtesy Environment Agency)

The Environment Agency in the UK identified the need to remediate and raise some of the flood embankments in Lincolnshire. The embankments lining the River Witham were built on a peat foundation, and this created problems of stability and settlement. Using traditional earthen materials, the side slopes would have had to be 1:4 to maintain stability, and the consequential widening of the embankment's footprint meant that the base of the embankment would encroach on existing 11 kV powerlines and a drainage ditch.

The Environment Agency together with Bullen Consultants and May Gurney Construction devised a scheme where tyre bales were used to reduce the embankment's footprint by steepening the side slopes. This had the effect of reducing costs and saving space. The plan involved stabilisation of the levee by widening the crest to 4 m, re-profiling the embankment, berm reinstatement and toe protection (Figures 9.107 and 9.108). It is estimated that, when complete, the scheme will have used over a million tyres. As the scheme was a pilot project, the Environment Agency set up a water monitoring programme involving regular surface water quality monitoring, which showed that tyre-derived leachate from the scheme was minimal.

9.13.5.2 Deep soil mixing

One of the consequences of climate change is the perceived need to raise levees as a result of the anticipated rise in sea levels and storm surge levels. The historic construction of houses on or close to the landward toe of levees in the Netherlands over the centuries has complicated this process. If structures have been built close to the landward toe of a levee, then the dike cannot be raised by extending the toe further landwards.

A number of research projects have been carried out in the Netherlands to investigate innovative methods for crest raising. One of the methods investigated was the use of deep cement mixing (de Kant and Wiggers, 2009). In this technique, deep soil mixing is used to strengthen the levee's foundation. The strengthening is carried out using rotary augers (Figure 9.109) which create inclined soil/cement columns through the landward slope of the levee and the soft deltaic soils before terminating in the underlying dense Pleistocene sand (Figure 9.110). In plan view, the blocks are created at regular intervals with non-stabilised soil in between. The spacing of the soil–cement blocks is a critical design issue and is determined by limit equilibrium slope stability calculations and finite element modelling.

Figure 9.109 Deep soil mixing to facilitate levee raising (after de Kant and Wiggers, 2009, courtesy Keller Ltd and Royal HaskoningDHV)

In the Netherlands, where the soft soils have a high organic content, it was found that water-injection during mixing, and the use of a relatively high percentage of blast furnace cement binder (200 kg/m^3 to 300 kg/m^3) were important factors in enabling the treated ground to reach to required strengths.

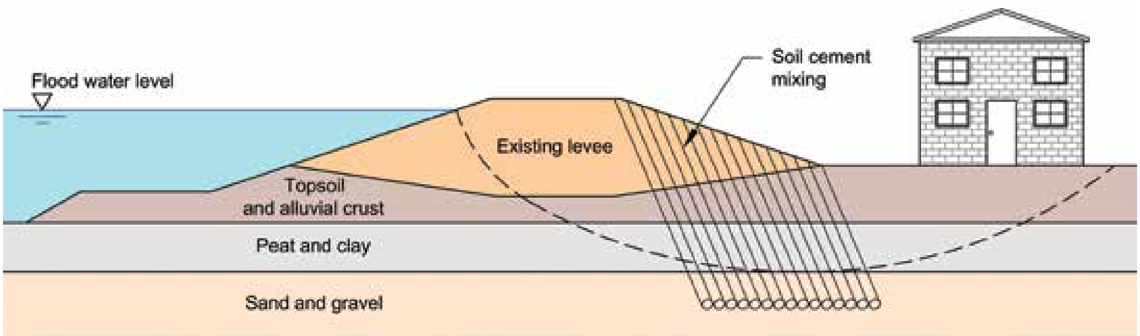


Figure 9.110 Deep soil mixing to stabilise levees (after de Kant and Wiggers, 2009, courtesy Royal HaskoningDHV)

One benefit of using cement-stabilised columns can be to reduce the flow of water beneath the levee. In this case, the columns need to be contiguous rather than formed into blocks (Figure 9.111).

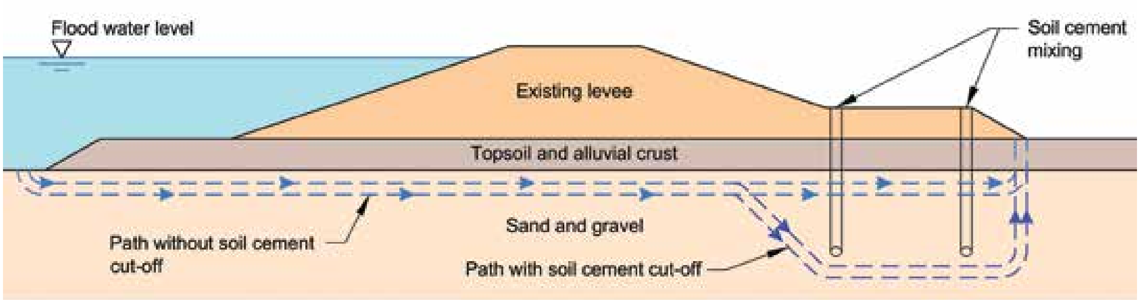


Figure 9.111 Deep soil mixing to control seepage (after de Kant and Wiggers, 2009, courtesy Royal HaskoningDHV)

9.14 SPILLWAYS

9.14.1 Introduction and background

The key principles for design of a spillway are as follows:

1 **Capacity:** whether the spillway will release sufficient floodwater out of the river to fulfil its primary function of reducing river water levels upstream and/or downstream.

2 **Resilience:** whether the spillway structure will be able to carry the design flow for the anticipated duration without significant deterioration or structural failure. Spillways will operate infrequently and therefore their structure and performance should be robust given the erosive power of overflowing water and that malfunction may lead to serious and unpredicted flooding elsewhere.

In addition, it is vital to understand the likely destination for the water which passes over the spillway and whether the water can be contained and managed safely in that location.

These considerations should inform the choice of spillway location, as discussed earlier (Section 9.5.1.4). They should also inform the hydraulic design (Section 9.14.3) and general issues of civil engineering design (Section 9.14.4).

Section 9.5.1.4 explained that spillways on riverine levees are of two possible functional types (Degoutte *et al*, 2012):

1 **Security spillways** protect neighbouring levees from damage and breach by concentrating and controlling overflow during an event that exceeds the return period water level for which the rest of the levee has been designed. As explained in Section 9.5.1.4, by being designed to resist overflow erosion, these spillways allow controlled discharge of water into areas of relatively low risk. The overflow at spillway locations reduces river water levels and thereby reduces the chance that neighbouring levee sections will overflow. Security spillways would only be expected to function rarely. In extreme cases, security spillways can be operated by demolishing discrete lengths (or 'fuses') of a levee, for example by excavation or controlled explosions.

2 **Bypass spillways** are normally designed as part of a flood control system and control water levels within a river system by diverting part of a high flow into either an alternative channel or a safe area of temporary storage. These structures generally operate more frequently than security spillways, and work under reasonably well-controlled situations. An example of a bypass spillway is shown in Box 9.56.

While the roles played by the above two types of spillway are different, the basic mode of hydraulic operation is the same.

Sections 9.14.5 to 9.14.8 discuss the most important types of spillways in more detail.

Box 9.56 *An example of a bypass spillway on the Rhône at Vallabrègues (Degoutte et al, 2012)*

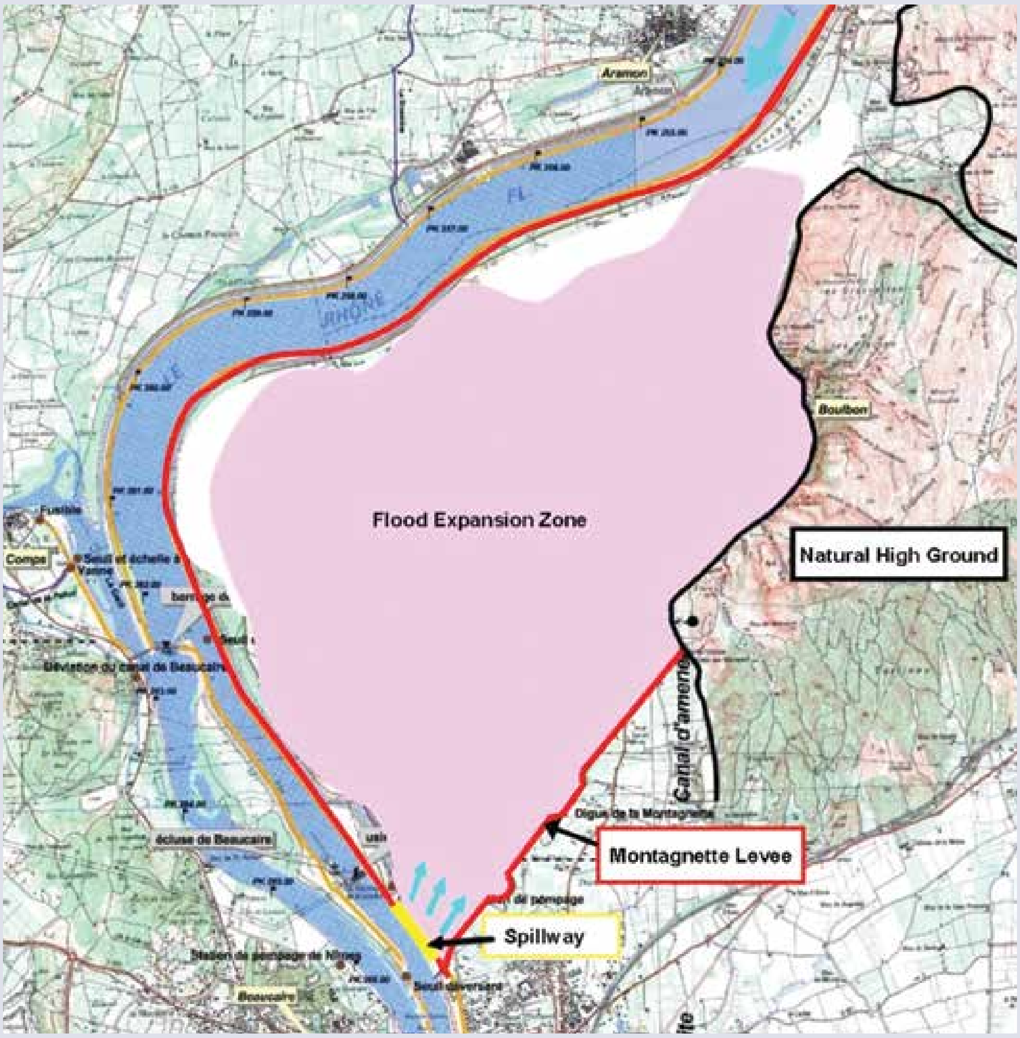


Figure 9.112 *Use of a spillway on the left bank of the Rhône at Vallabrègues (Degoutte et al, 2012)*

Figure 9.112 shows the location of a spillway on the left bank of the Rhône at Vallabrègues. It was constructed in 1969 and protects the town of Tarascon. The spillway discharges into a flood expansion zone which is bounded by levees and natural high ground and consists mainly of agricultural land. The spillway itself is a calibrated weir, 500 m long and with a crest level of +10.45 m. It starts to overflow when the rate of flow of the river approaches 8500 m^3/s (1 in 10 year events). When the flow rate reaches 10 500 m^3/s (1 in 50 per year event), the water level in the flood storage area reaches +11.7 m (coloured pink). In the 1 in 1000 per year event (14 160 m^3/s) the water level in the storage area reaches +13.2 m (pink and white).

Source CNR Symadrem, background map provided by IGN

9.14.2 Hydraulic design of spillways

Hydraulic design of spillways comprises two interacting components:

- assessment of the impact of the spillway on the flood hydrograph
- calculation of the flow behaviour at the spillway itself.

The design process is an iterative one between these two processes and may involve various kinds of computational models and even physical models for final optimisation.

For the assessment of the *impact of the spillway on the flood hydrograph*, hydraulic models developed during site characterisation (Chapter 7) are expanded to assess the effect of spillway/fuse plug components of a levee system on the magnitude and timing of discharges (flow hydrographs). Figure 9.113 shows this as the 'hydrograph with spillway'. The capacity of the spillway/fuse plug section is determined by the depth of water above the overflow section crest, the length of section that overflows and the length of time for which overflow occurs. Water diverted by overflow reduces the discharge rates in the main conveyance system. The effects of an adequately designed overflow section prevents the riverward water level from exceeding top of levee elevations along other parts of the levee. However, water levels on the landward side of the overflow section experience increased water levels as depicted in the lower stage hydrographs in Figure 9.113. Blue shaded segments of hydrographs in Figure 9.113 are diverted by overflow at the spillway. The yellow segment represents where diverted water returns to the river. Depicted spillway hydrographs would be obtained if there were no limit on the spillway, but in some cases may be controlled by rising water levels on the landside where there is limited storage capacity.

For the assessment of the overflow behaviour at the spillway/fuse plug itself, the weir equations presented in Section 8.2.2 provide a reasonable estimate for overflow discharge when the levee embankment or spillway configuration is in the form of a weir. Determining the crest level and the length of the spillway then becomes an iterative process related to other aspects of the design.

However, there are some other cases that need consideration:

1 Spillways may be fitted with various types of gates, to control and regulate the flow into the spillway outlet. The discharge characteristics of the gates and associated structures, and their operation, will determine the amount of water passing over the levee.

2 Fuse plugs are segments of a levee embankment designed to a lower crest elevation to permit overflow. In some cases, fuse plugs incorporate provisions for erosion and eventual breach of the embankment, which may be assisted by explosives (Chapter 6). Analysis of fuse plug overflow sections is complex and involves assessment of the rate of breach development using the tools discussed in Section 8.10.

Spillway/fuse plug detailing for levee systems will typically involve an iterative process to achieve a balance between spillway performance and required spillway structural requirements with respect to unit discharges, frequency of use and resulting erosive forces.

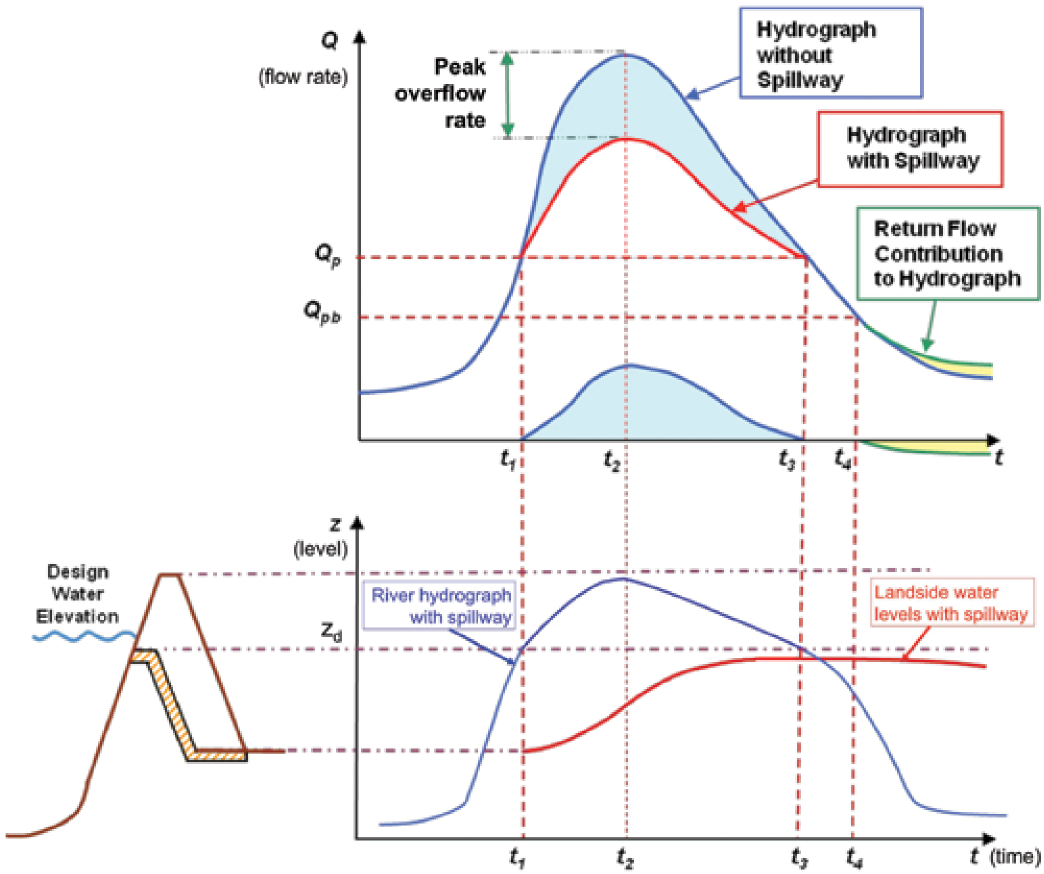


Figure 9.113 Effect of spillway/fuse plug on flood hydrograph (after Degoutte et al, 2012)

With intended overflow sections and with overtopping of embankments where wave activity on the waterside slope is limited (such as those associated with small lakes or river flood defences), **threshold discharge conditions** and design discharge are usually related to events with a defined probability of occurrence (or risk). However, where embankments are subject to substantial wave activity on the waterside slope (lakes, estuaries or large river systems with considerable wind fetch), overflow conditions are likely to be caused by a combination of extreme water level and wave action. In such cases, overflow discharge will fluctuate, and the value of peak design discharge for protection measures is a matter of engineering judgement. Owing to the random nature of wind-generated waves, the local peak discharge intensity when a particular section of the embankment is overtopped by a large wave could be between one and two orders of magnitude larger than the time-averaged mean discharge intensity (see tools in Sections 8.2.1 and 8.2.2).

For a **bypass spillway**, the maximum flow that can be allowed to stay in the river system compared with the anticipated flood hydrographs, determines the volumes of water and the rates of flow that need to be discharged. Hence, as discussed above, the crest level and the length of the bypass spillway may be determined, ideally identifying a minimum length of weir that can efficiently fill the flood expansion zone at an appropriate rate (Degoutte *et al*, 2012).

For a **security spillway:**

- the **crest level** of a security spillway can be set to match that of adjacent levee segments. However, a safety margin of 10 cm to 20 cm can be applied to give confidence that the spillway will overtop before the unprotected levees (Degoutte *et al*, 2012). This is particularly important if the levees either side of the spillway may be subject to ongoing settlement
- **energy dissipation** of any water crossing the spillway must be provided by appropriate spillway toe details and the flow will need to be channelled away from the area. Some form of stilling basin to promote a hydraulic jump so that the downstream water level is at least 0.5 m high is suggested by Degoutte *et al* (2012). Note that in Figure 9.114, the difference in height between the crest of the levee and the crest of the spillway is exaggerated for ease of presentation, but may well be less than 0.5 m.

9.14.3 Civil engineering design of spillways – general

The spillway surface should be designed to carry the range of possible overflows without failure or significant deterioration given their the anticipated durations. This requires consideration of:

- levee surface details
- structural integrity of both the levee and the surface protection system
- durability of the materials
- all interfaces (eg drainage or bedding layers between the spillway surfacing and the body of the levee).

Spillways consist of three main parts:

- a threshold that defines the crest level across the weir
- a slope that carries the water over the landward side of the levee
- a stilling basin that diffuses the energy of the spilled water at or close to the toe of the levee.

Spillways need to be designed to carry varying volumes of water:

1. During a minor flood, a relatively small volume of water needs to be discharged, so the spillway crest could be short and only marginally lower than the rest of the levee crest.
2. During a major flood, a much greater volume of water needs to be discharged, requiring either a longer spillway (which would be expensive) or a lower spillway (which would then spill water more frequently than may be ideal).

To solve this challenge a number of options have been developed which are discussed in the following sections:

- a fixed threshold (simple and robust but inflexible)
- a variable threshold such as a fuse
- the use of adjustable gates
- a maze type threshold (such as a piano key weir).

9.14.4 Simple threshold spillways

These are the simplest and most robust types of spillway. A simple threshold spillway consists of a levee type structure which has been reinforced on the crest and on the landward slope and at the landward toe, to provide a greater level of resistance to the overtopping forces. It can consist of:

- a massive spillway structure built out of reinforced concrete or roller compacted concrete
- a robust surface covering placed on top of a levee
- a grass spillway.

These subtypes are discussed in the following sections.

9.14.4.1 Massive spillway structures

The following massive spillway types have been used successfully in the past:

1 Conventional reinforced concrete spillways

These structures are designed as conventional reinforced concrete structures. They generally have a vertical upstream face and a spill way on a slope of about 1:1 (see Figure 9.114). The reinforced structure includes the stilling basin, and construction joints are usually provided at 10 m to 15 m intervals along the spillway alignment to avoid shrinkage cracking. The threshold can be rounded to increase hydraulic efficiency.

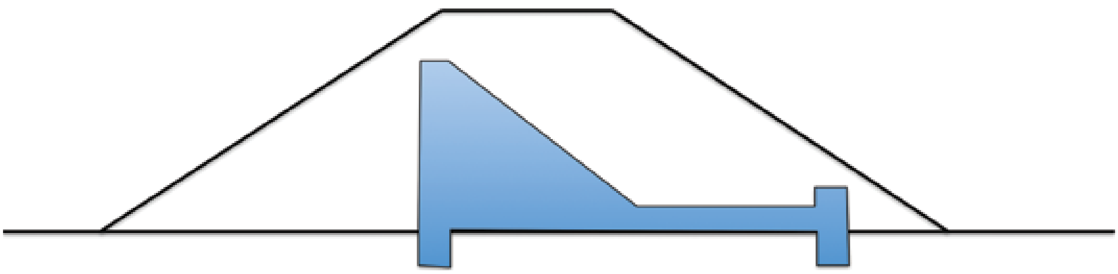


Figure 9.114 Reinforced concrete spillway (after Degoutte et al, 2012)

The quality of the foundation of reinforced concrete spillways is of great importance. It must not be subject to significant differential settlement and the structure must be able to resist high hydraulic gradients (both structurally and in terms of under seepage, uplift and internal erosion).

2 Roller compacted concrete spillways

Roller compacted concrete (RCC) spillways are increasingly used for dam construction (ICOLD, 2003). They are less expensive to construct than conventional massive weirs but are normally of sufficient strength and resilience.

An RCC weir will normally have symmetrical upstream and downstream slopes which are 1:1 or flatter, as this facilitates construction (Figure 9.115). RCC weirs will be less resilient than reinforced concrete weirs and may be subject to some erosion during overflow. However, any such erosion should be relatively minor and generally acceptable in comparison with the cost saving offered by such a construction methodology (Degoutte *et al*, 2012).

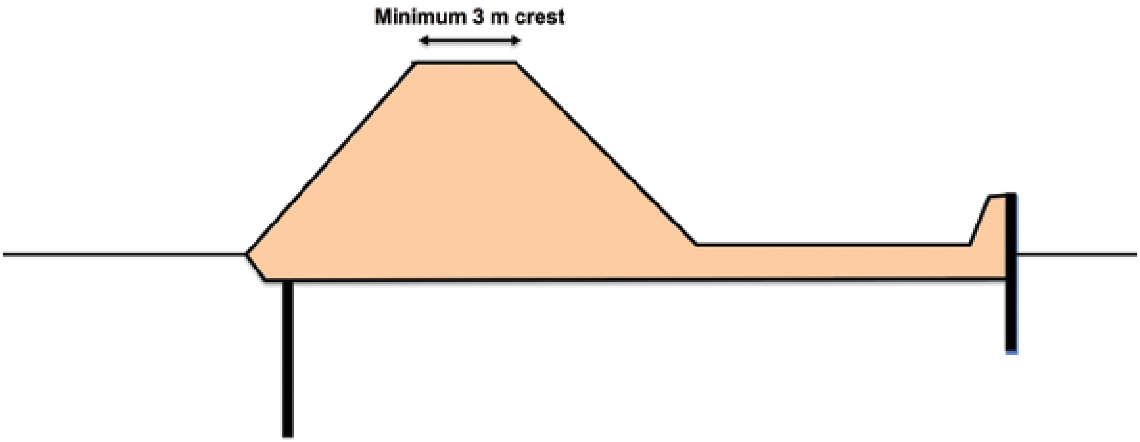


Figure 9.115 Roller compacted concrete spillway (after Degoutte et al, 2012)

Given the nature of the construction of RCC spillways, a cut-off structure of some kind may be required beneath the spillway to control under-seepage, uplift and internal erosion. The connection between the RCC weir and the rest of the levee will require a vertical interface and this is usually achieved through either a robust gravity structure or anchored bulkheads. Similarly, if the stilling basin is also constructed using RCC, then a sheet pile wall may be needed to support the end of the structure.

Figure 9.116 RCC Spillway (from Abdo and Adaska, 2003, courtesy Portland Cement Association)

Owing to the nature of their construction, RCC spillways are potentially tolerant of some foundation settlement but the implications of foundation performance must still be considered in the overall design. An example of an RCC spillway from Brownwood Country Club Dam in the USA is given in Figure 9.116.

9.14.4.2 Protected earth embankment

An alternative to a purpose-built spillway is to use robust surfacing materials for spillway construction. In this case, the detail of the interfacing between the surfacing and the underlying levee embankment will be of critical importance to its behaviour during overflow.

The nature of the surface reinforcement and the detail of the interface with the rest of the levee will depend on many issues including:

- the height of the levee above the surrounding ground
- the volumetric flow rate and the velocity of the overflow

- the proximity of the stilling basin and the nature of the hydraulic loading including turbulence and the possible locations of any hydraulic jumps
- the duration of the overflow.

The benefits of using robust surfacing materials rather than creating a solid weir include cost savings, reduced construction time and disruption and reduced foundation loads (and hence less risk of levee instability and reduced settlement). The downside is that they are lighter construction elements and hence they are more prone to damage during a flood. They are also critically dependent on the detail of the construction interfaces.

One particular benefit of most surface covering systems is that they can be covered by sacrificial topsoil and allowed to grass over. This grass cover will improve the look of the spillway and can be replaced after a flood if it is washed off. However, grass cover of this nature will hide the spillway surface from inspection. Periodic strips of some of the sacrificial grass cover to check on the condition of the spillway surface could be included in the O&M manual.

Commonly used surfacing materials include:

- reinforced concrete slabs
- rockfill concrete
- rip-rap
- reno mattresses
- stepped gabions
- pre-cast concrete blocks.

1 Reinforced concrete slabs

Reinforced concrete slabs are commonly adopted for providing surface protection for relatively short spillway weirs. A typical cross-section through a reinforced concrete slab spillway is given in Figure 9.117. Figure 9.118 then shows a reinforced concrete slab spillway construction, and Figure 9.119 shows a completed spillway.

Reinforced concrete slab spillways are normally constructed out of cast-in-situ concrete slabs, which can be of good quality and durability if formwork is used. They are relatively maintenance-free in comparison with other systems (such as concrete rip-rap, gabion baskets or reno mattresses). However, they are less effective at dissipating energy during overflow than other spillway surface materials, and they can be vulnerable to distortion due to settlement and foundation displacement, which can affect performance.

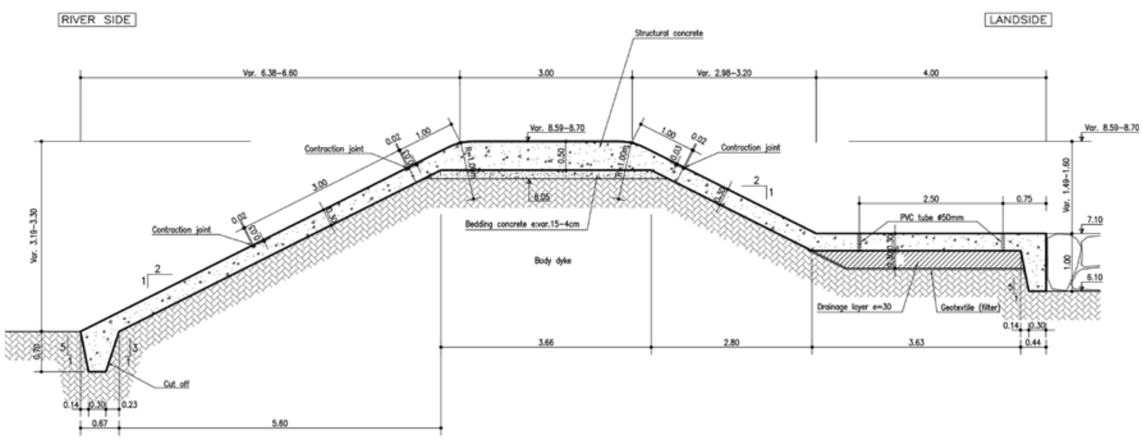


Figure 9.117 Reinforced concrete slab spillway (150 m long), River Lez, France (after Degoutte et al, 2012)

Figure 9.118 *Construction of reinforced concrete spillway slabs (after Degoutte et al, 2012)*

Figure 9.119 *Reinforced concrete slab spillway River Lez, France (after Degoutte et al, 2012)*

2 Rockfill concrete

The use of concrete coated rip-rap is common in France for long spillways. The system avoids the need for construction joints and can support a hydraulic load of 1 m and can carry overflows having a velocity of up to 8 m/s (Degoutte *et al*, 2012). However, the use of rockfill concrete creates an irregular crest profile and so a separate reinforced concrete beam is required for the threshold. A typical cross-section for a rockfill concrete spillway is shown in Figure 9.120, and Figure 9.121 shows stages of construction.

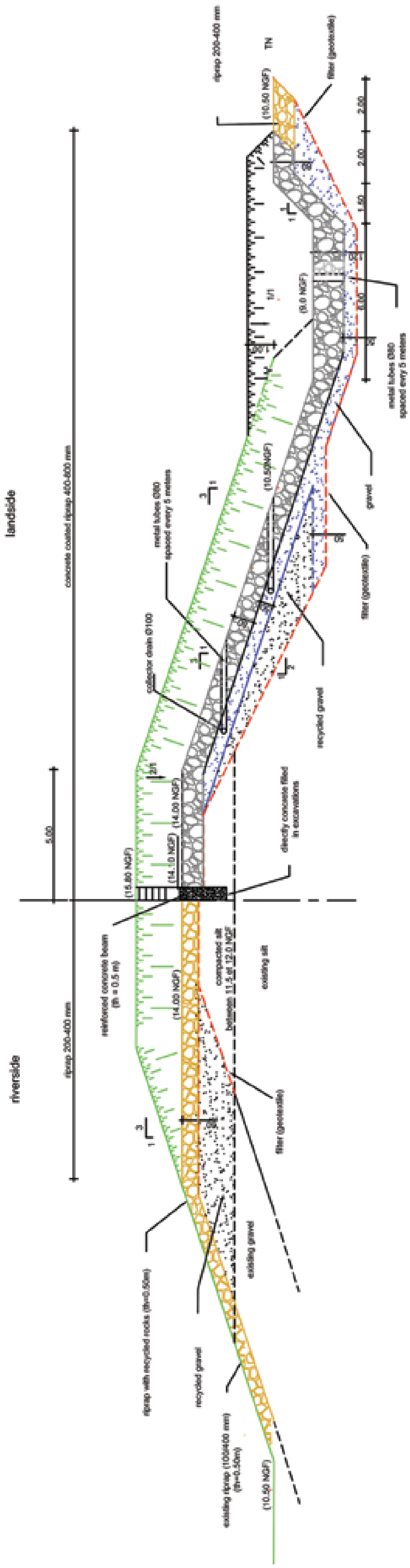


Figure 9.120 Typical cross-section of the spillway on the left bank of the Rhône at Comps (after Degoutte et al, 2012)

The performance of the spillway relies on the concrete locking the rockfill into place (Degoutte *et al*, 2012). If the concrete is damaged or it cracks, individual rocks could be lifted out of the spillway surface during the design event, and this process could induce turbulence and further damage.

An important element of the construction detail (Figure 9.120) is the drainage layer beneath the rockfill concrete. This layer prevents the build-up of pore pressures beneath the rockfill concrete layer, thereby reducing the risk of uplift during an overtopping event. It is usual to place a geotextile filter below the drainage layer to prevent fines being washed out of the embankment fill material and into the drainage layer, causing settlement. The performance of granular filters and geotextiles is discussed in Section 8.5.5.

a Preparation of drainage layer

b Construction of reinforced concrete threshold

c Placement of concrete

d Completed spillway

Figure 9.121 *Construction of rockfill concrete spillway (Degoutte et al, 2012)*

3 Rip-rap

Armourstone can be used to protect the spillway surface and can be sized using the formulae given in Box 8.12.

The filter system under the armourstone is also important (Degoutte *et al*, 2012). Any geotextile should be able to offer both a filter function (to avoid the entrainment of fines out of the levee body) and a protective function (against surface erosion of the levee body). The filter system should be placed under the armourstone. Armourstone should be placed carefully to avoid tearing or puncturing the geotextile. An underlayer of stone between the geotextile and the armourstone:

- provides good contact between the geotextile filter and the levee fill material
- facilitates drainage
- provides a base onto which the armourstone can be laid without damage to the geotextile.

Each of the material interfaces (levee fill to geotextile, geotextile to underlayer, underlayer to armourstone) should be designed according to the standard filter rules (Section 8.5.5).

4 *Reno mattresses*

Reno Mattresses can be used to create a resilient downstream slope for a spillway. As with concrete rockfill, Reno Mattresses will produce an uneven crest and so a pre-cast concrete threshold is usually adopted (Figures 9.122 and 9.123).

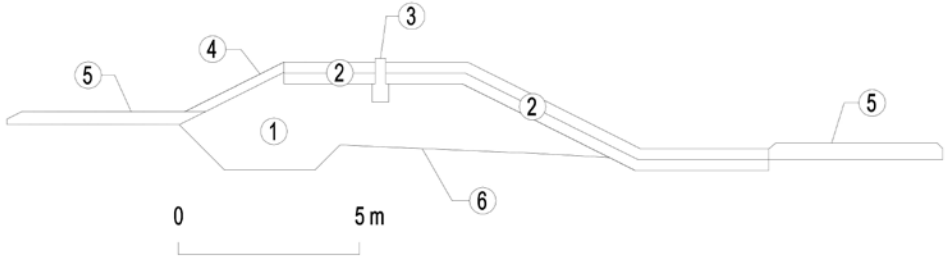


Figure 9.122 *Typical cross-section through Reno Mattress spillway (Degoutte et al, 2012)*

Figure 9.123 *Lunel spillway on the Vidourie River, France, (Degoutte et al, 2012)*

As with any permeable spillway surface, there is a possibility that fine soil particles can be washed out of the underlying levee fill. A separation layer such as a granular filter or a geotextile filter should therefore be provided beneath the Reno Mattress. A layer of free-draining stone may also be required between the gabion baskets and the geotextile layer to facilitate drainage and prevent uplift.

Reno Mattresses should only be considered for use in small spillways, with a maximum hydraulic head at the crest of 0.7 m and a maximum velocity of 6 m/s (Degoutte *et al*, 2012). The Reno Mattress basket will also deteriorate over time, and allowance should be made for potential loss of material from within the basket. For this reason, the water depth on the slope should be limited to 50 per cent of the median diameter of the stone mattress for a slope of 1 vertical to 3 horizontal, 70 per cent of the median diameter for a slope of 1 to 4 and 90 per cent for a slope of 1 to 5. Further restrictions may be needed in areas of turbulence.

5 *Stepped gabions*

Gabions can be used as an alternative to Reno Mattresses. When arranged in tiers (Figures 9.124 and 9.125), they can offer an improved level of energy dissipation. As they are installed in smaller units than reno mattresses, they can be placed with smaller construction plant can be replaced more easily if damaged or deteriorated. As with reno mattresses, gabion baskets require filter/separation layers beneath them.

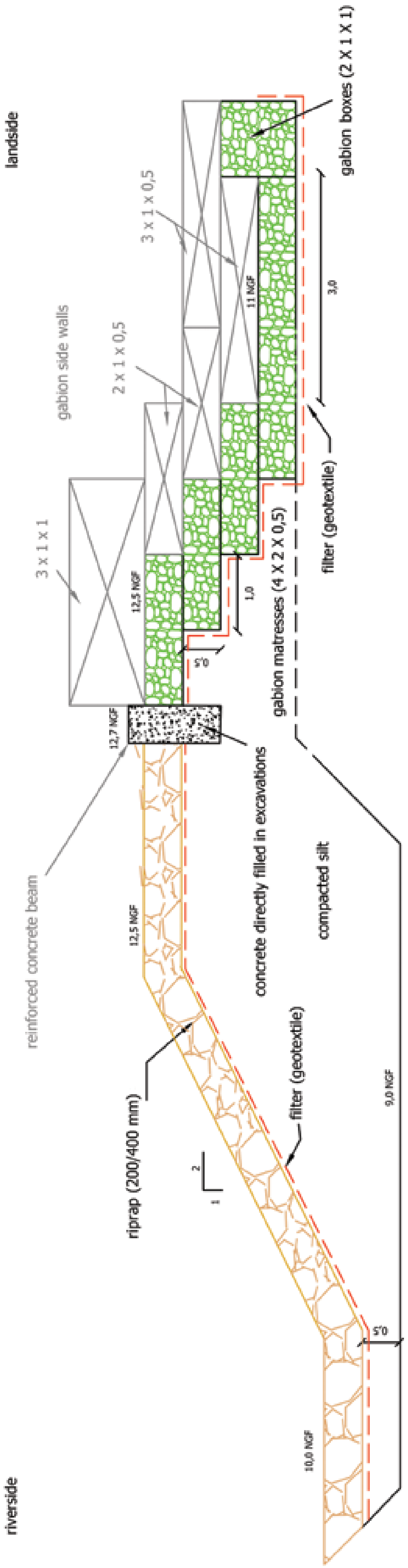


Figure 9.124 Stepped gabion weir downstream of Comps, France (after Degoutte et al, 2012)

Figure 9.125 Stepped gabion weir downstream of Comps, France (Degoutte et al, 2012)

6 *Pre-cast concrete blocks*

Concrete block systems (or ACBs) have historically been used in spillways for levees and dams for flood storage reservoirs, where high shear stresses are predicted. Under conditions of steady laminar flow, well laid ACBs can have an erosion resistance of 10 m/s (30 ft/s) or more (Hewlett *et al*, 1987). ACBs are generally 10 cm to 25 cm (4 in to 9 in) deep with an average open area of 20 per cent.

While ACB spillways have performed well under test conditions, a marked deterioration in performance occurs when blocks are not placed accurately (Hewlett *et al*, 1987), steps of approximately 25 mm between individual blocks leading to local turbulence and a significantly accelerated process of failure. So, ACBs should only be used where accuracy of placement can be guaranteed and differential settlements will not disrupt the surface with time.

Anchoring selected concrete blocks is not recommended as a way to reduce the potential for uplift of the system as there have been failures of such systems and no clear guidance on the subject is yet available.

9.14.4.3 Grass spillways

In contrast to the structural spillways described above, simple grass-covered spillways can also be used to convey water into a flood expansion zone during potential flood periods. Such spillways work best when conveying water at relatively low velocities and for short durations. They are therefore best suited to small rivers and rural, low-risk environments.

Grass spillways have the benefit of being simple and generally cost-effective structures.

The requirement that overtopping flow rates are low means that grass spillways commonly have long spillway lengths. This can be a disadvantage, as differential or localised settlement along a long crest can mean that some parts of the spillway could overtop significantly earlier than other locations. This lower area then becomes vulnerable to preferential erosion because of the locally higher velocities. For this reason, grass-covered spillways are best suited as adaptations to existing levees rather than being used for new levees.

A way to allow slightly higher velocities over a grass spillway is to provide a reinforced grass cover. The design of reinforced grass protection systems is discussed in Section 9.6.1 and an example of its application is given in Box 9.57.

Box 9.57 ***Example of reinforced grass spillway at Aller Moor, Somerset, UK***

Aller Moor in Somerset, UK, provides an example of a grass spillway. The spillway is 550 m long and it is located on the right bank of River Parrett. It is designed to convey flood flows from the River Parrett to the River Sowy flood relief channel.

Historically, uncontrolled overflow during flood conditions occurred at low spots. This overtopping flow threatened to lead to the formation of scour holes and threatened a breach.

The scheme adopted was to marginally lower the crest of the spillway and raise banks downstream. The objective of the works was to initiate overflow at a known location from where the overtopping flow could be managed.

The spillway crest was designed to comprise a 500 m long spillway with a crest height 500 mm lower than the surrounding levees. The crest and the downstream face of the levee were reinforced to resist the overflowing velocities. A detail of the cross-section is shown in Figure 9.126.

Velocities on the landward face of spillway were determined to be within acceptable tolerances for grass slopes as stated in Hewlett *et al* (1987). However, since the spillway may carry a discharge flow for several hours, it was decided to reinforce the slope with a 3d geotextile (Enkamat 7020), which will be covered with grassed topsoil.

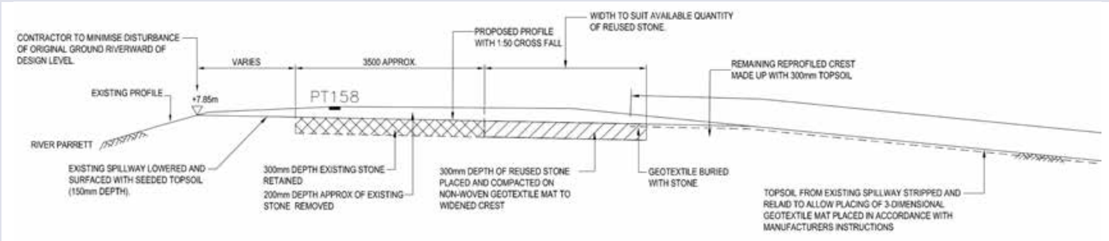


Figure 9.126 ***Aller Moor spillway cross-section (courtesy Black and Veatch Ltd and Environment Agency)***

Photographs of the spillway at Aller Moor are provided in Figures 9.127 and 9.128, the latter showing a picture of the spillway during a flood event.

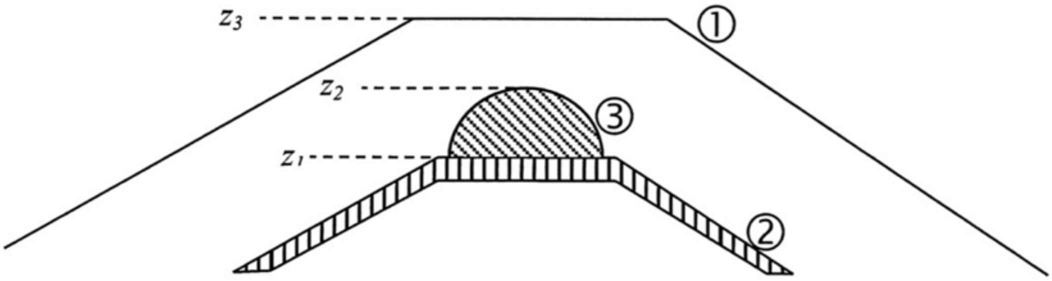
Figure 9.127 ***Looking downstream along spillway crest (courtesy Black and Veatch Ltd and Environment Agency)***

Figure 9.128 ***During the flood of November 2012 (courtesy Black and Veatch Ltd and Environment Agency)***

9.14.5 Variable spillway thresholds

As discussed in Section 9.14.3, the threshold level of a spillway can be difficult to design efficiently. If the threshold is too low, it will overtop frequently and fill the flood expansion zone too often. Conversely, if it is too high, it will require a spillway of great length to discharge a sufficient volume of water to adequately control water levels in the river during a flood. Variable thresholds have the benefit that they prevent the spillway from overtopping during normal conditions, but then offer a substantial capacity during an extreme event.

The above principle is illustrated in Figure 9.129. In reality, this is a powerful but dangerous concept. The net efficiency of such an approach must be set against the risk of a performance failure during an extreme event. The failure of such a system could result in floodwater not being discharged safely, and this could lead to considerable and uncontrolled flooding elsewhere. Safety mechanisms should therefore be included with a suitable level of redundancy and proportionate to the risk of operational failure (Royet and Meriaux, 2004).



Key

z_1 = crest level of the levee (element 1)

z_2 = crest level of the spillway (element 2)

z_3 = adjustable threshold (element 3)

Figure 9.129 Principle of a spillway equipped with an adjustable threshold (from Royet and Meriaux, 2004)

Adjustable threshold devices commonly take one of the following four forms:

- erodible soil fuses
- removable thresholds (flashboards and needle timbers)
- inflatable thresholds
- toppling/collapsible thresholds.

These variable thresholds are discussed as follows. Gated structures (including removable plates) are discussed separately in Section 9.14.6.

9.14.5.1 Erodible soil fuses

Earthfill fuse plugs are designed to contain water up to a certain level at which they begin to overflow. The depth and duration of overflow causes erosion of the earthfill, and the rate of erosion depends on the nature and condition of the material used to construct the fuse. The complete removal of the fuse plug by overflowing water is possible but depends on actual flow conditions and embankment characteristics. This unpredictability is one of the main disadvantages of the system.

Earthfill fuse plugs are relatively inexpensive to construct, but often require a large amount of space. They may be an integral part of the levee or they may be added to concrete spillway structures. Where no concrete structure is provided below the soil fuse, erosion depths may be excessive (in the order of 30 m on the Mississippi River), so the future cost of reconstruction must be balanced against the increased cost of constructing a more resilient spillway.

Mechanical activation of soil fuses can be carried out during a flood (or, preferably, in advance of an anticipated flood reaching the spillway). This can be carried out by removal of earthfill using heavy equipment or even by using explosives (Figure 9.130 and Section 6.4).

When designing soil fuses, the following issues should be addressed:

1 A large amount of space may be required for these systems to work effectively.
2 Their long-term reliability may be uncertain, because of changes to the properties of the soil fill material over time (this could either be a stiffening or a softening).
3 The impact of spilling the floodwater on the flooded area should be considered as well as the impact of the river itself.
4 The cost, logistics and reliability of the implementation of mechanical activation should be considered.
5 The cost and time needed for reconstruction should be determined.

Examples of the use of soil fuses in levees in France over the past 140 years are given in Box 9.58.

Figure 9.130 ***Earthfill embankment fuse plug after activation using explosives, Birdspoint-New Madrid floodway, Mississippi River (courtesy USACE Memphis District)***

Box 9.58 ***Soil fuses in French levees***

Soil fuses have been installed on some rivers in France since the 1870s (for example, Comoy on the Loire). These structures consist of grass-covered sand placed on top of a masonry spillway as shown in Figure 9.131. A more recent example can be found Reyran in France (Figure 9.132). Few of these structures have been tested and so they are unproven in a flood. An exception is a fuse weir located at Comps on the right bank of the Rhône, which was overtopped by a flood in 2002. This fuse consisted of two parallel bunds of gravel constructed on top of a layer of armourstone. In September 2002, the top of the fuse was exceeded by about 40 cm and this caused part of the fuse to be eroded; the underlying rockfill was not damaged. At the same location in 2003, a higher flood level quickly eroded to the rockfill shell at one location (causing some damage to the rockfill) but took nearly 20 hours to erode the rest of the sandy fuse even though the head of water at the crest exceeded 50 cm. The slow unravelling of the fuse described above was possibly because it consisted of two separate barriers. However, it does show that it is very difficult to have confidence in how such a fuse will work and how efficient it will be at releasing the desired volume of water from the river.

Figure 9.131 *Jargeau weir with its 715 m long, 1.5 m high, grassy bench fuse (Degoutte et al, 2012)*

Figure 9.132 *Soil fuse at Reyran, France (Degoutte et al, 2012)*

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9.14.5.2 Removable thresholds (flashboards and needle timbers)

Flashboards are devices frequently used to provide temporary flood protection in urban areas. In these cases, flashboards perform as temporary flood walls. Flashboards can also be used to provide a means of adjusting the crest elevation in spillways. Most flashboards are wooden boards supported by steel sections attached to the sill of the spillway. The flashboards may be removed mechanically (by hand, mechanised plant or by being allowed to fail through rotation of the supports).

Flashboard extensions of the spillway crest elevation are usually designed so that they can be adjusted to various levels. However, adjustment can only be accomplished prior to a flood unless elaborate bridge structures are used to support equipment and personnel needed to remove the boards, such as at Bonnet Carrè control structure on the Mississippi (Box 9.59).

Box 9.59 Bonnet Carre control structure, River Mississippi, USA

This is a hybrid structure that uses the flashboard concept. Vertical wooden timbers (needles) can be removed at a predetermined water level by a crane standing on a bridge structure above the spillway (Figure 9.133). This configuration permits removal of the vertical timbers in a sequence that permits some regulation of total flow passing through the spillway (Figure 9.134).

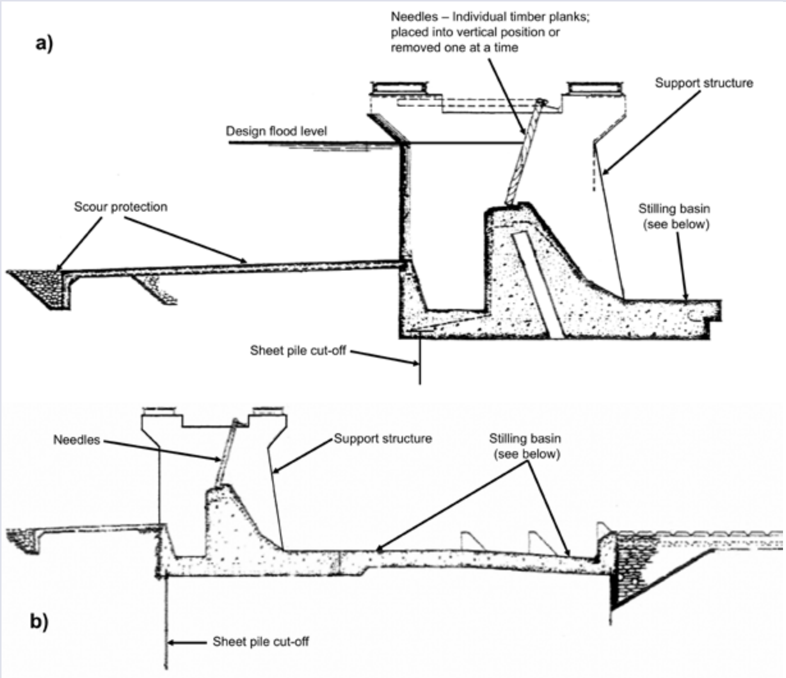

Figure 9.133 Spillway through levee with needles shown: (a) gate details (b) stilling basin (courtesy USACE New Orleans District Archives)

Figure 9.134 Cranes removing needle timbers to open spillway, Bonnet Carrè control structure (courtesy USACE New Orleans District Archives)

9.14.5.3 Inflatable thresholds

These thresholds consist of an inflatable flexible tube constructed out of reinforced rubber (approximately 10 mm thick). The inflatable thresholds can be inflated in advance of a flood and then deflated when a decision has been made to allow water over the spillway (Figure 9.135). This system is known to have been used in the USA, France and Japan (Degoutte *et al*, 2012).

The inflatable tubes can be filled with either water or air. Water-filled tubes are heavier than air-filled tubes and take longer to inflate. Water-filled tubes are capable of supporting a head of 1 m to 1.5 m of water while air-filled tubes can support up to 2 m of water over lengths of approximately 100 m. The tubes are fixed to the body of the levee structure by means of metal plates bolted to mooring lines in the sill. The air supply lines are usually embedded in a locating beam.

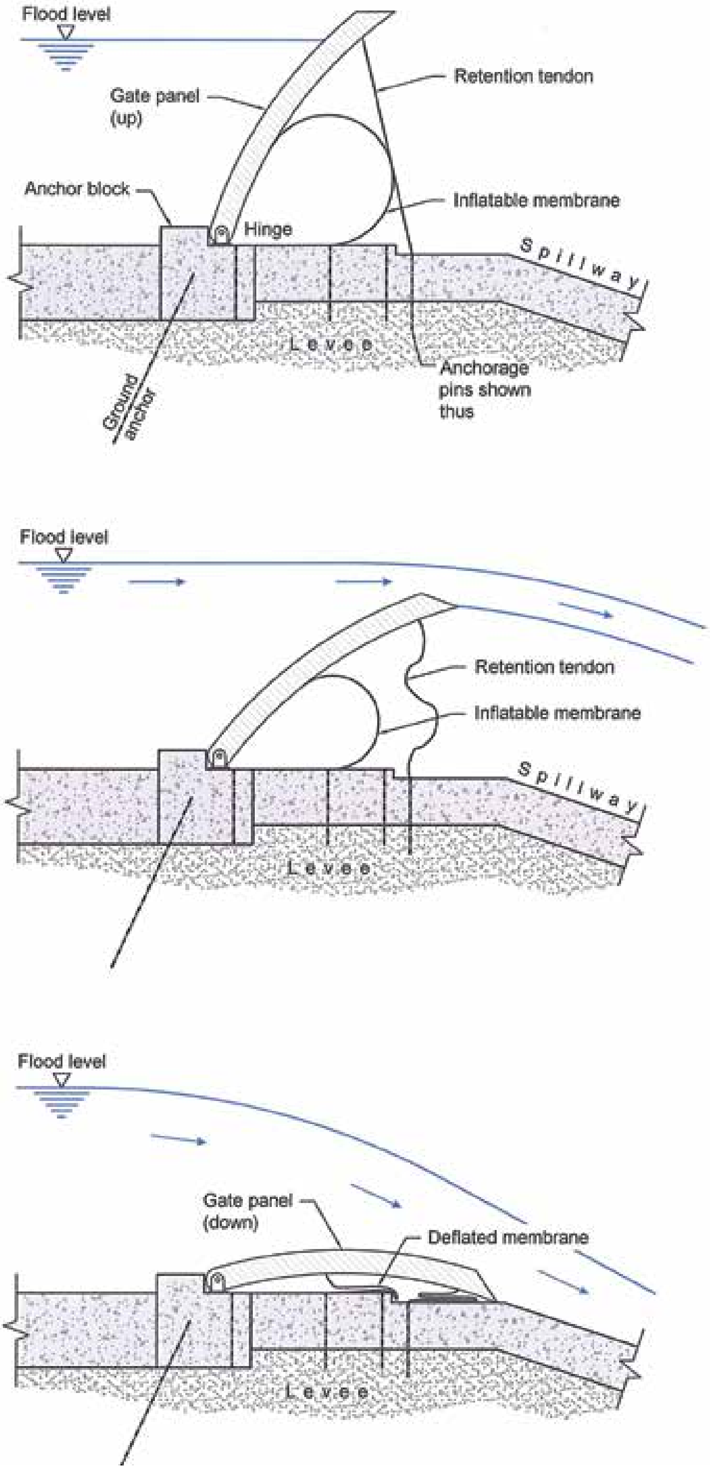

Figure 9.135 Schematic diagram of inflatable threshold (after Degoutte, 2006)

Prior to a flood, for rivers that exhibit slow response times, the inflatable tubes will normally be deflated as this avoids the risk of vandalism and reduces the risk of UV damage. The tubes can be housed in guttering within a crest slab. Consisting of rubber at least 10 mm thick, these tubes are resilient to rodents.

During a flood, where there is a risk of the spillway crest being overtopped, the gutter covers are removed and the tubes are inflated using motor-compressors. It is important that this procedure is carried out as part of a controlled flood management system. Once a decision is taken to allow the spillway to overtop, the inflatable thresholds can be deflated in a controlled manner.

After a flood, the inflatable tubes just need to be checked for condition and then repacked into the gutters. This can be particularly beneficial if two floods occur in close succession. If the tubes are not used, they should be inspected and tested every two to five years. These tests can be coupled with an overall system and management test for preparedness for flood emergencies.

Inflatable thresholds have the advantages of flexibility and reliability. Feedback from owners and operators of such systems, suggests that inflatable systems exposed permanently to the atmosphere will have an effective life of about 40 years while tubes housed in specially constructed guttering will potentially last for up to 100 years (Degoutte *et al*, 2012).

A variation of the inflatable threshold is to place a metal plate on the upstream side of the inflatable tube. In this case, the purpose of the tubes is to support the metal barrier rather than to support the water. The tubes therefore do not need to be continuous. This process was invented and patented in the USA by Henry Obermeyer. A structure of this type with a variable threshold height of 1.3 m was constructed by VNF in France at Auxonne in 2010, replacing a needle weir on the River Saône (Figures 9.136 and 9.137).

Figure 9.136 Inflatable metal-flap thresholds at Auxonne (Degoutte et al, 2012)

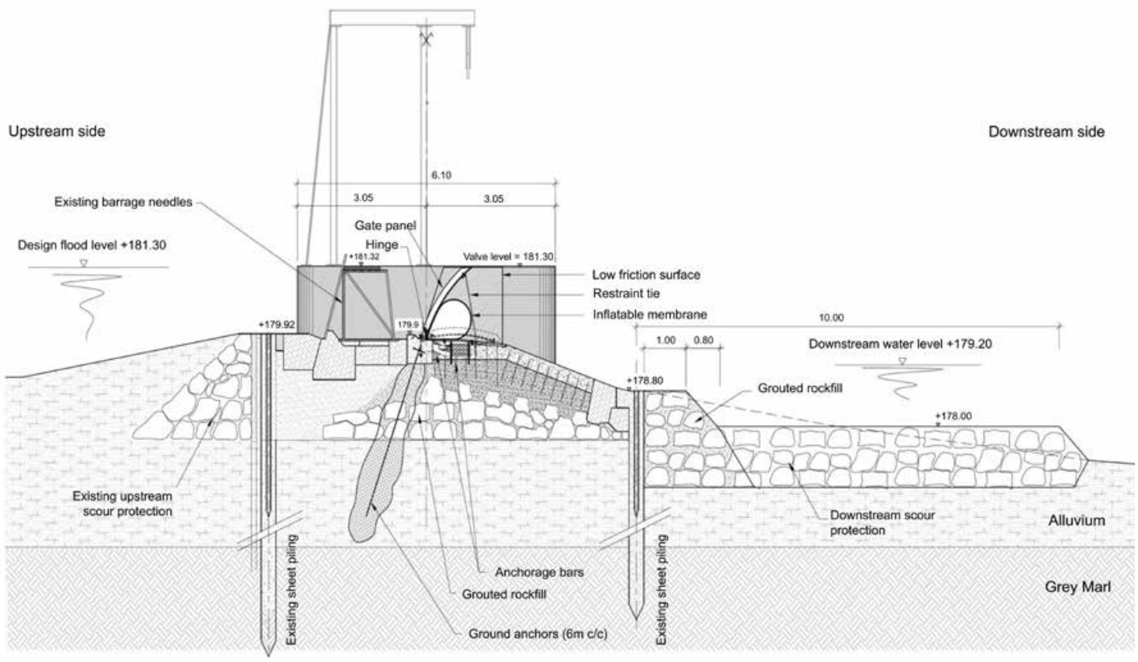

Figure 9.137 Detail of inflatable metal-flap thresholds at Auxonne (after Degoutte et al, 2012)

9.14.5.4 Toppling/collapsible thresholds

A limitation of inflatable thresholds is that they have to be operated manually in response to a flood event. This leaves them vulnerable to malfunction or operator error. One approach to reducing this risk is to design a structure that will topple when the retained water reaches a precise predetermined level. The advantage of such a mechanism is that it relies on robust characteristics such as the weight of concrete and compacted ballast and the density of water. The point at which the structure topples can therefore be determined with a reasonable level of accuracy (Figure 9.138).

The blocks can be engineered with upstream or downstream seals depending on whether a zero-uplift or total-uplift design is used. The zero-uplift design uses an upstream seal and is best suited to cases where there is no overflow over the blocks. In situations where the blocks will overflow, the total-uplift design using a downstream seal is more suitable. Blocks may incorporate a piano key or labyrinth configuration to increase overflow capacity if desired.

Important issues to be considered when designing concrete toppling thresholds include the following:

- the hydraulic performance of the blocks is important if the blocks are designed for overflow
- a good seal at the base is important to control uplift
- aeration of the overflow nappe needs to be considered
- construction detailing is important so that blocks overturn without sliding
- separation between blocks is required so there is no mechanical or hydraulic interference between adjacent units.

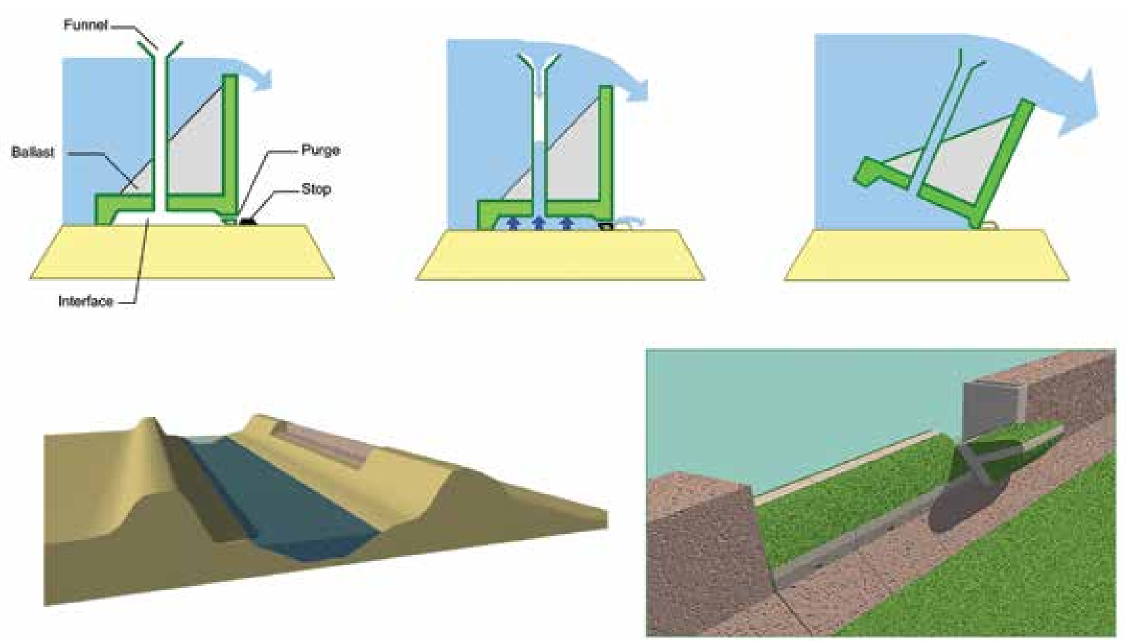

Figure 9.138 Toppling thresholds (courtesy Hydroplus)

Such an approach has been used for dam spillways (Box 9.60) and is considered to be reliable for toppling at predefined water levels (Degoutte *et al,* 2012). Owing to this reliability, it is possible to design the system so that the number of structures that topple depends on the level of the floodwater (this allows a high level of control over the rate of overtopping at different flood levels). Toppling threshold structures are generally not reusable (Degoutte *et al*, 2012).

Box 9.60 Toppling thresholds at Allan Dam in France

The toppling threshold principle has been adapted by incorporating a concrete tower (Figure 9.139). This arrangement increases reliability of the system by preventing debris accumulation and vandalism. In Figure 9.139, Reno Mattresses are being constructed to improve resilience to the overtopping flow.

Figure 9.139 Toppling thresholds at Allan Dam (Degoutte et al, 2012)

A variation on the toppling block system has been developed in Switzerland by the Federal Office for the Environment (FOE) and is shown in Box 9.61 along with an example implementation.

Box 9.61 *Toppling block system, Aa Engelberg River, Switzerland*

The scheme consists of pre-cast concrete slabs that stand vertically and are normally supported by a bund of gravel on the landward side of one of the levees on the River Aa Engelberg. The spillway mechanism requires that water flows over the top of the upright pre-cast slabs and then erodes the gravel bunds as shown in Figure 9.140. This removes the passive support for the slabs, which then topple over, releasing a larger flow of water. The pre-cast concrete slabs are of limited height (0.5 m) and therefore topple easily and relatively simultaneously. This system worked well during the floods of August 2005 (Figure 9.141).

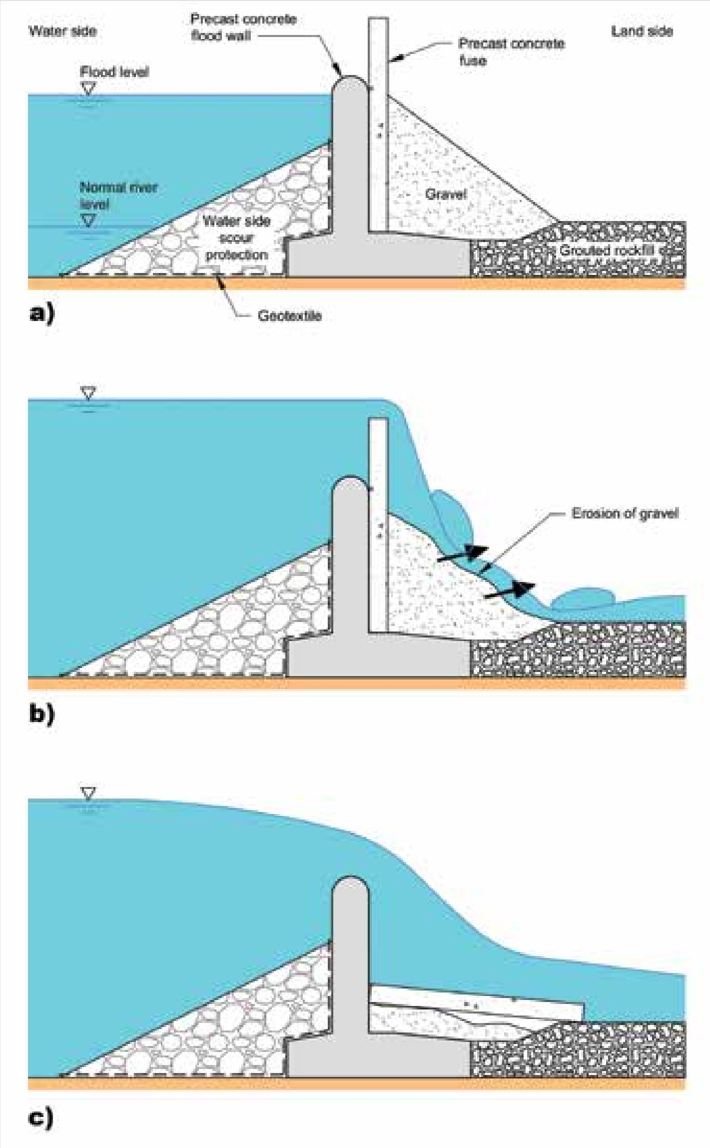

Figure 9.140 *Principle of operation of collapsing weir Aa Engelberg River, Switzerland (after Degoutte et al, 2012)*

Figure 9.141 *Weir fuse on the right bank of the Aa Engelberger (a) before flood, and (b) after flood (Degoutte et al, 2012)*

9.14.6 Gated spillways

Gated spillways provide a means of regulating discharge over a levee through the use of operable gates. A fixed threshold of reinforced concrete forms the base spillway level. The reinforced concrete structure extends down the landward slope and includes a stilling basin to dissipate high energy from the discharge through the gate outlets. The principal types of gates used to regulate discharge in reservoir spillways (including vertical sliding sluice gates, radial or tainter gates and roller gates) are used for these spillways, which are normally reserved for the largest of rivers. However, vertical sliding gates are most often used for large levees owing to their relatively infrequent use and their lower cost than other gate types.

Gated spillways usually include one or more gate bays separated by piers. These piers commonly support a bridge or walkway that facilitates the process of gate opening (Box 9.62). The piers contain the necessary hardware required to retain the gates and any equipment needed to adjust gate settings. The gates may be engineered to permit overflow in extreme conditions.

Particular issues to be considered during the design of gated spillways include the following:

- flows through the spillway must be regulated, and the sluice must be capable of conveying the required capacity
- the water levels under which the spillway will operate must be predefined from both a flood management and a safety point of view
- there is a requirement for substantial energy dissipation downstream of the gates
- the method of operation needs to be carefully considered (manual or automatic)
- the type of gate needs to be assessed from both a cost and an operational point of view
- the availability of personnel to safely operate gates at all times is important (floods usually happen at inconvenient times!)
- full life costings need to include construction costs, maintenance costs and operational costs.

Box 9.62 Morganza control structure, Mississippi River, USA

Figure 9.142 shows a cross-section through the Morganza control structure on the Mississippi River, Figure 9.143 shows the control structure under construction and Figure 9.144 shows the sluice in operation.

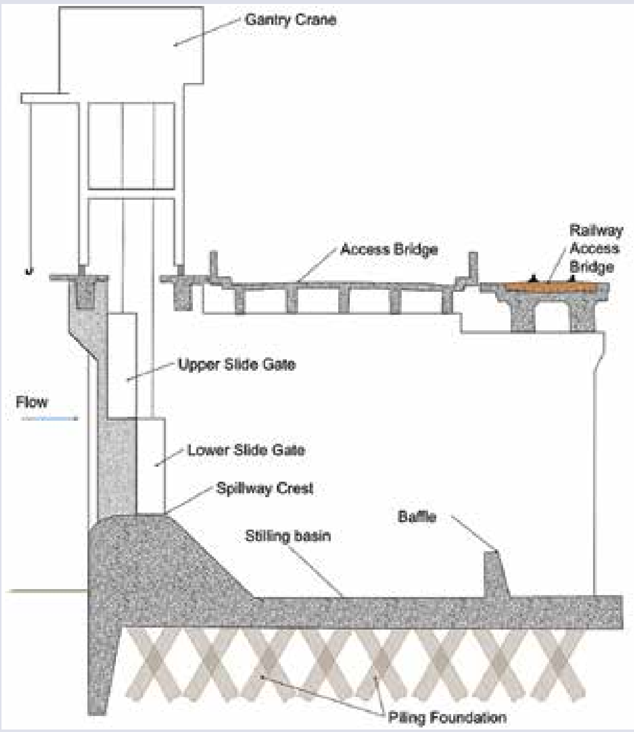

Figure 9.142 Spillway through levee with sluice gates shown (USACE, Morganza control structure, Mississippi River) (courtesy USACE New Orleans District Archives)

Box 9.62 ***Morganza control structure, Mississippi River, USA (contd)***

Figure 9.143 ***Construction of Morganza control structure, Mississippi River (courtesy USACE New Orleans District Archives)***

Figure 9.144 ***Morganza control structure discharging floodwaters during 2011 flood on Mississippi River; note the vigorous discharge (courtesy USACE New Orleans District Archives)***

9.14.7 Alternative spillway configurations

Much of the discussion on spillways so far has considered spillways as linear structures. While this has generally been the case historically, the growing pressure on space, together with the environmental impact of large engineered structures means that there is increasing pressure to design innovative structures that provide an effective spillway but use the available space more efficiently. Examples include the piano key weir, duckbill spillways (see Degoutte *et al*, 2012) and the maze threshold (Figure 9.145).

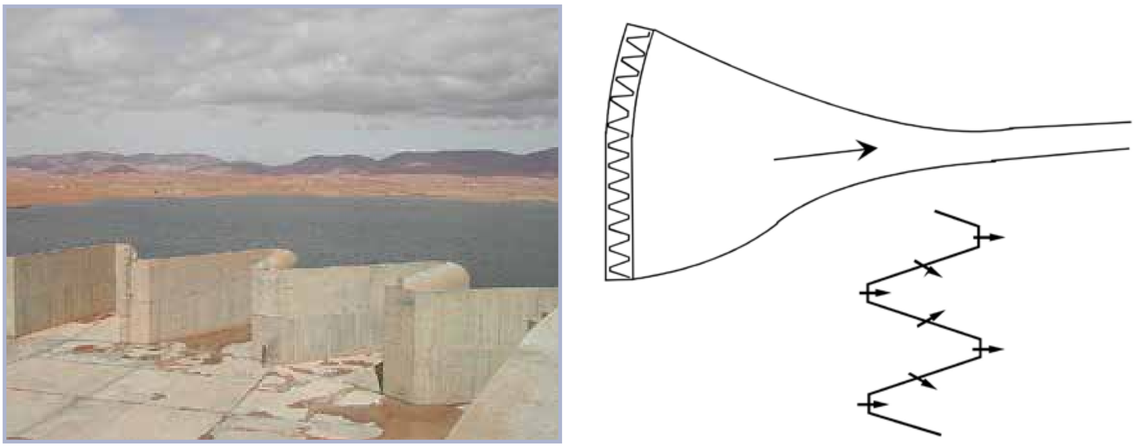

Figure 9.145 ***Plan view schematic of a maze type threshold (courtesy Frederic Laugier, EDF)***

9.15 ASSOCIATED STRUCTURES

9.15.1 Introduction

Associated structures are hard structures that are constructed in or around levees. They can be necessary to enhance the flood defence function of the levee, or they can be required for other functions. This section does not address these other requirements, but only focuses on the design work needed to ensure that the associated structures do not compromise the levee's flood defence function. The following types of associated structures are discussed in their respective subsections:

- crest walls
- embedded walls

1 2 3 4 5 6 7 8 9 10

- pipes, conduits and culverts
- spillways.

In engineering terms, levees are relatively soft and flexible structures. This brings advantages in that they are able to settle and potentially distort as the underlying ground consolidates and compresses. However, problems can be created if rigid structures such as crest walls, pipes and spillways are incorporated into the levee (see FEMA, 2009). Particular problems to be avoided by appropriate design details include:

- crest walls cracking, rotating or separating as a result of differential settlement
- embedded walls attracting high hydrostatic loads and collapsing as a result of lateral displacement during a flood
- differential settlement between piled structures and the body of the levee, which can create voids and cause a hydraulic separation which could lead to seepage and erosion
- poor compaction of levee fill material around pipes and culverts, leading to hydraulic separation and preferential seepage which in turn could cause erosion and piping.

Hydraulic separation (Section 9.7.1) is the potentially dangerous process by which a flow path is created (sometimes suddenly) between a rigid structure and poorly compacted or low-strength fill material by the action of the pressure of the floodwater. It is not straightforward to address, and so the number of interfaces between rigid elements and fill should be minimised. Further, the issue of hydraulic separation should be considered holistically rather than on an element-by-element basis. For example, historically, anti-seepage collars were commonly provided on pipes through embankment dams, but this is now considered bad detailing (Johnston *et al*, 1999), as the disadvantage of being unable to achieve good compaction around the collars outweighs any advantage from a longer seepage path length (this is discussed further in Section 9.15.4).

Detailed guidance on the design of crest structures can be found in the Hurricane and storm damage risk reduction system design guidelines (USACE, 2012a). This document was updated following the events of Hurricane Katrina; it incorporates many of the lessons learned from the performance of levees and levee crest structures during that event.

9.15.2 Crest walls

Crest walls are used to raise the level of flood protection without widening the footprint of a levee (levee cross-section options are discussed in Sections 3.2 and 3.3). They can do so without significant loss of crest width, avoiding the need to widen the base of the levee and thereby limiting the cost and impact of levee raising. Crest walls can be stand-alone structures, which are solely supported by the levee and only rely on a toe or a heel to provide part of the lateral resistance to the design water loads (the remainder coming from base friction). Alternatively, they could be composite structures which are supported on piles or sheet piles which in turn penetrate through the levee. An example of a composite crest wall constructed following Hurricane Katrina is presented in Box 9.63.

The design process broadly follows that set out in the levee design flowchart (Figure 9.3) and includes consideration of:

- whether future levee raising will be required and the implications for the form of the wall
- performance of the structure if overtopped, including external erosion of the levee body
- stability against overturning, sliding and rotational failure and seismic action
- deformation of the wall relative to the levee during the design flood event – if necessary, additional measures should be provided to avoid damage (eg a stiffer structure or increased horizontal stiffness through the use of a raking pile support system etc)
- differential settlements
- the constructability of the wall – if pre-cast crest wall segments are to be used, consider how they will be transported to site and lifted into place (Chapter 10).

The key design issues are external erosion, hydraulic separation, stability and differential settlement. These are discussed in the following subsections.

9.15.2.1 External erosion

Protection against external erosion of the surface of the earthen levee on the landward side of the crest wall is critical. This topic is discussed in Section 9.11.2 and supported by principles of external erosion protection in Section 9.6.

9.15.2.2 Hydraulic separation

Hydraulic separation can occur between the crest wall and the earthen levee. Two examples of this include the following:

1. Crest walls that are not embedded may be founded on bedding material which can be subject to seepage and internal erosion.
2. Crest walls supported on piles will not settle as the levee settles. This may create a potentially hidden gap between the underside of the crest structure and the levee, through which floodwater may flow potentially causing internal erosion and/or flooding.

The likelihood of piles promoting hydraulic separation can be reduced through the use of short (settlement reducing) piles rather than long rigid piles. A downstand from the base of the wall, such as a concrete heel or a sheet pile wall that is adequately fixed into the pile cap, can be used to reduce the risk of hydraulic separation for such a structure. In existing structures where voiding or hydraulic separation is possible, regular inspection work should be carried out, and consideration should be given to remedial measures such as grouting or the installation of a physical cut-off on one side of the crest structure.

9.15.2.3 Stability

Crest walls should be designed for stability against overturning, sliding and rotational failure during the design flood (cf. Sections 9.9 and 9.10), taking account of the hydrostatic, hydrodynamic and uplift forces acting on them (Section 8.9.1). If a gravity type wall sits on the crest of a levee, then most of the hydraulic resistance to the flood level is provided by the levee, and the crest wall will only have to resist the hydrostatic and hydrodynamic pressures acting on the wall itself (including any seepage-related uplift on the underside of the wall – see Sections 8.3 and 9.7). For block-type systems, inter-block sliding should be considered as well as sliding on the base.

When carrying out stability analyses for a levee with crest structures, it is suggested that stability calculations which deal with the body of the levee are carried out first, and that separate analyses are carried out subsequently to check the stability of the crest structure. Methods for carrying out stability analyses for crest structures are given in Section 8.9. It is important that the interaction and relative stiffness of the earthen levee and the structural elements is considered as part of the design process. For example, differential settlement could cause gaps to open between wall units (see Section 9.15.2.4).

In earthquake-prone regions, the performance of the levee and crest structure should also be checked when subject to the design seismic event during a non-flood situation. The design should be adjusted, if necessary, to provide adequate stability during this situation (Section 9.9.4).

Box 9.63 ***Lakefront Airport T-walls, LPV 105, New Orleans, Louisiana***

LPV 105 is part of the flood protection that lies to the north of New Orleans, along Lake Pontchartrain. The flood protection is required to protect the city from storm surge that can enter Lake Pontchartrain from the Gulf of Mexico. Levees and I-walls in the LPV 105 reach were constructed in the 1970s and raised again in 1990s to achieve a 100-year level of protection. Prior to Hurricane Katrina, the top of flood protection varied from 3.3 m to 4.3 m above MSL. Portions of the flood protection were overtopped during Hurricane Katrina, which resulted in severe scour on the protected side, in addition to contributing to the inundation of New Orleans. Following Hurricane Katrina the 100-year level of protection was recomputed and it was determined that the top of the flood defences had to be increased to 4.7 m above MSL (USACE, 2007).

In order to achieve the new level of flood protection, the existing levees and I-walls had to be replaced with T-walls. Increasing the height of the levees was not possible because of limited space. Increasing the height of I-walls was not possible because of limits on the heights of I-walls established following Hurricane Katrina. In order to construct the T-walls, the I-walls were removed and the levees had to be partially degraded to provide enough width to drive piles and to accommodate the full width of the base slabs. The foundation of the T-walls consisted of battered, steel H-piles and a sheet pile cut-off wall. A typical cross-section of the T-wall used is shown in Figure 9.146. A portion of the completed T-wall with scour protection in place can be seen in Figure 9.147.

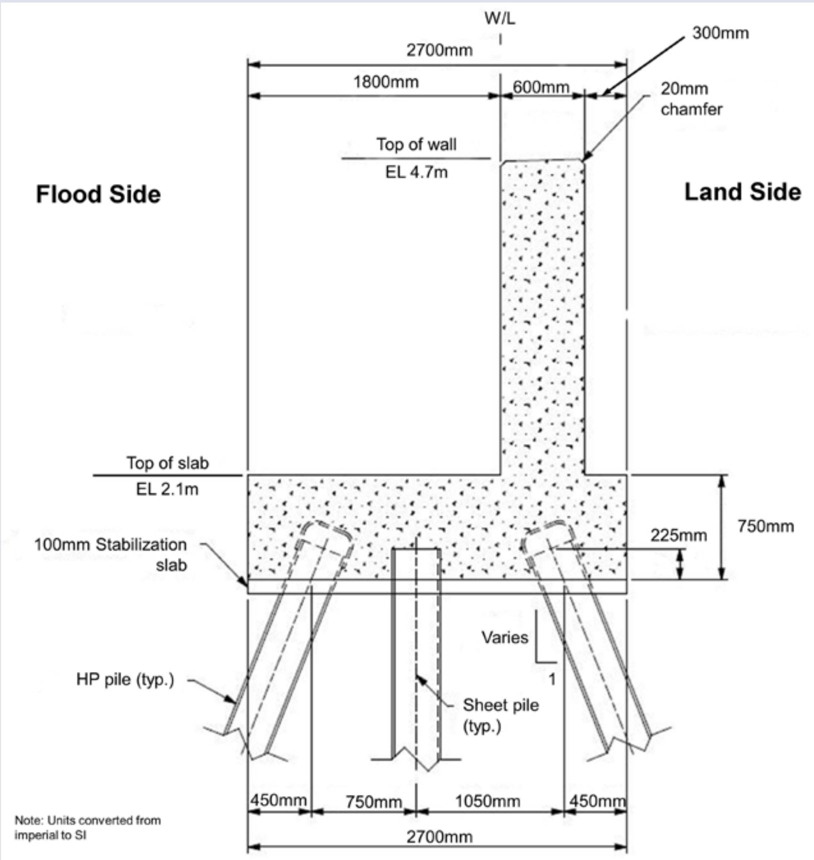

Figure 9.146 ***Typical T-wall cross-section in LPV 105 (courtesy Barry Fehl/URS)***

Figure 9.147 ***Portion of LPV 105 constructed T-wall (courtesy Barry Fehl/URS)***

9.15.2.4 Differential settlement

Relative orders of magnitude of movements of the crest structure compared with levee movements throughout the design life should be considered, given their implications for robustness, the structural stresses within the members, hydraulic separation and the need for regular maintenance to ensure water tightness (Section 9.12).

Non-embedded structures, such as crest walls, should not experience the same problems as embedded vertical walls (Section 9.15.3) as they do not interact with the levee body in the same way. They tend to behave as more flexible structures and will, to a certain extent, move with the levee. This creates its own problems: reinforced concrete walls, for example, are produced in lengths that are manageable for construction. However, because these units are rigid in themselves, they are prone to differential movements as the levee moves over time (as shown in Figure 9.148). This can lead to serious serviceability problems, leaving the wall vulnerable to seepage through the joints and localised erosion which could cause a local failure during a flood situation.

Figure 9.148 Differential movement between pre-cast crest wall units (courtesy Mike Wallis)

The designer must carefully consider the relative stiffnesses of the levee and crest wall, and must find a way to accommodate the resulting differential movement. There are two fundamental options for resolving this: either a carefully considered structural solution or a detailed method of monitoring and maintenance over time (such as periodically checking and replacing the joint sealant between individual wall elements).

An important design decision for crest walls concerns how to deal with differential settlement: the fundamental choice between a structural solution (higher construction costs but low maintenance), and an approach with emphasis on monitoring and maintenance (vice versa). This requires consideration of whole life costs and of other issues such as ease of access for maintenance.

9.15.3 Embedded walls

In a similar way to crest walls, embedded walls can be used to raise the level of flood protection without widening the footprint of a levee. They can be installed relatively quickly and can be left standing proud of the levee as a simple sheet pile structure, or they can be encased in concrete. Their apparent simplicity is the reason for their relatively common use but their function and performance is often misunderstood, potentially with disastrous consequences.

The design process broadly follows that set out in the levee design flowchart (Figure 9.3) and includes consideration of:

- whether future levee raising will be required and the implications for the form of the wall
- external erosion of the levee body to landside or waterside leading to loss of support for the wall
- stability against deep rotational failure, wall rotation and seismic action
- deformation of the wall relative to the levee during the design flood event – if necessary, additional measures should be provided to avoid damage (eg a stiffer structure or increased horizontal stiffness through the use of a raking pile support system)
- differential movements
- the constructability of the embedded wall (Chapter 10) including the need for use of land-based or floating plant and approach to pile installation.

The key design issues are external erosion, hydraulic separation, stability and differential settlement. These are discussed in the following subsections.

9.15.3.1 External erosion

Embedded walls can be adversely affected by erosion of the levee in two situations:

- on the water facing side due to fluvial or coastal erosion
- on the landward side due to overtopping flow.

Protection against external erosion of the surface of the earthen levee is discussed in Sections 9.6 and 9.11.2 and design tools are provided in Section 8.4.

9.15.3.2 Seepage and water pressures

Embedded walls create a sharp barrier to the flow of water and to the establishment of a smooth phreatic surface within the levee. As a result, while the embedded wall may be subject to the full hydrostatic action on the waterside side of the wall, it may be subject to a much reduced hydrostatic pressure on the landward side. While this effect may be beneficial in relation to inhibiting seepage through the levee during a flood, the structure should be designed to support that head difference.

9.15.3.3 Stability

Stability calculations for the embedded wall acting as a cantilever under flood loading should take account of:

- the groundwater profile (Section 9.15.3.2). Seepage analyses should be carried out for this purpose (Section 9.7)
- displacements as a result of the difference between the water pressure loads on either side of it (Section 9.15.3.2). The differences in stiffness between the wall and the surrounding levee fill may cause this cantilever wall to displace relative to the ground, opening a gap between the wall and the soil and thereby allowing the full hydrostatic pressure to act on the waterside of the wall. If the wall has only been designed to support the hydrostatic pressure above the levee crest, it could fail, as happened in New Orleans during Hurricane Katrina (Box 9.64 shows further details)
- reduction of the landward **passive resistance** arising from the landward slope of the levee.

The design of embedded retaining walls is a major topic in its own right, and for further information readers are referred to Gaba *et al* (2003) and Eurocode 7.

The possibility of deep rotational instability beneath the toe of the wall should be checked.

In earthquake regions, the performance of the levee and crest structure should be checked for the design seismic event during a non-flood situation. If necessary, the design should be adjusted to provide adequate stability during this situation.

Finite element or finite difference analyses can be used to investigate the interaction of all of the elements, but simpler calculations of the independent components discussed above should always be used as a cross-check.

9.15.3.4 Differential movements

Potential movements of the levee and the embedded structure relative to each other should be evaluated for all potential situations throughout the design life. Orders of magnitude of relative movements between the embedded wall and levee should be evaluated and the implications of these relative movements in terms of robustness, the structural stresses within the members and the need for regular monitoring and maintenance to ensure water tightness.

9.15.3.5 Location of embedded wall in relation to levee crest

By definition, crest walls are usually installed on the crest of a levee. Depending on the detail of the design, they may be installed:

- **On the waterside side of the crest:** This location maximises the proportion of levee on the landward side and hence maximises the passive resistance provided by the levee crest, landward slope and any landward berm. The crest width available for vehicular access is largely maintained and located on the landward side of the levee, so emergency and maintenance plant can operate at high water levels. However, if the levee's resilience to waterside erosion is limited, erosion could reduce the support provided by the waterside slope to the retailing wall.
- **In the centre of the crest:** This location has the benefit of optimising the passive support to the embedded wall both during the flood and in the post-flood rapid draw-down situation (Section 9.9). However, unless the crest is very wide, such a location will inhibit the passage of vehicles along the crest for O&M or emergency response.
- **On the landward side of the crest:** This location maximises the space for the river in flood and provides the greatest resilience to fluvial or coastal erosion. However, the design of the embedded wall is the least efficient at this location as the slope on the landward side will provide little passive resistance for the wall. Another downside with this location is that maintenance or emergency vehicles will not be able to gain access along the crest during a flood.

Box 9.64 Failures of embedded crest structures

The failure of the I-walls in New Orleans stemmed from a lack of understanding of the interaction between the levees and the relatively rigid embedded walls (ASCE, 2007 and Brandon *et al*, 2008).

Figures 9.149 and 9.150 illustrate the effect on the failure surfaces of the water-filled gap.

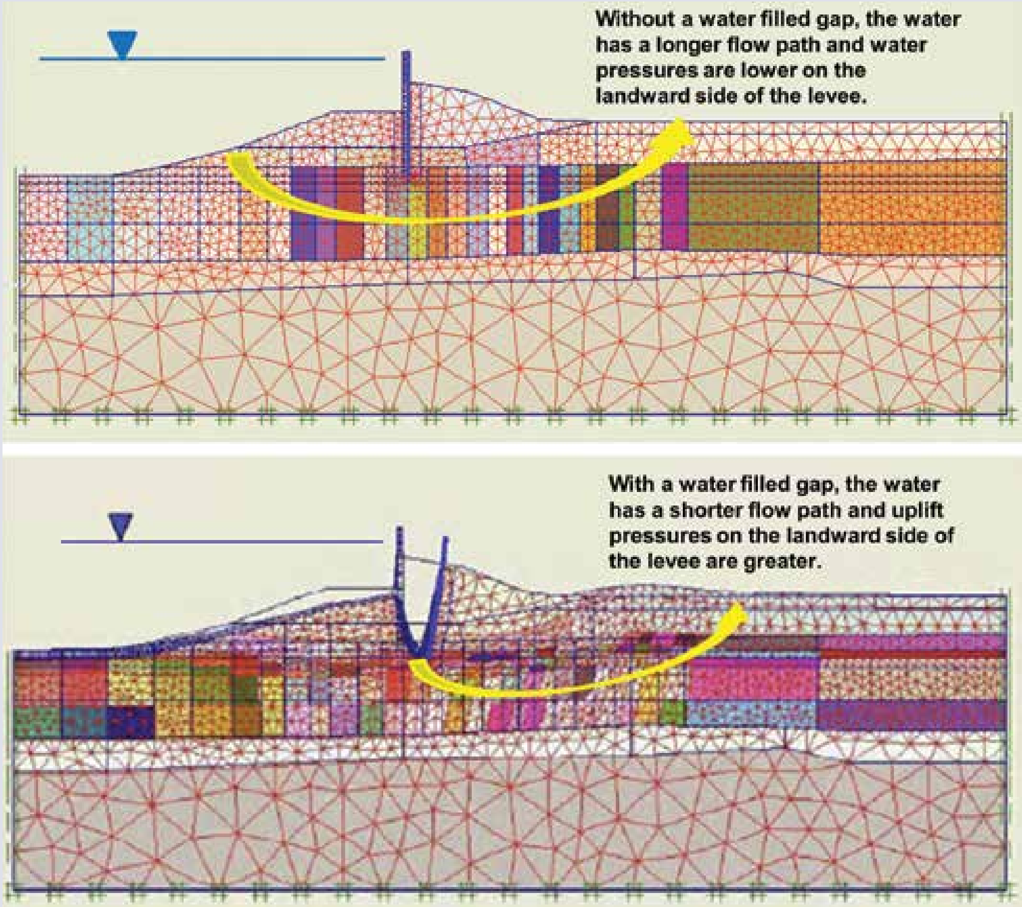

Figure 9.149 Failure of I-walls during Hurricane Katrina, (courtesy USACE)

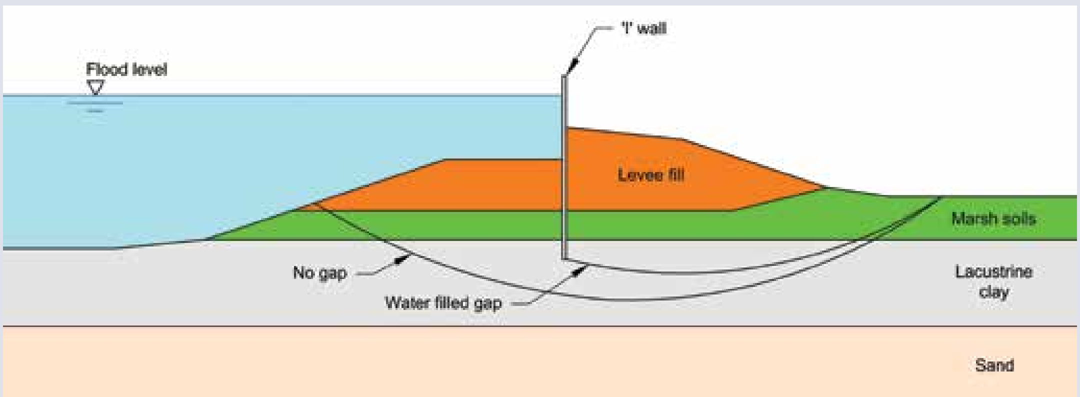

Figure 9.150 Failure of I-wall structure, New Orleans (after Brandon et al, 2008)

9.15.4 Pipes, conduits and culverts

9.15.4.1 Introduction

Pipes, conduits and culverts are sometimes needed to carry liquids or gases through or under levees. The preferred solution is to carry the pipe or conduit over the levee and avoid disturbance of the levee body altogether – Section 9.15.4.2 gives guidance for design approaches to achieve this.

If this is not possible, then the levee designer has to deal with a number of challenges related to leakage, differential settlement, seepage and erosion due to increased turbulence. Section 9.15.4.3 sets out how to deal with these challenges in detailed design.

Finally, Section 9.15.4.4 gives guidance on the replacement of pipes that have reached the end of their functional life.

9.15.4.2 Pipe crossings up and over existing levees

The disturbance of the levee body by pipes, conduits and culverts is undesirable in principle because it can negatively affect levee performance, but potentially also the utility function of the pipe itself. This section gives an example of the US approach to designing crossings over levees.

In the USA, current trends are for third party operated pressurised utility pipes to go up and over existing levee embankments with crest excavations not extending below the freeboard zone. Where a burial depth exceeding the freeboard limit is required, additional fill is placed on the levee crest. Figure 9.151 is a cross-section of a pipe crossing up and over a levee which required additional fill.

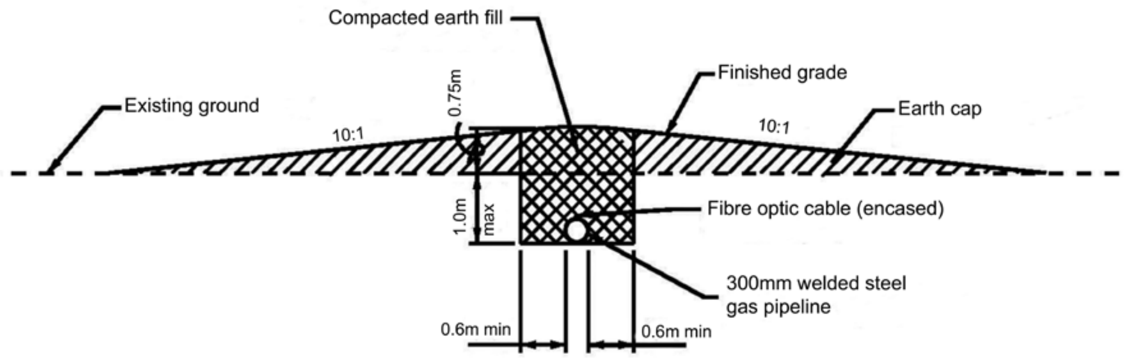

Figure 9.151 Cross-section view of a pipe crossing at crown of levee (courtesy City of Paducah, Kentucky, Paducah Levee System)

Figure 9.152 Utility pipes crossing up and over Paducah Kentucky Levee System (a) and the Cincinnati Ohio Levee System (b) (courtesy City of Paducah and Metropolitan Sewer District).

Figure 9.152 presents photographs of an overbuild area on the crown of a levee. These pipes are typically designed to have a **shut-off** type valve on the landside and riverside of the levee situated within 3 m to 15 m (10 ft to 50 ft) of the levee toe. This is to allow for isolation of damaged or defective portions of pipe that could cause an issue with internal erosion of the levee embankment during a flood event.

Once installed, newly placed pipelines are usually pressure tested to check that the joints are watertight. These pipes do not typically require any casing pipe to be installed around the exterior of the line because of the high elevation of the levee crest, or because these lines are installed in the freeboard zones. Box 9.65 describes the pipe joint testing procedures adopted in the USA for pre-cast concrete pipes.

Box 9.65 Pipe joint testing in the USA

In the USA, pipe joint tests for pre-cast concrete pipe must meet the standards of ASTM C1103-3 (2009) for pipes up to 24 in (0.61 m) or to ASTM C 1214-02 (2009) for pipes up to 36 in (0.91 m). It is noted that there are other methods for performing pipe joint testing such as air, smoke and water, using national standards.

Backfill around pressurised utility pipes can use:

- flowable fill to some distance out from the levee toe
- impermeable soils with an estimated hydraulic conductivity less than 1×10^{-7} m/sec.

Non-pressurised utility lines (electrical, fibre optic, telecommunications etc) are installed in a similar manner. These lines are installed in a protective conduit, with the interior of the conduit sealed with an approved material to ensure that floodwater does not pass through the space between the conduit and the utility line.

An example of an inverted siphon crossing a levee in France is provided in Figure 9.153. This detail was designed in accordance with the guidance set out in Box 9.66 (Section 9.15.4.3). Note, in particular, that the surface of the levee is protected by reinforced concrete for 5 m on either side of the pipe and that the pipe is exposed on both the landside and the waterside of the levee to facilitate access and maintenance.

Other elements of pipe crossings are shown in Figures 9.154 and 9.155. Again note that the pipe is well protected in the finished structure and accessible for maintenance purposes.

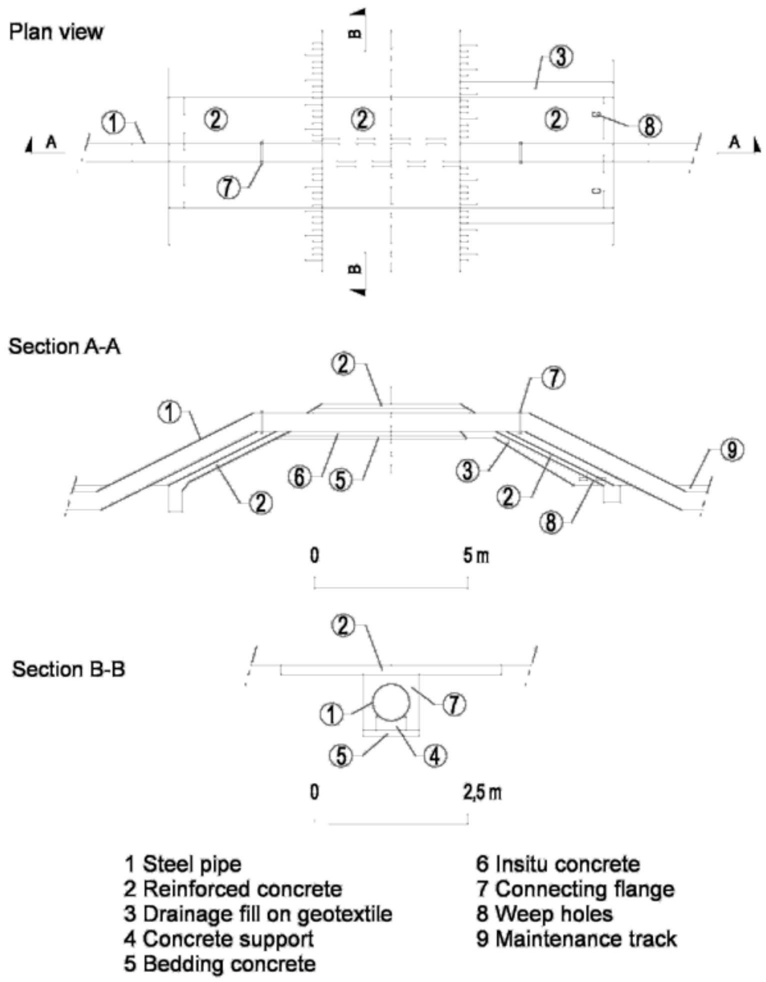

Figure 9.153 Cross-section showing the detail for an inverted siphon crossing a levee (courtesy Symadrem and ISL)

Figure 9.154 Pipe crossing of Rhône river in France (courtesy Symadrem)

Figure 9.155 Landward side of pipe crossing of Rhône river in France (courtesy Symadrem)

9.15.4.3 Pipe, conduit and culvert crossings through levees

If it is not possible to carry the pipe, conduit or culvert over the levee, there is a need for careful design to secure the flood defence function of the levee. This section sets out the key design issues and gives recommendations and examples for detailed design.

External erosion

The structural arrangement of the inlet and outlet structures can increase turbulence of the flow of water along the levee sides, particularly during a flood event. This can lead to local erosion of the levee surface. Allowance for local increases in the size or capacity of surface protection systems (Section 9.6) should be provided in such locations.

Seepage and internal erosion

Problems with material selection and construction detailing can allow **preferential seepage** through the levee adjacent to the pipes or conduits. In particular, it is very difficult to ensure good compaction and high contact stresses at the interfaces. If the contact stress drops below the water pressure at the design flood, then hydraulic fracture and hydraulic separation can occur. If this happens, a crack can open up along the interface, providing a leakage path with potential for internal erosion. This in turn may lead to softening, the formation of voids and ultimately collapse. Detailed guidance and examples for this issue are provided further in this section.

As shown in Figures 9.156 and 9.157, poor compaction of fill materials around a pipe can promote hydraulic fracture, seepage and internal erosion. Note that this process can happen quickly as hydraulic fracture is the immediate opening of a pathway for water rather than a slow process of deterioration. FEMA (2009) gives examples of situations where such failures occurred on the first filling of reservoirs contained by embankment dams. The same failure mechanisms can occur in large levees under flood conditions.

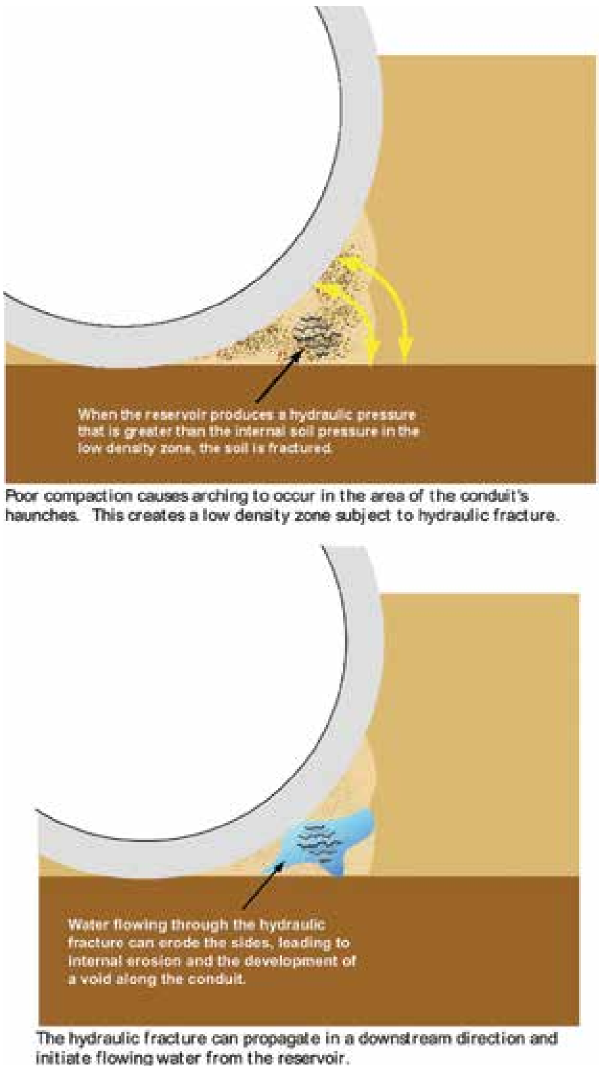

Figure 9.156 Poor compaction under pipe haunches causing seepage and internal erosion (from FEMA, 2009, courtesy FEMA and US Bureau of Reclamation)

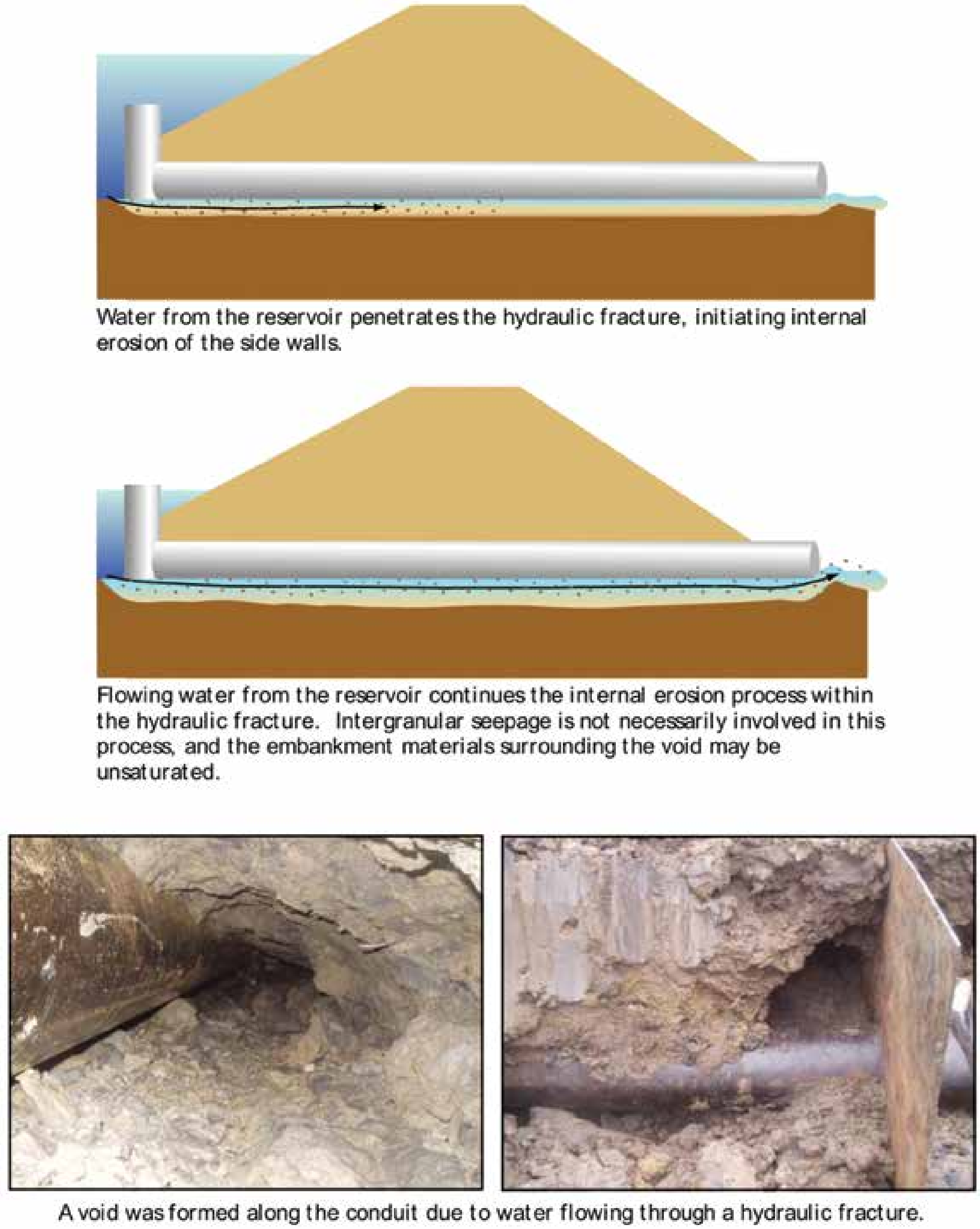

Figure 9.157 Internal erosion around pipes caused by poor compaction and hydraulic separation (from FEMA, 2009, courtesy FEMA and US Bureau of Reclamation)

On the other hand, as shown in Figure 9.159, care is required to avoid applying too much compactive effort as this can open up a void and water pathway under the pipe.

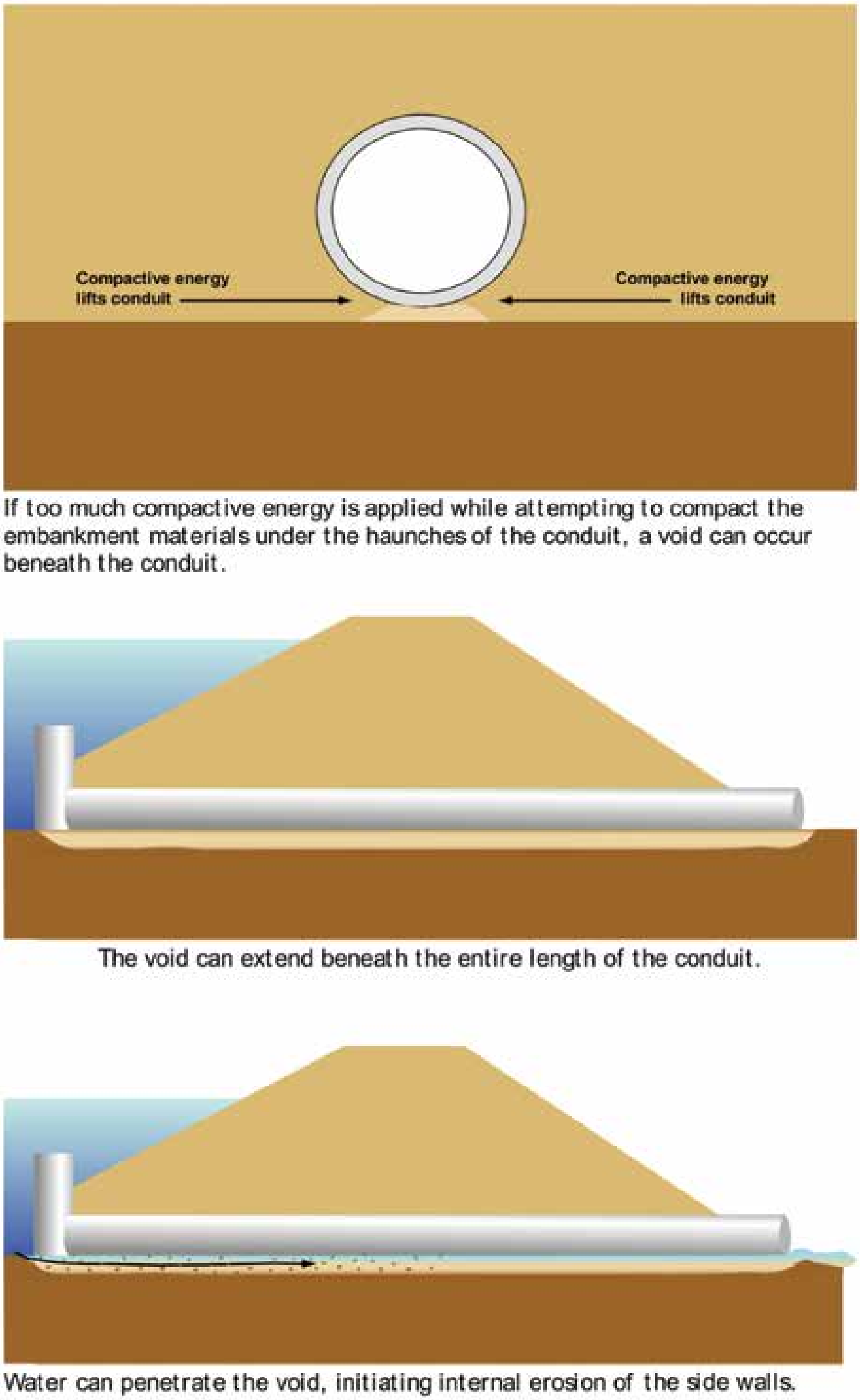

Figure 9.158 Excessive compaction promoting hydraulic separation (from FEMA, 2009, courtesy FEMA and US Bureau of Reclamation)

The pipes can also **leak**, particularly if they are old or have deteriorated. This leakage can increase phreatic pressures within the levee, potentially causing softening or instability. It can also cause internal erosion.

Like other 'associated structures', pipes, conduits and culverts can be affected by **differential settlement** because they are stiff inclusions within a comparatively soft and flexible levee. For example, the crest of a levee will normally settle more than the side slopes or the levee toes. Differential settlement can damage the pipe, conduit or culvert, especially around joints (causing leakage which in turn can affect the levee). It can also cause hydraulic separation, leading to seepage.

To solve seepage problems, pipes through levees and embankment dams used to be provided with anti-seepage collars. This is now not considered to be good practice, as the disadvantages of being unable to achieve good compaction around the collars outweighs any advantages of a longer seepage path length (Figure 9.159).

Figure 9.159 Difficulties of compacting around anti-seepage collars (from FEMA, 2009, courtesy FEMA and US Bureau of Reclamation)

Modern practice on dams is to have no collars but to incorporate alternative details into the design. For small levees, these details may be less important because the hydraulic head will be lower, but it is recommended that the issues are considered for all levees and that appropriate measures are incorporated into the design if necessary:

1 For rigid pipes, the difficulties associated with compacting beneath and around the pipe can be overcome by constructing a concrete cradle or using concrete bedding up to approximately 25 per cent of the pipe diameter (Figure 9.160). Any joints in the concrete bedding should coincide with joints in the pipe. In order to facilitate compaction around the cradle, sharp edges should be avoided and sides should be sloped at 1H:10V or flatter.

Figure 9.160 Pre-cast concrete pipe using bedding as support (from FEMA, 2009, courtesy FEMA and US Bureau of Reclamation)

2 For flexible pipes (eg HDPE) cradles should be avoided as they inhibit the movement required to mobilise the pipe's strength. Consideration should therefore be given to fully encasing the pipe. The use of a filter diaphragm or collar (Figure 9.161) should be considered.

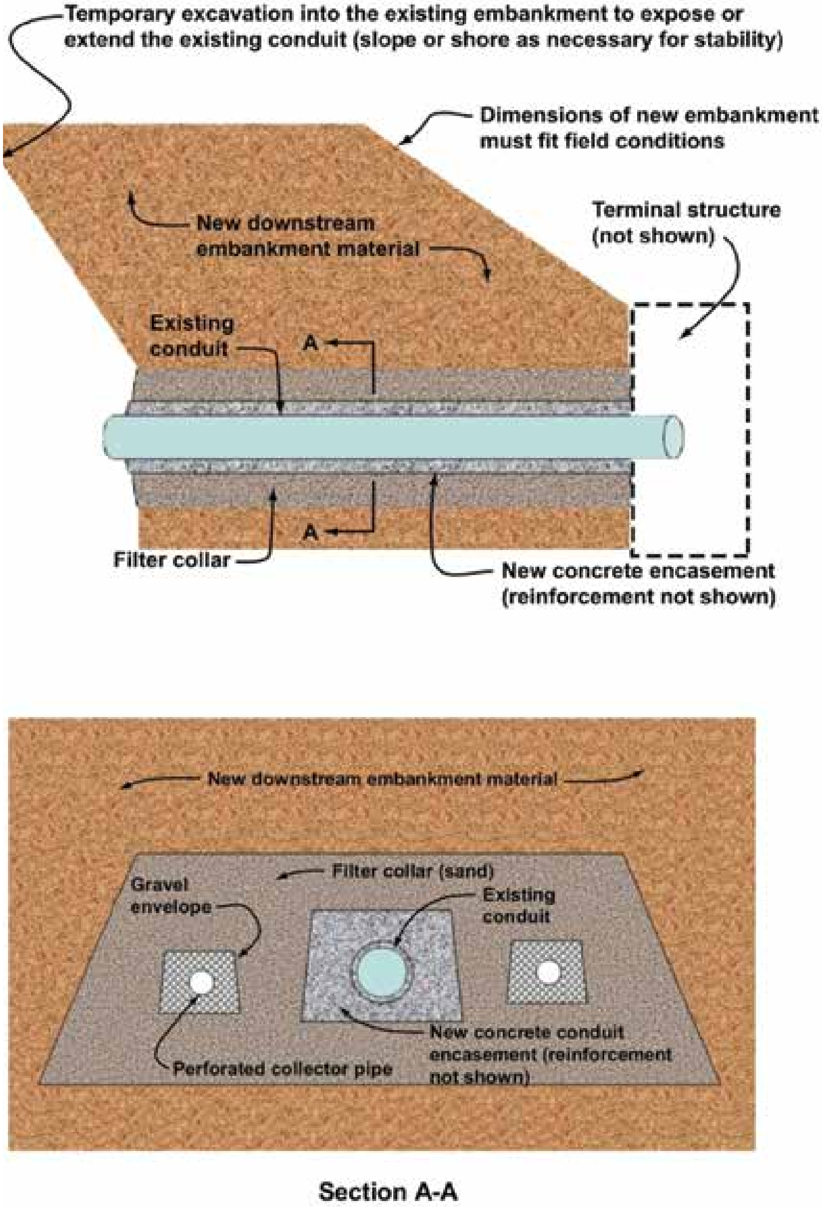

Figure 9.161 Sand filter collar details (from FEMA, 2009, courtesy FEMA and US Bureau of Reclamation)

3 It is easier to achieve good compaction and maintain a positive contact stress if the vertical walls of concrete structures such as culverts are battered at say 5V:1H.

In France, Symadrem have produced guidelines for the design of crossing structures such as pipes (Tourment *et al*, 2012). These are set out in Box 9.66.

Box 9.66 *Symadrem guidelines for pipes and culverts crossing levees*

Guidance for avoiding internal erosion around the structure

1 Where possible, the pipe system should use an inverted siphon to decrease the hydraulic gradient and the pressure within the pipe.

2 Siphon pipes should be exposed, where possible, on the landward slope of the levee to facilitate access and to allow maintenance without damaging the levee.

3 The crossing profile should be chosen to have no impact on the levee geometry.

4 Protection against hydraulic separation and internal erosion should include:

   i a cut-off device or collar (concrete beam) on the upstream side

   ii a filter (sand or geotextile) around the pipe on the landside to prevent internal erosion occurring at the interface between the fill and the pipe; the filter should be protected by a draining backfill which is then covered with topsoil or a reinforced concrete shell (with weep holes).

5 The design should be based on the Lane rule (Section 8.1.5.2).

6 For concrete structures, concrete should be placed directly into excavations to avoid poor compaction adjacent to the structures and to increase the contact area.

Guidance for dealing with high flood levels

- a robust external closing or locking device should be provided on the upstream side of pipes or culverts that open at low water levels to avoid high pressures occurring on the inside of the pipe or culvert.

Guidance for normal operating conditions

- pipes located on both sides of the crossing levee should be removable to allow maintenance and repairs without damaging the levee
- the levee slopes 5 m either side of a pipe crossing should be surfaced using concrete to reduce the likelihood of erosion and to avoid the growth of vegetation close to the pipe
- protection against external erosion around the outside of the pipe should be incorporated into the design.

The examples below (from Aramon in France) provide details of hydraulic structures crossing levees. In both of these examples, the design has provided increased resistance to seepage adjacent to the structures and hydraulic separation by lengthening the potential seepage path. The lengthening of the seepage path has been achieved by:

- incorporating concrete beams into the foundations by casting concrete directly into the excavated trench (rather than trying to compact adjacent to the structure)
- using sloping side walls to improve compaction of the fill against the walls
- adding bituminous paint onto the wall to increase the contact between the fill and the wall.

A cross-section and a plan of the crossing are provided in Figures 9.162 and 9.163 respectively. A photograph of the construction is shown in Figure 9.164 and the completed structure can be seen in Figure 9.165.

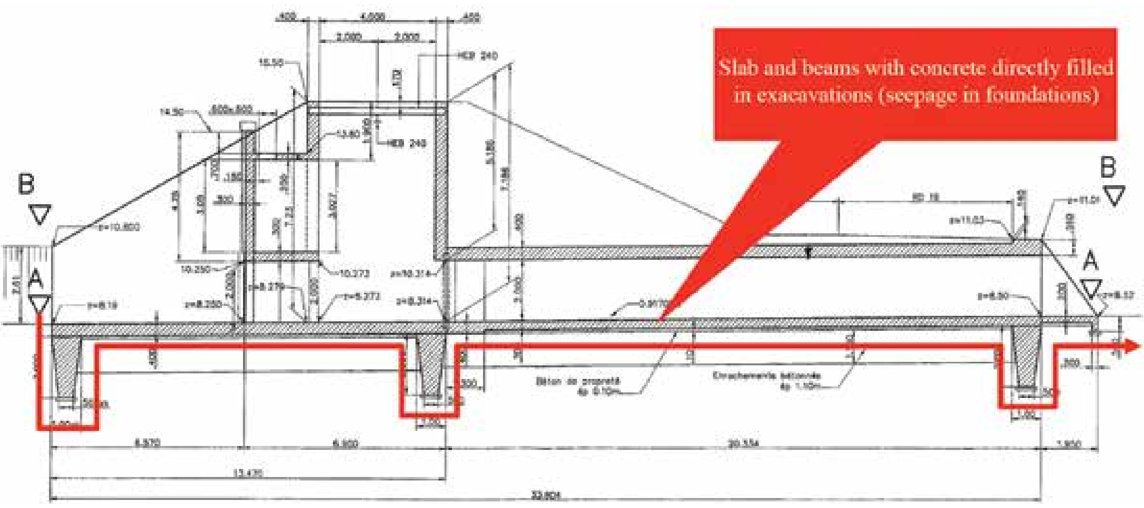

Figure 9.162 Cross-section through hydraulic crossing structure at Aramon (courtesy Thibaut Mallet)

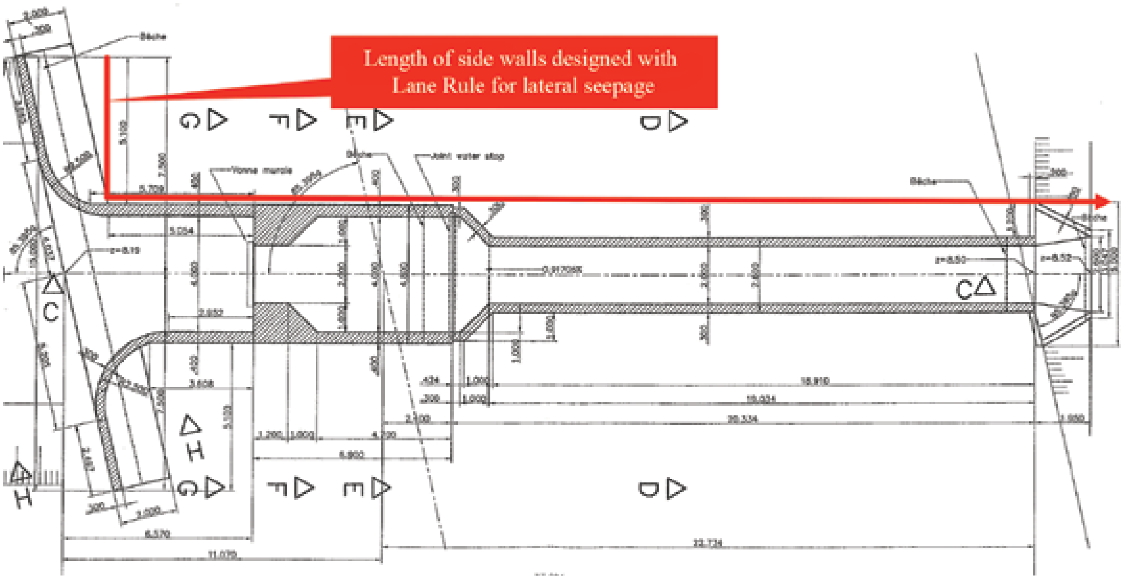

Figure 9.163 Plan of hydraulic crossing structure at Aramon (courtesy Thibaut Mallet)

Figure 9.164 Construction of hydraulic structures crossing at Aramon (courtesy Thibaut Mallet)

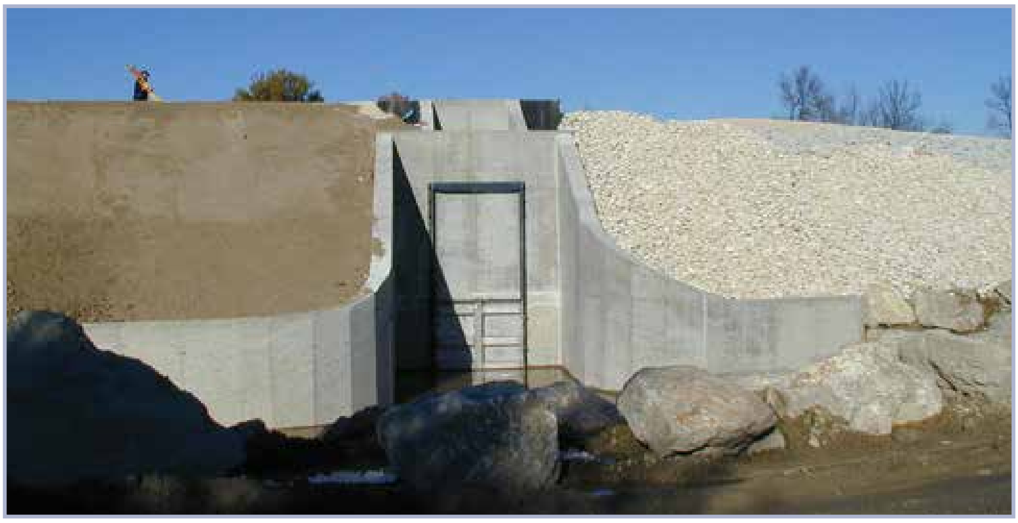

Figure 9.165 Completed structure at Aramon (courtesy Thibaut Mallet)

For pipes or culverts passing through or under levees, USACE (2000) recommends the installation of a drainage fill around the inlet one-third of the pipe length on the landward side of the levee, as shown in Figure 9.166.

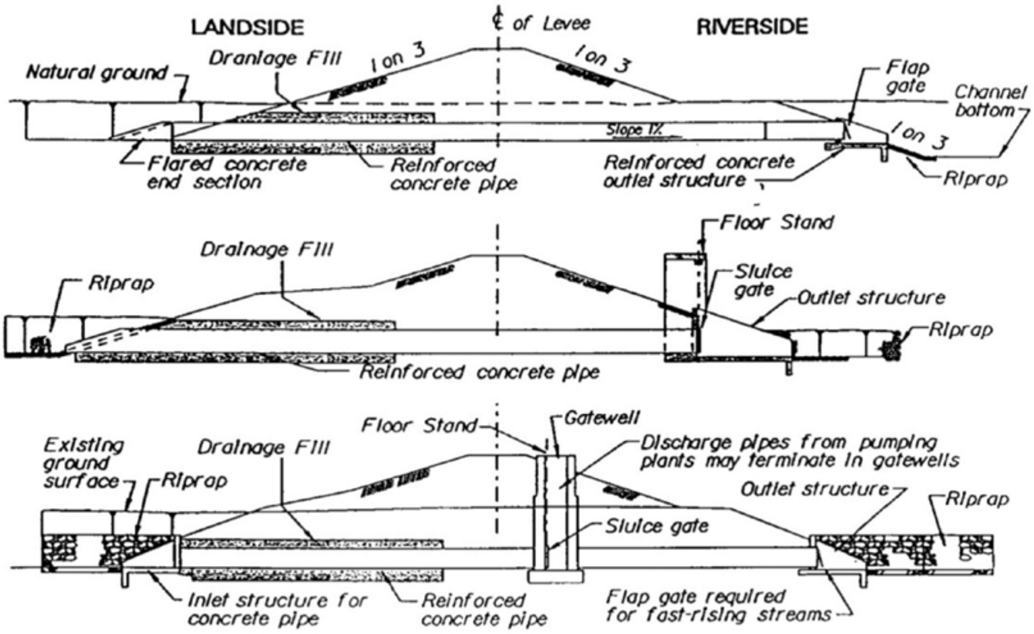

Figure 9.166 Typical sections, drainage structures through levees (from USACE, 2000)

Pipe joints

A risk-based approach is recommended to decide whether pipes should be placed in culverts or laid directly in the fill. Where a pipe with spigot and socket joints is laid directly within the fill, geotextile should be wrapped around the outside to prevent migration of fines through the joints into the pipe.

Pipe structure and durability

Pipes and culverts can corrode, lose capacity due to the build-up of sediment or debris, deform, crack or spall due to movement or overloading. The structural design of pipes and culverts is beyond the scope of this handbook, but guidance on the general issues of pipe and culvert design is provided by FEMA (2009) and Balkham *et al* (2010).

Use of directional drilling for pipe installation through existing levees

In the USA, the use of directional drilling to install pressurised pipes under levees is discouraged due to concerns about seepage and piping in the disturbed zones around the installed utility pipes. This type of trenchless technology requires an evaluation of soil and groundwater conditions, pipe entry and exit locations, construction procedure, allowable uplift pressures, on-site quality control and quality assurance monitoring during construction operation, grouting of the pipe annulus, backfilling of any excavated areas and repair and reinstatement of the construction-staging areas. General directional drilling guidance is provided by USACE (2000). Guidance for construction of pipelines beneath levees using directional drilling is provided by Staheli *et al* (1998). Guidance for construction of pipelines using microtunneling methods is provided by Bennett *et al* (1995).

9.15.4.4 Replacement of old pipes and culvert details

Need for replacement

A pipe or a culvert will be deemed to have reached the end of its useful life following a survey that shows it to be beyond economical repair. This survey may be based on 'man-entry' or remote camera inspections (Section 5.4.4, Box 5.28). Following a decision that the pipe or culvert needs to be replaced, a replacement solution needs to be identified on the basis of cost, practicality, performance and safety (from the point of view both of the operatives working on the replacement and the increased risk of flooding during the replacement works).

The choice of solution has to be influenced heavily by the reason why a replacement pipe or culvert is required. It may be that the structure has already far exceeded the original design life and has undergone a process of deterioration over this period. Alternatively, it might be a relatively new pipe or culvert that has undergone a significant and unexpected deterioration over a relatively short period of time as a result of settlement, seepage or internal erosion. The damage might be a symptom of a failure, or a process of deterioration that would not be fixed by repairing the pipe or culvert.

Data requirements

The following information is normally required for, or gathered as part of, the process of designing a replacement pipe or culvert:

- as-built drawings of the original pipe or culvert and of the new pipe or culvert after the repair or replacement, which should include longitudinal sections, cross-sections, typical details and end details
- pipe video/sonar/laser inspection reports and condition assessment reports for the pipe or culvert before repair, which should highlight any defects within the original pipe
- for applications of trenchless technology to repair/rehabilitate a pipe, a manual or laser cloud mandrel of the pipe, performed before liner pipes are purchased and brought to the site
- survey data from each end of the pipe such as global positioning system (GPS) data, which will be used to appropriately size the liner pipe (both maximum diameter and maximum segment length) that can be accommodated within the host pipe.

Selection of liner pipe

The following three slip-liner pipe materials are known to have been installed and to have performed successfully:

- **high-density polyethylene (HDPE):** solid wall HDPE pipes (ASTM D3350–12) with smooth interior and exterior surfaces are generally used. Pipe segments are joined either by butt fusion (ASTM D3261–10a and D2657–07) or by use of push-together joints with interlocking machined groves using approved sealant
- **polyvinyl chloride (PVC):** spiral wound pipe is installed using continuous PVC profile strips (ASTM F1697–09) that are wound around a form at the pipe end. Contrary to ASTM F1697–09, for levees, composite profile strips made of extruded PVC and a ferrous element necessary to provide long-term structural strength of the pipe are not considered acceptable. The joint consists of a single, mechanical interlock between profile strips (supplemented with sealant) that is created continuously as the profile is wound into the pipe. Neither the outside diameter of the pipe nor the internal diameter of the pipe should change at the location of the joint. In the USA, the joint should meet the performance and testing requirements of ASTM F1697–09
- **glass fibre reinforced plastic (GFRP):** segmental solid wall GFRP (ASTM D3262–11, cell classification Type 1, Liner 2, Grade 3) is normally used. The pipe joints should be designed so that neither the outside diameter of the pipe nor the internal diameter of the pipe change at the location of the joint. The joints should be watertight over the range of head pressure expected for the pipe. In addition, no grout should be able to penetrate the pipe from the outside during installation. In the USA, the joints should meet the performance requirements of ASTM D4161–01.

A suitable liner pipe should be selected for installation based on a detailed evaluation of the following issues:

- an assumption that the host pipe is or will be in a state of complete deterioration and incapable of providing any structural support to the liner
- the flow capacity of the slip liner
- live loads (normally only considered when the pipe installation passes under vehicular traffic or other known heavy loads)
- levee settlement and horizontal displacement deforming the pipe or culvert to the levees long-term profile after settlement
- appropriate safety factors or partial factors (from local or national codes of practice or other well-established design procedures)
- design life, which should equal or exceed that of the other structures which form part of the levee, a suggested minimum design life for replacement pipes being 50 years.

Methods of construction/installation

A number of methods that can be used to replace damaged pipes or culverts are described in Section 10.5.5.4.

Post installation inspection of replaced/rehabilitated culverts/discharge pipes

After pipe replacement or rehabilitation is complete, the pipe should be (video) inspected and its condition evaluated in accordance with the specification for the works and any relevant national standards. For example, pipe inspection in the USA should normally be carried out in accordance with the National Association of Sewer Service Companies (NASSCO) Pipeline Assessment Certification Program (PACP) standards. In the UK, similar standards are issued by the Water Research Council (WRC). Apart from anything else, these inspections will constitute a good baseline condition of the pipe for comparison with subsequent inspections for condition assessment purposes.

Decommissioning: sealing and abandonment of obsolete culvert/discharge pipes

When hydraulic analysis of the interior drainage area adjacent to a levee dictates that a pipe is no longer required as part of the flood risk management system, the pipe or culvert could (and probably should) be decommissioned, because leaving a horizontal drainage path through or beneath a levee creates a risk of future seepage or leakage.

Two methods are commonly used to deal with this situation.

1 Complete removal of the pipe, any bedding and both headwalls from the levee embankment

The excavation should then be filled with suitable, properly conditioned, compacted, fine-grained soil. The fill should be the same basic material from which the levee was constructed. The excavation should be properly benched in accordance with good practices for earthwork construction. This will help to achieve a good connection between the new fill and the existing material in order to achieve a good join and to avoid a single plane of weakness.

2 Complete filling of the pipe with a cementitious grout containing a shrinkage compensating admixture or concrete

Prior to filling, the pipe should be (video) inspected to check that there are no joint separations or holes that could result in the loss of levee material into the pipe or the loss of grout from the pipe into the levee. If it has been determined that the pipe is intact and that there are no voids or holes, then the next step will be to clean the inside of the pipe or culvert with high pressure water jets to remove any slime or earth accumulation within the pipe. Wastewater from this process may need to be captured and treated before being allowed back into the river system. The process of grouting of the pipe may be accomplished by a general contractor if there is adequate access. In general, a very high slump concrete with suitable admixtures to prevent segregation and volume change should be used as the concrete may have to travel long distances. The contractor should compute the volume of the empty pipe so the degree of filling completion can be determined. If the pipe appears to be filled but the actual placed volume is less than the computed pipe volume, then there may be a void in the pipe that has not been filled. If this is the case, then a higher level of monitoring of the sealed pipe over time may be necessary to check that there is no ongoing seepage. In some cases, it may be necessary to identify the location of any void (for example by geophysical investigation) before determining the appropriate course of action.

9.16 DESIGN INPUT – CONSTRUCTION AND OPERATION STAGES

9.16.1 Introduction

The requirement for design input does not finish when the construction documentation is complete. Designers should stay involved during both the construction stage (Chapter 10) and the operations and maintenance stage (Chapter 5).

9.16.2 Design input – construction stage

Input from the designer during the construction stage of a levee project may include:

- provision of technical input and direction to the construction stage during the execution of the works. This will involve providing appropriate documentation to inform construction (Box 9.67), but also may involve monitoring the quality of the construction operations
- monitoring by the designer by:
    - assisting in checking that the levee works are being constructed in a manner that is consistent with the documented design
    - reacting to the ground conditions encountered during the works and to highlight the need for any modification to the original design.

Box 9.67 USACE engineering considerations and instructions for field personnel (ECIFP)

An ECIFP is a brief document outlining the engineering considerations used to formulate and design. It includes the project discussions on why specific designs and materials were selected and any features requiring special attention. The document provides insight and background necessary to review submittals and resolve minor construction problems without compromising design intent. An ECIFP is used to transmit special design concepts, assumptions and instructions on how to construct unique design features and is the means of communication and co-ordination between design and construction personnel for preconstruction and preparatory meetings, submittal reviews, shop drawings, samples, certifications and test results.

The main areas requiring careful control and monitoring by the designer for levee earthworks construction may include:

- compliance of the earthworks materials (eg gradings, plasticity, organic content) with the specifications (Section 9.13.1)
- construction sequence (particularly for the case of construction on soft ground where a lack of control during construction could lead to a significant failure) – see also Section 10.5
- achievement of the specified levels of compaction (Section 10.13.2) through a combination of field and laboratory testing
- checking of the details of construction, especially the deposition and compaction of filter materials, the construction of composite structures and the detailing of interfaces between different types of structures
- quality of construction and installation of any associated structures (Section 9.15)
- quality of the finished surfaces of the levees, including any external erosion protection measures (Section 9.6).

Familiarity with the design and the site conditions suggest that the designer should also be involved in the following:

- **monitoring of earthworks behaviour** during and after construction (Section 7.9.8), especially for large new-build levees or levees adaptations constructed on soft ground. For such levees, the installation of monitoring instrumentation and the recording of phenomena such as settlement, horizontal deformation and the generation of excess pore pressures in the ground may be an important part of the construction process and may require the designer to be involved in both care of the monitoring exercise and the timely interpretation of the acquired data
- **reinterpretation of the site conditions** based on previously unforeseen conditions which may become apparent during the construction stage (eg a short length of levee undergoing much greater settlement than adjacent sections). To avoid construction delays, the designer will need to react quickly to the changed ground conditions, identify the implications for the design and adapt the design accordingly
- **updating of the design report and the O&M manual:** as explained in Section 9.3, these reports should be viewed as 'living' documents that are amended and updated as necessary. The type of information that needs to be added to either or both of these documents is:
  - as-built records (drawings and specifications)
  - cross references to instrumentation and monitoring record reports (which should be appended to the design report and, if necessary, to the O&M manual)
  - main conclusions from instrumentation and monitoring exercises and recommendations for ongoing monitoring, particularly for problem areas
  - notes, sketches and plans of areas of unforeseen or difficult ground conditions that caused problems during construction
  - notes on difficult construction conditions where the quality of construction could not be properly monitored and controlled.

9.16.3 The observational method for levee design

The observational method (Nicholson *et al*, 1999) is an emerging project delivery system that can be used to produce cost-effective designs in situations where the prediction of geotechnical behaviour is difficult. It is typically employed, for example, in the construction of tunnels, deep excavations and temporary works. However, the method could present an alternative to traditional levee design and construction approaches, potentially saving time and money without compromising safety or the quality of the end-product. It is scenario-based, requiring the project design team to anticipate a range of likely events and associated outcomes during levee construction and to bound the likely and possible outcomes and to put in place a range of contingency actions to manage scenarios within those bounds. It is also risk-based and requires that designers understand and use risk-based criteria and limit-state designs rather than traditional deterministic designs and factors of safety.

In the observational method, the following must be developed before construction is started:

- the range of possible behaviours during construction with definition of acceptable limits
- a plan of monitoring to determine whether actual behaviour is within acceptable limits
- specification for monitoring equipment with sufficiently rapid response times
- a plan of contingency actions by the constructor should levee behaviour fall outside acceptable limits.

Use of the observational method is limited to the construction stage for levees because

- it will not be possible to test the key design assumptions relating to performance during a flood within the construction period
- the performance of a levee during a flood event should not be dependent on contingency actions carried out in response to monitoring.

In order to be effective, the observational method may require more extensive site investigations to better define conditions, reduce uncertainty and assist in definition of design and safety cases for potential responses during construction. Having good quality geotechnical data in combination with the observational method allows the designer to employ less conservative design assumptions in terms of the characteristic geotechnical parameters for design. While this may increase design costs, it should result in lower construction costs and shorter construction times and should reduce uncertainty.

An extensive discussion of the benefits of the method is given by Nicholson *et al* (1999). For levees, particular benefits of the observational method may include:

- shortening of construction times through an optimisation of the rate of earthwork construction
- reduction of pause periods for consolidation and strength gain.

An example of employing an observational method during levee construction is described in Box 10.27.

Early constructor involvement in combination with the observational method during the design stage can also help to develop responses to the range of possible scenarios, further reducing uncertainty. This should also help to improve the reliability of both cost and schedule estimates and to enhance teamwork in overall project delivery.

9.16.4 Design input – operations stage

The designer's role potentially continues after the completion of the construction stage:

- to review performance problems after construction and recommend remedial solutions
- to evaluate potential improvements to the operation of a levee system, identified during operations and maintenance.

It is also desirable that any improvements in design approach identified during operations and maintenance are disseminated in order to benefit designs of new levees or levee improvements.

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10 Construction

Courtesy Association Départementale Isère, Drac, Romanche

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CHAPTER 10 CONTENTS

10 CONSTRUCTION

Chapter 10 describes organisational and practical aspects of levee construction, identifying key constraints from environment, ground and hydraulic conditions

Key inputs from other chapters

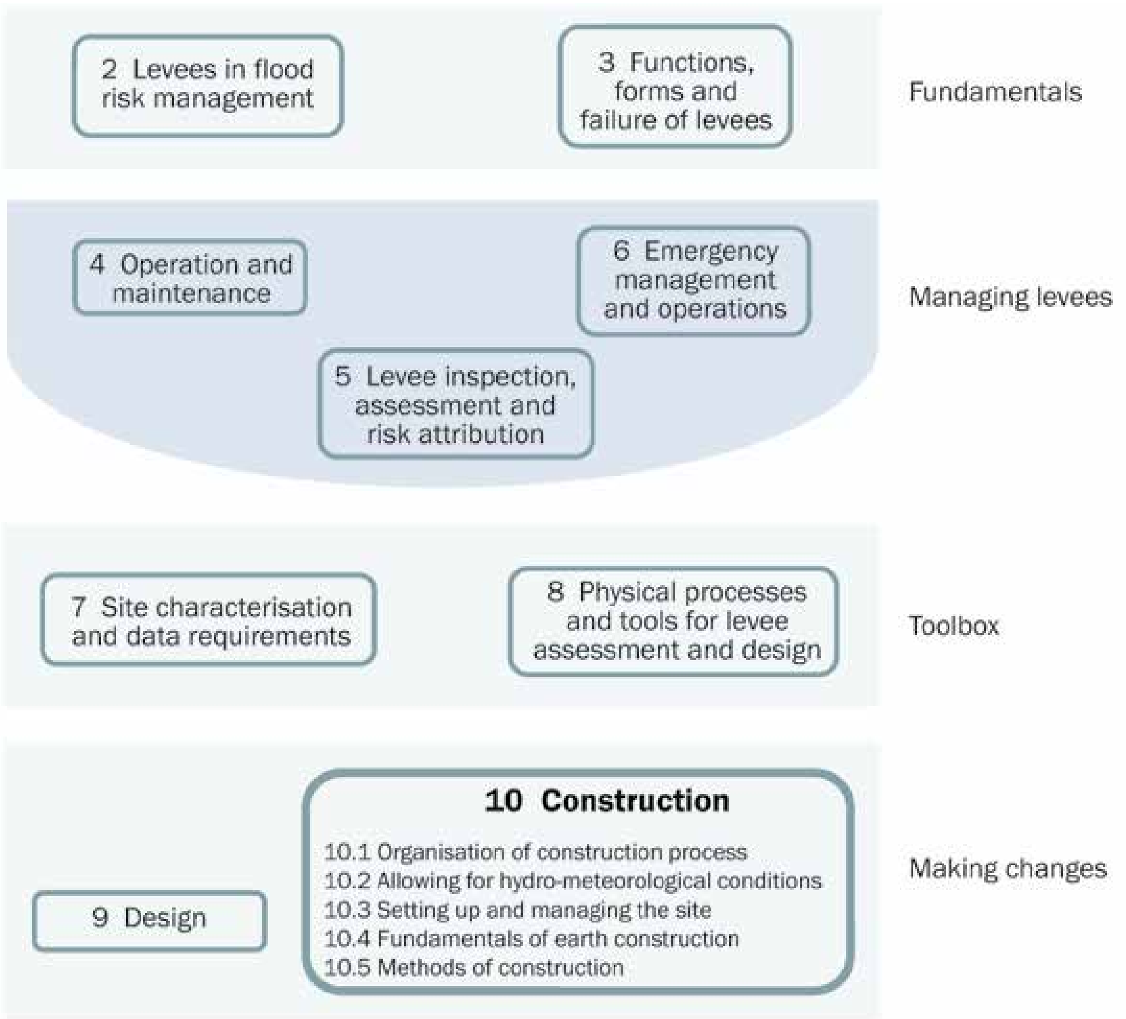
- Chapter 7 ⇨ **site characterisation and data requirements**
- Chapter 9 ⇨ **design requirements for construction**

Key outputs to other chapters

- **constructability of the design** ⇨ Chapter 9
- **data related to construction** ⇨ Chapters 4, 5, 6 and 9

Note: The reader should revisit **Chapters 2 and 3** throughout the levee life cycle for a reminder of important issues.

This flow chart shows where to find information in the chapter and how it relates to other chapters. Use it in combination with the contents page to navigate the handbook.



CHAPTER CONTENTS AND TARGET USERS

This chapter is divided into five sections, providing an overview of levee construction.

Organisation of construction process

Section 10.1 describes the context of the construction process. Important issues such as planning a project, the management of quality, environment and safety are addressed along with the data relevant to construction.

Allowing for the hydro-meteorological conditions

Section 10.2 describes the hydro-meteorological conditions that influence all construction activities and should always be considered in order to work safely, to limit flood risk for the leveed area and the work area and also to guarantee the workability of the soil being used for construction.

Setting up and managing the site

Section 10.3 describes the set up and management of a levee construction site. The importance of allowing for local residents and the environmental context is detailed as well as good practices for site management, the management of materials, access routes and traffic, archaeological remains and utilities.

Fundamentals of earth construction

Section 10.4 describes the fundamentals of earthworks construction, including the availability and suitability of earth materials and how they are handled by equipment to carry out elementary operations.

Methods of construction

Section 10.5 describes methods of construction at the scale of the whole levee. These methods include the construction of a levee test section, the stages of a levee construction for new build, adaptation, repair and decommissioning, the instrumentation and the integration of non-earthwork features into levees.

10.1 ORGANISATION OF CONSTRUCTION PROCESS

Section 10.1 sets the context of the construction to give a clear understanding of procurement approaches, inputs from the various design stages and legal responsibilities (see Section 10.1.1). Then details are given related to planning the project (see Section 10.1.2) and managing the risks to the construction programme (see Section 10.1.3). Finally, the framework to ensure quality, occupational health and safety, environmental management, and data acquisition and management are introduced respectively in Sections 10.1.4 and 10.1.5.

Note

Throughout the handbook the term 'constructor' is used instead of 'contractor' to avoid describing the construction stage only in the context of a traditional contract. Although 'contractor' is more commonly used, 'constructor' can be used in every situation. In a similar fashion, 'designer' is used instead of 'engineer' due to the limited number of situations 'engineer' can be applied. In the event the designer is not in charge of supervising the works, the person in charge is called the project manager.

10.1.1 Setting the context

A constructor requires three types of information in order to plan the levee construction:

- type of contract
- inputs from the design stages
- licenses and permits that have been obtained and those that must be sought.

10.1.1.1 Procurement approaches

Levee construction is normally procured by some type of construction contract. There are a variety of contract types available and in use by government agencies and private constructors. Table 10.1 lists some advantages and disadvantages of using each contract type to procure levee construction services. This table also suggests the type of levee projects, which typically favour each procurement approach. See Section 10.5 for a broader description of the types of levee construction.

Table 10.1 Typical procurement approaches

Contract type ***Suggested levee project scope***	**Advantages**	**Disadvantages**
Design-bid-build Suitable for risky or complex new levee or adaptation, using qualified design agents, where execution time is not critical and budget is a priority.	Separate design and construction agents with individual specialties. Increased opportunity for qualifications screening and competitive bidding of agents.	Involves delays for sequential awards. Requires owner to co-ordinate between designer and constructor. No constructor involvement during design phase.
Early constructor involvement Suitable for risky or complex new levee or adaptation, where execution time is critical and budget is not a priority.	Separate design and construction agents with individual specialties. Constructor involvement during design phase. Shorter execution time due to concurrent awards.	Requires owner to co-ordinate between designer and constructor.
Design-build Suitable for simple new levee, adaptation or repair, where execution time is critical and budget is a priority.	Owner deals with a single entity. Shorter execution time.	Owner has less control over design process and specification.
Design-build-fund-operate Suitable for levee building with low financial risk or adaptation, where owner funding options are needed.	Owner deals with a single entity. Shorter execution time. Owner has more funding options, including amortising costs until user fees are generated.	Owner cedes control over the entire process to the selected constructor.

Table 10.1 Typical procurement approaches (contd)

Cost reimbursable Suitable for emergency repairs or very simple adaptations, or repairs with low design requirements.	Owner can direct changes as work progresses.	Owner assumes risks of cost and programme performance.
Direct labour Suitable for emergency repairs, or very simple adaptations or repairs with low design requirements.	Owner has unfettered control over all project details.	Owner assumes all risk of cost, programme, and technical performance.

One of the options in Table 10.1 is early constructor involvement. The advantages and disadvantages of this approach are set out in more detail in Table 10.2. Where the disadvantages outweigh the advantages, the levee owner and the designer should consider the direct employment of an individual with appropriate construction experience.

Table 10.2 Advantages and disadvantages of early constructor involvement

Advantages	Disadvantages
Cost and construction duration savings for levee projects that are: • large or complex • have challenging ground conditions • require technical inputs from many professional disciplines • need additional proposed options.	Creation of unfair tendering conditions for levee construction contracts by the constructor that provides early input gaining early information on critical project issues.
Shortened construction programmes for levee projects that have constrained schedules.	Inadvertent tendency for a construction company to identify construction techniques that suit its own equipment and staff.

10.1.1.2 Inputs from the design stages

The main inputs into construction from the design are the following:

- specified objectives for the levee, both when delivered to the owner at the end of the work, as well as during the construction phases
- information on environmental constraints, on construction methods and site management
- main characteristics of the levee: alignment, layout, geometry, stability, serviceability and durability
- the overall construction programme, in particular any allowable delays. Table 10.3 presents some examples of ways of dealing with contract programme constraints and delays to the programme.

Depending on the procurement approach, the constructor may be more or less involved in the design.

The owner and their designer may also have established a public relations strategy during the design phase, for example:

- some public meetings may have been organised to discuss the project programme, goals, construction methods, impacts on local transportation systems, and interim and long-term benefits to local residents
- regular updates on project progress via local news media outlets may have been issued.

During the construction phase, the constructor is likely to be strongly involved in the implementation of this strategy.

Table 10.3 Examples of ways of dealing with constraints and delays

Issue	Contract information	Constructor good practice
Inclement weather	• the contract documents should clearly define what are considered weather delays • the constructor's programme should account for 'normal' weather days when construction will not be possible.	• constructor should protect equipment at all times and should make arrangements if inclement weather is expected • forecasted weather delays can be obtained by the National Weather Service and included in the contract duration.
Adverse impacts to environmental or cultural resources	• the contract documents should identify any environmental or cultural resource issues that are known at the time of tender.	• proactive partnering between the constructors, owners, and owners' representatives could expedite and address impacts.
Funding interruptions	• the owner should fully disclose to the constructor the funding arrangements and how the constructor will be compensated on a monthly basis.	• proactive partnering between the constructors, owners, and owners' representatives could address funding issues.

10.1.1.3 Permits and licenses

Levee project licences (permanent permissions) and permits (time and location limited) are usually required by government organisations that authorise civil works projects and provide guarantees for funding. After completion of the plans and specifications, central and/or local governmental organisations or departments review the project plan and provide general permits and licenses in order to comply with the applicable regulations before construction of the levee.

It is important to emphasise that these permits and licences often have conditions attached such as those presented in the following sections.

These general permits or licenses are obtained by the levee owner or operator, or the construction manager. The constructor may be responsible for obtaining additional permits and licenses required by their method of working or alternative solutions. This latitude is provided in the expectation that the constructor may propose acceptable alternative methods or materials for the construction process that result in reduced cost or shorter construction time, or that are more environmentally friendly. The involvement of an experienced constructor with demonstrable local knowledge will reduce the risk of delay due to licenses and permits.

Many permits can be obtained by the constructor soon after award of contract. However, unexpected problems and constructor proposed solutions or alternatives may require additional permits or licenses from local or central government organisations. Permitting issues that arise after a constructor has initiated work can be considered a significant consideration to successful and timely completion of project. Failure to anticipate the impact of the time and data requirements to satisfy legal and environmental requirements of a permitting process can result in adverse impacts to construction schedules and potential legal actions and resulting fines for the constructor or claims to the levee owner.

Before mobilisation, or after, as necessary, the constructor should contact the levee owner to explain their intentions, and the local planning authority and relevant regulatory organisations to determine legal requirements for licenses and permits for all scheduled construction activities that vary from the original plan. Application for permits and licences should be initiated well in advance to provide adequate time to complete the permit process. Extensive data collection and analyses may be required to accommodate constructor proposed construction activities or acquisition of additional land.

Constructors may be granted permits and licences for the following aspects of the construction project.

Permits and licences for use of additional land

Proper land acquisition is critical to the success of the construction project and should be planned accordingly. There are many reasons for such land acquisition:

- permanent loss of land due to flood protection works
- temporary loss of land due to temporary works
- temporary loss of land due to flood storage.

The constructor may determine that lands proposed for levee construction by the owner are inadequate to facilitate the preferred construction activities. Constructor preferred routes and lands may be subject to restrictions set by a local planning board. However, planning boards may issue temporary permits under special circumstances that include landowner consent, financial reimbursement guarantees and security bonds. Temporary permits and or licenses may be necessary for:

- rights-of-way and permission to construct borrow pits and temporary material sorting areas in wetlands or near residential areas
- construction of haul roads across flood ways
- expansion of rights-of-way adjacent to levees for increased vehicle access and turnabout
- additional lands required for temporary materials unloading and storage capabilities in wetlands or adjacent to residential and other developed areas
- alternative borrow and disposal areas not designed by the owner or local planning boards that will provide more material or will shorten haul time.

Good practices include:

- the owner will have carried out pre-application discussions with the regulatory authorities and should initiate the permit process upon completion of a preliminary set of plans that show the alignment of the levee and the location of major structural water control features (expansion or larger footprints should be considered when planning the project and adequate land should be purchased for the permanent works)
- at the start of construction planning, the owner or the constructor carries out dialogue with permitting/licensing authorities to establish requirements and duration of approval period
- upon completion of the final engineering plans and specifications, field conditions may have been discovered that require alterations to the preliminary plans provided to government agencies that issue building permits.

Permits for special equipment and material

In the course of levee construction, specialised construction equipment, or scheduling activities may require permits not anticipated during the planning process or previously secured by the owner. Permits may be required:

- to operate mobile crushing equipment (noise and dust abatement issues)
- to transport large equipment on specialised carriers
- for waiver of restrictions that limit vehicle size, wheel weight and type to use city and rural traffic roadways
- to cross railway lines and bridges
- for temporary construction activities, or within or adjacent to an existing road or utility
- for temporary road closure to transport equipment.

Environmental permits and licences

The construction manager or designer will have carried out an environmental assessment and environmental impact study before the construction project begins (see Chapter 9). This should comply with all appropriate national and international environmental regulations and should be as detailed as possible to avoid future delays and costs. It is important that the constructor be given strict guidelines to follow that concern easements, access, construction techniques, construction working seasons and hours, and materials acceptability based on the results of the environmental studies. Any environmental considerations on the site that need to be protected by the constructor, or that may necessitate special working arrangements should be clearly identified in the environmental assessment and included in the construction contract.

Local, regional or national requirements for permits are often required:

- to work in vicinity of local residents and of the environment (noise and light level requirements)
- for water and air quality requirements (eg permits to abstract or discharge water, permit for use/storage of fuels/hazardous materials, permits to process materials)
- to process materials
- for vessels to use waterway.

Good practices include:

- carry out a review to identify all permits and licenses that may be required to construct the work
- carry out dialogue with permitting/licensing authorities to establish requirements and duration of approval period
- upon completion of the final engineering plans and specifications, field conditions may have been discovered that necessitate further permits and licenses to those originally considered.

10.1.1.4 Roles and responsibilities

There are a number of considerations and risks that can apply to a levee project. It is important that these are addressed at the appropriate stage of the project, and that there is clarity as to who takes responsibility at each stage.

Table 10.4 identifies the:

- key construction stages
- key levee construction activities in each stage
- project parties that take lead responsibility and provide key input for these considerations.

Table 10.4 Key construction stages, activities and issues, and lead role responsibility

Key stage	Key activities	Lead role
Consultation and construction planning	• environmental assessment • levee operational requirements • permitted work hours • light/noise restrictions • operation issues during construction • development of the programme • development of project risk register • development of construction management plan • public consultation.	• designer • constructor. with input from: • owner/operator • project manager • regulator • public works utility • public.
Construction input during design	• develop acceptance criteria • constructability review with constructor • develop levee construction methodology • development of the construction programme • development of construction risk register • development of the initial budget cost • construction trials, in particular proposed embankment material.	• designer • constructor. with input from: • owner/operator • regulator • project manager.
Tendering and award of contract	• issue of tender documents • address constructor questions • contract administration protocols • required permits acquired for the construction of the levee	• designer • construction manager. with input from: • owner/operator • project manager • designer.
Mobilisation/start-up	• access to the levee site • environmental considerations (flora and fauna, dust, noise, runoff etc) • flood management operations during construction • safety plan review and approval • design considerations meeting.	• construction manager • constructor. with input from: • owner/operator • designer.
On-site construction	• communication protocols • review and approval of submittals • weekly/monthly meetings • management of construction programme • management and mitigation of project risks • measurement and payment (contract administration) • levee foundation inspection/approval • inspection of major electrical and mechanical equipment • independent evaluation of construction management activities.	• construction manager • project manager. with input from: • owner/operator • designer • public works utility.
Acceptance of work	• specifications definition of ‘acceptance’ and ‘substantially complete’ • quality assurance • final inspection • commissioning of major electrical and mechanical equipment.	• construction manager • project manager. with input from: • owner/operator • designer • regulator.
Handover and post-construction	• completion of snagging and outstanding works • site restoration • environmental compliance testing • as-built drawings • technical documentation of construction • training • transfer of works • resolution of contract issues • final payment/release of claims	• construction manager • constructor. with input from: • owner/operator • designer.

Levees are critical in the protection of the public from catastrophic flood losses, yet by their nature involve variability in site conditions and construction materials. Accordingly, good practice for levee construction relies on roles and responsibilities, which address unique levee characteristics of high risk and changeable conditions.

Construction of levees involves the conventional earthwork and heavy construction industry, in addition to specialists involved with waterways, environmental and flood protection projects.

Many owners follow the tenets of the 'total design and total construction' process where persons from the design and construction teams are integrated in both project phases. The responsibility for risk management is transferred from the design team to the construction team when the project is tendered. However, representatives from design should be involved in the construction phase to confirm the design intent is met. Depending on the legal framework, the responsibility rests with the owner (principal) or with the constructor. Risks can be technically related to executing the design (project features and adjacent structures) and can be related to the process (stakeholders, public). Risks can manifest themselves as delays, additional costs from claims, damage to adjacent property, and loss of life.

The perception of risk during construction may be very different, depending on the position of the audience. The designer, for example, may have prepared the contract documents based on a set of assumptions that may or may not be borne out during construction. So, contracts should be written to make clear the ownership of risks such as:

- potential limitations in geotechnical investigations, since differing site conditions if encountered will affect both design and construction
- potential for more severe weather delays than allowed for under the contract.

10.1.2 Project planning

Construction of a levee involves many different engineering and construction disciplines. The definition of the construction process requires input and co-ordination of responsible professionals with experience in construction of levees. Construction issues and challenges are best resolved by an experienced multi-disciplinary team that includes the hydraulic and geotechnical engineers from the design team, the owner designated project manager, the construction manager representing the constructor, constructor's staff with experience in heavy construction, and input from local government agencies familiar with local regulations and environmental restrictions. The multi-disciplinary team should reach agreement and recommend best procedures and methods to overcome the identified challenges with the least impact on the local residents and environment.

The objective when planning the levee works should be to develop a robust construction programme for the levee project. This should use realistic/achievable outputs based upon real experience and take account of the project and planning considerations to provide the assessment of the duration of the actual works. The earlier these planning considerations are fully understood during the development of the project the sooner a realistic budget or cost can be achieved.

The key personnel to input into establishing realistic/achievable outputs are those who have actual knowledge and experience of construction of levees, and where possible should include an experienced constructor's operational and planning staff. It is also valuable to have input from the owners' local maintainers to understand the local considerations that may affect the project.

The programme should be defined progressively in co-operation with the owner:

1. The constructor should produce a detailed baseline construction programme before work starts.
2. The owner and constructor should review and agree that the baseline programme is accurate and reasonable.

3 Any programme impacts can then be evaluated from the baseline programme.

4 Proactive partnering between the constructor, owner, and owner's representatives should help to expedite resolution of any scheduling or subcontractor performance issues.

5 Financial penalties (so-called 'liquidated damages') may be imposed by the owner for constructor caused delays.

Key steps in planning the levee construction works are discussed as follows.

10.1.2.1 Project specific programme constraints

The programme needs to be suitable for the procurement approach (in particular any key dates and allowable delays) and needs to take account of the considerations, key dates and risks that are likely to affect the construction works and the methods and resources that are to be used. Obtaining some permits and licences can generate some delays in addition to the delays linked to physical constraints.

Levee construction is unique because the project site can include varying soil and ecosystem conditions, multiple land ownerships and uses, the potential for causing unintended flooding consequences and legal requirements that can pose significant construction challenges. Special considerations for a construction project to allow for are:

- hydro-meteorological considerations, including the constraints for works, flood risk for the leveed area and for the work area and specific risk to earth structures (as presented in Section 10.2)
- constraints from local residents and the environment (habitats, flora and fauna) that can limit the working hours and require some measures to preserve air and water quality (see Section 10.3.1)
- access routes and traffic (see Section 10.3.2)
- archaeological remains and utilities (see Section 10.3.3)
- availability and suitability of materials (see Section 10.4)
- methods of construction (see Section 10.5).

Without a basic understanding and appreciation of these considerations, the construction project will likely result in significant delays, unexpected high cost and/or even damage or loss of material, equipment and environmental values. Some examples of different types of considerations and risks are given in Table 10.5.

10.1.2.2 Checking the constructability

Although the designer should ensure in general terms that the levee project is viable for construction, the levee constructor should check the 'constructability' based on a review of project objectives, funding and programmes and their own experience, capabilities, materials and equipment. The review should include a detailed understanding of the purpose of the completed project, its operational function, and the risk associated with each phase of construction. A determination of levee constructability requires a higher level of planning and knowledge of the site than most construction works. The construction will obviously be influenced by requirements such as:

- assessment of the environmental constraints
- assessment of borrow areas and evaluation of alternative borrow material sources
- secure and alternative site access ways
- programming of construction phases to minimise risk of flooding adjacent lands and violating environmental regulations regarding issues such as sedimentation and erosion control.

Table 10.5 Construction sequence considerations

Construction sequence issues	Good practice
• working around environmentally sensitive areas (eg no work in a particular area during the mating season of a threatened or endangered species).	• identify all sequencing considerations during the planning phase and highlight them in the contract so the constructor has adequate information to sequence the work activities • sequence construction in environmentally-sensitive or fragile areas based on information contained in environmental documents.
• availability of land for the project.	• ensure proper title is available for purchased land when planning the work (note that in the USA, some levee projects cover a very large geographic area with multiple construction contracts. This requires that the land acquisition be staged and sequenced).

• rate of loading of embankments to permit dissipation of pore water pressures.	• considered methods for phasing construction of earthwork on soft soil:
	  ◦ increasing the height in stages, with a period of consolidation between the stages
	  ◦ increasing the height in one stage, with controlled squeezing of soft soil layers
	  ◦ increasing the height in one stage, after soil improvement.
	• additional measures to minimise delay during phasing of earthwork such as:
	  ◦ temporary application of extra weight to accelerate settlements
	  ◦ the introduction of vertical drains in order to improve the consolidation process
	  ◦ stability improvement by using geotextiles, geogrids or geotubes
	  ◦ construction by using light materials
	  ◦ 'undercutting' or over-excavation of foundation to remove soft or unsuitable materials and replacement with suitable embankment.

• potential for reduction in cross-section of the river channel, leading to flooding • vulnerability to flood erosion of the partially completed works.	• if there are certain work items that should be sequenced such as installation of temporary flood protection prior to levee excavation, it should be clearly defined in the contract • consult hydraulic designers to ensure that sequencing does not create adverse flow conditions.
• programming to provide flood control benefits as early as possible.	• maximise the benefits of each segment of the levee project. Ideally, each levee reach should provide flood protection benefits. Long levees may intercept high ground elevations that would provide full or partial design flood protection before completion of the final segment of a levee scheme.

10.1.2.3 Developing the methods, resources and corresponding construction outputs

Once the constructability has been checked, a realistic plan can be developed that includes:

- adequate logistics
- development of appropriate construction procedures
- careful selection of construction equipment
- planning appropriate flood protection measures to minimise risk of increasing flood levels upstream and downstream of the project and reducing erosion damage to partially constructed features.

The stages of construction of a levee, as further described in Section 10.5, are similar to those for many other major civil engineering projects, but some considerations may have particular significance on a levee project. A typical example of construction sequencing is the phased raising of the levee when working on soft soils, to avoid loss of stability during the construction phase.

Labour, material and equipment should be analysed and planned taking into account the project requirements. If the constructor's planned construction activities are not properly sequenced, the levee project may be delayed and costs increased significantly. Table 10.6 provides some examples.

Table 10.6 Material/equipment/labour availability considerations

Material/equipment/ labour availability issues	Good practice
Availability of necessary materials and resources for the project	• projects should be scaled to ensure that equipment and labour shortages are not encountered. However, if shortages are anticipated all project selections should be based partially on ability to provide equipment and labour • if possible, projects in the levee system should be phased and programmed to avoid peaks in demand for material, equipment and labour • during the planning process, preliminary material quantities should be used to analyse the local supply chain for delivery and price. Analysis should focus on effects of increasing demand with current supply chain • consider pre-ordering particular material, prior to the start of the contract, and provide these materials to the constructor.
Alternative materials and methods	• the owner may also allow the constructor to submit a request for variance or value engineered alternative to the specified material in the contract.

Further considerations are given for embankment material availability and suitability in Section 10.4.

10.1.2.4 Defining the associated costs

The most reliable way of developing a realistic construction budget for any flood alleviation project is to fully resource the developed construction programme. Generic 'schedule of rates' budgeting is unlikely to provide meaningful information.

Resourced programme based budgets are more reliable because the programme will be based on the:

- various specific considerations, such as seasonal working
- outputs derived for the actual methods and planned equipment
- actual anticipated sources of materials
- construction contracting environment (high cost versus low cost environment).

The process for arriving at a programme based cost is illustrated in Figure 10.1, and is summarised as:

1. Establish costs of the actual resources, durations and quantities from the resources programme.
2. Base the site overhead costs (sometimes referred to as 'the preliminaries') on the actual forecast resources and durations.
3. Assess residual design risks and construction risks and appropriate allowances established.
4. Include realistic constructors overhead profit risk.
5. Include allowances for escalation.

Experienced construction staff are essential to provide input into the costing of the works.

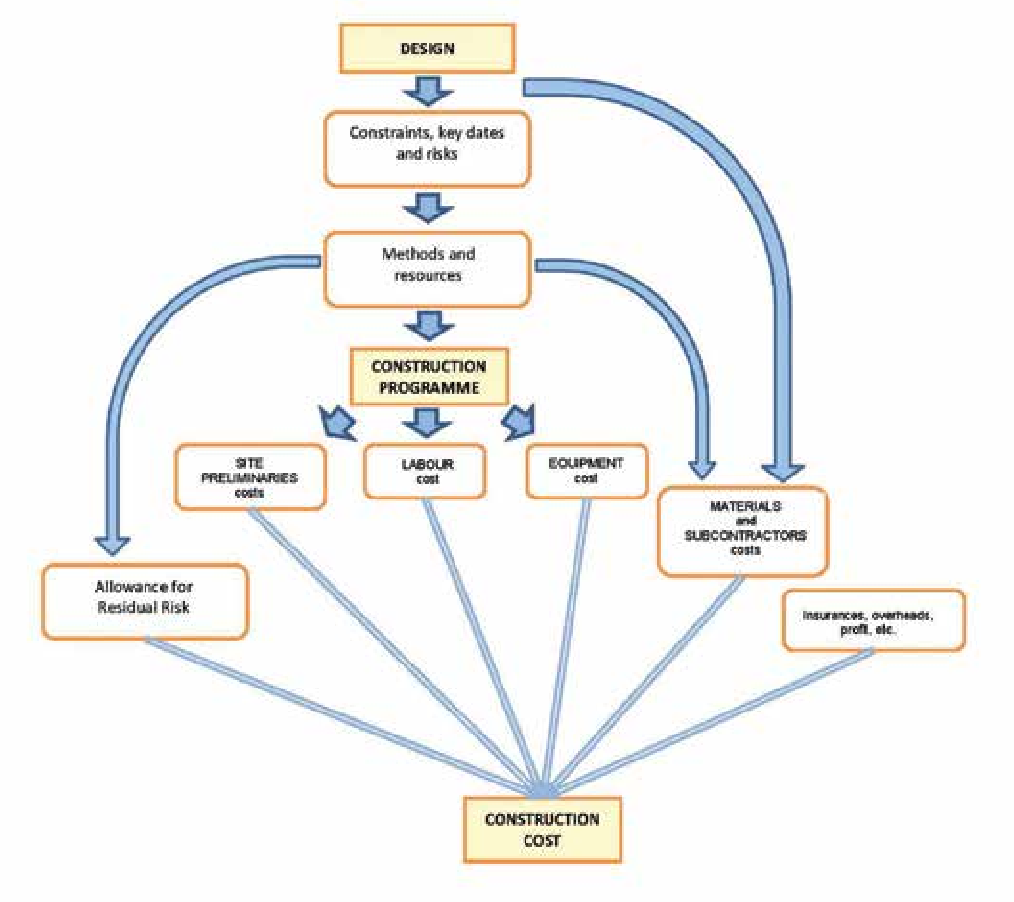


Figure 10.1 Development of the construction programme and project cost (courtesy Bam Nuttall)

10.1.3 Managing risks to the construction programme

Risk management during the construction phase should be based on continuously monitoring risks in time. This will enable a swift response to the development of existing risks, the emergence of new risks, and/or a change of risk acceptance by stakeholders and/or the public.

During the construction phase, levee projects rely on careful co-ordination between the designer, project manager and constructor to accurately assess and react to changed site conditions. Levee projects involve the prediction, assessment, confirmation and manipulation of geotechnical and other site specific elements, many of which are hidden by nature. Site condition assessment occurs before and during construction phase, so design often needs to be adjusted to conform to differing conditions. When a significant change is detected during construction, additional parties are often involved including the owner, regulators, public and funding agents.

A number of the risks that can arise on levee construction projects are set out in Table 10.7, together with examples of good practice for dealing with these risks. Some of the risks in this table can be dealt with by advance planning. Others relate to unknown or changed conditions as it is often not feasible to inspect and plan for all existing conditions due to the size of large projects and/or the complexity and uncertainty associated with very old levee systems. Unforeseen conditions can also stem from archaeological remains and utilities, good practice for these is discussed in Section 10.3.3.

In the event of delays and programme overruns, it is preferable for all the stakeholders to negotiate an additional period of time that still allows the design requirements to be met than to accelerate the construction and jeopardise the eventual performance of the levee. So, it is important that the constructor has a multi-disciplinary team adaptable enough to react if the programme requires modification.

Table 10.7 Good practice in dealing with some typical levee construction risks

Risk to construction	Good practice
Hydraulic design	There are a variety of negative outcomes associated with poor hydraulic design including, but not limited to, scour, erosion, head losses through pump station screens, inadequate pump capacity etc • modelling program to confirm design results can often be money well spent early in the construction phase • if required, physical flow test.
Unusual soil conditions	Seeking out experts with experience dealing with unusual soil conditions can improve the chances to deal with settlement and other issues.
Unforeseen ground conditions	• proper geotechnical investigations should be performed during design phase. The foundation should be mapped and documented by a geologist • constructor should perform a full site investigation before starting work and identify any inconsistencies from the contract drawings • the owner, owner's engineer, and the constructor should be involved as the foundation for the levee is excavated and exposed • if foundation conditions are not as anticipated, the project design team should be proactive in assessing whether the design assumptions are affected. Immediate action is required in the case of changed site conditions to avoid major delays in the construction programme • implement a robust and thorough project risk assessment and monitoring process.
Unexploded ordinance (UXO)	• a detailed review of historical records • ground radar and/or employment of UXO experts where risk is perceived to be significant.
Structure and foundation Scouring and/or undermining/overtopping	Use of splash pads inside the line of protection or proper design and installation of toe protection can reduce the risks associated with scour, undermining, or overtopping the structure both during and after construction. The same is true for temporary structures that may be constructed as part of the project, but are removed before contract completion.
Excessive or differential settlement	Implementing a thorough geotechnical investigation and testing program can address the potential for settlement. During construction, measures such as surcharge fill placement can speed settlement. Wick drains, pile foundations and expansion joints in structures, and other measures can minimise settlement impacts.
Seepage	Use of continuous sheet pile cut-off walls is one measure that can help ensure integrity of the line of protection.
Concrete integrity	Cracking and minimum strength: ensure that the concrete meets specified compressive strength requirements through a rigorous QC testing program, that an adequate amount and type of reinforcing is installed for crack control, and that the proper minimum amount of concrete cover is provided to the reinforcing materials through the inspection program.
Testing and commissioning	Levee systems typically contain a number of mechanical and electrical systems that, if not procured and installed properly, can delay the online date for the project, result in increased cost to the owner or constructor, and reduce overall project effectiveness. • develop a testing and commissioning plan early in the process • dedicate a commissioning manager to oversee this effort • document testing results and corrective actions and co-ordinate witness testing with the owner or their representatives • take corrective actions, as necessary, before turnover or start of warranty.
Social and political risks (such as actions of non-governmental associations that impact on the works)	• thorough consultation and public liaison with all persons and organisations who may be affected by the works or who may affect the works • follow the appropriate risk allocation procedures.
Unforeseen environmental issues	• adaptation of the construction programme should be agreed with the relevant stakeholders • Box 10.1 gives an example of a levee construction project impacted by fauna change following start of construction.
Changed requirements (including owner instructed change)	• anticipating by choosing adaptable and/or robust designs during the design phase of the project • early involvement of the constructor.

Box 10.1 Responsibilities for managing site changes during levee construction

Figure 10.2 Caracaras (courtesy Glen Tepke)

During construction of a flood control project in Florida US, a pair of caracaras, an endangered bird species, began nesting in the project area. In order to comply with environmental law, the construction equipment and activities were severely restricted during nesting season, impacting the programme. Managing this change involved the combined efforts of the owner, regulator, designer, constructor and project manager.

10.1.3.1 Use of a risk register

A good way of managing risks (and accepted good practice) during construction is the creation and use of a risk register. This is a register of risks that not only documents the thought process of identifying and recording potential risks to the construction project but also sets out risk mitigation options and can be referenced and updated throughout the project, ie a 'living document'.

Before entering into a detailed assessment it is important for the constructor to recognise that higher strategic assessments may be necessary such as:

- establishing the overall project objectives and the business risks that could have an impact on the achievement of these objectives
- whether these business objectives are met or will be compromised by entering into the project.

Once this analysis has been completed, steps 1 to 9 here, which are focused on the construction risk, can follow:

1. Identify hazards and risks.
2. Consider the ownership of the risks.
3. Assess the likelihood and consequences of these risks.
4. Identify control/mitigation measures.
5. Assess the residual risks including new risks that are created by the mitigation measures.
6. Estimate the cost of the mitigation measures.
7. Estimate the net benefit of the measure.
8. Select and implement beneficial mitigation actions.
9. Monitor and review the process/feedback into the cycle.

Simm and Cruickshank (1998) describe construction risk assessment and these steps in detail – the results of which should be recorded in the risk register. An example of a partially completed risk register is shown in Table 10.8, which includes the outputs of steps 1 to 5. More extensive risk registers can include steps 6 to 9.

Table 10.8 Example of a partially completed risk register

No	Risk description	L	C	R	Risk mitigation measure	Residual risk		
						L	*C*	*R*
1	Severe flooding of site beyond what might reasonably be expected	Moderate	High	Undesirable	Improve flood mitigation measures – consult designer(s) on possible construction sequencing to reduce risk of inundation of site. Plan for rapid removal of staff, plant and equipment from site. Assess flood risk on a daily basis.	Low	Low/ medium	Acceptable
2	Uneven/excessive settlement of fill materials	Moderate	Medium	Undesirable	Consult results of ground investigation for each levee segment before fill placement, and monitor each placement, consult site engineer and/ or designer to check acceptability or advice on remediation.	Moderate	Low	Acceptable
3	Excessive sediment discharge from site to water body	High	Medium	Unacceptable	Discharge dewatering water to settlement pond. Monitor water quality at outlet.	Low	Low	Acceptable
4								

Notes
L = likelihood, C = consequence, R = magnitude of risk

Care is needed when combining likelihood and consequences to take account of the importance of the risk. Although the product of likelihood and consequence gives the mean impact of risk, in many cases the variance can be very significant. Typically, high consequence, low-likelihood risks will be more important to manage than low-consequence high-likelihood risks even though they have the same expected risk impact.

10.1.4 Quality, health and safety, and environment management

A successful levee construction project includes the safe completion of the build to the required standard(s) with minimal effect on the environment. The quality, occupational health and safety, and environmental requirements for levee construction should be to the standard required for all critical public infrastructures. A construction company or organisation should be able to demonstrate that it has an effective management system (or systems, preferably certified) in place for quality, occupational health and safety, and environmental management requirements.

In levee construction the effective management of activities and data are critical for:

- verifying the use of the proper types and use of construction materials
- verifying the proper use and correct application of construction equipment and methods
- realising the design objectives and expectations
- legal compliance and other requirements (eg policy).

Quality management is primarily established to assure the levee is constructed in accordance with the specification, with the constructor processes and procedures, and complies with minimum industry standards and regulatory and other requirements. Successful construction provides flood risk reduction, while defective construction can lead to undesirable consequences – both for the constructor, and the client. The performance required will generally be specified by the designer, while further prescription of specific management requirements may be outlined within the construction contract. Levee constructors should have a quality management system (QMS) in place and ideally this should follow ISO 9001:2008 or equivalent.

Occupational **health and safety management** is primarily established to assure the levee is constructed in a safe manner in accordance with industry standards and regulatory requirements, and company policy, processes and procedures. Levee constructors should have an Occupational Health and Safety Management System (OHSAS) in place and ideally this should follow OHSAS 18001:2002 or EM 385-1-1 (as used by USACE) or equivalent.

Environmental management is primarily established to assure the levee is constructed with minimal impact on the environment. Levee constructors should have an Environmental Management System (EMS) in place and ideally this should follow ISO 14001:2004 or equivalent.

Good practice is to base each of these three types of management systems on the 'plan-do-check-act' (PDCA) cycle, which includes:

- establishment of policy
- planning
- implementation and operation
- checking
- management review.

General requirements of the three types of management systems include evidence and records of:

- policy (each for quality, occupational health and safety, and the environment)
- internal audit
- management review
- responsibility and authority
- document control
- operational controls
- records
- training awareness and competence
- monitoring and measurement
- non-conformance corrective action
- preventive action
- communications
- continual improvement.

10.1.4.1 Quality management

The specifications and construction contract documents should clearly detail the quality management requirements for inspection and acceptance of the work as part of a **quality management plan**. Such a plan is generally designed to accomplish compliance with contract specifications and specified minimum industry standards. Box 10.2 shows a typical outline plan.

Box 10.2 ***Field office quality plans, USACE***

- purpose and scope
  - establishes annual operating quality plan
  - period covered
  - applicability
- workload
  - contracts underway
  - anticipated contracts
- organisation
  - description
  - chart
- staffing
  - current
  - required (keyed to workload)
- responsibilities
  - general
  - specific
- training
  - needs analysis
  - planning
- pre-award
  - design review conferences
  - input to constructor quality specifications, schedule requirements etc
  - buildability, constructability, operability (BCO) reviews
  - plan-in-hand reviews
- post-award
  - quality surveillance
    - participation in phases
    - problem solving
    - deficiency monitoring
  - quality testing
    - policy
    - facilities
    - schedule
  - reporting.

The owner, designer, construction manager and constructor all play key roles in ensuring that a high standard of quality management is maintained during levee construction, so differing and changed conditions are detected and accounted for in the constructed levee. Figure 10.3 provides an example of organisation for the quality management of a project with more complex owner representative third tier quality verification.

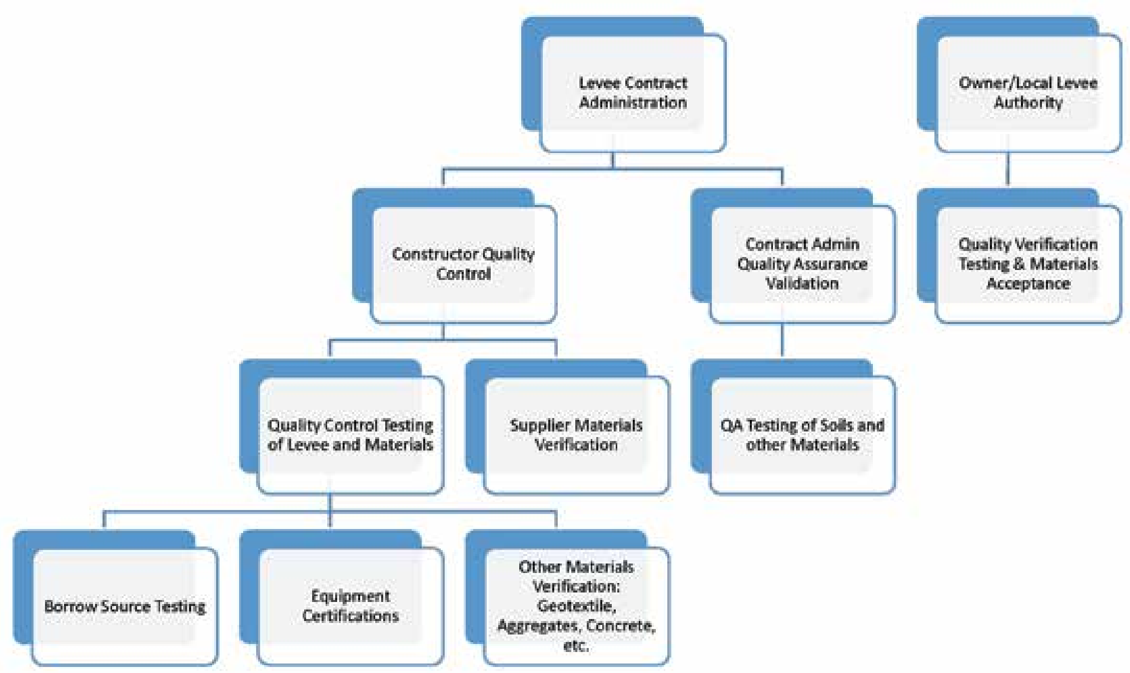


Figure 10.3 ***Example of quality management hierarchal system***

Table 10.9 identifies the basic quality management staff responsibilities that are critical to the successful completion of a levee meeting good quality practices. Box 10.3 presents the quality procedures for a levee project in the Netherlands.

Table 10.9 ***Example quality management staffing and responsibilities***

Levee construction staffing	Quality management responsibilities
Project manager	• project quality
Construction manager or contract manager	• construction contract quality
Owner quality manager	• quality audits • quality management documentation
Design site engineer	• quality compliance communication • quality manager duties (if/when assigned)
Constructor quality manager	• quality compliance (including corrective action) • quality management documentation
Constructor's supervisor	• quality compliance communication • constructor quality manager duties (if/when assigned)
Testing agency	• sampling and testing for compliance verification

Box 10.3 ***Coastal levee quality: strengthening a weak link in Noordwijk, the Netherlands***

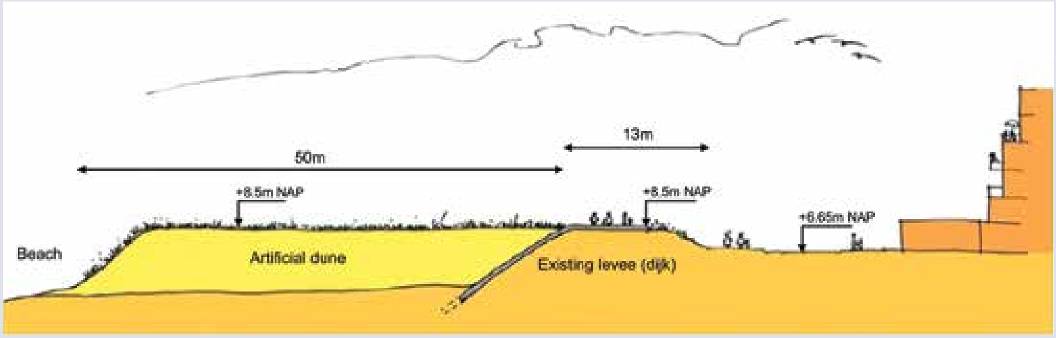


Figure 10.4 ***'Levee in dune', constrution to facilitate spatial development of the boulevard in Noordwijk, the Netherlands (courtesy Marco Veendorp, Arcadis)***

The quality procedure of the Netherlands design work was based on a system engineering approach. The requirements regarding the 'objects' and 'functions' were documented during the whole process, including clear and traceable reasoning for keeping up, dropping or modifying the requirements, so the design met all the needs. Working with the system engineering based quality system (that fully complied with ISO 9001 and ISO 14001 systems) highlighted the conflicting interests or requirements so that these could be addressed very early in the design process. The quality of the design was also dependent on early stakeholder involvement so there were no additional/surprising requirements during the design process or construction works. The quality check (own check, peer check) was based on a set of objective criteria determined before the work. The quality of the construction works was checked by a supervisor, which verified the plans of the contractor, submitted before the works. The deviations in the construction were dealt with (if necessary) on a daily basis in a committee consisting of the contractor technical staff and the consultant.

The engineering details of this project are given in Box 9.1.

The constructor should have arrangements in place to ensure that the specific features of a levee construction are carried out in accordance with the contract and any regulatory requirements. For example these features might include procedures for:

- drawing plan checking and verification
- site/operational activities such as ground clearance, earthworks, including compliance with design profiles, site security and access
- selection of equipment and materials suppliers
- instrument calibration and testing
- specialised treatment methods
- storage and disposal of waste
- corrective action.

Construction sampling and testing is required to verify contract compliance. Instrument calibration is an important aspect of quality control for which records should be kept. All instruments should be 'in-test' and withdrawn from use if either their calibration has expired or their measurements become suspect. The quality management plan should also state a procedure for the resolution of 'rejected' work (or 'non-compliance') and subsequent corrective actions.

One such checking process is that of the USACE's three phase inspection process (see Box 10.4), which works towards quality implementation from inception of construction to completion and verification of the levee project.

Box 10.4 Phased inspection process, USACE

Three phase inspection process

1 Preparatory phase:
   - a Review plans and specifications.
   - b Transmittals involved for approvals, items not approved, materials on hand for the work to be performed.
   - c Compliance with approved submittal required by contract.
   - d Check that all equipment to be used has been inspected for safety.
   - e Compare materials on site to submittals.
   - f All preliminary work is complete and approved.
   - g QC will discuss procedures for accomplishing work.
   - h Review any samples, materials, and test sections.
   - i Activity hazard analysis will be discussed as well as other safety related items to include language barriers.

2 Initial phase:
   - a Identify full compliance with the contract.
   - b Establish level of workmanship.
   - c Resolve all differences.
   - d Check safety.

3 Follow-up:
   - a Verify full compliance with the contract.
   - b Resolve any remaining differences.
   - c Document final acceptance.

10.1.4.2 Health and safety management

Occupational health and safety is a significant component of legal risk exposure for any organisation – especially in the construction industry. The general health and safety management system requirements for a construction site follow those listed in the introduction to this section. However, the focus is on risk reduction and legal compliance, so additional requirements include:

- hazard identification
- risk assessments and determining controls
- emergency preparedness and response
- incident investigation.

The construction of levees normally involves heavy machinery and exposed working conditions – both are hazardous to staff working on site. Public access to the construction site should be restricted and any visits permitted during construction (eg for education or information dissemination purposes) carefully controlled.

The management system should emphasise preventive action to avoid incidents happening in the first place by appropriate staff training (such as in the use of heavy machinery) and raising awareness of construction site hazards and poor practices (to avoid, and to prevent injury and ill health). A site incident log should be kept and regular reviews made to monitor the types of incidents that occur. Measures should be instigated to reduce the likelihood of reoccurrence.

Working with levees, which are often near open water, can bring other health and safety risks normally unrelated (or less often) to machinery or construction working. Waterborne diseases should feature in the hazard identification, as well as the risk of drowning in adjacent waterways or other water hazards.

Further reading

CIRIA has produced a *Site safety handbook* (Bielby and Gilbertson, 2008), which is an easy reference guide for those who work on construction sites.

In addition to good practice, it will also be necessary to comply with national regulations regarding occupational health and safety at work (the UK position is given as an illustration in Box 10.5).

Box 10.5 Occupational health and safety regulations, UK

There are over 50 health and safety regulations that may apply to work on construction sites in the UK. Regulations most affecting the design and management of work are the Construction (Design and Management) Regulations 2007 (CDM2007) and the Management of Health and Safety at Work Regulations 1999 (the Management Regulations). CDM2007 places duties on the client and all parties who are resourcing, designing and preparing for works on site. It requires all parties to co-operate with each other and those on adjoining sites, and to co-ordinate their activities to ensure works are carried out with health and safety as an integral part of their management process.

The Management Regulations cover many health and safety matters. The most pertinent with regard to site safety and site health is that an employer must provide those working with details of the preventative and protective measures to carry out the activity safely (ie a safe method of working or method statement). They must also provide information of the risks to health and safety identified by a risk assessment to prepare that safe method of work (ie a suitable and sufficient risk assessment).

Both the Management and CDM2007 Regulations require that staff report anything considered to be a health and safety issue to managers or supervisors.

10.1.4.3 Environmental management

The general requirements of an EMS for a construction site follow those listed in the introduction to this section. However, the focus is on protection of the environment from site activities and sustainable use of resources in construction, and so additional requirements include:

- identification of significant environmental aspects, including hazards to and impacts from the project
- legal requirements (including licences, permitting and consents)
- risk assessments and determining controls
- emergency preparedness and response (including pollution prevention)
- incident investigation.

In large construction companies, a main board member may be responsible for directing and reviewing corporate environmental protocols and responsibilities. In smaller companies, this responsibility may be held by the managing director. It is relevant to appoint someone to be responsible for providing corporate advice on environmental legislation, good practice and the company's environmental policy.

Environmental management planning

An **environmental management plan** (or site environmental plan) can arrange the management and reporting procedures for a project to mirror those of an EMS. The primary purpose is to focus on the environmental issues specific to the site. Environmental management plans are an effective way of employing the principles of EMS at the site level. These may include legal requirements, a requirement of the organisation's environmental policy, or the contract may well impose specific or general environmental requirements on the contractor (and/or subcontractors). In either case, project and site managers will need to acquaint themselves, through the environmental management plan, with the environmental requirements of the contract or their company, and will need to ensure awareness of these issues among their staff and workforce.

The benefits of environmental management plans are that it:

- introduces a planning phase before undertaking the project

- raises awareness and focuses training
- records environmental performance during the construction phase, allowing for modification and improvement of working practice
- provides for site specific purchasing policies
- provides for transport policies, including selection and maintenance of site plant/vehicles
- allows careful control of working hours
- helps to minimise energy use
- helps to minimise water use
- improves waste management, storage, reuse and recycling
- creates strategies for dealing with sensitive features and areas such as archaeology, ecology, and nature conservation sites, including designated areas (eg Sites of Special Scientific Interest)
- helps subcontractor management.

The site environmental plan should be accessible and regularly revised. All site staff, third parties and visitors to the works should be made aware of its existence and importance to the safeguarding of the local environment.

The level of detail in this plan will depend on the complexity and size of the development and should define lines of communication between all staff and third parties (ie regulators), as well as providing contact and emergency details. It is important to define the environmental responsibilities of all personnel within the site management structure, including those involved in monitoring initiatives. Once determined, the names, roles and responsibilities of staff should be recorded, along with the environmental procedures in place for dealing with potential issues.

Environmental responsibilities should be encouraged at all levels in an organisation since it only takes one careless act or period of inattentiveness to cause damage. Appropriate training of staff and a clear definition of responsibilities will help to reduce the potential for accidents to occur. Safeguarding the environment should be regarded as part of normal day-to-day activities rather than as a chore or burden. 'Top management' staff should lead by example.

On most projects, regardless of their size, the 'site manager' (or site agent) has principal responsibility for environmental management. This should include defining, monitoring and controlling activities that either have or potentially have impacts on the environment. The site manager may delegate some or all responsibilities to a suitably qualified and experienced representative to act on their behalf. Responsibilities will also include auditing environmental practice, liaising with regulatory authorities, and informing and monitoring subcontractors. Many companies have taken the opportunity of using their safety inspectors as environmental inspectors as well. The benefits are that a pool of expertise is developed and good practice can be easily shared around sites. All site staff should be charged with following good practice and encouraged to provide feedback and suggestions for improvements to management.

It is very useful to provide site staff environmental training alongside the induction to health and safety on site. The sooner staff are made aware of the identified risks on site, and the correct working practices to be followed, the better the chances are of avoiding an emergency or environmental problem. For training purposes, checklist and guidance cards could be provided to site staff to help them in their daily work. These cards can be kept in the central site office or in work huts, vehicles and on notice-boards (if laminated) for easy use. In order to be effective, adequate time should be set aside to inform staff about the environmental issues that are relevant to their site and to their work.

Further reading

CIRIA, in conjunction with the Environment Agency, has produced a video, leaflet and poster set that may be useful for on-site training – *Building a cleaner future* (CIRIA, SP141) and also published the *Environmental good practice on site – pocket book* (Charles and Wadams, 2012) as a useful reminder for construction staff. It also produced *A simple guide to controlling risk* (CIRIA, 2002).

The management responsibilities presented here should also apply to all 'subcontractors' on the project, whatever their size. Overall responsibility for environmental management however, normally resides with the main contractor. Good practice for managing subcontractors includes:

- when selecting subcontractors, ask them to present proof of their past record in achieving good environmental practice. Do they have an EMS? Is it certified by an accredited body? Check for any environmental prosecutions and take them into account
- ensure that subcontractors are aware of the environmental management requirements and obligations of the site and project, before starting work
- if the subcontractor works frequently with the main contractor, then it is common for the main contractor to invite (or require) them to attend environmental training sessions
- include environmental controls from the project specification in the subcontract, and encourage the use of method statements, to ensure good environmental practice.

Environmental training topics that should be considered include:

- the reasons for adopting good practice
- good practice in dealing with potential pollutants (eg oil refuelling, handling of paints and solvents)
- how to manage wastes
- how to manage materials and components on site to reduce waste
- emergency procedures and contact details
- choice of working methods, and sources of advice
- relevant legislation
- choice of plant
- importance of good housekeeping
- personal responsibility/liability.

10.1.5 Data acquisition and management for construction

This section deals with the data necessary before, during and after the construction (see Section 10.1.5.1) and the way to manage these data (see Section 10.1.5.2).

10.1.5.1 Construction data

Data related to construction can be distinguished in two categories:

- construction data inputs, especially from design and survey
- construction data outputs produced during the construction (which provide the owner good knowledge of the completed works).

Data inputs

Before starting construction of a levee, preliminary surveys are essential not only for the design process, but also to provide a clear understanding of the current site conditions.

Conditions on a levee alignment may have changed since completion of the design and compilation of plans and specifications. Recent or unforeseen changes could result in complications for a constructor. The condition of site should be examined before construction. Site conditions that differ from the original plans and specifications need to be clearly documented and presented to the levee owner. Useful information that is applicable to pre-construction investigation and documentation of differing conditions along levee alignments is provided in Section 5.4. Chapter 7 provides approaches for acquiring information on site conditions.

Good practice is for a team of experienced designers and constructors to review previous field reconnaissance and records. This way of proceeding will in the end lead to a better management of project planning and cost, instead of simply transferring all responsibilities at a certain stage to the constructor. Site inspections should be performed to obtain up-to-date knowledge of the construction sites and the surrounding area. Team members should review and discuss the project site and adjacent areas with the objective of discovering any new conditions that could pose problems for constructors.

Topographic maps, soil and geological maps, site condition photographs with notes, and aerial photographs can be essential tools supplemented with current surveys. Pertinent information on existing construction in the area should also be obtained. This includes design, construction, and performance data on utilities, highways, railroads, and hydraulic structures. A geographic information system (GIS) can be used extensively in a range of projects types. GIS is capable of compiling large multi-layered databases, interactively analysing and manipulating those databases, and generating and displaying resultant thematic maps and statistics to aid in engineering management decisions and for co-ordinating construction activities. GIS developed contour maps showing geologic structure elevations can assist with planning construction activities.

Successful levee construction works rely specifically on sufficient and accurate surveys in order to avoid problems with the actual construction Before and during construction work, additional surveys should be performed by the constructor in order to clearly define the specific work requirements. Specific issues concerning the gathering of information to support construction activities and in the updating of survey information may include:

- ensuring accuracy of benchmarks
- checking accuracy of previous construction as-built drawings
- identifying location of critical utilities, roads, railway, hydraulic structures, existing drainage features etc
- including local knowledge of foundation conditions
- checking alignment of proper access roads

Figure 10.5 presents a typical example of plan and section surveys for a levee system.

Data output

Some completed levee works require acceptance surveys, as well as verification surveys on behalf of the owner or responsible agency. The surveying methods may include all of the following required activities:

- horizontal co-ordinate system
- vertical datum
- accuracy of surveys for:
  - controls
  - layout
  - quantity calculations
  - as-built documentation.

Levees are engineered earthwork structures, subject to varying subsurface conditions and construction material properties. As such, the documentation that relates to earthwork criteria and ensuing design adjustments are of critical importance. Box 10.6 summarises the construction phase documents, which are important for levee projects.

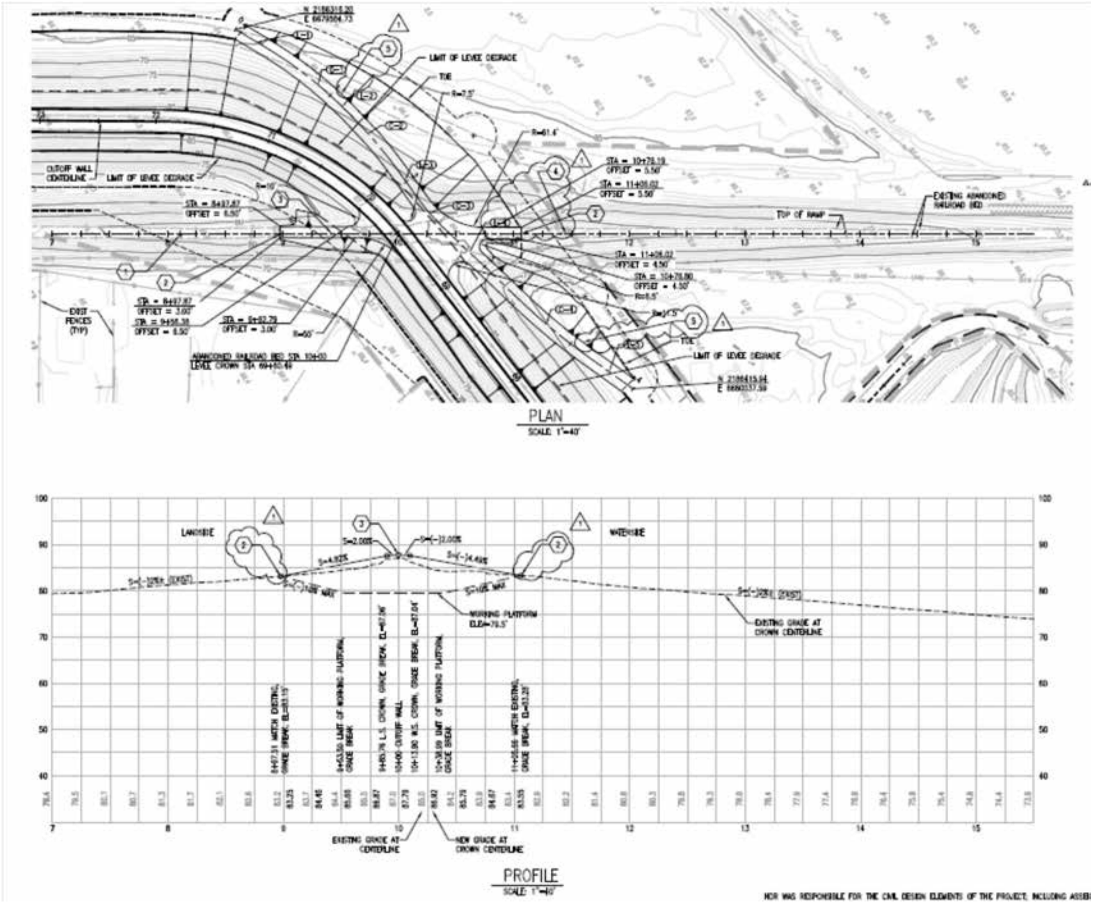


Figure 10.5 Examples of plan and section survey for Marysville Levee, CA, USA (courtesy USACE, Wielputz)

Box 10.6 Important levee construction documents

- design criteria requiring construction phase confirmation
- soil and earthwork material analysis and testing reports
- revised design calculations for differing site conditions
- pore pressure monitoring reports
- settlement monitoring reports and revised settlement prognosis
- cover material maintenance instructions
- non-earthwork facility operating instructions.

The majority of the data produced during construction should be provided to the owner/manager at the end of construction.

Example of construction data used for maintenance

During the construction phase, the owner, designer, and construction manager are primarily responsible for ensuring that data management meets the elevated standards associated with levees. Long-term maintenance of levees relies on accurate and obtainable records of design-basis and physical conditions. Proper record keeping and data management during the construction phase is of paramount importance for future operation, maintenance and improvement design activities. Box 10.7 highlights an example of levee assessment savings that resulted from conscientious preparation and archiving of construction as-built documents by the designer and owner.

Box 10.7 As-built data and archiving responsibilities during levee construction

The designer's detailed land survey of post-construction elevations for the Lock Haven, Pennsylvania, USA levee were part of the as-built documentation. Fifteen years later, the integrity of the levee was confirmed and certified by comparing resurvey data to the owner's as-built archives. Made possible through careful data management over the life cycle, this assessment method was less costly and less invasive than geotechnical testing.

Figure 10.6 Lock Haven Pennsylvania Levee (courtesy Buchart Horn, Inc)

10.1.5.2 Data management during construction

Data management principles are integral to the life cycle of levee projects as presented in Chapter 5. Data management during construction phase plays an important role in assuring that the levee designs are appropriate for actual site conditions, and that as-built information is recorded for future maintenance, design and construction activities. This section describes good practice to ensure reliable documentation and record keeping while levees are being constructed, modified or rehabilitated.

Good practice for levee construction relies on the confirmation and documentation of site and as-built conditions during construction phase. Contract documents and construction work plans define who is responsible for documenting the conditions, and what type of records are required. A typical levee construction documentation programme includes the data and documents listed in Table 10.10. The table refers to other relevant portions of this handbook, where the information for each data type is described in greater detail.

Table 10.10 Data produced and used during construction

Type of data	Description of data and its use in construction	Links
Assurance agreements	Description: • deeds and easement agreements and conditions. Use: • required for most major levee construction • affects levee construction general conditions including limits of work and requirements for supplemental land lease by constructor.	Chapter 9
Land and easements	Description: • site characterisation during design phase. Use: • during levee construction, data is used to confirm compliance of observed site conditions and construction test data with the established design basis.	Chapter 9 Chapter 10 (Sections 10.1 and 10.3)
Design site analysis and characterisation reports	Description: • designer notes regarding the basis of design and recommended levee construction considerations • designer instructions regarding changes to design resulting from changed site conditions and constructed work. Use: • affects levee construction general conditions, method and quality management plan.	Chapter 7 Chapter 9

Table 10.10 Data produced and used during construction (contd)

Design memorandums	Description: • local and national environmental, land use and construction code permits. Use: • affects levee construction general conditions, method, programme and quality management.	Chapter 9
Permits	Description: • project programme (schedule). Use: • affects levee construction general conditions and method.	Section 10.1
Project programme (schedule)	Description: • accident prevention and emergency response plans including flood and jobsite hazards. Use: • affects levee construction method.	Section 10.1
Emergency, health and safety plans	Description: • written quality control and quality assurance programme addressing workload, organisation, staffing, job responsibilities, training, work methodologies, testing and documentation. Use: • affects levee construction general conditions, method, programme and quality management.	Chapter 6 Chapter 10 (Section 10.1)
Quality management plan	Description: • documentation of constructor's material, supplies, equipment and testing. Use: • during levee construction phase, this data confirms compliance with design basis and construction documents • following levee construction, this data provides written records for future operations, maintenance, condition assessments, repairs and adaptation designs.	Section 10.1
Constructor shop drawings and submittals	Description: • site characterisation during construction use Use: • during levee construction phase, this data confirms compliance with design basis and construction documents; and if it indicates differing conditions, triggers construction phase design reviews and revisions • following levee construction, this data provides written records for future operations, maintenance, condition assessments, repairs and adaptation designs.	Section 10.2, 10.3 and 10.4
Construction phase material analysis and testing reports	Description: • documentation of observed levee construction conditions during construction phase. Use: • during levee construction phase, this data provides written records for administration activities such as payment requests and quality monitoring • following levee construction, this data provides written records for future operations, maintenance, condition assessments, repairs and adaptation designs.	Chapter 7 Chapter 10 (Section 10.4 and 10.5)
Construction inspection reports	Description: • documentation of levee constructed condition including changes to design, site conditions and constructed work. Use: • provides written records for future levee operations, maintenance, condition assessments, repairs and adaptation designs.	Section 10.1

Table 10.10 Data produced and used during construction (contd)

As-built drawings	Description: • specifies method and frequency of levee operation, inspection, maintenance and condition assessment • typically developed during construction phase. Use: • provides written records for levee operations and maintenance.	Section 10.1
Operation and maintenance instructions	Description: • documentation of construction contract general conditions • specifies levee construction methodologies, materials, supplies and equipment. Use: • during levee construction phase, this data provides written records for administration activities such as payment requests and quality monitoring • following levee construction, this data provides written records for future operations, maintenance, condition assessments, repairs and adaptation designs.	Chapter 4 Chapter 9
Contract administration documents	Description: • documentation of observed levee construction activities, with emphasis on providing construction feedback for improving future levee designs. Use: • provides written records for future levee design and construction projects.	Section 10.1
Lessons learned file	Description: • deeds and easement agreements and conditions. Use: • required for most major levee construction • affects levee construction general conditions including limits of work and requirements for supplemental land lease by constructor.	Section 10.1

10.2 ALLOWING FOR HYDRO-METEOROLOGICAL CONDITIONS

Levee construction is affected by hydro-meteorological conditions in the coastal and fluvial environment (water levels variations, waves, currents, winds, ice etc) in two ways:

- they restrict work by affecting operations (Section 10.2.1)
- they create a risk of flooding, both to the construction site and the leveed area (Section 10.2.2).

In Section 10.2.2, two main considerations are addressed:

- hydro-meteorological conditions may lead to flooding of the construction site and damaging the partly completed works, construction plant and equipment
- the construction site and activities may temporarily increase the flood risk in the leveed area, by increasing hydraulic loads (water level) or decreasing the levees strength (by removing the revetment or digging in the levee, for example for a drainage feature/culvert etc).

Note

Environmental restrictions may mean that preferred working periods with more benign hydro-meteorological conditions cannot be adopted (Section 10.3.1.1).

10.2.1 Working in coastal and fluvial environments

It is important to plan for hydro-meteorological conditions given:

- variability and severity of the conditions that could potentially arise
- risks of loss and damage
- associated delays, costs and impacts if contingencies are not in place.

10.2.1.1 Construction risks associated with coastal and fluvial environments

Coastal and riverine environments present a number of weather, river and sea condition driving forces that need to be taken into account during construction.

Wind chill and driving rain can be significant factors for site staff and can restrict all land-based plant operations. High rainfall can also affect working conditions, adding to flooding of the site and damaging concrete and other materials.

Winds can be particularly strong at the coastline. Winds are of the order of 10 per cent faster over water than over land and the lack of shelter when the winds are from offshore will exacerbate this effect. Wind can also have an important effect on floating craft. The effect of strong and rapidly varying wind-speeds and local pressures can cause significant motion, affecting operations. The local wind climate will also drive the wave climate, with the exception of swell conditions that originate from more distant storms.

Water level variations (due to discharge variations, tides or other factors such as storm surges, wind and wave set up, and seiches) will define what works can be carried out in the dry and the available access time. They can:

- prevent sea borne deliveries from reaching the site
- greatly restrict land access to site
- flood the works where excavations or cofferdams are used
- affect the wave and current climate that can penetrate into a construction site from the open sea.

Water levels and other hydraulic actions can be linked, especially for wave propagation at the shore. In simple terms: **higher tides = larger waves**. This means that at high tide, during the elevation of water levels due to storm surge and wind and wave set up, the site is at its most vulnerable. Chapter 7 provides more information on this subject.

It is essential to obtain forecasts early and plan the works accordingly. On coastal sites construction operations must work with the tide and on the river may have to accommodate higher water levels.

Waves can:

- affect deliveries reaching the site (eg preventing barges from leaving port, causing them to 'run for shelter', delaying them in transit and preventing them from unloading)
- damage plant (due to beaching, overturning, striking the works etc)
- damage temporary and incomplete works where permanent protection is not yet in place
- draw-down beach levels which can affect the works, deliveries and expose contaminated materials
- result in poor placement of material, which may affect the environment.

Currents can:

- affect the ability of a vessel to hold position offshore or on a river
- affect the ability of a vessel to safely approach the site, especially in restricted water depth
- affect the ability to place materials within tolerance

- erode partially completed works
- apply loading on temporary works
- affect the incident wave conditions
- increase turbidity of the water, resulting in damage to flora and fauna.

10.2.1.2 Planning levee construction for hydro-meteorological conditions

Levee construction is particularly susceptible to the extremes of temperature, precipitation, relative humidity, river flow and tides. These conditions may significantly impact construction timing and operations. Inclement weather may require a temporary suspension of construction activities that could significantly affect the project programme. Temporary construction features such as haul roads, construction equipment and materials unloading and storage sites could be at risk if not properly designed.

Before going on site, the contractor should produce a plan, which addresses key issues in relation to weather and hydraulic conditions. This plan should take account of the following:

- the amount of time (downtime) for which land-based or marine plant will be unavailable for work due to:
    - excessive current or wave action causing motions that make it impractical to operate plant. For example, many items of plant are restricted to operating where wave heights do not exceed about one metre
    - limitations on access to the point of construction activity due to high water levels (land-based plant) or inadequate water depth (marine plant), including the effect of waves.
- whether temporary protection would be required for the partly completed works. This can entail:
    - phasing the work so that partial completion of more robust parts of the permanent works protect the more vulnerable parts
    - physically protecting or reinforcing vulnerable parts during times when storms are anticipated.
- whether it is appropriate to completely shut down construction activity for the season in which most severe weather or flooding occurs.

If the duration of the construction is much less than the life span of the levee, the level of protection may typically only be designed to withstand a one in 10-year return period event. However, the obligation to protect the population from flood risk means that temporary protection measures may be needed for much more severe events, for example 100-year flood events (and even in some instances up to 1000-year events). The issue of allowing for flood risk in construction is discussed in Section 10.2.2, and Chapters 2, 5 and 9 provide more guidance on this subject.

For such project planning, the format in which wind, wave, water level and current data will be needed takes two forms:

- normal conditions or ‘climate’ (ie statistical presentation of the data based upon historical record, showing the range of conditions that can be anticipated at a known location; used for planning purposes to ascertain how the works could affect, or be affected by, the environment). Ideally such information will have been prepared for the owner in advance and will include information on:
    - the proportion of time different wave height thresholds are exceeded (‘storms’) and the variability of the length of that exceedance
    - the proportion of time different wave height thresholds are not exceeded (‘calms’) and the variability of the length of that non-exceedance.
- extreme conditions (ie statistical maximum conditions).

Table 10.11 sets out some detailed examples of hydro-meteorological effects on levee construction and ways of managing these.

Table 10.11 Examples of hydro-meteorological considerations for earthen structures

Hydro-meteorological issues	Good practice
• levee work should not be performed during heavy rain seasons or extreme drought conditions • embankment moisture content is critical for proper levee construction.	• contact appropriate authorities to secure local tidal, weather and flood discharge records. Seasonal weather and tidal stage records should also be acquired to ensure construction of haul road and temporary drainage facilities are placed at safe elevations • avoid starting work during adverse conditions. Strong consideration should be given to programming construction for months with historically best weather conditions, eg in the period April through October in the Netherlands and the UK, when rainfall and relative humidity should be most favourable.
• river levees/flood walls should not be built/ repaired during peak flood seasons or high water events without taking appropriate precautions • storm and hurricane protection levees/flood walls should not be built during the storm and hurricane season.	• contact appropriate authorities to secure local tidal, weather and flood discharge records (see Example 2 in Box 10.8). Some agencies can provide probability analyses of the expected frequency and duration of tidal events • planning is required to programme projects to avoid extreme weather events such as droughts or monsoons • detailed sequencing is required for seasonal flood protection construction in order to ensure that levees/flood walls are operational during peak seasons • temporary flood protection may be constructed if no other options are available, however, this is a costly alternative, which has varying results • organise early warnings including appropriate response plans to deal with extreme storms etc outside this 'closed time frame' • constructors and principals should clearly identify the expected weather delays in the contract before work. Average rain days can often be obtained from the appropriate national source (see Example 1 in Box 10.8).
• levee work should not be performed during extreme cold weather.	• planning is required to programme projects to avoid extreme cold weather, especially if there is a risk of ice jam causing river levels to rise (see Figure 10.7) • flood wall concrete work should not take place during extreme cold weather. If no other options are available, certain construction techniques such as heating elements, variable concrete mix designs, enclosed pours etc may be used.

Figure 10.7 Ice jam raising river levels (courtesy USACE)

10.2.2 Limiting flood risk during construction

Levee projects only achieve the final level of flood risk reduction after all features have been completed, so have elevated risks of catastrophic life and financial loss during the construction process. Hydraulic engineers should review all the proposed phases of construction, carrying out any necessary additional hydraulic modelling. Levee repair projects may require special provisions, such as fast track execution and temporary protection arrangements. Table 10.12 presents most frequent considerations and associated good practices to minimise flood risk to both the levee works and the leveed area.

Box 10.8 ***Examples of hydro-meteorological considerations***

Example 1: USACE provides a table of 'adverse weather days' that shows the average number of days of inclement weather for any given month. Those data are compiled from historical rainfall records are useful in determining a realistic construction programme.

Example 2: A very large watershed in Florida, USA has many large lakes that are controlled by spillways and canals. A heavy rainfall event (often associated with a hurricane) can cause the lakes to rise significantly with peak levels occurring days or weeks after the peak of the rainfall event. Water managers will direct discharges as required to prevent lake levels from reaching flood damage elevations. Those discharges result in high floodway water surface elevations and high velocities for many miles downstream of the lakes.

Table 10.12 ***Flood risk management good practice during construction***

Flood risk management issue	Good practice
Level of flood risk in the leveed area may inadvertently be increased by levee construction activity across floodplains and tributaries to the watercourse such as: • temporary work areas • haul road excavations or embankments • partially constructed levees or associated structures • repair of structures and utilities • restrictions of existing flow channels. These can result in increased flood levels and flow velocities.	During levee construction, the construction manager and constructor must recognise (and where possible avoid) activities that increase the risks to life and property from flooding in the leveed area by: • evaluation by hydraulic engineers of floodwater level and wave data for the area and generation of long-term climate predictions (see Sections 7.2 to 7.5) • investigation of water releases anticipated from upstream hydraulic installations • construction of temporary works such as bypass channel(s) to ensure that water levels during construction do not exceed pre-project elevations • careful programming and staged execution to minimise impact on hydraulic regime, including appropriate sequencing of construction of embankments, culverts, and structures. In riverine environments, this includes the need to work from upstream to downstream to ensure provision of 'superiority' of structures and upstream embankment protection (as discussed in Section 9.5) • simultaneous advancement of protection on opposite sides of the watercourse (note that in some circumstances this can have detrimental effects, see Example in Box 10.9) • temporary measures, as illustrated by Figures 10.8 and 10.9, to maintain proper levels of protection during construction. Temporary defences may be the only option available in a coastal situation • minimisation of temporary water level rise due to construction works, and avoidance of unmanageable situations such as bank erosion • contingency plans to ensure public safety in the event of an unforeseen flood event (see Box 10.10).
During construction, the following may be weakened or damaged by the high water levels, flows and waves associated with river, tidal and coastal flood events: • existing levees being repaired or rehabilitated • partially constructed new levee segments • associated new water control structures.	Before construction, the owner and designer should formulate plans to minimise risk to the works. This may include: • selection of an appropriate sequence of construction • provision of temporary flood protection or drainage to levees (see Figure 10.11) • contingency planning for flood fighting during construction, such as temporary stockpiles of embankment material and other flood fighting measures.

Table 10.12 Flood risk management good practice during construction (contd)

Vulnerability of plant, equipment, materials, personnel and part-built works to flood events: • **materials stockpiles:** floodwaters could damage or wash away materials • **equipment parking:** floodwaters could irreparably damage construction equipment • **debris storage:** the constructor may be legally responsible for collection and disposal of construction debris scattered by a flood event.	General mitigation measures include: • obtaining advance meteorological forecasts • development of contingency plans • provision of temporary protection measures • safe access and emergency egress • only store material that are impervious to water, such as rip-rap, in the floodplain • store levee fill and other materials that may be sensitive to water damage in locations avoiding critical floodplain areas such as the main floodway or upland tributary drainage channels • locate construction equipment parking areas at elevations above flood level with access at all times during flood events • choose higher ground for disposal or storage of debris, particularly from clearing and grubbing operations.

Box 10.9 Example of flood control during construction

For the Rio de la Plata Flood Control Levee in Puerto Rico, the Levee Contract Phase 1 called for construction of levee segments on the east and west side of Rio de la Plata and a channel to increase flow capacity between the levees. The levee segments and channel would be constructed on both sides of the channel at the same time.

The Rio de la Plata floodway has complex flow patterns under flood conditions. Flood flows from the main river spill over the east bank upstream of a small rural town and return to the flood channel downstream of the town. Hydraulic numerical modelling of the proposed Phase 1 construction scenario showed that construction of the east levee segment would block the historical return point of the out of bank floodwaters and would result in an increase in depth and duration of flooding. The owner decided to modify the Phase 1 contract scope to eliminate risk of east bank flooding.

Figure 10.8 Example of temporary flood protection during construction (courtesy USACE)

Figure 10.9 ***Temporary flood protection built to protect adjacent community during reconstruction of the existing flood protection (courtesy USACE)***

Box 10.10 ***Ensuring public safety during levee construction***

Figure 10.10 ***Flooded construction site (courtesy BAM Nuttall Ltd)***

Flooding during levee construction directly affects the owner and the public in terms of public safety, project programme and project budget. During construction of a flood storage reservoir in the UK, an inland flood inundated the site (see Figure 10.10). The levee designer and constructor co-ordinated efforts to maintain public safety during construction.

Figure 10.11 ***Temporary drainage ditch dug to ensure proper drainage during construction (courtesy USACE)***

10.3 SETTING UP AND MANAGING THE SITE

Section 10.3 provides information necessary for setting up and managing the site. This includes many considerations that will affect the programme at the stage of its definition as well as during the construction.

A safe and environmentally friendly approach is presented for site management, including:

- consideration of the particulars of the site (see Section 10.3.1)
- defining access routes, managing site traffic and alternative delivery methods (see Section 10.3.2)
- managing archaeological remains and utilities (see Section 10.3.3).

10.3.1 Managing constraints from natural and human environment

This section introduces the constraints associated with the natural environment (see Section 10.3.1.1) and the inhabitants (see Section 10.3.1.2) along with good practices to solve the issues relative to:

- noise, vibration and lighting (often linked with working hours limitation as presented in Section 10.3.1.3)
- air and water quality (practices will be adapted to avoid pollution as presented in Section 10.3.1.4).

10.3.1.1 Natural environment constraints

There are numerous types of constraints that can have a significant impact on the construction project, especially:

- sensitive area protections and protocols
- protected and endangered species.

Sensitive area protections and protocols can imply restrictions on sound, duration, working days and hours, accessibility of terrains, vibrations, dust, temporary changes of groundwater level. Box 10.11 provides a list of these protections and protocols. The constructor should identify the measures to be employed and obtain approval of them from the construction manager or appropriate authorities. One example of a measure is appropriate net fencing (see Figure 10.12). Box 10.12 illustrates the permits and licence issues in a coastal environment with dunes.

Box 10.11 Sensitive area protections and protocols

International designation of sensitive areas requiring special protection or protocols will vary, but typically may include:

- European sites
- National Parks
- Ramsar sites
- Sites of Special Scientific Interest (SSSI)
- Special Protection Areas (SPA)
- Special Areas of Conservation (SAC)
- archaeological sites
- monuments and protected landscapes
- biodiversity habitats and species
- polluted areas
- wetlands
- environmental compensation areas
- tree preservation orders.

Figure 10.12 Net fencing of a sensitive area (courtesy David St. Marie)

Box 10.12 Examples of consequences from lack of environmental permits in a coastal environment during construction

Surge barriers and natural and artificial sand dunes mitigate the impacts of winds and waves from infrequent storm events. Vegetation on the dunes provides erosion resistance due to wind and waves. Dunes and beach areas are also used by reptiles, water fowl and sea and land birds for habitat and nesting.

Construction activities that require intrusion into or that are in close proximity to a coastal dune may result in destruction of dune vegetation and produce odours, high noise levels and direct contact with construction staff personnel. This may result in damage to nesting sites, excessive disturbance to nesting animals. It is clear that the constructor should obey all existent regulations, although it should be the responsibility of the levee owner to indicate the constraints that can limit the operations for the constructor.

Many countries have instituted rules to protect coastal dunes and the habitat they provide. Legislation prevents removal or disturbance of dune material, and interference with nesting animals. Central government may impose heavy fines for unauthorised activities on or close to dunes. Some examples are provided by the USA:

Example 1: in Mississippi "any person who removes a plant commonly known as 'sea oats' or 'uniola paniculata' from the shores of this state shall be guilty of a misdemeanour and shall, upon conviction, be fined not more than Five Hundred Dollars ($500)." (Mississippi Code, 2010).

Example 2: in Florida a fine of up to $1000, up to one year in jail, or both, for misdemeanour convictions for cutting, harvesting or destroying sea oats (Florida Statutes, 2012).

A constructor may be held financially and/or criminally responsible for damages. Consultation with the appropriate government organisations responsible for granting licenses and/or permits for construction in coastal areas should be initiated as soon as possible. A clear construction plan, programme and description of activities should be provided to local government organisations by the constructor. If a permit is issued, the constructor should request a written description and map showing areas into which construction vehicles and personnel must not venture. Construction plans and schedules can then be formulated.

Protected and endangered species including their habitats and migratory routes can imply:

- identify work calendar considerations due to any relevant spawning or migratory species (see Box 10.13)
- provision of alternative habitats.

The identification of any flora and fauna that may be affected by the works should have been highlighted within the environmental impact assessment (EIA) carried out by the construction manager. These may include endangered or protected species as illustrated by Figure 10.13.

For protected species it is important to provide good practice methods or devices (temporary or permanent) to maintain the continuity of ecological corridors that are crossing the project or access routes (for example, the installation of a culvert to provide an animal crossing under the construction access road).

Similarly, the constructor will need to put in place such protection measures as are necessary to prevent damage or disruption of the habitats. Protective measures may also be required for other reasons.

The species that are at risk of being affected by the works can vary throughout the season due to migration, nesting, breeding, spawning etc. For example, nesting birds may be protected by law from disturbance. If works during bird breeding season are unavoidable a license may be needed if nesting birds are to be disturbed. Advice should be sought from an ecologist.

Box 10.13 provides an example of the seasonal considerations for protected mammal and reptile species in the UK.

Figure 10.13 Lizard, an endangered species in the Netherlands (courtesy Harry Mols)

10.3.1.2 Liaison, participation and consents with authorities and residents

Liaison with environmental regulatory and land-use planning authorities, site neighbours and the public is essential when setting up and managing a site. The wide range of users and activities at marine or riverine sites can mean that construction projects in these locations are particularly vulnerable to complaints. Landowner considerations are not detailed here as they have been presented in Section 10.1.1.3 relative to permit and licenses.

Establishing good relations with environmental regulatory and land-use planning authorities should include:

- identifying the extent of the liaison already undertaken at the design phase
- identifying from the specification, other contract documents, and consultation with the engineer/architect any special environmental requirements that may be required (see Section 10.1)
- identifying any existing contacts
- making plans to establish working relationships with each appropriate organisation
- identifying and assigning responsibility to appropriate site staff to undertake the necessary liaison during the construction phase.

Early contact should be made with the relevant environmental regulatory authority. If possible, arrange a site visit with all interested parties as soon as possible. In this way contacts can be made, issues identified and works can be suitably planned. Also training and induction courses should be timed to occur around the same period, so that information on habitats and other sensitive features can be passed directly to staff.

Establishing good relations with site neighbours (including local residents, businesses, fisherman, the tourist industry etc) could include:

- public meetings to explain the construction of the project and its potential impacts
- regular meetings with local representatives groups
- an exhibition in a suitable local venue
- setting up liaison with local schools
- on large long-term projects, a newsletter, website, up-to-date notice-boards or regular bulletins on progress, providing details of the proposed timing of disruptive activities.

Good public relations

Good public relations are vital in the drive to complete a project with the minimum disturbance to neighbours. Experience has shown that members of the public tend to complain less if they know

Box 10.13 Example of seasonal considerations calendar for protected species in the UK (courtesy BAM Nuttall)

Protected mammal and reptile species		**Jan**	**Feb**	**Mar**	**Apr**	**May**	**Jun**	**Nov**	**Aug**	**Sep**	**Oct**	**Nov**	**Dec**
	***Badgers*** **Legislation:** Protection of Badgers Act 1992 **Statutory authority:** Natural England, Countryside Council for Wales or Scottish Natural Heritage **Requirements:** see guidance on interpretation of disturbance License is required to close and for translocation of badger setts **Guidance:** English Nature (2001a) *Badgers and development*	No work close to setts						Badger exclusion licensing season					No work close to setts
		Best time for field surveys				Still OK for surveys but they won't be as effective						Best time for field surveys	
		Construct artificial sett		Territorial bait surveys can be done									Construct artificial sett
	***Water Voles*** **Legislation:** Schedule 5 of the Wildlife and Countryside Act 1981 **Statutory authority:** Natural England, Countryside Council for Wales or Scottish Natural Heritage **Requirements:** no provisions for issuing license for the destruction of burrows for development and maintenance to disturb them while using such a place **Guidance:** English Nature (2002) *Water voles – the law in practice*	Avoid disturbance										Avoid disturbance	
				Best time for surveys									
					Best time to work					Best time to work			
	***Great Crested Newt*** **Legislation:** Schedule 5 of the Wildlife and countryside Act 1981, The Conservation (Natural Habitats, &c.) Regulations 1994 **Statutory authority:** Natural England, Countryside Council for Wales or Scottish Natural Heritage **Requirements:** license required for any works likely to affect newts and their habitats. Licenses are issued by Natural England, WDA and Scottish Natural Heritage **Guidance:** English Nature (2001b) *Great crested newt mitigation guidelines*	No work in inhabited ponds	Best time for licensing and surveying									No work in inhabited ponds	
					Best time for terrestrial work								
	***Bats*** **Legislation:** Schedule 5 of the Wildlife and countryside Act 1981, The Conservation (Natural Habitats, &c.) Regulations 1994 **Statutory authority:** Natural England, Countryside Council for Wales or Scottish Natural Heritage **Requirements:** works likely to disturb bats must be licensed. Issued by Natural England, WDA and Scottish Natural Heritage **Guidance:** Natural England (2011) *Focus on bats and bats* Bat Conservation Trust (2003): *Bats and trees and bat boxes – how to make them and where to put them*	No work in hibernation roosts					No work in breeding roosts					No work in hibernation roosts	
	***Reptiles*** **Legislation:** Schedule 5 of the Wildlife and countryside Act 1981 **Statutory authority:** Natural England, Countryside Council for Wales or Scottish Natural Heritage **Requirements:** works likely to affect reptiles such as smooth snakes and sand lizards require licenses. Issued by Natural England, WDA and Scottish Executive **Guidance:** English Nature (1998) *Facts about reptiles*	Avoid work										Avoid work	
					Best time for capture								

Box 10.13 ***Example of seasonal considerations calendar for protected species in the UK (courtesy BAM Nuttall) (contd)***

<table>
<tr><th></th><th></th><th>Jan</th><th>Feb</th><th>Mar</th><th>Apr</th><th>May</th><th>Jun</th><th>Nov</th><th>Aug</th><th>Sep</th><th>Oct</th><th>Nov</th><th>Dec</th></tr>
<tr><td rowspan="3"></td><td rowspan="3">Frogs and toads
Legislation: Schedule 5 of the Wildlife and countryside Act 1981
Statutory authority: Natural England, Countryside Council for Wales or Scottish Natural Heritage
Requirements: work likely to affect Natter jack toads or disturb them or damage their habitat requires licenses. Licenses issued by Natural England, WDA or Scottish Nature Heritage
Guidance: English Nature (1998) Facts about amphibians</td><td colspan="4">Do not disturb – hibernation</td><td colspan="3">Do not work with Natterjack toad or spawn without license</td><td colspan="2"></td><td colspan="3">Do not disturb – hibernation</td></tr>
<tr><td colspan="2"></td><td colspan="6">Best time for pond surveys</td><td colspan="4"></td></tr>
<tr><td colspan="2">Best time to work in ponds</td><td colspan="6"></td><td colspan="4">Best time to work in ponds</td></tr>
<tr><td rowspan="2"></td><td rowspan="2">Hare
Legislation: Ground Game Act 1880, Hare Protection Act 1911
Statutory authority: Natural England, Countryside Council for Wales or Scottish Natural Heritage
Requirements: limited protection</td><td colspan="2">Best time to work</td><td colspan="7">Breeding season</td><td colspan="3">Best time to work</td></tr>
<tr><td colspan="12">Survey possible all year</td></tr>
<tr><td rowspan="2"></td><td rowspan="2">Otter
Legislation: Wildlife and countryside Act 1981, The Conservation (Natural Habitats, &c.) Regulations 1994
Statutory authority: Natural England, Countryside Council for Wales or Scottish Natural Heritage
Requirements: it is an offence to kill, injure, disturb or damage their habitat. License is required for works within 30 m radius. Licenses are issued by Natural England, WDA and Scottish Natural Heritage
Guidance: SNH (2008) Otter and development</td><td colspan="5">No work close to otter activity</td><td colspan="4">Best time to work</td><td colspan="3">No work close to otter activity</td></tr>
<tr><td colspan="12">Surveys possible all year</td></tr>
<tr><td rowspan="2"></td><td rowspan="2">Red squirrel
Legislation: Wildlife and countryside Act 1981, The Conservation (Natural Habitats, &c.) Regulations 1994
Statutory authority: Natural England, Countryside Council for Wales or Scottish Natural Heritage
Requirements: it is an offence to kill, injure, disturb or damage their habitat. Licenses are issued by Natural England, Countryside Council for Wales or Scottish Natural Heritage</td><td colspan="12">If spotted stop work, they are rare and sightings need to be reported</td></tr>
<tr><td colspan="6">Breeding season</td><td colspan="6"></td></tr>
<tr><td rowspan="2"></td><td rowspan="2">Dormouse
Legislation: Schedule 5 of the Wildlife and countryside Act 1981, The Conservation (Natural Habitats, &c.) Regulations 1994
Statutory authority: Natural England, Countryside Council for Wales or Scottish Natural Heritage
Requirements: license required for works that are likely to affect dormice and their habitats. Licenses are issued by Natural England, WDA and Scottish Natural Heritage
Guidance: Bright et al (2006) The dormouse conservation handbook</td><td colspan="12"></td></tr>
<tr><td colspan="5">Nest surveys during hibernation</td><td colspan="3"></td><td colspan="4">Nest surveys during hibernation</td></tr>
</table>

what is happening on site. Public liaison is particularly important if operations that are likely to cause disturbance are going to be carried out for any length of time. Try to explain the efforts that are being made to limit the impacts of operations through phasing and other control measures. Use hotlines, newsletters, notice-boards and viewing stations to encourage an understanding of the development, the costs and efforts involved, and to minimise confusion and discontent. Establishing good public relations is easier if the site staff understand the project and its effect from the public's perspective. Training should be appropriate to the size, nature, and type of activities carried out and should emphasise the key environmental aspects and impacts of the operation, and methods for their mitigation. In particular, the need to be sensitive to local communities and aware of sensitive environmental assets should be stressed.

10.3.1.3 Working hours – noise, vibration and lighting

Site working hours can create considerable concern and annoyance to local residents, in particular due to noise and light pollution. Excess noise and artificial lighting of construction activities during night hours may seriously disrupt nesting wildlife. In addition, other indigenous species such as bats, owls, reptiles etc may be adversely affected by artificial light and noise from nocturnal construction activities.

Noise and vibration

Levee construction can be more susceptible to risks associated with noise restrictions due to:

- the inherent nature of the works (eg excavation, loading, hauling, compaction, rock unloading and piling works)
- the fact that water is acoustically 'hard' (ie sound waves move over water rather than penetrate)
- the often close proximity of the public.

Conflict can arise due to the need to maximise the use of floating or marine plant and the need to optimise the time windows available for convenient hydraulic conditions (in particular tidal windows in the coastal sites). This is often against accepted working hour standards and can lead to a high level of public complaint. On some projects, the working hours for noisy operations are defined within the contract document, although there may be opportunities for extending working hours in consultation with the levee owner. Extensions to working hours may be critical to achieving an effective and efficient construction programme (particularly on projects that require tidal working), their effect on the public should be carefully considered and extensions outside sociable working hours avoided as far as practicable. When extended working is necessary, affected members of the public should be advised in advance of the unsocial work hours, its duration and the reasons for the work at that time.

In addition to adjustment of working hours, when work is taking place the following measures should be considered:

- effective noise suppression for all plant, including ensuring all vehicle noise reduction equipment and silencers are fully operational
- schedule intrusive activities at less sensitive times within the allowable working day based on establishing patterns of behaviour of residents (including the tourist industry and business) and the public carefully. For example, schedule deliveries outside the morning and evening 'rush hour'
- consider effect of noise on the natural environment, such as on birds in the breeding season.

Vibration due to noisy plant is a further issue and adjacent landowners, residents, and business owners etc will often claim for any negative impact of the project on their properties and businesses. Where possible, agreement on reasonable vibration damage arrangements should be sought before the project starts. Depending on the project scope, setting, locale, and local regulatory environment this might include agreement on:

- use of noise meters, parameters to be monitored and administrative procedures

- pre-construction structural monitoring and assessment
- a vibration monitoring program, if appropriate.

Lighting

Lighting is essential for many activities to maximise working hours, for the use of machinery, and to provide suitable working conditions. Lighting can also be used as a deterrent to vandals. However, light can be a source of annoyance to local residents so it is important to keep site lighting at the minimum brightness necessary for adequate security and safety. The lighting should be located and directed so that it does not intrude on any properties nearby and the use of infrared lighting for security should be considered. Wildlife can also be disturbed by artificial lighting. Box 10.14 gives an example of a law regarding light restrictions in the USA.

Box 10.14 ***Example of requirements for nocturnal construction activities***

> In South Carolina, USA, the law requires lights visible from the beach to be extinguished or shielded from 1 May to 31 October, because of the potential impact on the behaviour of a local protected turtle species. The penalty for not doing so can be a fine of $895 per incident.

10.3.1.4 Pollution avoidance – air and water quality

Construction activities must comply with any statutory requirements or local regulations for air and water quality. Local restrictions on water and air quality may vary with location along a levee alignment.

Air quality

Licenses and permits may be required for:

- operation of equipment that emits fumes
- disposal and/or burning of construction debris and vegetation from clearing and grubbing activities
- waste material processing such as bio-remediation/composting.

Good practices are:

- **dust reduction:** spray water to minimise dust from roadways
- **truck bed covers:** soil and debris in trucks should be covered during transport close to residential areas
- **avoid on-site burning:** of site clearance raisings and refuse near businesses and residences
- **use of:** fuel efficient engines and regular servicing of plant and equipment.

Water quality

Licenses and permits may be required for:

- abstraction of groundwater, when dewatering an excavation
- discharge of construction wastewater to a natural watercourse or sewer
- operation of equipment that emits fumes
- disposal and/or burning of construction debris and vegetation from clearing and grubbing activities
- waste material processing such as bio-remediation/composting.

Pollution avoidance strategies are required when working in coastal or fluvial locations. In addition, emergency action plans should be put in place to deal with potential pollution incidents before setting up the site.

Good practices are:

- flow meters to quantify water discharges from site
- management of rainfall runoff – provide turbidity barriers, silt fences and/or trash traps to filter dust etc in runoff from construction site and detention ponds for storage and settlement of solids from runoff. Installation and maintenance of isolated truck and equipment washdown areas with sediment recovery etc
- management of wave-borne sediment using techniques such as geotextile filters
- spillage control – ensure that all liquids are appropriately stored to prevent spillage, also ensure that spillage prevention caps, valves etc seal properly and that storage tanks are adequately banded
- provision of specialist equipment to tackle oil spills in the event of any spillages
- use of materials and chemicals that are environmentally neutral or biodegradable (eg lubricants and hydraulic oils)
- waste – ensure that appropriate containers are available for the collection and disposal of all wastes
- wastewater control – ensure that all wastewater produced on site is disposed of appropriately and cannot enter controlled waters.

If land or water become contaminated through the spillage of oils, lubricants or other substances, remediation will be required. This will take place after the initial clean-up and will require advice from the relevant environment authority and/or specialist consultants.

10.3.2 Access routes and traffic

10.3.2.1 Defining accesses

Access routes consist of permanent or temporary routes to the levee from outside the construction area, and ramps that provide safe transit from the natural grade to the levee crest (Figure 10.14). Safe access routes are critical to ensure that construction materials can be delivered on time and labour and equipment costs are kept within budget. Regular maintenance of haul or access roads needs to be performed to minimise the risk of delivery interruptions during construction.

Figure 10.14 Levee access road in the USA (courtesy USACE)

Inadequate access routes can severely affect overall project programme and pose potential safety hazards. The constructor should study possible access routes, both overland and by water, to and from the levee construction site, the borrow pits and the materials handling areas. Adequate access routes will exist, or temporary roadways that meet the demands of construction equipment and are acceptable to any local parties affected.

The use of public roads for site access may be restricted (ie within the consents for construction). Such restrictions may include weight and width controls, parking controls, steps to minimise pedestrian conflict and low headroom access routes. Even if these aspects are covered within the contract, local police and the local authority should be consulted when addressing potential traffic issues and agreeing on a workable site access that does not compromise public safety.

Acceptable access routes should:

- have the ability to withstand construction equipment high wheel loads
- provide adequate space for movement and manoeuvre of heavy equipment required for construction
- provide a safe working environment, and ensure the safety of the public
- not violate local planning restrictions on noise, vibration and air quality
- not create significant interference to normal traffic flow
- have adequate clearance between the roadway and overhead electric power and communication utility lines, with clear warning signage and demarcation
- not exceed local environmental restrictions (for example, spillage of material from transport vehicles, air quality, noise limits, or operation time limits)
- provide access along the project right of way and other off-road alternatives.

Alternative access roads should be designated in the event that they become necessary to accommodate constructor proposed changes to the plan or in the event that primary ways become unavailable.

Table 10.13 provides more detailed access considerations, but general good practices include:

- proactive traffic plan co-ordination with the local municipality, highways authority or regulatory body to ensure no delays are anticipated during the project
- development and implementation of a public affairs plan using multiple media approaches (ie, meetings, websites, call-in numbers etc)
- starting permitting process as early as possible, involving all necessary parties and regulatory agencies
- acquisition of all necessary access agreements/permits and compliance with any restrictions imposed
- all areas required for construction activity should be acquired before construction and should ideally accommodate all future development of the levee system (see Section 9.5)
- consider compulsory purchase of access and construction areas as an approach of last resort, unless there are long-term maintenance benefits. In some cases, compulsorily purchased land can be re-let for farming under licence after construction is completed.

Table 10.13 Access considerations

Access and events issues	Good practice
Permitted routes (roads, waterways, locations for jetties)	Take account of considerations on the access routes to the site, such as: • no heavy traffic through particular built up areas • acceleration/deceleration lane to be constructed at highway access point • turning restrictions into the site access • weak bridges/restricted headroom/narrow roads/sharp bends • navigation considerations.
Access points	• each access point to the construction site should be identified, the agreed route to the nearest main road, and the routes to be used by lorries to access the road network • where possible, the access should be arranged so that lorries enter and exit the site in a forwards direction.

Table 10.13 Access considerations (contd)

Public rights of way	• alternative routes are preferable to mixing the public with construction plant • safe provision for footpaths and tracks crossing the site or site access • keep crossing points clean and clear of obstructions • adequate sign-posting of changed or alternative routes and associated information provision on length of disruption or any permanent changes.
Special localised traffic considerations may arise such as traffic control during peak traffic times, high traffic volume roadways, evacuation routes etc	• planning and co-ordination with the highway authorities, municipalities and local project team are necessary steps to avoid disruptions in traffic during the construction process.
Inter-relation with other construction projects	• developing an understanding of the interrelation and phasing of the construction work activities on the respective projects • shared access if possible and relevant.

10.3.2.2 Managing site traffic

It is important to manage site traffic, because it can cause delays to local traffic and create a safety hazard both on and off site. People living and working near the site are often annoyed by emissions, noise and the visual intrusion of queuing vehicles. An organised site with well-managed traffic activities (and storage of materials as close as possible to where they are needed) can provide a positive perspective to local residents. If a levee is constructed or modified in an urban area, this can be an important issue because of the potential for issues such as traffic tie-ups for residents, trip delays, delayed deliveries to the site, loss of access during critical construction times. To minimise these impacts consider the following:

- when ordering deliveries, ensure that all drivers are aware of traffic restrictions at and around the site
- plan the timing of deliveries to avoid vehicles waiting outside the site boundary
- load and unload vehicles off the highway, where possible
- designate queuing areas where several deliveries are likely to take place over a short period
- mitigation of construction traffic queues and their impacts, particularly in the summer:
  - instruct drivers to switch off engines when vehicles are waiting
  - avoid queuing outside buildings, as windows will most likely be open
  - where possible try to avoid waiting vehicles reducing the amenity and recreation value of surroundings areas
  - in urban areas consider allocating a waiting area some distance from site and only calling in deliveries when access to the site is clear
  - consider the use of in-cab communication systems to maintain control over lorry movements.

Site staff car traffic often annoys the public and so the following should be considered:

- arranging designated parking areas
- prevent staff parking in unsuitable areas and ensure that restrictions are obeyed
- implementing a park-and-ride or car-share scheme
- avoiding monopolising public car parking areas, especially those used during the summer by high numbers of visitors to the area.

Sometimes construction sites are blamed for disturbance caused by vehicles that are not associated with the site. To avoid this it may be helpful if site vehicles display some visible identifying marks. While this may not be appropriate for individual deliveries it can be done for the main contractor’s vehicles and for regular delivery vehicles.

10.3.2.3 Alternative delivery methods

When undertaking certain projects, delivery of materials may be possible by sea or river and should be investigated because of the clear advantages of minimising traffic disruption and because overland delivery of construction equipment or materials can pose significant delays and permit restrictions. However, there is a risk that the sea or river conditions will disrupt deliveries.

The constructor may determine that an adjacent watercourse would provide a cost effective method of delivery of materials or equipment. Regulation and control of waterways is usually presided over by local planning organisations.

Constructors should notify the owner of their intentions, and contact the river and harbour regulatory organisations to determine legal requirements, data required and time involved in securing licenses and permits for transport vessels to accommodate construction purposes. Special measures may be required to minimise risk of pollution of waterways and docking areas. Licenses may be required for:

- boat and barge transport companies operators
- temporary barge and boat vessel safety and operational certification
- construction of temporary docking, anchorage, and unloading facilities near the construction site (see Figure 10.15). Permits should show any prescribed separation between a temporary dock and the navigation channel.

Figure 10.15 Transport of material over the river Ijssel for the reconstruction of the quay Pothoofd in Deventer, the Netherlands (courtesy Harry Mols)

10.3.3 Managing archaeological remains and utilities

Archaeology

Historically early settlements were along watercourses. Such archaeological remains can be very precious, especially in old urban areas and need be dealt with carefully during construction. For example:

- along rivers in the USA, Native American settlements and burial grounds are often encountered
- in the Netherlands many levees are hundreds of years old and contain relevant archaeological artefacts as illustrated in Figure 10.16.

Figure 10.16 Historical city wall in the Netherlands (courtesy Harry Mols)

It is vital to investigate at an early stage, preferably during planning and consultation, to avoid disruption to construction activities, and unwelcome pressure on the ensuing archaeological excavation (Table 10.14).

Table 10.14 Archaeological considerations

Archaeological issues	Good practice
Archaeological sites	• carry out thorough desk study of historical records and maps • consult local museum or archives holders, regional archaeological groups and if applicable national archaeological and historical organisations • archaeological dig prior to construction works • where archaeological finds may be anticipated briefing and raised awareness of the construction team • specific construction methodology or phasing • consider full time archaeological watching brief.
Historically significant (protected) sites	• carry out thorough desk study of records and maps • consult local museum or archives holders, regional archaeological groups and, if applicable, national archaeological and historical organisations • briefing and raising awareness of the construction team • specific construction methodology or phasing • full time expertise on site.

Utilities

Failure to identify all local utilities (pipes, cables, bridges etc) can pose considerable problems for excavation and movement of large construction equipment (cranes, scrapers etc) during construction.

The constructor should verify the types, locations and acceptable clearances (below and above grade) from utilities such as electric communications and power lines, water and gas lines, and drainage structures crossing or in close proximity to the project features. Recommended procedures should be agreed upon to minimise risk to those features during construction. If utility relocation is a possible alternative representatives from the utility should be included in detailed planning of the relocation procedure. Table 10.15 presents some utility considerations.

Table 10.15 Utility considerations

Utility issues	Good practice
Known utilities in the site area	• utility crossings can be found in most levees/flood walls and proper co-ordination with the utility owners is required for a successful project. Certain requirements for construction around utilities may delay or halt construction activities • in some cases the utilities have to be realigned before or during the levee reconstruction project.
Unknown utilities in the site area	• thorough investigation of utility owner records • thorough inspection of the site (see Chapter 5) • non-intrusive ground investigation, such as ground radar and cable detection equipment.

10.4 FUNDAMENTALS OF EARTH CONSTRUCTION

Section 10.4 focuses on the specifics of earth material works. This section covers:

- availability of materials (see Section 10.4.1)
- suitability of the material (see Section 10.4.2)
- soil testing and the corrective actions (see Sections 10.4.2.1 to 10.4.2.4)
- elementary operations (vegetation management, excavation, fill, treatment and compaction) presented in relation with the equipment (see Section 10.4.3).

10.4.1 Availability of materials

Material availability at the project site or at local borrow areas is a primary factor in the success of a project. Preliminary design efforts to define available borrow material can be very limited in scope, leaving the responsibility to the construction management and constructor to clearly identify sufficient acceptable borrow material to complete the project. This may require additional sampling and analysis to identify sufficient and satisfactory materials needed in order to ensure the feasibility of project success.

Good practices are:

- the design/construction documents should summarise all investigations and testing completed by the owner's engineer
- the design documents should summarise all assumptions regarding the processing that the constructor will be required to perform to achieve proper moisture and density limits in the completed structure
- approved primary material sources as well as secondary sources should be identified before the start of the project
- agreements with material sources should be put in place soon after construction contract is executed.

Materials from the existing foundation, or from local or transported borrow sources need to comply with the design's quality and environmental suitability requirements. For economic and environmental reasons, material availability should be identified in the preliminary investigations at or near the project site, minimising haul distances. At times it may be necessary to use marginal or poor quality materials when more suitable soils are not available. When using sub-standard materials, the designer may require differing treatment, blending, extraction and/or compaction criteria.

Owner furnished material could be used if local availability is scarce or if cost prohibitive. If there is a risk that costs may increase due to increased demand, the owner should investigate pre-ordering material before the start of the contract and providing materials rather than constructor supplied materials.

10.4.2 Suitability of materials

It is important to note that suitability of materials may be based primarily upon compliance with soil classification characteristics or index properties (for suitable index properties, see Sections 7.8 and 9.12). While the material behaviour may comply with grain size and plasticity requirements, soil, having unsuitable environmental characteristics such as heavy metals, may be allowed for use under special treatment processes. Some specialised treatment processes of unsuitable soil index properties or unsuitable environmental properties may include innovative improvement methods such as identified in Section 9.12.

The use of sustainable materials, or using all available local materials including recycled materials, while further minimising hauling operations, can be managed and constructed so as to improve environmental acceptance of the levee works. Maximising the use of local materials, including recycled materials for environmental sustainability of the levee works, supports the 'cradle to cradle' concept.

Figure 10.17 shows an example of borrow material for levee construction in New Orleans, USA.

Figure 10.17 Hydraulic excavator handling soil (courtesy USACE)

Concerns for properly constructing a levee project can include some or all of the following issues:

- differentiating suitable and unsuitable soils
- evaluating pre-approved borrow sources
- determining limits of borrow source
- revealing borrow materials variations
- de-watering
- excavation, hauling, and handling requirements
- excavation and fill volume measurements
- stockpiling limitations at the project site
- blending or treatment processes, where required
- logistical aspects/limitations.

Methods for satisfying these issues generally require the proper evaluation of the available soils and good selection of borrow material that is compliant with design expectations. The activities required to properly plan and prepare for the levee earthworks operations will include the following:

- identify the contract specified borrow sources

- verify allowable soil classifications
- identify the soil types available
- investigate potential for expansion of the available borrow sources, where needed
- determine any measures necessary to process the borrow soil to acceptable moisture content before hauling
- determine any limitations of the project site for delivery and processing of soil
- where required, determine equipment and processes needed for allowable blending operations
- investigate additional borrow sources if needed.

Having determined and defined the available soils for use in the project, the next step should be to identify the equipment required for compacting the selected materials to the required degree. This may involve both of the following actions:

- conduct one or more field test sections (see Section 10.5.1) and determine roller pattern
- conduct testing in accordance with the contract requirements for compliance verification.

Constructors will generally perform these actions as early as practicable to better define equipment needs and the processes that will comply with the contract specifications.

The constructor may be provided information such as shown in Figure 10.18, identifying available borrow with initial testing for moisture and classification of a number of samples. The constructor is likely to be required to conduct further sampling and testing for current moisture content and for density testing of all the approved soil types, which allows for planning excavation, drying, transporting, spreading and compacting materials on the levee. Testing results enable the constructor to control this process, to ensure compliance with the contract specification. When the volume of borrow material is insufficient in quantity for the levee project, the constructor will need to conduct additional sampling to determine acceptable materials in an extension to the borrow area. This may involve removal of unsuitable materials and dewatering.

***Figure 10.18** Borrow plan for levees in New Orleans, USA (courtesy USACE)*

Moisture control of soil is important in order to determine when materials are acceptable for delivery from borrow locations to the placement site. It is generally best to carry out any adjustments that may be necessary to the moisture content of the material at the borrow area, where typically there is more area for spreading, drying, and mixing. Carrying out the material preparation before delivery to the project site will avoid costly disruption to the project.

10.4.2.1 Soil testing and corrective actions

Test methods and frequency plans are used to provide information on the control of compaction rate, water content and its variation during construction of the embankment. These are further specified in the levee construction contract in order to verify compliance with the various other materials used in the project. Test methods and frequency plans are project specific, depending on the geometry and design of the embankment, the available materials, and the design that has been specified. Testing frequency is usually established in the contract specification.

Construction methods include various phases of testing for a levee project in order to ensure compliance with design expectations. The following phases of testing are necessary to ensure that minimum quality standards are met, while further testing will be necessary for critical activities where noted:

1 Preparatory phase testing:
   - a On-site borrow pit and at off-site borrow source testing.
   - b Existing materials testing including foundation materials testing.
   - c Identify the soil types available and amount of acceptable borrow material available.
   - d Materials compliance testing.
   - e Determine on-site soil and imported borrow soil characteristics related to compaction:
     - i Material suitable, marginal, or not suitable, standard or modified compaction efforts, maximum dry density and optimum moisture content, specific gravity etc.
     - ii Test section performance and verification.
     - iii Identify the equipment required for spreading and compacting the selected materials to the required compaction effort.

2 Initial phase testing:
   - a Identify full compliance with the contract.
   - b Establish level of workmanship.
   - c Resolve all differences.

3 Follow-up phase testing:
   - a Validate full compliance with the contract.

Within each identified borrow source, it is important to determine all the existing soil characteristics to identify suitable material, marginal quality soils, or soils that are not suitable. Once the quality of the available soil is determined and accepted for use, the material must be further evaluated for determining the behaviour under compaction. Compaction behaviour is identified initially from laboratory standard or modified compaction efforts, where the maximum dry density and optimum moisture content is determined for each given borrow sample of soil (see Chapter 7). Other laboratory testing such as classification testing and sometimes specific gravity determination is further conducted in order to properly identify the characteristics of the soil. It is important to note that during construction, a source of borrow material frequently changes, which may raise concerns about its suitability, and for understanding control for differing compaction characteristics of the changing soils. Establishing frequency of testing is important in order to monitor changes in soil borrow characteristics that can affect the construction performance of the material (see Chapter 7 and Section 10.4.2.3).

To gain an understanding for the various sampling and testing required for levee construction, the scope of typical testing for investigation (see Chapter 7), analysis tools and design (see Chapters 8 and 9), and post construction validation (see Chapter 7) the reader is referred to Table 10.16.

Table 10.16 Typical construction test methods (images courtesy USACE and Fugro)

Type of test	Photo	Purpose	Description
Sampling (Section 7.9.7)		Verify acceptable materials	Sampling soil
Sample reduction (Section 7.9.7)		Reduce sample to size required by test method	Reduction of soil samples sizes by quartering or splitting to minimum mass for individual tests
Sieve analysis or grading (Section 7.8.3.1)		Determine retained and passing no. 200 (75 µm) sieve	Materials finer than no. 200 sieve in by washing
Sieve analysis or grading (Section 7.8.3.1)		Determine clay, silt, sand and gravel sizes	Particle-size analysis of soils. (used with or without hydrometer analysis where specified)
Sieve analysis or grading (Section 7.8.3.1)		Determine sand and gravel sizes	Particle-size analysis after wash no. 200
Unit weight (Section 7.8.3.1)		Determine mass per unit volume	Unit weight and voids

Table 10.16 Typical construction test methods (images courtesy USACE and Fugro) (contd)

Specific gravity (Section 7.8.3.1)		Determine density in relation to density of water	Specific gravity of soils
Atterberg limits (Section 7.8.3.1)		Determine liquid and plastic conditions of fine grained soil	Liquid limit, plastic limit and plasticity index of soils
Moisture content (Section 7.8.3.1)		Determine moisture or water content of soil.	Laboratory determination of moisture content of soil (oven drying)
Moisture-density relationship (Section 7.8.3.2)		Determine density of soil at varied moisture conditions to select maximum dry density and optimum moisture	Moisture-density relations of soil and soils aggregate mixture using 2.5 kg Rammer and 300 mm drop (standard effort) or 4.5 kg Rammer and 450 mm drop (modified effort)
Field wet density, sand-cone (Section 7.8.3.1)		Determine in-place density of soil and relate it to its respective moisture-density relationship	Density of soil in place by the sand-cone method
Field wet density, nuclear (Section 7.8.3.1)		Determine in-place density of soil and relate it to its respective moisture-density relationship	Density and moisture of soil and soil-aggregate in place by nuclear methods (shallow depth)
Field wet density, drive cylinder (Section 7.8.3.1)		Determine in-place density of soil and relate it to its respective moisture-density relationship	Density of fined grained soil by drive cylinder method

Table 10.16 Typical construction test methods (images courtesy USACE and Fugro) (contd)

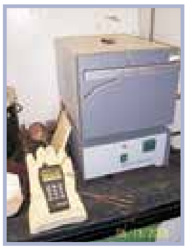
Field moisture content, nuclear (Section 7.8.3.1)		Determine field in-place moisture of soil	Moisture content of soil and soil-aggregate in place by nuclear methods
Field moisture content, direct heat (Section 7.8.3.1)		Determine field in-place moisture of soil	Moisture content of soil by direct heat (hot-plate) method
Field moisture content, microwave (Section 7.8.3.1)		Determine field in-place moisture of soil	Moisture content of soil by microwave method
Organic content (Section 7.8.3.1)		Determine amount of organic material in a soil	Organic content determination by furnace
Salt content (Section 7.8.3.1)		Salt content determination of soil per litre soil moisture by chemical analysis	The salt content may not be too high, because fine particles in the clay may slowly dissolve (dispersion) in changing fresh-salt water conditions. This will lead to a lower erosion resistance
Chalk content (Section 7.8.3.1)		Chalk content determination of soil by mass loss in hydrochloric acid treatment	The erosion resistance of clay will decrease if the chalk content gets too high
Soil classification (Section 7.8.2)		Determine classification of a soil by testing for particle size and Atterberg limits	Soil classification – requires grain-size determination and Atterberg limits or other test results for classifying soils

Table 10.16 Typical construction test methods (images courtesy USACE and Fugro) (contd)

Hardness indication according to Sowers (Section 7.8.3.3)		Determine field consistency of a soil	Simple field identification test to estimate hardness and consistency
Unconfined compression (Section 7.8.3.3)		Verify soil strength	Determining unconfined compressive strength of constructed materials
Permeability (Section 7.8.3.5)		Verify material permeability	Falling head rising tail hydraulic conductivity in a flexible wall permeameter used to determine construction compliance

10.4.2.2 Levels of testing for various levee construction

Construction of levees can be based on simplified earthwork specifications, typically for development of non-critical structures such as back levees or berm. For some of these non-critical construction works and repair work, the designer may simply require satisfactory soils with an allowable range of moisture contents and compacted in-place with a specified number of passes for a particular piece of compaction equipment. For this type of specified construction, minimal testing may be prescribed. The testing required for such works may merely include soil classification and water content determination testing, without requirements for density verification.

Construction of more critical components are likely to require a more prescriptive earthwork construction method that focuses on performance criteria and compliance with an extensive range of quality control and quality assurance testing. Such levee construction for new, repaired, and raised levees may require most of the tests noted in Table 10.16. Minimum performance criteria may be established for allowable soil type, allowable moisture content range, and minimum density required. Different borrow soils or sources may require varying degrees of effort at the levee placement sites in order to condition the soils to meet the acceptable criteria that is compliant with the design expectations. Further information on the advantages and disadvantages of using method specifications compared with performance specifications is given in Section 9.12.

Obtaining the required results for the specified testing will take a length of time that varies from hours to days. Waiting for these acceptance test results may at times conflict with the programme rate of placement of levee materials, and in these cases it may be prudent for the designer to allow for more expedient test methods for determining moisture and compaction. In all cases standardised methods should be used to avoid conflicts between various testing agencies.

10.4.2.3 Testing frequency

Testing frequency is generally specified directly within the contract documents for both quality assurance and quality control activities. The frequency of testing will vary with the complexity of the project, with unexpected variations in materials, and as a consequence of any failing results. Table 10.17 identifies some commonly used frequencies for various testing (see Section 7.9.7).

Table 10.17 Types of tests and common frequency

Type of test and purpose	Frequency		
	Backfill and levee	*Sub grade*	*Backfill for culvert trenches, walls, culverts, building perimeters*
Field density with moisture content **Purpose:** determine in-place density and moisture in relation to design expectations	Two per lift for each increment or fraction of 1600 m² placed during each eight hour shift	Two per lift for each increment or fraction of 800 m² placed during each eight hour shift	• culverts and utility trenches: one per lift for each increment or fraction of 150 lineal metres of backfill • walls and building perimeters: one per lift for each increment or fraction of 60 lineal metres of backfill
Grading with Atterberg limits (from compacted material) **Purpose:** determine characteristic of soil for classification and compliance	One per five field density tests		
Moisture-density relationships w/ grading, Atterberg limits, specific gravity, and classification (from bulk sample) **Purpose:** determine performance characteristics of soil and its relationship to determined soil classification	One per five field densities (with not less than one per type of material) for the first 25 field density tests. Thereafter, one additional test for each change in material		

10.4.2.4 Failed tests and resolutions

Test methods are used to provide information on the control of compaction rate, water content and its variation during construction of the embankment. Test methods further identify results for verification of project specified compliance for levee construction. Test methods follow industry standards such as Eurocode, British Standards (BS) or American Society for Testing and Materials (ASTM) Standards and are typically identified in the construction contract specifications.

Non-compliance with the prescribed minimum testing standards can sometimes lead to questions concerning validity of data used for verification of materials and for compliance with expected design criteria. Analysing data for construction compliance can at times reveal discrepancies in sampling, preparation of materials, measurement of sample parameters, procedural mistakes, and computational errors. Potentially errors can lead to an additional requirement for sampling under the direction of quality assurance or quality control in order to verify proper material compliance.

The review of test reports by experienced materials personnel can reveal discrepancies in the data, where engineering judgment may be necessary and can potentially include involvement of the design engineer. The consequences of failed results require careful analysis and a decision taken based on comparison with the specified requirements, visual examination and engineering judgment. Correction depends on acceptability but is very specific to the material location and problem encountered. Corrective action to prevent recurrence is an essential part of quality assurance and quality control. Table 10.18 identifies test methods, common discrepancies, and possible resolution.

Table 10.18 Types of tests, common discrepancies and possible corrective actions

Type of test	Common discrepancies	Possible corrective action
Sampling disturbed samples	• sample not representative.	• re-sample.
Sampling undisturbed samples	• sampling disturbance • transporting disturbance • trimming disturbance.	• analyse test results for potential degree of disturbance • re-sample.
Grading distribution	• improper sample size • inaccurate measure of mass • insufficient sieving • inaccurate calculations • poor quality equipment • readability of equipment insufficient • small grains get lost during sampling.	• re-sample • where possible recalculate data • use undisturbed samples instead of disturbed samples.
Atterberg limits	• improper sample preparation • inaccurate measure of mass • procedural deficiencies • inaccurate calculations • poor quality equipment • readability of equipment insufficient.	• re-sample • where possible recalculate data • use fall cone instead of Casagrande device to measure liquid limit • use three-points method instead of one-point method.
Organic content	• improper sample preparation • inaccurate measure of mass • procedural deficiencies • inaccurate calculations • readability of equipment insufficient.	• re-sample • where possible recalculate data • take roots and plant remains in account • use more accurate test method.
Soil classification	• inaccurate classification (visual method).	• re-evaluate test data • order additional testing • do classification in laboratory instead on field.
Moisture-density relationship	• improper sample preparation • inaccurate measure of mass • procedural deficiencies • inaccurate calculations • readability of equipment insufficient • improper curve fitting.	• re-sample • where possible recalculate data.
Lab moisture – oven	• improper oven temperature • inaccurate measure of mass • inaccurate calculations • readability of equipment insufficient.	• verify calibration oven • re-test.
Field moisture – nuclear	• calibration not conducted • correlation not performed • incompatible materials in soil • user is not certified.	• re-calibrate machine • determine standard counts • re-test • choose an alternate method appropriate for those materials.
Field moisture – microwave	• improper sample preparation • inaccurate measure of mass • procedural deficiencies • inaccurate calculations • readability of equipment insufficient • soil gets wet after testing.	• re-test • choose an alternate method appropriate for those materials • mix dry soil lumps with wet soil parts properly on solid bottom plate.

Field moisture – hot plate	• improper drying temperature • inaccurate measure of mass • inaccurate calculations • readability of equipment insufficient.	• adjust drying temperature • re-test.
Field wet density – nuclear	• calibration not conducted • correlation not performed • incompatible materials in soil • user is not certified.	• re-calibrate machine • determine standard counts • re-test • choose an alternate method appropriate for those materials.
Field wet density – sand replacement	• density of sand used not determined properly • apparatus not calibrated properly • surface calibration needed for uneven surfaces • sample volume too small • inaccurate measure of mass • inaccurate calculations.	• re-determine sand bulk density • re-test and perform surface calibration • re-test and obtain larger sample volume.
Field wet density – drive cylinder	• drive cylinder not calibrated for volume • inaccurate measure of mass • inaccurate calculations • wood or gravel encountered in sample.	• calibrate drive cylinders for volume • re-test • perform an alternate procedure.

Project adaptation after failed test results

Where equipment, staff, and procedures comply with specified requirements, and when testing results fail to meet minimum prescribed criteria, a non-conformance is generally issued. Failed testing results or identified non-conformances, require action by the quality management staff in order to resolve the issue by applying corrective actions. Consequences of failed results require analysis and a decision based on comparison with the specified requirements, visual examination and engineering judgment. Correction depends on acceptability but is very specific to the material location and problem encountered. Corrective action to prevent recurrence is an essential part of quality assurance and quality control.

The consequences and options toward failed tests are described as follows. A subdivision is made among material requirements, water content and compaction rate. To decrease probability of failed tests the testing procedure should be divided into more phases, from rough preliminary testing in the borrow area to final testing after placing and compaction. To diminish the consequences of failed tests the recovery options should be investigated in advance and described in the project plan. Tables 10.19, 10.20 and 10.21 identify failed test results and potential corrective actions.

Table 10.19 Options after encountering failed tests in soil (material requirements)

Test	Abnormality	Options
Erosion resistance	Too low	• do not use this soil as top layer.
Organic material content	Too high	• anticipate the settling of the levee by adding extra height • use a soil/subsoil layer to cover shrinkage cracks.
Salt content	Too high	• do not use this soil in fresh/potable water channels.
Chalk content	Too high	• anticipate settlement of the levee by adding extra height.
Percentages of fines (<2µm)	Too high	• apply soil/subsoil layer for development of grass cover.
Percentages of fines (<2µm)	Too low	• apply soil/subsoil layer for development of grass cover • check if soil is not too sandy.
Toxicity	Too high	• dig out contaminated material, and isolate • replace with acceptable material.
Extreme colouring/strong smell/impurities/sand lenses or layers	One of these features is present	• dig out contaminated material, and isolate • replace with acceptable material • set aside any soil loads with impurities for expert judgement.

Table 10.20 Options after encountering failed tests in clayey soils (water content)

Test	Abnormality	Options
Water content	Soil is too wet	• dry the soil in the sun/wind • dry the soil in not too thick deposits above groundwater • apply drainage layers or trenches • use soil from another source.
Water content	Soil is too dry	• break up dry lumps on a hard surface • add water and rework.

Table 10.21 Options after encountering failed tests in soil (compaction rate)

Test	Abnormality	Consequences/options
Compaction rate of top layer	Too low	• apply more compaction passes • anticipate settlement of the levee by adding extra height • anticipate instability of the levee by flattening slopes.
Compaction rate of deep layer	Too low	• dig out the upper layer and redo the compaction • anticipate settlement of the levee by adding extra height • anticipate instability of the levee by flattening slopes.
Compaction rate	Indistinct testing results	• perform multiple types of tests • test bigger samples.
Number of passes during compaction of material	Too low	• test the effect of a range of compaction passes • test impact of a range of compaction equipment • apply more compaction passes.

10.4.3 Equipment and elementary operations

Construction equipment used in the construction of levees is commonly found in other civil and site activities, such as roadways and bridges, utilities and construction of buildings as well. The major differences are that for levee construction, special considerations are required to ensure the permeability and stability of the embankment, and more generally to take into account the hydraulic environments.

Section 10.4.3 lists and describes common types of construction equipment used in the various steps of levee construction, including typical capacities, and advantages/disadvantages of each of the equipment types.

The equipment used for levee construction falls into five basic categories discussed in the following subsections:

- clearing and grubbing existing trees and vegetation and establishing vegetative cover (see Section 10.4.3.1)
- stripping and excavating, placing and spreading (see Section 10.4.3.2)
- loading and hauling (see Section 10.4.3.3)
- compacting the soil (see Section 10.4.3.4)
- treating the soil and installing sheet piling or cut-off wall (see Section 10.4.3.5).

There are some pieces of equipment (such as towed scrapers or self-propelled compactors) that can serve in multiple categories. There is also an entire range of marine and fluvial-based equipment (mostly barge mounted) that is similar in function to these land-based items.

Some of the underlying questions that should be addressed in equipment selection, which are specific to levee construction include:

1 Is the equipment of sufficient size and horsepower? Does it have the required stability to perform its intended function, including safely traversing adverse grades, unstable access roads etc?

2 Is the equipment capacity adequate to ensure efficient execution of the work? For example, are there enough trucks assigned to keep the spreading and compacting equipment working efficiently? Are there too many trucks, producing a queue and congestion on the embankment?

3 Is it compatible with the other pieces being selected? For example, is the loader capable of reaching over the sideboards of the dump truck?

4 Are the weight and dimensions of the equipment compatible with the size, type and level of the embankment being constructed? For example, is the roller type capable of achieving the desired density without rutting or shearing the embankment foundation? As trapezoidal embankments are raised and narrowed, it is often necessary to change to smaller, more agile equipment.

5 Can the equipment handle the levee fill material? Plastic clay will be excavated more easily by special attachments or teeth on bucket. The workability depends on the water content of the fill material. Driving with tyres on wet clay is not advisable. A sheepsfoot roller is not suitable for compacting of clay in cold and wet climates (the Netherlands, Norway etc), but may be useful in dry and warm climates (South of France, Thailand etc).

10.4.3.1 Vegetation management

Table 10.22 describes equipment that is used during activities in relation to vegetation. These include the clearing of trees and brush, and the equipment used to distribute vegetative cover seed and mulch products.

10.4.3.2 Equipment for stripping, excavating, spreading and placing

Table 10.23 presents equipment for stripping, excavating, spreading and placing of soil.

Stripping is normally the removal of 12 cm to 50 cm of organic soils, roots etc in order to expose underlying stable and inorganic layers to serve as the levee foundation.

Excavating is the removal of soil and rock materials, either to obtain materials for use elsewhere in embankments, or to create the desired geometry for construction of embankments and their associated features (drains, cut-off trenches etc).

Spreading is the distribution of materials in layers of uniform thickness (lifts) to construct levees and their associated features.

Placing relates to the placement of material without precise specification requirements.

Some equipment described in Section 10.4.3.3, such as tracked and wheeled loaders, can also excavate (from grade and stockpile) and place materials.

10.4.3.3 Equipment for loading and hauling

Table 10.24 describes equipment that is used to load and transport earth and rock materials to the embankment.

Other equipment already previously described, including scrapers, excavators and cranes with clamshell buckets, can also be used for loading.

10.4.3.4 Compaction

Table 10.25 describes equipment that is used to densify materials by the action of static (gravity) rolling, vibration, and/or kneading earthen materials to produce the desired density. In countries with a warm and dry climate other equipment may be preferable to that used in cold and wet countries

10.4.3.5 *In situ* soil treatment, cut-off walls and installation of piles

Table 10.26 describes equipment that is used to adjust moisture content of soils to enhance the compaction process, and/or to blend materials or incorporate additional materials. It includes also the installation of piles (sheet and H piling).

Table 10.22 Properties of equipment for vegetation management

Type	Photo	Uses in levee construction	Advantages	Disadvantages	Remarks	Rating
Chain saws ***courtesy USACE***		• felling trees and other woody vegetation.	Chain saws can is needed, especially for tight and congested sites to fell trees.	Very strict co-ordination and safety provisions are necessary.	Logging operations often accompany clearing if the trees to be removed are large or valuable.	Chain saws come with bar sizes ranging from 20 cm to 60 cm.
Skidders ***courtesy John Deere***		• removing and loading felled trees and logs.	Skidders can handle and load large logs.	Rubber tired skidders greatly disturb earth foundations.	Generally used as part of logging operations.	Skidders are generally 150 kW.
Tree shears and hydraulic axe ***courtesy Memphis District, USACE***		• cutting and handling trees.	Shears can both cut and handle trees. They can mitigate the danger of felling trees with chain saws.	Risks of falling tree to the equipment operator.	These are used when the clearing operation is large and widespread.	Shears are provided as attachments to hydraulic excavators or bulldozers. They can shear 20 cm to 60 cm trees.
Hydroseeding equipment ***courtesy Jerry Otto***		• used to distribute and sew grass seed, soil amendments, and mulch.	Used to sow seed, amendments, and mulch over large areas of completed embankment.	Operating on uneven terrain risks overturning.	Seed, lime, fertiliser and vegetable gum tack are the most common amendments to hydroseeding. Straw or hay mulch devices are mechanically driven.	Provided as appurtenances to water trucks or lorries, either directly mounted or on a trailer.

Table 10.23 ***Properties of stripping, excavating, spreading and placing equipment***

Type	Photo	Uses in levee construction	Advantages	Disadvantages	Remarks	Rating
Bulldozer (tracked) ***courtesy Stephanie Terry***		*Primary:* • excavating soil • ripping rock. *Secondary:* • compacting soils • moving/spreading materials • towing compactors • towing discs and ploughs.	Wide ranges of sizes and power available.	Must be transported over public roads by low loader.	Wide array of attachments (rippers, blades, etc) available.	Rated by flywheel power. Typically 50–600 kW.
Scraper (self propelled) ***courtesy Kevin McCollough***		*Primary:* • excavating soil and weathered or ripped rock. *Secondary:* • depositing and spreading embankment materials.	some models can self-load deposits and spreads in desired lift thickness.	expensive to operate dismantle to move (by low loader) over public roads.	Losing favour to more versatile use of hydraulic excavators.	Rated by flywheel power and capacity. Typically 250–450 kW and 10–35 m^3.
Scrapers (towed) ***courtesy Stephanie Terry***		Same as previous.	Can be used in conjunction with bulldozers.	Require some form of power take-off from companion equipment.	Same as previous.	Rated by capacity. Typically less than 10 m^3.
Hydraulic excavator (tracked) ***courtesy Stephanie Terry***		*Primary:* • excavating soil and rock • loading trucks • trench excavation. *Secondary:* • placing rock revetment and other embankment materials • unloading and placing pipe and other trench items.	• wide range of sizes and power available • can reach into tight locations and over/onto soft/unsuitable locations • available with a wide array of attachments for excavating, compacting, lifting, cutting etc.	• swing hazard for personnel and power lines • can be overturned if overloaded or when traversing unstable terrain • must be transported over public roads by low loader.	This is the one of the most versatile classes of equipment available for levee work.	Rated by flywheel power and bucket size. Typically 40–300 kW and 0.15–3.5 m^3.

Table 10.23 Properties of stripping, excavating, spreading and placing equipment (contd)

Excavator (wheeled) and backhoe/loader combinations ***courtesy Jacob Chavez***		Same as previous.	Can self-transport to and from job site.	Same as above. Wheeled excavators are normally used only where other wheeled equipment (trucks) can access.	Same as previous.	Rated by a variety of factors, including flywheel power and bucket size. 50–75 kW, 0.15–1.5 m^3.
Crane with dragline or clamshell bucket ***courtesy Stephanie Terry***		*Primary:* • excavating soils, especially below water level or in very soft terrain. *Secondary:* • lifting and placing pipe and associated items • loading materials • placing revetment.	Can be used to reach a wide radius of work area, including underwater.	Transport and erection are expensive and difficult.	Losing favour to hydraulic excavators with extended boom operations.	Rated by bucket size. Typically 1–2 m^3, but can be much larger for major dredging.

Table 10.24 Loading and hauling equipment properties

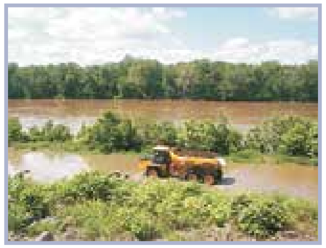
Type	Photo	Uses in levee construction	Advantages	Disadvantages	Remarks	Rating
Tracked loaders ***courtesy Kirk Dailey***		*Primary:* • excavate materials from grade or stockpiles. *Secondary:* • for very short distances can also transport, deposit, and spread materials.	• come in a wide variety of sizes and power ranges • can be adapted for excavating tough soils and weathered or ripped rock.	Must be transported over public roads by low loader.		Rated by flywheel power and bucket size. Typically 50–150 kW, 0.5–3 m^3.
Wheeled loaders ***courtesy Carlos Clarke***		Same as previous.	Same as previous.		Losing favour to tracked excavators.	Rated by flywheel power and capacity. Typically 50–1000 kW, 1–10 m^3.
Truck/lorry ***courtesy USACE***		*Primary:* • transport materials over public and haul roads. *Secondary:* • none.	• can travel public roadways • readily available in a wide variety of sizes and axle configurations.	Difficult access in rugged terrain or poorly maintained haul roads.	Most versatile and readily available method for material transport.	Rated by flywheel power, number of axles, and hauling capacity (weight or volume). Typically 50–500 kW, 1–3 axles, 1–25 m^3 or 1–70 tons.
Off-road dump truck ***courtesy USACE***		*Primary:* • transport materials on site. *Secondary:* • none.	• can traverse rugged terrain and steep grades • high volume and speed capability.	Cannot be used on public roads.	Available in articulated (cab/driver vs. load) configurations for added manoeuvrability and safety.	Rated by capacity. Typically less than 1 m^3.

Table 10.25 Compacting equipment properties

Type	Photo	Uses in levee construction	Advantages	Disadvantages	Remarks	Rating
Static smooth drum roller ***courtesy USACE***		***Primary:*** • compact granular materials. ***Secondary:*** • sealing embankment materials to prevent water intrusion.	Versatile for use on a wide range of materials Available in a wide range of sizes and types.	Prone to tipping and sliding on rough terrain.		Rated by flywheel power, number and type of axles or drums, and weight. Typically 50–100 kW, 5–40 tons, one to three steel drums and/or pneumatic axles.
Static tamping foot (sheepsfoot) roller ***courtesy Jerry Otto***		***Primary:*** • compact and mix clay materials. ***Secondary:*** • ensure penetration and bonding to underlying lifts.	Provides both compactive and mixing effort.	Usually must be ballasted and carefully controlled (speed and number of passes) to ensure optimal compaction.	These can be found in towed, self-propelled, and combination rubber tyre/tamping foot designs.	Self-propelled models are rated by variety of factors, including flywheel power, weight, and number/type of drums/wheels. Typically 100–300 kW, 10–40 kg, one or two drums. Towed models are rated by weight, typically variable up to 45 tons See static smooth drum and self-propelled ratings for more detail.
Vibratory rollers (both smooth drum and tamping foot) ***courtesy Stephanie Terry***		***Primary:*** • compact a wide range of soil and aggregate types. ***Secondary:*** • same as previous.	Can be used with or without vibration Generally lighter and more transportable than static rollers.	Often misused on soil types and moisture conditions, which are unfit for vibratory action (silts, high moisture content material etc).		Rated by flywheel power, weight, number of drums or axles, and available centrifugal or dynamic force. Typically 20–150 kW, 10–20 tons, one to two steel drums and/or pneumatic axles, and 2000–20 000 kg.
Rubber tyre rollers ***courtesy Joseph Koester, USACE***		***Primary:*** • compact a wide range of soils. ***Secondary:*** • large models (50T) also used to 'proof roll' foundations to ensure adequate bearing capacity.	Provide some mixing/kneading action for fine grained soils Good choice for compacting granular drainage sands and gravels.	Proof rolling models are large and difficult to manoeuvre.	These are available in towed or self-propelled models. This category also includes towed rollers with large rubber tyres and frame which can be ballasted (typically in the 40 ton range) for 'proof rolling' of foundation soils and embankments.	Rated by flywheel power, weight, and wheel configuration. Typically 50–150 kW, 10–20 tons, with three to four tyres on each axle.

Table 10.25 ***Compacting equipment properties (contd)***

Tracked equipment ***courtesy Pierre Hingle***		***Primary:*** • compact soils which are amenable to static compaction. ***Secondary:*** • seal embankments against water intrusion • Provide 'grouser' surface texture on embankment slopes to retard erosion.	These activities (compacting, sealing, grousing) can be incidental to excavation, stripping, and spreading.	Compactive effort limited by ground pressure and static weight of equipment.	Many homogeneous levees composed of high plasticity clay require only bulldozer compaction.	Rated by flywheel power (bulldozers and tracked loaders above). Ground pressure exerted may also be calculated using operating weight (5000–100 000 kg) and track contact area (1–6 m^2).
Self-propelled compactors ***courtesy Michael Siu***		***Primary:*** • compact a wide range of soils. ***Secondary:*** • spread materials.	Dual use machines.	• care must be exercised to limit activity to either spreading or compaction to ensure uniform embankment density • easily misapplied to wrong soil types.	These are available in a wide variety of drum types, configurations, weights, and sizes.	Rated by flywheel power, weight, number/type of drums/wheels, and presence or absence of vibratory capability. Typically 150–400 kW, 15–40 tons, one to three tamping or smooth drums/feet or tyres. Some models also offer vibratory compaction.
Hand-operated compaction equipment ***courtesy USACE***		***Primary:*** • compact embankment soils where the use of larger equipment might overload or otherwise damage nearby structures and pipes.	Can be more closely controlled than larger wheeled and tracked equipment.	• limited compactive effort and area of coverage • thinner embankment lift thicknesses are required.	Photo depicts what are commonly called 'jumping jack' impact compactors (on the right) and a small remotely controlled roller (on the left).	Varies. Rated by compactive force, typically 9–35 kN.

Table 10.26 *Properties of equipment for in situ soil treatment and installation of piles*

Type	Photo	Uses in levee construction	Advantages	Disadvantages	Remarks	Rating
Disc harrows and ploughs ***courtesy Stephanie Terry***		***Primary:*** • aerate soils to adjust moisture content prior to application of compactive effort. ***Secondary:*** • mix varied and other non-homogeneous embankments materials. Scarify previously compacted lifts to ensure bond to subsequent lift.	Facilitate compactive efforts	Equipment can be crowded as embankment profile becomes tighter	These are mostly towed devices, but there are many variations for wheeled and tracked vehicles	Due to the wide variety of devices in this category, there are no typical rating values. The drawbar HP of the towing unit (tracked or wheeled vehicle) must be matched to the device being towed and the soil conditions
Water trucks, lime spreaders ***courtesy Pierre Hingle***		***Primary:*** • adjust moisture content. ***Secondary:*** • spread additional materials.	Can be accomplished on the embankment grade	Real time testing or visual indication are necessary for control		Rated by flywheel power, number of axles, and capacity. Typically 50 to 500 kW, two to three axles, 5000 to 15000 l
Soil mixing equipment ***courtesy Yvonne Gibbons***		• improve strength or decrease permeability of existing soils.	*In situ* soil mixing permits the improvement properties of soils at depth, without complete removal and replacement	In-situ soil mixing requires more sophisticated and indirect method for monitoring and control	Cement or bentonite are the most common types of soil amendments used in soil mixing	Generally furnished as hydraulically driven attachments to hydraulic excavators or cranes, and rated according to the specific type of material to be mixed, and the diameter and depth of mixing required
Pile drivers ***courtesy USACE***		• installing H or Interlocking sheet piling.	Pile driving equipment permits the insertion of steel and vinyl piles through soil horizons, to reach rock or other more desirable strata	These are dangerous operations, operating with high energy and at great heights	Both impact and vibratory energy types are used	Vibratory hammers are rated by drive force 20 to 9000 kN. Impact hammers are rated by drive energy 10 to 1000 kNm

Box 10.15 Specialised materials

Figure 10.19 Seepage mitigation with deep soil mixing in Marysville, CA, USA (courtesy USACE)

Seepage control is necessary in regions, such as those in Sacramento Area, where sandy soils require cut-off walls at times to depths of 35 m. This may include the use of soil blended with cement and bentonite, providing typically a one metre thick cut-off wall through the centreline of the levee, requiring the constructor to develop mixture designs with on-site soils in order to comply with designed strength and permeability. Such designs require sufficient lead time for completion of the analysis at various ages for cement hydration, and are likely to require the construction of a test section for verification of contract compliance.

10.5 METHODS OF CONSTRUCTION

Having discussed in the previous section earthworks aspects, this section now focuses on the methods of construction at the scale of the levee. Aspects covered include:

- construction of levee test sections (see Section 10.5.1)
- stages of construction (see Section 10.5.2), mainly for new build levees
- aspects that differ between new build, adaptation, repair and decommissioning works (see Section 10.5.3)
- monitoring of levee construction (see Section 10.5.4)
- integration of non-earthwork features into levees (see Section 10.5.5).
- construction approaches to repair pipes and culverts (see Section 10.5.6).

10.5.1 Levee test section

It is well understood that compaction changes the physical properties of soils. Desirable characteristics of compacted fill are low compressibility and high shear strength. In addition, low permeability is essential for compacted fill in water retention structures such as levees. As outlined in Table 10.16, there are several different laboratory compaction standards and many different types of compactive effort used in construction of compacted fills. To be used effectively, compaction must be tailored to the soil type, moisture condition and subsequent environment of the compacted fill. A large variety of mechanical equipment (see Section 10.4.3.4) is available for compaction of soils, but soil type and moisture conditions will often dictate the type of equipment and methods of use. The choice of compaction equipment depends also on the intended function of the compacted fill. The requirement for low permeability in water retention structures precludes the use of equipment or methods that produce layering or laminations in compacted fill. Substantial variations in density in individual lifts are also to be avoided if a homogeneous fill is desired.

There is no type of compaction device that is completely suitable for all type of soils and situations. It is good practice to construct a levee test section in the field to select the best equipment types and lift thicknesses and to validate that design assumptions and specification requirements can be achieved before starting full scale production of a levee. The size, weight, compactive effort, and type of compactive device, along with the type of material and lift thickness, are all variables that can be considered in a test section. A test section does not eliminate the need for good quality control and testing, but it provides valuable information to the constructor and the designer about the performance of the selected levee materials, and the equipment used to spread and compact those materials. Figures 10.20 to 10.25 show different stages related to the levee test section and the assessment of soil compaction.

Test sections are used to evaluate or verify:

- moisture conditioning processes
- efficient excavation in borrow areas
- acceptable lift thickness
- maximum size of rock allowed in earthen embankment lifts
- mixing processes to achieve homogeneous moisture
- applicability of compaction equipment
- required number of passes to achieve specified density
- moisture/density relationships in the field
- adequacy of quality control and quality assurance operations
- preparedness for changing weather conditions
- adequacy of methods for lift bonding and benching into existing levee
- evaluation of compaction around special features or structures.

Figure 10.20

Spreading of material into uniform loose lift thickness before compaction (courtesy USACE)

Figure 10.21

Spreading of material from borrow to determine appropriate lift thickness before compaction. Wooden lathe are used to control loose lift thickness (courtesy USACE)

Figure 10.22

Compaction of material with a towed tamping roller to determine acceptable range of moisture contents to achieve specified density (courtesy USACE)

Figure 10.23

Compacting impervious fill using a pad foot roller to determine acceptable lift thickness and number of passes to achieve required density (courtesy USACE)

Figure 10.24

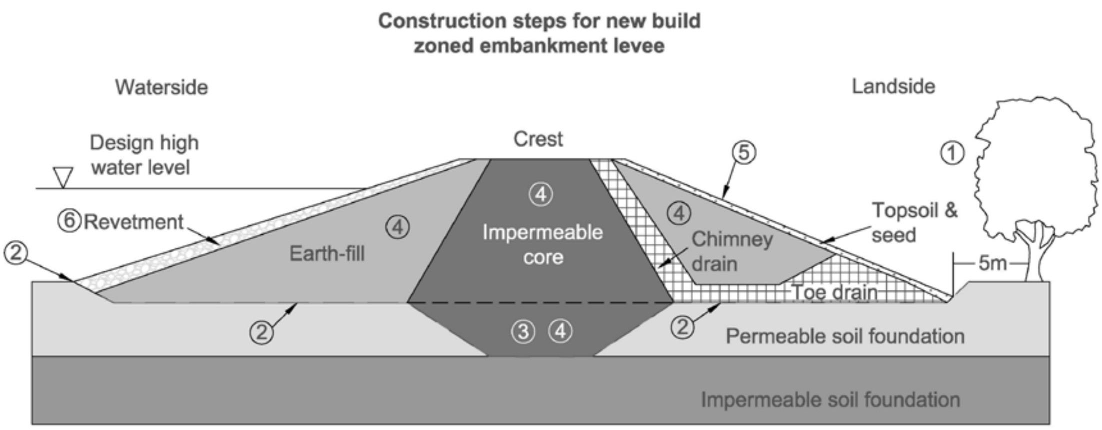
Verification of required number of passes of a self-propelled compactor to achieve specified density. This type of machine would count as two passes, since there are compaction drums on both axles (courtesy USACE)

Figure 10.25

Laboratory technician using a nuclear density gauge to measure field density after compaction (courtesy USACE)

10.5.2 Stages of levee construction

Figure 10.26 shows the steps involved in typical earthen levee construction. These construction steps are discussed further for a new build levee (see Section 10.5.3.1). Many of the steps also apply for levee adaptation (see Section 10.5.3.3) and levee repair (see Section 10.5.3.2).



Key:

1 Remove all trees and shrubs within levee footprint plus five metres. Grind or excavate all stumps and roots.
2 Strip topsoil 12–50 cm.
3 Excavate to shape existing topography.
4 Construct embankment zones' shapes as required.
5 Place topsoil and seed on landside slope (may also include measures to discourage digging fauna).
6 Place revetment on waterside slope (may require filter layer separation from embankment soils).

***Figure 10.26** Steps in construction of an example new build levee*

Typical steps involved in the construction of a levee as depicted in Figure 10.26, are further detailed as follows:

Step 1: site clearance involves the cutting of woody vegetation and large brush from within the levee footprint. Chain saws, skidders, and specialised attachments to the bulldozers and hydraulic excavators listed in Table 10.22 are commonly used for these activities. After the large trees are felled with chain saws, skidders are used to pull the larger trunk segments to areas where they can be loaded out for further processing. Treetops, smaller branches, and shrubs are usually piled for chipping, composting or burning.

Grubbing involves the unearthing and removal of large roots with diameters greater than 5 cm and is normally done using a root rake attached to a bulldozer or the teeth of an excavator bucket, combing and raking the existing ground to a depth of approximately 15 cm. Stumps are also pulled out using this process, or 'ground' into small pieces using a stump grinder.

Step 2: topsoil removal, or 'stripping', is the removal of organic soil layers from the previously cleared and grubbed levee footprint. This is normally done using bulldozers or excavators, as depicted in Table 10.23. The depth of material 'stripped' varies, but is typically 12–50 cm.

The objective is to remove any organic materials or 'topsoil' from within the footprint of the proposed embankment, as these materials are subject to undesirable consolidation and degradation, forming weak or unstable zones under the levee.

Step 3: excavation of earth and rock materials is then performed to remove wet, weak, or otherwise unsuitable materials from the levee foundation; to shape the existing topography to accept a stable levee cross-section, to provide for the installation of utility and drainage lines under or within the levee, or to generate 'borrow' materials for use in construction of the embankment. Excavation is performed using a variety of equipment types, as depicted in Table 10.23. The most common is a bulldozer, which can remove most overburden material, and 'rip' many forms of weathered rock. If hard rock excavation is required, hydraulic breakers or drilling and blasting techniques are employed to first break up the rock. Hydraulic excavators, tracked loaders, craned with draglines or clamshell buckers, and scrapers are also used for excavation, depending on the nature, location, and volume of material involved.

Step 4: embankment filling involves the controlled deposition, spreading, and compaction of earth and rock-fill materials to the extent required by the plans and specifications for the levee. The materials involved range from 'impervious' clay and silt materials to graded sand and gravel filters, to larger graded rip-rap used to provide scour protection. Scarce or expensive materials, such as impervious and sand filters, are often constructed as vertical zones at or near the levee centreline, and flanked by less expensive or more readily available materials (sometimes called 'common' or 'random' fill) to provide the balance of the trapezoidal earthen levee cross-section. Embankment materials are hauled to and deposited by equipment depicted in Table 10.24, spread by equipment depicted in Table 10.23, and compacted by equipment depicted in Table 10.25. It is good practice to overfill each layer, so that it is wider than the finished profile, to ensure full compaction at the edges of the cross-section. This step may also require trimming of excess materials, which fall outside of the desired limits of the levee cross-section.

Some of these activities, such as the removal of common fill, which may be inadvertently deposited in the impervious or sand filter zones, should take place as the embankment is raised. Others, such as the final shaping of the landside and waterside slopes of the levee, are done after the embankment is completed, in order to facilitate uninterrupted placement of embankment, equipment access, and ease of controlling the final lines and levels.

The case study in Box 10.16 describes a situation where existing levee materials had to be temporarily removed, and the levee cross-sections intentionally 'overbuilt' in order to provide for the construction of a stable cross-section, and access for equipment to deliver, spread, and compact the embankment materials. As the levee was completed, these overbuilt zones were removed, and final lines and grades were shaped.

Step 5: topsoiling and vegetation involves the deposition and spreading of a layer of organic soils (some of which may have been generated from the stripping operations previously described) on the exposed levee slope for the purpose of sowing, generating, and supporting a stand of grass. Topsoil is normally spread with bulldozers or hydraulic excavators. Sowing of seed, fertiliser, and soil amendments is normally done by hydroseeding, a process that distributes these items using water. Some type of 'mulch' (straw, hay, shredded paper) or a synthetic or biodegradable geocomposite is applied to provide protection from erosive rainfall while the seed germinates and propagates.

Reusing the site won topsoil, rather than importing new material, has the benefit of ensuring that any seed that may be within the topsoil is of local/native species. In areas where digging fauna are prevalent, measures may also be incorporated (see Box 10.16) to discourage these activities.

Step 6: revetment is normally composed of stone rip-rap, placed on areas where the flowing water or wave run-up might erode earthen embankments. Revetment can also be composed of concrete masonry or geosynthetics. Armourstone is normally placed by hydraulic excavators or crane/clamshells, after slopes are trimmed and shaped. It is often necessary to first place an intermediate layer of 'bedding' (ie gravel or smaller sized armourstone, often underlain by a geotextile) under the final layer to act as a filter to prevent the washing of finer materials from the underlying earthen embankment.

10.5.3 Types of levee construction

Good construction practices related to the four types of levee activities (new build, repair, adaptation, and decommissioning) have many common characteristics. They are derived and adapted from other earthworks activities such as foundation excavation and embankment construction, but they require more specialised and focused attention to details, as outlined in the following sections.

10.5.3.1 New build – creating entirely new levees

The primary objective in new build construction is to execute the work in a cost and time efficient manner to provide flood risk reduction in a logical and expeditious fashion, while maintaining environmental safeguards.

There are three methods for placing earthen materials for new build levee construction. Such levees can be homogeneous, zoned or enlarged, as outlined in Section 3.3.1.

1. Compacted (most desirable – zoned levee).
2. Semi-compacted (fill placed at natural water content, thicker lifts, less compaction).
3. Uncompacted (fill cast or hydraulic fill).

However, construction good practice for new build levees generally involves the careful preparation and examination of foundation conditions, followed by controlled placement and compaction of levee layers and zones, including logical sequencing of the construction as follows:

Foundation preparation considerations include:

- if the foundation materials are soft there are several alternatives for levee construction:
  - slow, phased placement of levee layers, allowing natural consolidation of the foundation by the placement of the fill
  - fast placement of levee layers, while monitoring and controlling the consolidation of underlying soft soils
  - removal or stabilisation of soft foundation soils, followed by placement of levee layers
  - installing, if appropriate, layers of sand or geotextile materials or drains to enhance the dissipation of pore pressures in the soft soils, followed by placement of levee layers.
- all foundations should be prepared by clearing trees and shrubs, grubbing up roots and stumps, and stripping away topsoil and other organic soils
- surfaces should be sloped to drain away from levee placements
- formal dewatering (using sumps or wells) may be required
- every effort should be made to avoid disturbing or rutting existing soils – compaction or other improvement of existing soils should be attempted only under the direction of the designer
- foundation inspected and approved by the construction manager or engineer/designer prior to fill placement when possible, work upstream to downstream and parallel to levee axis.

Work sequencing and borrow material considerations include:

- if both sides of the river or stream are to be protected, progress both sides of the watercourse simultaneously
- material from borrow areas should be conditioned at source to near optimum moisture content
- roots, oversized rock and other undesirable portions should be removed from borrow material at the source
- vary the routes of hauling equipment over foundations and levee layers to avoid rutting and over-compaction
- compaction using roller types suitable for the materials being placed while avoiding disturbance of adjacent or underlying soils.

Fill placement considerations include:

- make a distinction between fills of cohesive material (clay, loam, glacial till) and granular material (sands and gravels). Both materials need a different approach. For cohesive materials the most important is to control the moisture content carefully. Important aspects for granular materials are the grain size distribution, the grain shape and the shattering resistance
- for homogeneous levees, place finer grain material toward centre and coarser grain material toward slopes
- for zoned levees (cross-section includes more than one type or grading of material, see Figure 10.26), advance drainage layers and impervious core materials above and ahead of random or rock-fill shoulder materials to prevent cross-contamination
- monitoring instruments (settlement plates, water pressuremeters) should be installed before levee filling begins in order to obtain baseline measurements
- compaction should be done after each layer. Layer thicknesses will be determined by the nature of the material and the actual compaction equipment being used.

Other considerations include:

- ensure careful construction of the toe of the levee slope where it meets the ground. Proper compaction and keying of the levee material into the ground in this area is important to mitigate against seepage/internal erosion and mass instability
- depending on the design and foundation conditions, the crest of the levee may include extra height for settlement compensation
- settlement may occur during construction because of compressible foundations, dewatering, vibrations during the installation of any sheet piles, or other conditions. In addition to design considerations, this may require the constructor to provide more levee materials than the design cross-sections might indicate. In those cases it is important to monitor settlement plates, the levee, and crest heights at regular intervals.

Box 10.16 shows a new build levee for the city of Arles, France. This levee is built on top of a compressible foundation soils and in an area where animal burrowing is of great concern.

Note

The extraordinary measures taken to inspect and replace the foundation soils in the ‘anchoring system’ ditch (trench), to overbuild and trim the levee slopes for subsequent placement of the ‘protective grid’, and to monitor compaction and settlement of the levee soils.

Box 10.16 Example of new build levee construction, Arles France

The Syndicat Mixte Interrégional d'Aménagement des Digues du Delta du Rhône et de la Mer (SYMADREM) manages more than 200 km of levees that protect the Camargue region and especially the city of Arles. Following the important flooding event of the river Rhône in December 2003 (see Figure 10.27), part of the water had been trapped in-between infrastructure (railway, dike canal) and moved towards the city, creating a major flood of industrial and residential areas.

***Figure 10.27** Northern neighbourhood of Arles following December 2003 flood (courtesy EGIS)*

A new protection levee was built in the northern part of the city to protect the urban areas from floods. This levee, made with impervious backfill (see Figure 10.28), was built taking into account various constraints related to the site:

- compressible ground
- interface with the surrounding embankment structures with different nature and permeability
- several road and rail crossings of the dike.

The project design provided for an over-built levee crest and slope elevation to account for consolidation settlement of the foundation soils.

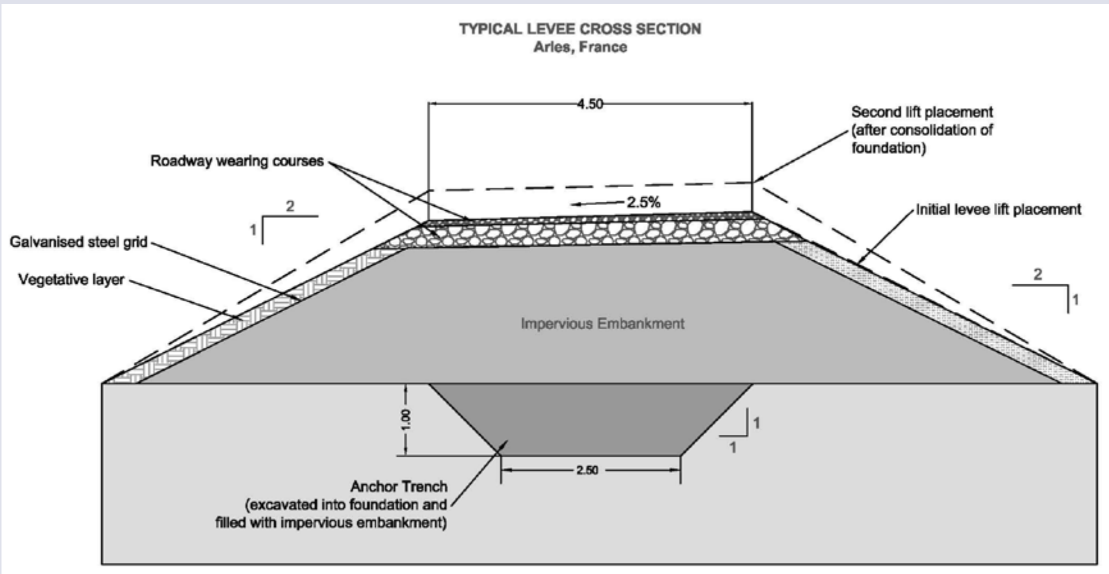


***Figure 10.28** Typical levee cross-section near Arles, France (courtesy EGIS)*

The construction of the new levee included several phases detailed as follows:

Phase 1 and 2: preparation of the area (site clearance and topsoil removal). All the vegetation of the project worksite was removed, the existing networks and the fences of agricultural parcels moved. The vegetated ground was scrapped off and provisionally stored for future reuse (see Figure 10.29a).

Phase 3: excavation of drainage ditches and anchor trench (see Figure 10.29b and c). The various drainage ditches were excavated first to secure good rainwater management of the worksite. An anchor trench was excavated and backfilled with impervious soils to seal the foundation. Localised drainage was also created for the low quality areas (eg ancient ditches) and subsidence instruments were installed on the bottom of the anchor trench.

Box 10.16 Example of new build levee construction, Arles France (contd)

***Figure 10.29** Preparation of site between agricultural parcels (a), excavation of anchor trench (b), develop localised drainage (c) (courtesy EGIS)*

Phase 4: construction of the impermeable embankment. After a testing campaign of compaction to determine the parameters of the material to be used, the anchor trench and the main part of the levee were backfilled with impervious compacted material (see Figure 10.30). Compaction and water content were measured regularly during the backfill operation. The earth filling was placed wider than necessary and then scraped with a mechanical excavator to ensure a uniformly compacted embankment and provide a suitable substrate for the grid and vegetative layers.

***Figure 10.30** Backfill of the anchor trench and body of the levee with impervious material (courtesy EGIS)*

***Figure 10.31** Installation of protection grid on the levee (courtesy EGIS)*

Phase 5: finalisation of the embankment and installation of a protection grid (see Figure 10.31), topping of the embankment with vegetable earth and seeding. The slopes of the levee were formed using an excavator with a regular slope of 1V/2H. Afterwards, they were protected from burrowing animals by the installation of a protection grid of galvanised steel. The grid was held in position by small backfilled trenches in the upper and lower part of the embankment, and metallic staples, especially in the places where two lengths of grid covered each other.

All the embankments were covered with a layer of topsoil (see Figure 10.32) put in place with an excavator or a bulldozer. To limit erosion due to water and rainfall runoff, shallow furrows were created perpendicular to the slope. Finally, hydroseeding was performed with a mix of plants adapted to the climate to stabilise the embankment.

Box 10.16 Example of new build levee construction, Arles France (contd)

Figure 10.32 Topping of the embankment with topsoil (courtesy EGIS)

Phase 6: creation of the maintenance and access tracks (see Figure 10.33), finishing of the embankment and installation of security equipment. The embankments of the levee were protected by the installation of a synthetic and biodegradable geocomposite, to help the grass grow and limit surface erosion. The supervision and maintenance tracks were constructed with granular material on the crest and at the bottom of the levee on both sides and then compacted. Finally, security equipment was installed to limit the access (fences, barriers). An aerial view of the completed project is shown in Figure 10.34.

Figure 10.33 Access road and security gate (courtesy EGIS)

Figure 10.34 Aerial view of completed project (courtesy EGIS)

10.5.3.2 Repair – restoring protective levels and dimensions of existing levees after damage or deterioration

The primary objectives of levee repair are to restore previously existing levels of protection and prevent further damage to the levee.

Good practice for repair of levees (if necessary in addition to those outlined for new build) are concerned mostly with restoring missing or damaged levee features using materials and methods, which do not present any additional risk to the levee or result in any added risk of flooding. Some important procedures are:

- keep a stockpile of emergency repair materials (sandbags, rip-rap etc) on hand
- if possible, avoid further and future damage risk by reducing loading (for example decreasing the hydraulic head by providing a seepage embankment or relief wells)
- remove adversely remoulded soil to eliminate slip surfaces
- after repairs are completed, monitor performance by topographical survey, flow measurements and visual inspection.

Box 10.17 describes an innovative approach for using selected methodologies and coarse granular fill, placed directly into the watercourse, to eliminate the need for expensive dewatering when restoring and reshaping a damaged levee.

Box 10.17 ***Using temporary decommissioning and innovative techniques to repair levee scour damage without dewatering in Isere, France***

The Alpine Department (AD) manages 250 km of levees in Isere. These levees are old, relatively narrow, and wooded. The toes of slopes are permanently under water, sometimes with great depths, due to scour holes (3 m to 7m deep). Remedial works on the river side are necessary to stabilise the slopes, which are too steep and often scoured. These works include filling the scours and caves, building a new slope or a berm, and protecting against erosion. The varying geometrical characteristics of the levee and the river require development of a range of construction methods to perform the repairs. Considerations include:

- steep slopes
- heights of the levees
- narrow crest
- toe of slope in the water
- cofferdam impossible to build during the works.

Work: building a berm of armour stone.

Object: filling the scour hole and stabilising the river-bank.

Means: embankment of waste (armour stone calibrated), 0/400 mm, insensitive to water, proper angle of friction, compression efficient, cheap, easy to recharge in case of erosion, suitable for straight dike reinforcement.

Implementation:

Phase 1: Supplying

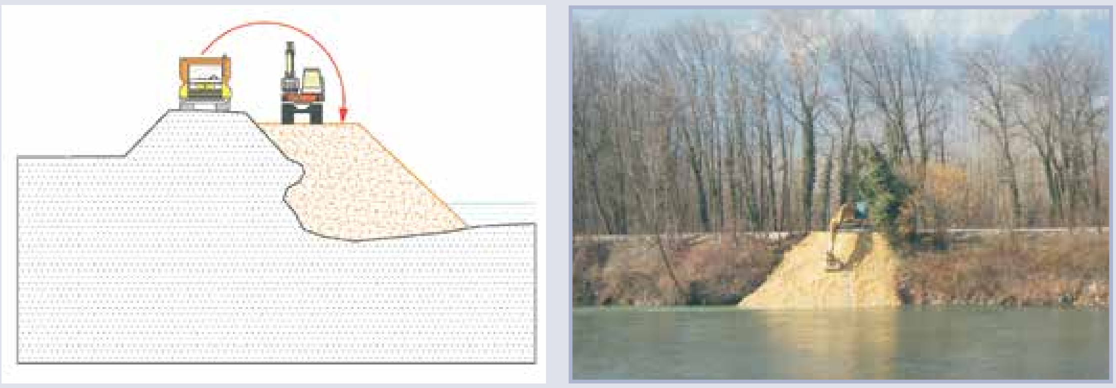


Figure 10.35 Ramp connecting crest to river

Box 10.17 ***Using temporary decommissioning and innovative techniques to repair levee scour damage without dewatering in Isere, France (contd)***

The armourstone is taken by a loader and placed over a width of 7 m, which allows the trucks to back down the ramp (note that beyond 300 m, another ramp is necessary) (see Figures 10.35 and 10.36).

Figure 10.36 ***Truck dumping armourstone (a), and loader placing armourstone (b)***

Phase 2: Shaping by a hydraulic excavator (see Figures 10.37 to 10.39)

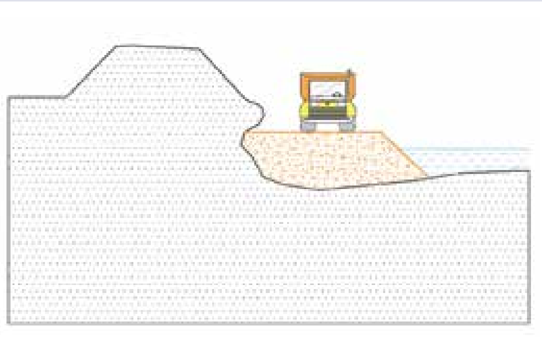


Figure 10.37 ***The excavator (working on the berm) uses the material to define the profile***

The excavator forms either a 4 m wide berm to provide access in the future for maintenance, later circulation or a 2 m wide berm if the slope will finally reach the crest (2 m is a minimum for the passage of a compactor).

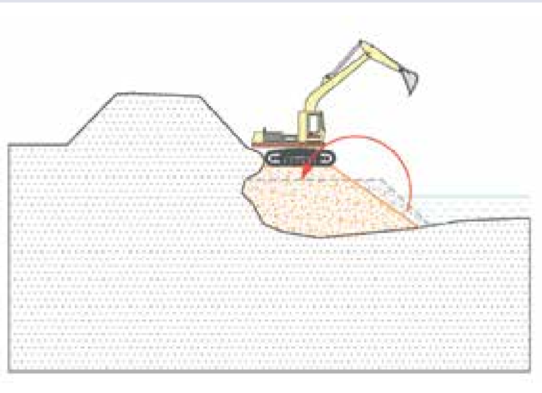


Figure 10.38 ***Completed berm created by excavator***

Box 10.17 ***Using temporary decommissioning and innovative techniques to repair levee scour damage without dewatering in Isere, France (contd)***

Figure 10.39 ***Excavator working on berm***

Phase 3: Completing work

In this case, it was achieved by filling with an excavator working from the crest (see Figure 10.40). Finishing can be done by dumping topsoil and seeding.

Figure 10.40 ***Excavator working from crest***

Photos courtesy ADIDR

10.5.3.3 Adaptation – raising, widening or strengthening existing levees

The primary objective of adaptation is to raise or strengthen the existing levee sections and elements without reducing the levels of protection or increasing the flood risk in an unacceptable way during construction.

Good practice for adaptation (if necessary, and in addition to those outlined for new build) is concerned mostly with ensuring proper bond with, and protection of, the existing levee, and levels of protection. Some important procedures are:

- surfaces of existing levee should be cleared of vegetation and roots and topsoil layers stripped (the constructor should avoid to strip the topsoil layer over the entire length of the levee at one time because it will make the levee vulnerable to the rain or to a flood)
- existing topsoil, rip-rap or other materials may be required for future use. They should be removed and stockpiled
- when increasing height on an existing slope, create a series of 'benches' (horizontal notches) in the levee to prevent the formation of preferential slip planes
- phasing for adaptation work should seek to uniformly raise the level of protection along the levee, working upstream or downstream
- temporary protective and emergency measures for wave and erosion protection may be required during levee construction
- avoid planning adaptation work during the high water season or during cold or rainy seasons
- maintain existing drainage features (ditches, drainage pipes etc) until new features are in place and fully operational. Clean and completely fill or plug all abandoned features including wells, piles, ditches etc.

Levees and dunes located in coastal environments are routinely damaged by the effects of wave forces and storm surge (see Figure 10.41). Good engineering, proper planning and quality construction methods should be used when rebuilding and restoring these levees and dunes for future storms.

The attempted adaptation of a coastal levee in Grand Isle, Louisiana in the USA is described in Box 10.18. In this case, the original sand (dune) berms were adapted, using a combination of sand and clay levee, accompanied by fencing and other measures, to protect against the erosive wave and wind forces of coastal storms. Unfortunately, severe coastal storms damaged the partially completed adaptation work during construction.

Box 10.18 Levee adaptation and repair of levees in coastal Louisiana, USA

Figure 10.41 Hurricane damages in Grand Isle, LA, USA (courtesy USACE)

In 1985, following multiple storm events, the USACE began multiple iterations of detailed designs to combat the erosion that followed large storm events. It wasn't until 1992, following Hurricane Andrew, that a carefully modelled system of breakwaters was determined to be necessary to effectively reduce the erosion rates (see Figure 10.42).

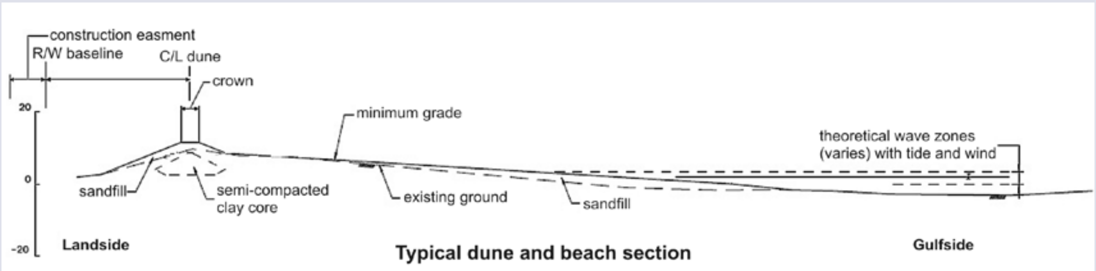


Figure 10.42 Typical dune and beach section for project along Grand Isle, LA, USA (courtesy USACE)

After Hurricane Katrina devastated the Grand Isle levee system in 2005, a project to fully repair and re-nourish the levee and beaches was undertaken (see Figure 10.43). The levees were designed with large berms and clay cores for increased stability (see Figure 10.42), but before the completion of these levee and dune repairs the region was again ravaged by Hurricanes Gustav and Ike in 2008. Unfortunately, only 2500 m of the 11 150 m of levee repairs were complete at the time of the storms and vegetation had not had a chance to establish, which resulted in the severe erosion to bare areas and flooding in the areas behind the unfinished levees.

Dredging operations took place offshore just outside of the project location and required sand material piped to the project site from the offshore dredge (see Figure 10.44 and 10.45).

Figure 10.43

Repairs to breakwaters along Grand Isle, LA, USA (courtesy USACE)

Box 10.18 ***Levee adaptation and repair of levees in coastal Louisiana, USA (contd)***

Figure 10.44

Dredging operations at offshore borrow site (courtesy USACE)

Figure 10.45

Sand placement operations at discharge pipe (courtesy USACE)

Planting, fertilising, and installation of sand fencing are necessary to protect the sand dune from wind-driven erosion (see Figure 10.46 and 10.47). Pedestrian walkways and emergency vehicle crossovers protect the sand dune by providing designated access points to the beach for pedestrians and vehicles without causing negative impacts, such as damage and erosion, to the protective sand dune.

Figure 10.46

Fully raised and vegetated levee section (courtesy USACE)

Figure 10.47

Vegetation planting operations (courtesy USACE)

Upon completion of the semi compacted clay cores several containment dikes were constructed around the segments of beach that were receiving the sand material. These dikes contained the dredge discharge into workable areas as well as prevented flooding of the surrounding community. Dozers were stationed inside the containment dikes and processed the incoming dredge material into the new dunes. Additional material processing occurred once the dunes reached the design elevations. This processing ensured proper condition for planting operations.

Unfortunately, a completely updated levee and dune system was not in place to protect the Grand Isle community from Hurricanes Gustav and Ike. However, it was obvious from subsequent inspections that even though new designs for the clay core were needed, areas with established vegetation withstood the brunt of the storm surge and survived much better than those without vegetation. These lessons learned will prove invaluable to future construction projects and to the community of Grand Isle.

Box 10.19 exemplifies the adaptation activities of an existing levee in Großenhain, Germany, damaged by a tornado and flooding. The repairs involved the restoration of the levee cross-section, but also included the removal of undesirable vegetation from the levee, followed by strengthening of the levee by soil mixing and the insertion of sheet pile cut-off walls.

Box 10.19 Levee repair in Großenhain, Germany

In 2010 a tornado (May) and a flood (September/October) caused severe damages at the levees of the watercourse Röderneugraben near the town of Großenhain, Saxony, Germany. Levee guidance does not allow trees on the levee or its slopes in this reach, however much woody vegetation was to be found. Many of the trees were uprooted during the storm due to the wind and scouring around the trunks on the waterside. The fallen trees formed obstructions for the following flood discharge (see Figure 10.48 and 10.49).

To remedy this danger the trees were cut and rooted out by means of a rotary hoe (see Figure 10.50 and 10.51). Where it was not possible to remove all trees the levee stability was ensured with sheet pilings or soil stabilisation methods along the levee centreline.

The following considerations were important for the measure:

- prompt recovery of the function of the existing levee
- uphold the same protection level
- no changes in levee cross-section dimensions
- no changes in groundwater flow and in flood discharge.

Figure 10.48

Uprooted trees fallen into the river and the foreland obstructing the water flow

Figure 10.49

Reduced levee cross-section (and reduced stability) due to uprooted trees

Figure 10.50

Cut trees to ensure the free flow in the river again. Stumps in the background on the right bank levee

Figure 10.51

Milling and cutting the stumps and refilling the stump holes in and besides the levee

For better stability and tightness performance some sections of the levee were installed with diaphragm walls of reinforced soilcrete and sheet piling respectively (see Figure 10.52 till 10.56).

Box 10.19 *Levee repair in Großenhain, Germany (contd)*

Figure 10.52

Milling-mix technique (FIM) on a levee. The sword of the soil cutting and mixing machine breaks up the soil and feeds in the binder suspension

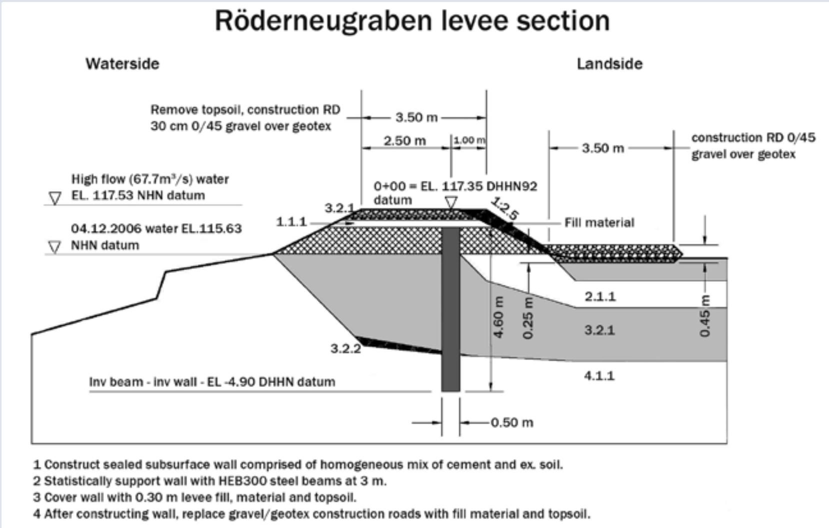


Figure 10.53

Cross-section of the 1800 m long reinforced levee section with stabilised soil diaphragm wall

Figure 10.54

Mixed in place technique (MIP) was used along a 6950 m long reach. On the left: triple auger to break up the soil and to mix in the binder suspension. On the right: placement of steel girders into the still fresh soil-cement mixture

Figure 10.55

Soil cement cut-off wall after placing the reinforcement steel girders in certain spacing derived from stability calculations

Box 10.19 ***Levee repair in Großenhain, Germany (contd)***

Figure 10.56 ***A 6250 m long sheet piling section of the levee. The sheet piling ends as well as the soil cement diaphragm walls below the final levee crest so that it will not be visible after completion of the refurbishment***

Photos courtesy E Bielitz, Saxon Dam Authority, Pirna, Germany

Box 10.20 exemplifies a levee adaptation, with addition of armourstone (see Case 2.1) and with creation of a berm and addition of armourstone (see Case 2.2).

Box 10.20 ***Building protection against erosion***

Case 2.1 Adaptation with addition of armourstone

The bank is correct, but the crest is too high to work directly from the top. The levee is excavated to shape a middle platform to allow the passage of a hydraulic excavator. The spoil is stored on the crest.

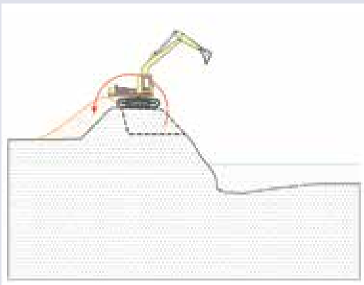


Figure 10.57 ***Excavator shapes levee to allow access***

The rocks are supplied from trucks backing down the platform.

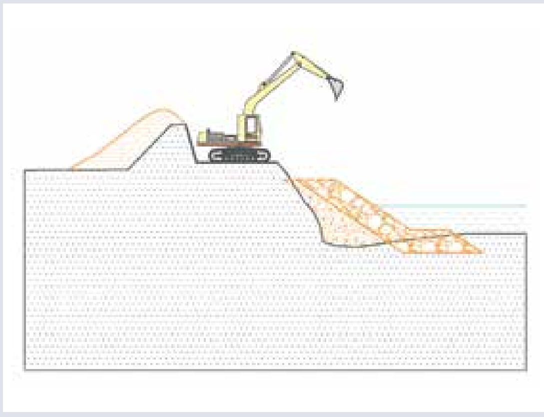


Figure 10.58 ***Truck supplying rocks for excavator to use***

Box 10.20 Building protection against erosion (contd)

Rocks are placed progressively by the excavator. Enlarging the platform with the thickness of the shell permits the secure passage of the trucks. Then, the crest is reconstituted.

Figure 10.59 Excavator placing rocks on levee slope

Case 2.2 Adaptation with creation of a berm and addition of armourstone

The bank must be stabilised by a berm (embankment slope too steep from erosion). In this case, a ramp is created (preferably upstream of the section) to be filled with rip-rap, according to Case 2.1.

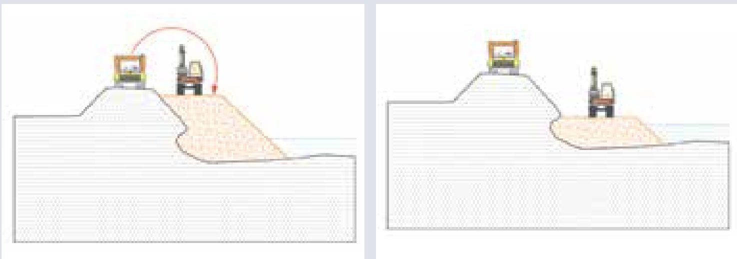


Figure 10.60 Truck supplies rock to excavator to create berm

Figure 10.61 Truck uses berm to supply excavator with rip-rap for placement

Once the 7 m berm is created, trucks can provide rocks by backing up to the edge of the berm.

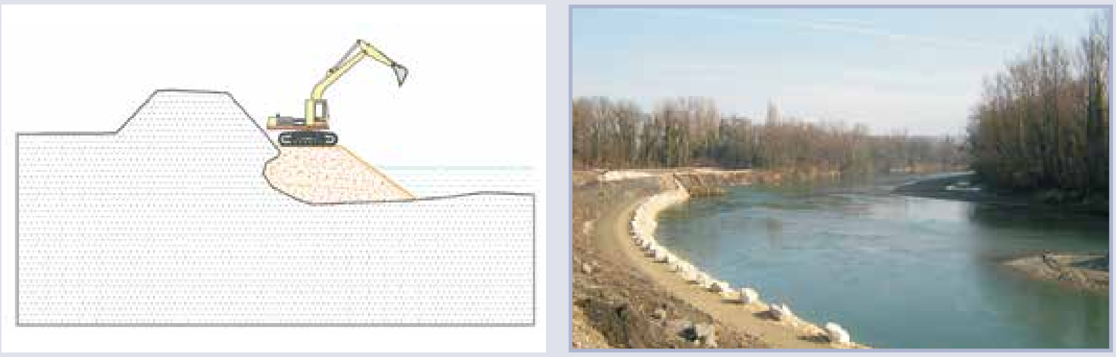


Figure 10.62 Excavator shapes the berm to its definitive size

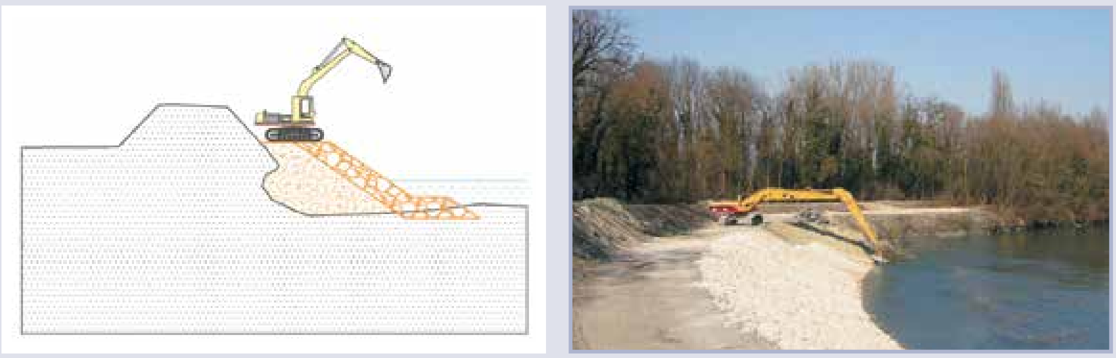


Figure 10.63 Excavator sets up the rocks brought by trucks

Box 10.20 Building protection against erosion (contd)

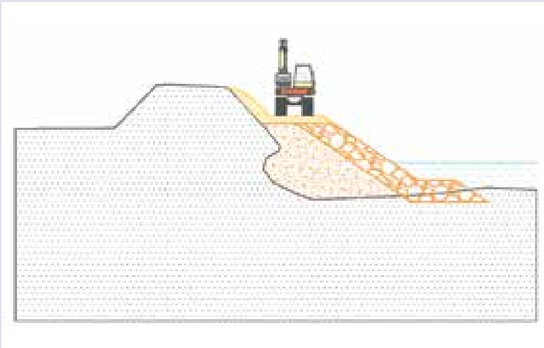


Figure 10.64 Completed levee slope

Equipment and rates

These works require particular equipment:

- hydraulic excavator: it is an excavator with a long arm (12.5 m operating length), so as to be able to bring the rocks under water without dropping them. The checking of the work under water is done with a graduation on the arm of the excavator
- a loader (wheel loader)
- a vibratory soil compactor
- trucks (6 × 8) at 5–6 rotations per hour.

Photos courtesy ADIDR

10.5.3.4 Decommissioning – removing or reducing the height of levees and other flood defence features

The primary objective of decommissioning is to remove the designated levee, features and/or segments to the designed level(s) in a uniform manner, while preserving adjacent features and remaining levee levels of protection.

Good practice for decommissioning includes:

- remove levee materials in uniform layers, working downstream to upstream
- provide temporary protection for levees exposed to erosion during decommissioning work
- restore, re-grade, and re-vegetate disturbed areas to prevent ponding and erosion. Be aware of environmental regulations during levee removal, especially in case of stone revetments, debris of asphalt levee roads and silt pollution of the watercourse.

The Isere example (see Box 10.17) includes (temporarily) reducing the level of protection in a methodical fashion. In this case, the crest level was temporarily removed, and ultimately restored, in order to provide for the safe passage of equipment during levee repair. The same methods used at Isere could be used for permanent decommissioning of a levee.

10.5.4 Instrumenting levee construction

Various amounts and types of instrumentation may be specified for a particular levee project. It is important to know that when these tools are specified for use during construction, they should be properly installed to provide the information that is required concerning the effects of construction loading, and post-construction behaviour of the levee. Measurements gathered from the instrumentation can potentially lead to essential corrective action and prevent failure or damage. The designer may also specify hold points in the data collection, before allowing or disallowing construction to continue.

Monitoring of instrumentation during construction (see Section 7.9) may include:

- seepage or groundwater monitoring wells for water level or pressure measurements
- settlement plates to monitor construction movement
- vibration and sound monitoring adjacent to homes or structures
- inclinometers or tiltmeters on structures.

Great care should be exercised to protect instrumentation during construction. If instrumentation becomes damaged the resulting loss of critical data may affect the construction progress or service of the levee. Vibration monitoring is one such method of measurement that can assist in combating unfounded claims of damage to nearby structures, as has been experienced on the Sacramento Levees in the USA (see Box 10.21).

Box 10.21 Construction monitoring

Levee construction activities adjacent to homes or other structures can require monitoring for effects of vibration due to the construction process. Use of heavy equipment in proximity of such structures will cause concerns about potential damage. Generally, vibration monitoring can assist in determining potential liability or alleviate liability for frivolous claims.

Figure 10.65
Vibration monitoring, Sacramento, USA

Box 10.22 presents the monitoring of levee fill placement on soft soil in Lake Marken for a levee improvement project in the Netherlands.

Box 10.22 Monitoring of levee fill placement on soft soil in Lake Marken levee improvement project, the Netherlands

If a levee is founded on thick layers of soft soil, the placement of fill will lead to excess pore water pressures in the soft soil which, if too high, may result in slope instability. Fill placement will also lead to settlement. Accurate settlement predictions are important to calculate the amount of fill required. For the placement of stone revetments and the building of roads it is important to know the anticipated residual settlement. Both excess pore water pressures and settlement can be calculated in advance, but they may contain various inaccuracies as a result of uncertainties about soil properties and soil layers. In this case the rate of raising the levee fill must be adjusted based on pore water pressure and settlement monitoring (see Figure 10.66a and b).

Careful consideration of the selection of monitoring points is essential. In the Lake Marken levee improvement project in the Netherlands two or three automatic water pressuremeters were used on each 250 m length of the levee embankment. These meters were placed in the soft soil layers, which are critical for levee slope stability. A row of about two to four settlement plates were placed

Figure 10.66
Settlement plates in the Lake Marken levee improvement project (courtesy Fugro)

Box 10.22 Monitoring of levee fill placement on soft soil in Lake Marken levee improvement project, the Netherlands (contd)


b

every 50 m. The meters are placed outside or on the edges of the crown of the levee to minimise disruption to the construction activities. Each meter is protected with a fence and marked with bright colours to avoid damage.

The fill placement to a height of about 5 m takes between one and three years in the Lake Marken levees. Geotechnical engineers frequently carry out settlement prediction and stability calculations based on the monitoring results during this period. To ensure a consistent set of monitoring data the process of data collection and processing should be well organised. In this way the construction period and amount of fill can be optimised without jeopardising levee safety.

Sources: TAW (1996), Van der Meer and Halter (2005), Hoffmans (2007)

10.5.5 Integrating non-earthwork features into levees

Section 3.4 provides detailed discussion of structures that might be included or encountered during the construction of earthen levees. This section provides examples of these features and good construction practices.

The primary objectives when integrating non-earthwork features into levees include:

- maintaining continuity of existing defences
- maintaining operation of the non-earthwork facility
- ensuring good compaction to achieve a good seal around non-earthwork features.

Concrete structures

Types of concrete structures used in conjunction with levees include:

- pump stations
- seepage collection vaults
- crossings/gates
- water control structures (sector gates, barge gates etc).

Good practice for concrete structures includes:

- foundations adequate to support the additional load (see Figure 10.67), either as strong as or stronger than the structure
- battered walls 1h:10v to ensure levee material can be compacted or pre-compact the levee material and then excavate locally and backfill entire trench foundation with concrete
- use of hand tamped/trench compactors within 1 m of structures
- no heavy equipment on or near the structure until concrete has reached 75 per cent design compressive strength.

Flood walls and seawalls

Flood walls (see Figures 10.68 and 10.69) and seawalls are non-earthwork defences that are provided where there is insufficient room to construct the levee footprint. These are generally constructed of concrete and/or sheet piling, but can take a variety of forms and material types. Good practice for flood walls and seawalls includes co-ordinating construction sequence with the designer to ensure that partially completed walls can withstand levee and (emergency) flood loading.

Figure 10.67 H-pile foundation for a closure structure foundation (courtesy USACE)

Figure 10.68 Concrete flood wall (courtesy USACE)

Figure 10.69 Flood wall riverside (courtesy USACE)

Composite structures

Composite structures are combinations of the previously mentioned non-earthwork features (see Figure 10.70). Good practice for composite structures includes:

- following the manufacturer's instructions and the designer's specifications for installing geotextile materials
- ensuring that construction phasing does not result in overload or unbalanced loading of adjacent features.

Figure 10.70 Concrete flood wall (background) and mechanically stabilised earth wall (foreground) (courtesy USACE)

Other non-earthwork features

This category includes a wide variety of features, which serve to provide or enhance the flood defence, or that occupy or encroach on the levee footprint or structure. Features include culverts, ditches, pipelines and swales, which transmit surface or subsurface drainage water, sewage, gas and electricity through the protection zone of the levee (see Figure 10.71). A comprehensive list can be found in Section 3.4. Design of these features is discussed in Section 9.13. A common construction-specific issue is the repair of pipe crossings and this is discussed in Section 10.5.6.

Figure 10.71 Sanitary sewer line penetration before levee construction (courtesy Lawrence Piazza)

The primary construction objective is to ensure that the levee seals around and against these features in order to avoid the creation of seepage paths and failure potential.

Good practice for other non-earthwork features includes:

- ensuring maintenance of levels of protection (see Section 10.2.2)
- checking the structural stability (with the designer) for levee and the non-earthwork features
- designing all cofferdams and other temporary support systems to withstand expected flood loadings and/or overtopping
- co-ordinating temporary losses of power and utility services and reinstatement of these supplies (especially electric and gas utilities)
- providing sufficient standby and bypass capability for pump stations and other essential utilities.

10.5.6 Construction approaches to repair pipes and culverts

This section discusses in detail the two most commonly used construction approaches to repair pipes through levees – open-cut pipe replacement and sliplining. Alternative methods that may be used where a special need exists, if approved by the responsible agency for the levee system and carried out by an experienced constructor are:

- cured-in-place pipe (CIPP)
- fold-and-form pipe
- spray-on lining
- horizontal directional drilling.

Methods of pipe rehabilitation involving pipe bursting and splitting, concrete relining, pipe ramming or pipe jacking are generally not recommended for use in levee systems.

10.5.6.1 Open-cut pipe replacement

This category includes a wide variety of features, which serve to provide or enhance the flood defence, or that occupy or encroach on the levee footprint or structure. A comprehensive list can be found in Sections 3.4.1.5 and 3.4.2.1.

Pipe or culvert replacement using an open-cut method is the most intrusive way to rehabilitate a failing pipe system. This approach involves excavating down through the levee to the pipe or culvert, removing and replacing or repairing the damaged elements before reinstating the levee. The benefits of open-cut replacement are as follows:

- the pipe can be fully inspected and repaired or replaced with confidence and to modern standards
- the work can usually be performed using conventional construction equipment and can be contracted to most competent earthwork constructors who have a high comfort level with the technique
- it offers flexibility through the opportunity to change size or alignment of the pipe if needed, and the opportunity to focus on damaged areas only or replace the whole pipe.

However, this technique carries risk that the levee will not be effective during the excavation, which means it should be performed outside of the flood season. Also, the excavation and replacement could create a weak point in the levee. Overall, if the pipe is shallow and traffic is not disrupted, pipe replacement may be less expensive than pipe repair.

Issues, methods and examples of detailed design are the same as for new pipes, see Section 9.14.4.

10.5.6.2 Sliplining

Sliplining is a trenchless method in which a liner pipe is installed into a larger deteriorated host pipe. Slip liner pipe materials, which have been installed and are known to have performed successfully, are described in Section 9.14.4.4 along with associated design considerations.

This approach may be used where the deteriorated host pipe generally retains its original internal geometry, with only limited roof sags, invert buckles or pipe bends. Advantages and disadvantages of the approach are set out in Table 10.27. When an embankment has signs of depression due to soil loss (presumably into the deteriorated pipe through holes identified by the inspection), then sliplining is not recommended and open-cut methods should be pursued.

Table 10.27 Advantages and disadvantages of sliplining

Advantages	Disadvantages
• sliplining is often significantly less expensive than pipe replacement using open-cut trenching. The cost of sliplining is mainly a function of pipe length and diameter and is not greatly dependent on the depth of pipe burial • the levee itself is not directly impacted by the works, the flood defence system remains intact throughout the works and much of the environmental nuisance created by the excavation and replacement is avoided • construction is simplified and the process of pipe replacement can be completed in a considerably shorter construction period • safety concerns associated with deep trenching, steep excavation slopes, work inside trench boxes and worker exposure to traffic will generally be reduced or even eliminated • less engineering design and documentation effort is required (for example, less surveying and fewer design calculations, drawings and specifications) • unknown issues associated with ground disturbance, opening levee sections and other risks associated with trenching and excavations are minimised.	• the slip liner should be of a smaller diameter than the original or host pipe; The host pipe should have sufficient clearance to pull or push the new pipe through the opening and also to allow sufficient clearance to facilitate proper grouting. Calculation may be required to verify that the clearance is acceptable • neither the existing pipe nor the replacement can be inspected as thoroughly as would be the case for excavation and replacement. Most importantly, the seal between the existing pipe and the slip liner cannot be inspected and verified.

The three most common methods of sliplining are:

- **continuous sliplining:** this is the insertion of a single slip liner pipe section into the host pipe. This may be accomplished by welding sections of high-density polyethylene extruded (HDPE) or glass-fibre reinforced (GFR) pipe together so that the entire slip liner pipe is pulled into the host pipe in one operation
- **segmental sliplining:** this is similar to continuous sliplining (see Figure 10.72), but in this case the individual pieces of pipe are lowered into place, joined to the preceding piece (threaded, snap-together, and welded joints are used), and pushed/pulled into the host pipe. Connections between pipe segments must neither increase the outside diameter of the slip liner pipe, nor reduce the inside diameter of the slip liner pipe
- **spiral wound sliplining:** this is most effective on short runs (less than 30 m) and is typically used when access is difficult such as from a man-hole (see Figure 10.73). Spiral wound installation performs well when the slip liner pipe has a significantly smaller diameter than the host pipe (more than 50 mm smaller) so as to avoid snagging. In this case, hydraulic capacity analysis should demonstrate that the use of a smaller pipe will not inhibit performance.

Figure 10.72 Typical installation of a segmental pipe liner on the New Albany Levee System in Indiana, USA (courtesy Christina Neutz)

Figure 10.73 Typical installation of a spiral wound pipe liner, Jeffersonville/Clarksville Levee System in Indiana, USA (courtesy Christina Neutz)

Sliplining installation

If water is expected to be flowing through the host pipe at any time during the rehabilitation process, appropriate measures should be taken to control or bypass the water so that sliplining installation can be performed in a dry state. The host pipe should be thoroughly cleaned before installing the slip liner pipe All debris, solids, roots, deposits, and any other matter that would preclude proper installation of the slip liner pipe and annulus grout should be removed. The cleaning water and any debris removed from the pipe or culvert should be handled so as to comply with environmental and sediment control regulations.

After cleaning the pipes, a mandrel should be pulled through the host pipes to check that the slip liner pipe will fit inside the host pipe. The mandrel length should be equal to the length of one of the joints in the liner pipe. The diameter of the mandrel should be 50 mm greater than the proposed slip liner pipe outside diameter, and the mandrel stiffness should be equal to or greater than that of the slip liner pipe. A segment of slip liner pipe with spacers added to increase its diameter may be used as a mandrel, but this test segment should not be used as a permanent slip liner pipe. Host pipes 900 mm in diameter and larger may be verified through man-entry (subject to health and safety regulations), a laser cloud mandrel, or mandrel proofing.

The necessary precautions to maintain line and grade of the host pipe and avoid flotation of the liner pipe should be taken during installation. This can be accomplished by providing cribbing on the top of the slip liner pipe (at 2, 10 and 12 o'clock positions) to prevent flotation during grouting and to maintain a minimum annular space around the slip liner pipe in all directions. Installation should be conducted so that the slip liner pipe invert is as low as possible in the host pipe while still maintaining the annular space.

Slip liner pipes may be sandbagged, sealed, and flooded with water during installation to reduce buoyancy. The pipe manufacturer's recommendations on loading should be followed during installation so that the pipes are not overstressed or damaged.

Corrugated metal pipe (CMP) with interior bituminous coating should not be slip lined during warm weather. The bituminous material becomes soft during hot weather and tends to stick to the slip liner pipe during installation, particularly on the slip liner pipe's leading edge.

Grout materials

As sliplining installations comprising solid-wall HDPE pipes or glass fibre reinforced plastic pipes should be designed to support all exterior loads, 'non-structural grout' similar to lightweight insulating grout can be used. However, a 'structural grout' should generally be used in combination with machine spiral-wound PVC liner. Typical recommendations for grout properties are given in Box 10.23.

Box 10.23 ***Typical USA recommendations for properties of grout used to seal sliplinings***

Grout should be mixed using ordinary portland cement and potable water. Pozzolans and other cementitious materials are permitted and fly ash (U.S. types C or F) may also be used. Admixtures are normally selected by the slip liner grout contractor to meet performance requirements, to improve pumpability, to control setting time and to reduce segregation. Admixtures should not be biodegradable.

Non-structural grouts should have a minimum 24-hour penetration resistance of 0.7 MPa (ASTM C403) and a minimum 28-day compressive strength of 2.1 MPa (ASTM C495). A foaming agent should be incorporated into the grout mix and the grout density should be 7 kN/m^3 ± 0.5 kN/m^3 (ASTM C138). Grout viscosity should be 20 seconds or less (ASTM C939) so that it can flow adequately through the annular space.

Structural grouts should have a minimum 28-day compressive strength of 3000 psi (21 MPa) (ASTM C942). Grout viscosity should be 35 seconds or less (ASTM C939) so that it can flow adequately through the annular space.

Grouting

The annular space at the ends of the pipe is typically sealed using Portland cement concrete to create bulkheads. The annular space between bulkheads is then low-pressure grouted through grout ports that extend through the bulkheads. Grout fills the annular space and then flows out of vent tubes that pass the distal bulkhead. Grout may also seal open joints, fill small voids around pipes and stabilise the surrounding soil mass. Figure 10.74 is a cross-section through a pipe showing the grout and air vent tubes, the bulkheads and grout zones. Further recommendations for the grouting process are given in Box 10.24.

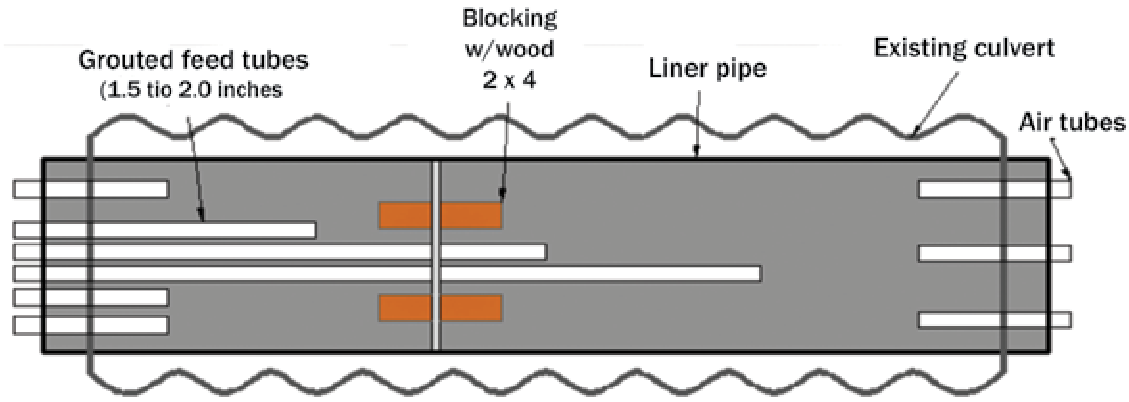


Figure 10.74 Typical plan view of a grouting setup when sealing the annular space between a host and liner pipe (courtesy Christina Neutz)

Box 10.24 Detailed recommendations for grout injection process

Grout injection

After the slip liner pipe is placed in the host pipe, bulkheads should be constructed at each end of the pipe in sequence from upstream to downstream. The bulkheads should be installed by placing a stiff (very low water-cement ratio) Portland cement concrete mix between the slip liner pipe and the host pipe. This mix should be placed by rodding and should be finished using a hand trowel to achieve a surface similar to the surrounding headwall concrete. Injection ports and exit vent ports (both usually PVC pipe) that extend through the bulkhead should be cast into place. The drilling of additional injection holes from the surface or through the liner pipe to facilitate grouting should normally be prohibited as this may damage the pipe or the surrounds.

The bulkheads should be hand-finished with ordinary Portland cement based grout. After a sufficient curing period, a soluble reactive silicate concrete treatment should be applied over the entire headwall surface, including the bulkheads. This will help to prevent issues with freeze thaw deteriorating the bulkhead material over time. Particular attention should be given to checking that the product is applied to the bulkhead grout properly.

Following construction of the bulkheads, the annular space between the slip liner pipe and the host pipe should be completely filled with the appropriate grout mix. The grout should be injected under low pressure, at one end of the pipe (preferably the outlet end), and should flow through the annular space towards the other end. The grout injection process should continue until all of the following conditions are achieved:

1 A volume of grout greater than the initial minimum estimated volume has been injected.
2 The exhaust grout recovered at each vent is no less than 85 per cent of the original density of the freshly injected grout.
3 The exhaust grout recovered at each vent is no less than 85 per cent of the original viscosity of the freshly injected grout.
4 The grouting specialist and any overseeing organisation recommend the ceasing of grouting operations.

The grouting system should have sufficient gauges, monitoring devices, and tests to determine the effectiveness of the grouting operation, and to ensure compliance with the slip liner pipe specifications and design parameters.

Checks should be carried out both by design and through survey to check that the external grout pressure does not collapse the new lining. The gauged grout pressure should not exceed the pipe manufacturer's recommendation.

The exposed ends of all slip lined pipes should be finished with a clean surface with no visible signs of grout vents, injection tubes etc. No hardened grout should remain inside the slip liner pipe invert after the completion of grouting operations.

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Glossary

Access way	A path or route that provides access to the levee from the leveed area and that must be useable also in case of flood and flood defence. See also *Escape route*
Active earth pressure	The horizontal stress exerted by a mass of soil on a retaining wall as the wall moves away from the soil.
Aeolian soil	Soil deposits that have been transported by wind.
Air-voids ratio	The ratio of the volume of air to the total volume of a mass of soil.
Alarm level	The level below crisis level. This is usually a predetermined value where the monitored levee parameter falls to within range of the crisis level, but has not resulted in systematic failure of the function being monitored. See also *Crisis level*
Allowable bearing capacity	The maximum bearing pressure that can be allowed on a foundation soil, usually in order to limit settlement.
Angle of internal friction	For a given soil, the angle on the graph of the shear stress and normal effective stresses at which shear failure occurs.
Angle of repose	The maximum angle, just before failure, of a slope composed of granular material.
Angle of shearing resistance	The ratio of effective shear and normal stresses mobilised at any state prior to failure.
Angle of wall friction	The angle of friction between soil and the surface of a retaining wall or bottom side of a foundation.
Anisotropy	A characteristic of soils which exhibit different properties such as strength, stiffness and permeability.
Annual exceedance probability (AEP)	Probability of exceeding a specified flow or level in any year (inverse of the return period for an annual maximum series).
Anthropogenic influences	General term used to describe the influence of man.
Appraisal	The process of assessing in a structured way the case for proceeding with a project or proposal. This is tied closely with Assessment.
Aquifer	A stratum of soil with relatively high permeability; a water-bearing stratum of rock or soil. See also *Confined aquifer*
Armourstone	Coarse aggregates used in hydraulic structures and other civil engineering works. A relatively large quarry stone or specially shaped concrete block that is selected to fit specified requirements of mass and shape, which is placed in a cover layer or under layer. A single stone is referred to as a piece of armour stone.
Artesian	A condition that exists when the water table piezometric surface lies above the ground level.
Asphalt	Description of all mixtures of mineral aggregates bound with bituminous materials used in the construction and maintenance of paved surfaces.
Assessment	The process of identifying, quantifying, and prioritising the condition, vulnerability, or risk associated with a system or components of a system.

Asset	Person, structure, facility, information, material, or process that has value. Generally, in this handbook, an asset is a raised defence, a structure, a watercourse, a channel, a culvert or a beach. Mainly during risk assessment, it can also indicate anything in the flood area that has value.
Asset management	Systematic and co-ordinated activities through which an organisation optimally and sustainably manages its assets and asset systems. This includes their associated performance, risks and expenditures over their life cycles for the purpose of achieving the organisations' strategic aims.
Astronomical tide	A periodic rise and fall in the level of the water in oceans and seas that are the result of gravitational effects of the earth, moon, sun and planets, without any atmospheric influences. See also *Tide*
At-rest Earth pressure (or Earth pressure at rest)	The horizontal stress developed in a mass of soil loaded in conditions of zero horizontal strain.
Atterberg limits	The water contents of a soil mass corresponding to the transition between a solid, semi-solid, plastic solid or liquid. Laboratory test used to distinguish the plasticity of clay and silt particles.
Barrage	Structure built in an estuary with the specific intention of preventing, or in some way modifying, tidal propagation. Synonym: estuary barrier (or coastal barrier) See also *Dam, Hydraulic control structure*
Base	Foundation area of a levee.
Beach	A deposit of non-cohesive material (eg sand, gravel) situated on the interface between dry land and the sea (or other large expanse of water) and actively worked by present day hydrodynamic processes (ie waves, tides and currents) and sometimes by winds.
Bearing capacity	The ability of soils to support applied foundation loads without shear failure.
Bearing pressure	The total stress transferred by a structure to the underlying ground through the foundation.
Bed forms	Mobile features on a seabed or a river bed (eg ripples, sand waves or dunes) resulting from the movement of sediment.
Bed load	Sediment transport mode in which individual particles either roll or slide or bounce along the bed as a shallow, mobile layer a few particle diameters deep. The part of the load that is not continuously in suspension. See also *Suspended load, Total load*
Bed shear stress	Stress acting tangentially to the bed, representing wave and current energy transfer to the bed.
Bench	A name applied to ledges that are shaped like steps or terraces cut into the side of a levee during construction to ensure a good interaction between two layers. See also *Berm*
Benefit area	See *Leveed area*
Benefits	The value placed on the reduced likelihood of flooding provided by flood defence assets.
Bentonite	Colloidal clay largely made up of the mineral sodium montmorillonite, a hydrated aluminium silicate.
Berm	A horizontal step in the sloping profile of a levee. See also *Bench*

Bitumen	A hydrocarbon binder. A virtually non-volatile adhesive material derived from crude petroleum that is used to coat mineral aggregate for use in construction and maintenance of paved surfaces.
Blanket	A layer or layers of graded fine stones underlying a levee or a breakwater, rock embankment or groyne. Its purpose is to prevent the natural bed material from being washed away.
Block size distribution	Sizes of armourstone pieces represented mathematically to reflect the relative proportions of smaller and larger pieces.
Body	The main part of an earth structure, whose main function is stability. For homogenous dams or levees, it also functions as water tightness. See also *Core*
Borrow pit	A site used to supply soils for earthwork construction.
Boundary conditions	Physical conditions, eg waves, currents and drifts, used as boundary input or constraint to physical or numerical models.
Breach	Any loss of material such that water could or does pass through the structure. See also *Deterioration, Break, Failure, Progressive failure, Sudden failure*
Breaching	Process of making a breach.
Break	Partial or total destruction of a levee. See also *Breach, Deterioration*
Breaking	Reduction in wave energy and height in the surf zone due to limited water depth. See also *Dissipation*
Breakwater	Structures constructed on the coastline as part of a coastal defence system or to protect beaches and/or harbours from the effects of wave action, coastal erosion or longshore drift. They can be constructed some distance from the coast, or with one end linked to the coast. They can be either fixed or floating. A breakwater structure is designed to absorb the energy of impacting waves. This is done either by using mass (eg with caissons) or by using a revetment slope (eg with rock or concrete armour units). See also *Dike, Groyne, Jetty, Levee*
Buildability	See *Constructability*
Bund	Mound of material, such as rock, gravel, sand, clay, gabions etc.
Canal	A large artificial channel, generally of trapezoidal cross-section, designed for low velocity flow. Its purpose is to convey water for navigation, hydroelectricity, irrigation or drainage.
Capillary action (or Capillarity)	Ability of liquid to flow against gravity where liquid spontaneously rises in a narrow space such as a thin tube, or in porous materials such as paper or in some non-porous materials such as liquified carbon fibre. Synonym: Soil suction
Capillary rise	The height to which water will rise above the water table due to negative pore water pressure (suction) or capillary action of the soil.
Capillary stresses	Pore water pressures less than atmospheric values produced by surface tension.
Catchment	The area from which precipitation and groundwater will collect and contribute to the flow of a specific river.
Caving	Process of losing material from a stream or river bank caused by different types of erosion. See also *Internal erosion, External erosion, Scour, Outflanking*

Channel

1. A natural or artificial waterway of perceptible extent that either periodically or continuously contains moving water or that forms a connecting link between two bodies of water.
2. The part of a body of water deep enough to be used for navigation through an area otherwise too shallow for navigation (synonym: sail line).
3. The deepest part of a stream, bay or strait through which the main volume or current of water flows.
4. A large strait, such as the English Channel.

Clay
A stiff, sticky sedimentary material that is soft and pliable when wet and consists mainly of various silicates of aluminium. Clay particles are smaller than silt, having a diameter less than 0.0039 mm. They possess electromagnetic properties that bind the grains together to give a bulk strength or cohesion.
See also *Silt, sand*

Climate change
Refers to any long-term trend in mean temperature, wind speed, drift rate and its consequences on the mean sea level, wave height, rainfall etc.

Closure structure
A structure intended to keep water from entering a leveed area, such as stop logs, earthen closure, gate or sandbag closure. The structure may be permanent or temporary.
See also *Stop logs, Demountable defence, Flood gate*

Coastal area models (2D and 3D)
Deterministic model that simulates temporal and spatial variations of hydrodynamic related parameters over a defined horizontal area.
See also *Two-/three-dimensional (2D or 3D) model*

Coastal defences
General term used to encompass both coast protection against erosion and sea defence against flooding.
Synonym: Coast protection

Cofferdam
A temporary structure used to enclose a construction area, and prevent soil or water from entering into it.

Cohesionless soils
Granular soils such as sands and gravels with values of cohesion close to zero.

Cohesive sediment
Sediment containing significant proportion of clays, the electromagnetic properties of which cause the sediment to bind together.

Cohesive soils
Clayey and silty soils that can be remoulded into balls or rolled into threads.

Colluvial soils
Soils deposited at the base of foothills via gravity or erosion.

Compaction
Volume change in soils that air, and in the case of cohesionless soils water, is expelled from the voids by mechanical action. In construction, compaction can be achieved by rolling, tamping or vibrating fill soils.

Condition assessment
An assessment of a coastal/fluvial flood/erosion defence structure to determine its condition from a structural, health and safety, and environmental perspective.
See also *Assessment, Condition appraisal*

Condition monitoring
Continuous or periodic inspection, assessment, measurement and interpretation of the resultant data to indicate the condition of the specific component. This will determine the need for some preventive or remedial action.
See also *Performance monitoring*

Consequence of failure
The (sum of) personal injuries, fatalities, material damage, environmental damages and other damages (eg cultural heritage) due to the failure of a structure or a flood defence system.
See also *Damage potential*

Consolidation
Volume change of a soil often leading to settlement as a result of the expulsion of air or water from a soil and the dissipation of excess pore pressure under sustained static loads.

Consolidation settlement	Settlement of a foundation due to squeezing out of water from the pores as the soil comes to equilibrium with the applied loads. See also *Immediate settlement*
Constructability	The extent to which the design of the building helps ease of construction, subject to the overall requirements for the completed building. Synonym: buildability
Contact erosion	Internal erosion at the joint of two soil materials. See also *Joint erosion*
Core	1 An inner, often much less permeable, portion of a structure (eg a levee). The fines content and upper sizes may be controlled. 2 A cylindrical sample of rock or soil extracted by coring. See also *Coring*
Cost benefit analysis	Method of economic analysis that assesses both costs and benefits of an intervention, design option or management process, estimating both costs and benefits in monetary units. This analytic technique is useful to compare alternatives.
Crack	A narrow break in the continuity of the material. See also *Fissure, Tension crack*
Creep	Time dependent deformations that occur in soil at constant effective stress without changes in volume and pore water pressure.
Crest	Highest elevation of levee, breakwater, seawall, sill or dam.
Critical circle	In a slope stability analysis, the slip circle that corresponds to the lowest factor of safety.
Critical depth	The water depth at critical flow in a given section of a mono-dimensional flow.
Critical flow	Free surface flow with minimum specific energy for a given discharge and a Froude number of unity. The water depth is known as the critical depth. See also *Froude number, Hydraulic jump, Subcritical flow, Supercritical flow*
Critical hydraulic gradient	The hydraulic gradient at which effective stresses becomes zero.
Cross-section	Vertical section of the levee perpendicular to the levee course/line. It includes outside and inside sections and is measured by surveying elevations with ranges across the levee from landside to riverside. Also depicts the shape of a watercourse by surveying elevations across and perpendicular to the direction of flow. See also *Planform*
Culvert	A closed conduit carrying a watercourse beneath an obstruction such as a road, railway or canal. The term 'closed' implies that a culvert has a hard soffit and invert. The term conduit implies the conveyance of water some or all of the time, but excluding tunnels and underpasses for vehicles, pedestrians and animals.
Current	Body of water that has a steady flow in a particular direction. See also *Flow, Discharge*
Current-refraction	Process by which wave velocity is affected by a current.
Cut-off wall	A wall of impervious material usually of concrete, asphaltic concrete, or steel sheet piling constructed in the foundation to reduce seepage beneath and adjacent to the levee. See also *I-wall, Sheet pile*
Dam	An artificial barrier built in rivers or estuaries that have the ability to impound water or liquid-borne materials for storage and/or control purposes. The dam is generally a permanent impoundment structure. See also *Barrage, Hydraulic control structure*

Damage potential	Property, goods etc that would be destroyed, damaged or affected in the case of potential flooding. See also *Stake, Vulnerability*
Datum	Any permanent line, plane or surface used as a reference to which elevations are referred.
Decommissioned (retired) levee	Levee that is identified by the operating authority as having lost its flood protection function.
Degradation	An irreversible process leading to a significant change in the structure of a material, typically characterised by a loss of properties (eg integrity, molecular mass or structure, mechanical strength) and/or by fragmentation. Degradation is affected by environmental conditions and proceeds over a period of time comprising one or more steps. See also *Deterioration, Weathering*
Demountable defence	Defences built above ground and supported by in situ structural foundations as part of a flood defence system. These are not normally in place but are put in place during a flood event or another event, for example to close a gate or to raise a levee. See also *Closure structure*
Density	The ratio of the total mass to the total volume of a unit of soil. Usually expressed as a unit weight where weight is interchanged with mass (unit: kg/m^3)
Depression	Relatively shallow and localised deviations in the crown or levee toe, often caused by vehicular traffic. See also *Rutting*
Desiccation	The process of shrinkage or consolidation of the fine-grained soil produced by increase of effective stresses in the grain skeleton caused by the natural drying of near-surface soils.
Design criteria	A set of conditions agreed by the developers, planners, and regulators that the proposed system should satisfy. See also *Design flood, Design storm, Protection objective, Level of protection*
Design flood	Hydrologic event(s) that is/are used to evaluate risk of overtopping, damage, or failure in consideration of defined design criteria. See also *Design storm*
Design flood level	Water level(s) referring to the design flood corresponding to local protection objectives. See also *Limit states*
Design profile	A geometric representation of a coastal or river structure, detailing the dimensions, shape and size.
Design standard	A set of engineering and/or planning procedures, policies and methodologies that are applied in the design of a system or its components.
Design storm	A hypothetical extreme storm whose waves coastal structures will often be designed to withstand. The severity of the storm (ie annual exceedance probability) is chosen in view of the acceptable level of risk of damage or failure. A design storm consists of a design wave condition, a design water level and duration. See also *Design flood*
Desk study	A preliminary investigation in which available information about a site is gathered and studied in order to characterise the site and identify ground related hazards in advance of any field investigations.
Deterioration	1 A gradual decline, as in quality, serviceability or strength.

	2 Decline in the material properties of some or all components of an asset caused by external agents (eg freeze/thaw) leading to a reduction in its structural strength. See also *Degradation, Weathering*
Deterministic	Descriptor of method or process that adopts precise, single values for all variables and input values, giving a single value output.
Development	Change of land use. Often, it is made up of housing, industrial buildings or new infrastructure. Where it is within the flood plain, it is of particular interest.
Dewatering	The removal of groundwater/surface water to lower the water table. See also *Drainage*
Diagnosis	Identification of the possible cause(s) of a failure or deterioration of a function, based on logical reasoning founded on a set of information coming from an inspection, a control or a test. By extension, a statement or conclusion from such an analysis.
Differential settlement	The vertical displacement due to settlement of one point in a foundation with respect to another point of the foundation. See also *Settlement*
Diffraction	Process affecting wave generation, where wave energy is radiated normal to the original direction of movement when it meets an obstruction. As the waves pass the obstacle the wave bends as it moves into the shadow of the obstacle.
Digital elevation models	A digital representation of ground surface topography or terrain exclusive of features (the earth surface). Synonym: Digital terrain models See also *Digital surface models*
Digital surface models	A digital representation of the ground surface that includes buildings, vegetation, and roads, as well as natural terrain features. See also *Digital elevation models, Photogrammetric analysis*
Dike	Flood protection linear structure that can be geotechnical works (levee), masonry, or concrete structure (flood wall). Also relates to sea dikes (breakwater) or the dikes along a canal or the auxiliary structure associated with a dam that serves to retain the reservoir. Also relates to river training structures. These structures are typically constructed using rock. Synonym: Wingdam
Discharge	The ratio of total volume of water flowing to a particular unit of time, normally expressed in cubic metres per second (m^3/s). Synonym: Flow rate (often abbreviated to flow)
Discharge section	Cross-section area of flowing water marked by the water level and the wetted perimeter perpendicular to the main flow velocity vector. Synonym: Flow section
Downstream	In the direction of or nearer to the mouth of a stream.
Drain	Part of a hydraulic structure whose function is to get water out of or across the structure. It can be made from coarse geotechnical material (or coarser than the rest of the body) or from geosynthetic materials. In a masonry or concrete structure it can consist of a boring (hole). See also *Filter, Slope drain, Toe drain, Drainage ditch*
Drainage	1 Process of dewatering a soil body, a structure, or a levee. 2 Removal of naturally occurring runoff from a watershed or basin by a waterway (canal, channel, or other conveyance mechanism).
Drainage area	Measured area within a drainage divide that contributes surface runoff to a given point on a stream.

Drainage ditch	Ditch parallel to the levee on the landside toe, collecting and discharging the seepage water. See also *Toe drain*
Draw-down	In subsurface hydrogeology, it is the change in hydraulic head observed at a well or aquifer, typically due to pumping. In surface water hydrology, it is the lowering of the water level in a man-made reservoir.
Draw-down rate	Water level lowering rate (velocity), eg after the flood peak.
Drowned weir flow	See *Submerged weir flow, Subcritical flow*
Durability	The ability of a material to retain its physical and mechanical properties when exposed to actual loading during the service life.
Earth pressure	The force per unit area exerted by soil on a retaining wall.
Earthworks	Earthworks are structures created through the deposition, compaction, and shaping of quantities of soil or rock used as fill materials. See also *Embankment, Levee*
Eddy	A vortex-type motion of fluid flowing partly opposite to the main current.
Effective porosity	Drainable pore volume fluid in porous media (soils) most commonly considered representing the porosity of a rock or sediment available to contribute to fluid flow through the rock or sediment.
Effective stress	The portion of the total stress that is supported through grain-to-grain contact of the soil. It is the stress in a soil mass that is effective in causing volume changes and in mobilising the shear strength arising from friction. It is the difference between the total stress and the pore water pressure.
Elastic deformation	Deformation caused in a soil due to a change in loading, where the soil recovers completely when the load is removed.
Elevation	The vertical distance above or below a local or national datum.
Embankment	Fill material, usually earth or rock, placed with sloping sides and with a length greater than its height. See also *Earthworks*
Empirical modelling	Computational modelling using empirical relationships. See also *Numerical model, Hybrid model*
Encroachment	Any permitted, authorised, or unauthorised structure that is within the easement area of the flood risk mitigation device and is not a part of the device itself.
Energy grade line	An imaginary line showing the total head or the sum of the elevation, pressure and velocity heads, of a flow relative to a datum. The slope of the energy grade line is the energy gradient. See also *Energy head, Hydraulic grade line*
Energy head	The total energy per unit weight of fluid expressed in metres of water above a geodetic datum. Also known as ‘head’. See also *Hydraulic head, Velocity head, Energy grade line, Specific Energy, Hydrodynamic force, Hydrodynamic pressure, Pressure head*
Engineered fill	Soils used as fill, such as retaining wall backfill, foundation support, dams, levees, slopes etc that are selected, deposited and compacted in accordance with engineered specifications. See also *Earthworks, Rockfill*
Engineering inspection	A detailed investigation of any asset to determine its underlying condition or performance, including any structural faults. Synonym: *Engineering survey*

Engineering properties of soil	Engineering parameters of a soil such as permeability, shear strength and consolidation (as distinct from index properties).
Engineering survey	See *Engineering inspection*
Epoch	A period of time. Used in levee management plans and other strategic documents to refer to the three time periods when considering future change: short-term (0 to 20 years), medium-term (20 to 50 years) and long-term (50 to 100 years).
Equipotential	For a flow net, lines connecting points of equal total head. Equipotential lines are usually drawn so that the interval, or equipotential drop, is constant. Equipotential lines intersect flow lines and impermeable boundaries at right angles.
Escape route	Way to leave the polder/hinterland in the case of emergency (emerging levee failure). Must also be usable during flooding of the landside area behind the levee. See also *Access way*
Estuary	A transition zone between river environments and maritime environments subject to both marine influences, such as tides, waves, and the influx of saline water; and riverine influences, such as flows of fresh water and sediment.
Event	An occurrence meeting of specified conditions (eg water level, wave height and period) in relation with the characteristics of the flood defences. By extension, the result of these conditions on the landside area (eg volume of water overtopping or overflowing, water depth, velocity of the current). See also *Joint probability*
Excess pore pressure	That increment of pore water pressure greater than hydro-static value, produced by consolidation stresses in compressible materials or by shear strain. Excess pore pressure is dissipated during consolidation. See also *Hydrostatic (pore) pressure*
Exit gradient	The hydraulic gradient near an exposed surface through which seepage is moving.
External erosion	Process by which particles are removed from a surface by the action of wind, flowing water or waves. See also *Internal erosion, Wear, Weathering*
Facing	A coating of material for architectural or protection purposes, eg stonework coating or an impervious coating on the waterside slope of the levee. See also *Revetment*
Factor of safety	The ratio of a limiting value of a quantity to the design value of that quantity.
Failure	1 Gradual decline (deterioration) or sudden decline (break) of the structure of a levee or of its foundation, leading to the inability to achieve its function. See also *Failure modes, Deterioration, Break, Breach* 2 Inability to achieve a defined performance threshold for a given function, in particular for flood defence. See also *Limit states, Progressive failure, Sudden failure*
Failure envelope	For a given soil, the graph of the shear stress and normal effective stresses at which shear failure occurs.
Failure modes	Description of one of any number of ways in which a levee or flood defence system may fail to meet a particular performance indicator.
Fetch (length)	Relative to a particular point (on the water surface or the banks), the length of the area of water surface over which the wind can blow to generate waves at the point. The fetch length depends on the shape and dimensions of the fetch area and is measured parallel to the expected wind. The longer the fetch length and

	the faster the wind speed, the larger and stronger the wave will be.
Filter	Layer or zone consisting of geotextile, geomembrane, sand, gravel, or other granular or fibrous material, preventing the fine materials from being washed through the voids of another layer or to avoid particle transport in the case of seepage. See also *Underlayer, Graded filter, Granular filter, Filter layer, Filter zone, Open layer, Slope drain, Drain*
Filter layer or zone	Layer or zone with certain grain size distribution or geotextile to avoid particle transport in the case of seepage. Synonym: Layered filter
Filtration	Function of preventing the migration of particles between two layers. This function can be accomplished by a special part (filter) or it can be accomplished naturally by the granular properties of the two zones.
Flap gate/valve	A top-hinged gate designed to close when downstream water level exceeds the upstream water level. Frequently used for drainage outfalls into tidal waters and rivers to prevent backflow. Synonym: Check valve
Flood	1 Discharge of water beyond the mean discharge under conditions of high water level. A flood is described by its probability of not being exceeded, its hydrograph, max discharge, duration, and volume. 2 An inundation (by overflowing or overtopping) that comes from a river, a sea or other body of water and causes or threatens damage. Also, any relatively high stream flow overflowing or overtopping the natural or artificial banks in any reach of a stream. See also *Storm event*
Flood defence asset	An asset that would by its failure increase the likelihood of flooding from any main river and/or the sea to people, property or infrastructure (eg levees, flood walls and other raised defences, closure structures, pumping stations).
Flood defence system	The system of levees and associated structures that protects a previously floodable area from floods up to certain conditions. See also *Benefit area*
Flood duration	Duration of the elevated water level and discharge above some threshold. See also *Persistence of storms*
Flood gate	1 A roadway or railroad closure structure that can be of varying types, eg swing gate, trolley gate and rolling gate. See also *Closure structure* 2 A means of controlling, varying or stopping a flow in a pipeline. They can be of different sorts (eg sluice gate or flap gate) and be housed in a gatewell. See also *Gatewell*
Floodplain	Land on either side of a river or behind the coastal defences that is below the highest defined flood level.
Flood Risk Management System (FRMS)	A system consisting of those flood defence assets that relate to main river or sea flooding, upon which an entity may choose to exercise operational or direct enforcement powers, and that contributes to managing flood risk to a discrete location.
Flood wall	A hard structure (eg masonry or concrete) with purpose to contain water. It can be either associated with levees in a flood protection system (function similar to an earthen levee), or be used as a structure to protect the embankment or placed on the top of a levee.

Floodway	Path that flowing water takes during a flood. Also, a designated flowage area intended to convey discharges that exceed a certain level.
Flow duration curve	Graph showing the proportion of time during which discharges are equalled or exceeded.
Flow force	Force on completely or partially submerged bodies due to the approaching flow of water. See also *Hydrodynamic force*
Flow line	The path that water will follow when moving from an area of high pressure to an area of low pressure in a seepage analysis.
Flow net	A graphical analysis of seepage flow in a mass of soil to estimate flow quantities and pore pressures.
Flow pattern	Modelling of the flow net.
Flow quantity	The total volume of water flowing in a seepage analysis.
Flow rate	See *Discharge, Stream flow*
Flow section	See *Discharge section*
Forcing	The natural processes that activate hydro- and morpho-dynamics (eg winds, waves, tides).
Forecasting	Forecasting is the process of making statements about events whose actual outcomes (typically) have not yet been observed. A commonplace example might be estimation of the expected value for some variable of interest at some specified future date.
Foundation	A component of an engineered structure that transmits a structure's forces into the underlying soil or rock. Related to levees, the levee generally rests directly on the ground without an engineered foundation, where the ground itself is the foundations. In some case, particularly if the soil has poor properties, then a blanket (eg Fascine mattress) can be used as foundation. See also *Substrate, Founding depth*
Founding depth	The depth below the ground surface where the base of a foundation is located. See also *Foundation*
Fragility	The likelihood of particular defence or system to fail under a given load condition. Typically expressed as a 'fragility curve' relating load to likelihood of failure. Combined with descriptors of deterioration, fragility relationships enable performance to be described over time. See also *Design standard*
Freeboard	1 The height of the lowest point of a structure above still water level at the maximum level of a given event. 2 The increment of levee or flood wall height added to the design flood height to increase the likelihood of the design event being contained without the levee or flood wall overtopping.
Free surface flow	Flow with a free water surface at atmospheric pressure and exposed to the air. See also *Full flow, Pressure flow*
Friction angle	See *Angle of internal friction*
Froude number (Fr)	A dimensionless ratio between inertia and gravity forces in a fluid, or between mean velocity and wave celerity. Froude number is unity for critical flow, greater than 1 for supercritical flow and less than 1 for subcritical flow.
Full flow	Flow in a closed conduit in which the water surface just reaches soffit level, but does not flow under pressure. See also *Free surface flow, Pressure flow*

Functional analysis	Analysis of a system, environment and components, based on its/their (main) functions.
Gabion	Generic name given to a revetment system consisting of stone contained in steel or polymer mesh. Types include box gabions, gabion mattresses and sack gabions.
Geology	The science that deals with the dynamics and physical history of the earth, the rocks of which it is composed, and the physical, chemical, and biological changes that the earth has undergone or is undergoing.
Geomechanics	The geologic study of the behaviour of soil and rock. The two main disciplines of geomechanics are soil mechanics and rock mechanics. The former deals with the behaviour of soil from a small scale to a landslide scale. The latter deals with issues in geosciences related to rock mass characterisation and rock mass mechanics, such as applied to tunnel design, rock breakage, and rock drilling. Many aspects of geomechanics overlap with parts of geotechnical engineering. Modern developments relate to seismology, continuum mechanics, discontinuum mechanics, and transport phenomena.
Geomembrane	A kind of geosynthetic material, which is impermeable.
Geomorphology	Describes the characteristics of all the features on the earth, in particular the river, estuary, lake or seabed forms and systems and examines the processes sustaining them.
Geophysical survey	The systematic collection of geophysical data for spatial studies. This process produces images of features (such as archaeological and geotechnical) that are hidden below the ground surface. A great variety of sensing instruments may be used, and data may be collected from above or below the Earth's surface and from aerial or marine platforms. See also *Ground investigation*
Geophysics	Quantitative physical methods for exploring structures and properties beneath the Earth's surface. A variety of methods and instruments are available using natural or artificial sources generating, eg electromagnetic/seismic waves or static/dynamic electric/magnetic fields and corresponding sensors at the surface, on/below water or in boreholes to record the response of the subsurface.
Geosynthetics	Generally polymeric products used to solve civil engineering problems. The term is generally regarded to encompass eight main product categories: geotextiles, geogrids, geonets, geomembranes, geosynthetic clay liners, geofoam, geocells (cellular confinement) and geocomposites.
Geotechnique	The branch of civil engineering that deals with the mechanical behaviour of soils, rocks and earthwork materials. It adopts the principles of soil and rock mechanics to evaluate the stability and performance of both natural soils and man-made earthen structures.
Geotechnical instrumentation	Instruments used to monitor phenomena such as deformation, pore pressures and stress within the ground.
Geotextile	Any strong synthetic fabric used in civil engineering, as to retain an embankment to stabilise soils, retain soils, prevent the mixing of dissimilar soils, provide a filtering function, pavement support, subgrade reinforcement, drainage, erosion control and silt containment.
Graded filter	Filter consisting of different layers with different grain sizes. See also *Filter, Granular filter*
Grading curve	See *Particle size distribution*
Grain size distribution	See *Particle size distribution* See also *Block size distribution*

Granular filter	A bed of granular material incorporated in a levee and graded as to allow seepage to flow across or down the filter zone without causing the migration of the material from zones adjacent to the filter. See also *Filter, Graded filter*
Ground investigation	The sub-surface part of a site investigation including sampling and field testing and with associated laboratory testing and factual reporting. See also *Geophysical survey*
Groundwater	Water that is below the surface of the ground in the saturated zone.
Grout	A material used to fill voids, and seal joints. It is usually composed of a mixture of water, cement, sand, and sometimes fine gravel. It hardens over time much like mortar.
Groyne	Narrow, roughly shore- normal structure built to reduce longshore currents, and/or to trap and retain beach material. Most groynes are made of timber, rock or concrete, and extend from a seawall, or the backshore, onto the foreshore and occasionally further offshore. See also *River training structure* Synonym: Spur-dike
Habitat	The area or environment where an organism or ecological community normally lives or occurs.
Hazard	1 A situation, physical event (eg flood or storm), phenomenon or human activity with the potential to result in harm. 2 Probability for a dangerous phenomenon to occur with a given intensity.
Head loss	The difference in head between two points due to friction or other features that result in energy loss (eg a transition, step, constriction, expansion, or bend). See also *Energy head*
Headwall (of a culvert)	The retaining wall at a culvert inlet or outlet that provides support to the embankment. The headwall is normally at right angles to the culvert barrel, but may be skewed. The headwall may have wingwalls at an angle to the headwall that provide support to the channel sides and form part of the transition from channel to culvert and vice versa.
Height of levee	Vertical measured difference between the landside levee toe and the highest point of the levee crest.
Heterogeneous soil	A mass of soil with highly variable index and engineering characteristics. Antonym: Homogeneous soil
Homogenous soil	A mass of soil where the soil is of one characteristic having the same engineering and index properties. Antonym: Heterogeneous soil
Hybrid model	Model that adopts a combination of empirical and deterministic modelling approaches. See also *Numerical model, Empirical modelling*
Hydration	The introduction of water to a substance.
Hydraulic conductivity	Ratio of flow velocity to driving force (hydraulic gradient) for viscous flow under saturated conditions of a specified liquid in a porous medium.
Hydraulic control structure	Gated or fixed structure used to regulate the discharge through, over, or under a flood protection work. See also *Barrage, Dam, Spillway, Weir, Flood gate*
Hydraulic grade line	See *Hydraulic head, Energy grade line*
Hydraulic gradient	1 Quotient of drop in hydraulic energy and distance of flow.

	2 In a structure or in soil: the hydraulic gradient is the difference between two or more hydraulic head measurements divided by the length of the flow path.
Hydraulic head (piezometric head)	Hydraulic head (or piezometric head) is a specific measurement of water pressure in units of length above a given geodetic datum. See also *Energy head, Hydraulic grade line*
Hydraulic jump	Abrupt rise in water level when flow changes from supercritical to subcritical, accompanied by surface disturbance and air entrainment and an associated dissipation of energy. See also *Critical flow*
Hydraulic performance	Performance of a levee (system) in terms of protection again hydraulic events (flood, storm).
Hydraulic roughness	Measure of the amount of frictional resistance water experiences when passing over land and channel features. The roughness can be expressed according to Manning (n) or Strickler (kSt). An increase in the n value will cause a decrease in the velocity of water flowing across a surface. See also *Manning's equation*
Hydraulics	The scientific study of water and other liquids, in particular their behaviour under the influence of mechanical forces and related uses in engineering.
Hydrodynamic force	Forces due to currents and waves on a completely or partially submerged body. See also *Hydrodynamic pressure, Flow force*
Hydrodynamic (or hydraulic) pressure	The pressure exerted by water (whether at rest or moving) on a surface or structure. Hydraulic pressure has the units of force per unit area and is calculated for water at rest as the product of the depth of water and its density. The pressure can differ for water in motion. See also *Hydrostatic pressure, Pressure head*
Hydrogeology	Area of geology that deals with the distribution and movement of groundwater in the soil and rocks of the Earth's crust.
Hydrograph	Graph showing the variation of discharge or water level over time.
Hydrology	Science of the hydrological cycle, including precipitation, runoff and fluvial flooding.
Hydrostatic pore pressure	Pore water pressures exerted under conditions of no groundwater flow where the magnitude of pore pressure increases linearly with depth below the groundwater surface.
Hydrostatic pressure	The pressure exerted by water at rest on a surface or structure. The product of the depth of water and its density. See also *Hydrodynamic (hydraulic) pressure*
Immediate settlement	The settlement of a foundation occurring immediately upon loading. See also *Consolidation settlement*
Impermeable	Will not allow water to pass through. Synonym: Impervious
Impervious	See *Impermeable*
Incident wave	A wave moving towards land or to a structure.
Index properties	Attributes of a soil such as moisture content, void ratio, specific gravity, Atterberg limits and grain size distribution, which are unaffected by remoulding that soil (as distinct from engineering properties).
Infiltration	1 The entrance of groundwater into a structure (eg sewer, culvert or pipeline) through breaks, defective joints, or porous walls.

	2 The penetration of water through the soil from surface precipitation, stream or impoundment boundaries.
Infrastructure	Collective term for a group of assets needed for the operation of a society or enterprise or the services and facilities necessary for an economy to function. It includes physical resources, services and information technology facilities, networks and assets that, if they were disrupted or destroyed, would have a serious effect on the health, safety, security or economic well-being of citizens or the effective functioning of government. Examples include roads, railways, public services, power supplies and telecom equipment.
Internal erosion	The movement of soil particles as a result of chemical actions and/or unbalanced seepage forces produced by percolating water. See also *External erosion, Wear, Weathering, Piping, Joint erosion, Contact erosion, Retrogressive erosion*
Isotropy	A characteristic of soils that exhibit the same properties such as strength, stiffness and permeability in all directions. Isotropy is often the result of engineering approximation rather than a true soil property
I-wall	Sheet pile driven vertically into the ground. See also *Sheet pile, Cut-off wall*
Jetty	A structure extending into a body of water that protects a harbour or coastline from the effects of currents and tides. See also *Breakwater*
Joint erosion	Internal erosion in the joint between soil and concrete/masonry.
Joint probability	The probability that two or more specific outcomes will occur in an event. See also *Event*
Landside	Refers to the side of the flood defence structure opposite to the waterside. Antonym: Waterside
Layered filter	See *Filter layer*
Leakage	Unwanted discharge of fluid. See also *Seepage, Resurgence*
Leveed area	Area behind the levee that is not flooded, or in which the flooding is reduced or delayed due to the levee/flood defence system.
Levee	Raised, predominantly earth, structures (sometimes called flood defence embankments or dikes) whose primary objective is to provide protection against fluvial and coastal flood events along coasts, rivers and artificial waterways that are not reshaped under normal conditions by the action of waves and currents. Levees form part of flood defence systems that may also include flood walls, pumping stations, closure structures, natural features etc. See also *Dike*
Level of protection	**For a levee:** the maximum event that, with a high degree of assurance, will not result in levee failure subsequently inundating the leveed area. This maximum event can be associated with a probability of occurrence. **For a flood defence system:** the maximum event that, with a high degree of assurance, will not result in defence system failure subsequently inundating the flood defence area. This maximum event can be associated with a probability of occurrence. See also *Design criteria*
Levee segment	The division of a levee based on some determined parameter such as ownership, composition etc.
Life cycle cost	Total cost of managing an asset over its design life (or service life), ie the

	assumed period of time after construction or refurbishment when an asset meets or exceeds its functional performance requirements with anticipated maintenance but without major repair being necessary. See also *Whole life cost*
Limit states	1 Conditions under which a structure can no longer perform its intended functions. 2 The boundary between safety and failure for a structure. The limit state function Z=R-S is a function of the structure's strength (R) and loading (S) for a particular failure mode. Failure will not occur if the limit state function is positive. Generally two types of limits state are distinguished: ultimate limit states (ULSs) are related to the safety of the structure and they define the limits for its total or partial collapse. Serviceability limit states (SLSs) represent those conditions that adversely affect the expected performance of the structure under normal service loads. See also *Failure*
Liquefaction	Describes a phenomenon whereby a saturated soil substantially loses strength and stiffness in response to an applied stress, usually earthquake shaking or other sudden change in stress condition, causing it to behave like a liquid.
Liquidity index	A measure of the relationship between the current water content of a soil and its consistency limits.
Liquid limit	The water content above which the soil will flow like a liquid, but below which it will have a plastic consistency.
Lithology	Refers to rock type and composition.
Maintenance	All activities whose purposes are to maintain or restore a system in a state or in given safety or working condition, to perform a required function. It includes preventative maintenance and repairs (exclusive options). Generally it consists in repairing or replacing the components of a structure whose life is less than that of the overall structure, or of a localised area that has failed or will fail.
Maintenance area	Stripe/sector at both toes of the levee that should be kept clear for monitoring and maintenance.
Manning's equation	An empirical formula for estimating flow in open channels, or free-surface flow driven by gravity. See also *Hydraulic roughness*
Marsh	An area of low-lying wetland in which the level of water is generally shallow and often fluctuating. The water may be either standing or slow-moving. In contrast to a swamp, in which there is an abundance of woody plants, the plants in a marsh are mostly herbaceous. Reeds and rushes dominate the vegetation of marshes.
Maximum dry density	A soil property obtained in the laboratory from a compaction (Proctor) test. It is the density of compacted soil at 100 per cent compaction under the particular level of compaction applied.
Meandering	A single channel characterised by a pattern of successive deviations in alignment that results in a more or less sinusoidal course.
Mean normal stress	The mean value of the three orthogonal stresses.
Mean sea level	The average level of the sea over a period of 12 months, taking account of all tidal effects (see tides) but excluding surge generated by meteorological effects. Variation in mean sea level may well occur in the longer term.
Mean wave period	The mean period of the wave defined by zero-crossing analysis of a wave record.

Mechanism	The fundamental processes involved in or responsible for an action, reaction, or other natural phenomenon (these mechanisms can lead to breach, collapse, settlement and other failures modes).
Median annual flood	Flood with an annual exceedance probability of 50 per cent (return period two years), defined as QMED by the Institute of Hydrology (1999).
Metadata	Definitional data that provides information about or documentation of other data managed within an application or environment. It is used to document data about data elements or attributes, (name, size, data type etc) and data about records or data structures (length, fields, columns etc) and data about data (where it is located, how it is associated, ownership etc). Metadata may include descriptive information about the context, quality and condition, or characteristics of the data.
Modular flow	State of flow over crest of weir or other control structure in which the upstream water level depends on the discharge but is independent of the water level downstream of the structure. Antonym: Submerged weir flow See also *Supercritical flow*
Moisture content	The ratio between the mass of water and the mass of soil solids.
Monitoring	Systematic recording over time to establish trends in data. See also *Performance monitoring*
Monochromatic waves	See *Regular waves*
Morphology	The plan form and cross-section shape of a watercourse. See also *Geomorphology*
Mud	Wet, soft earth or earthy matter, on the ground after rain, at the bottom of a pond, or along the banks of a river.
Multi-criteria analysis	The use of more than one factor, with different units of measurement or appraisal, to judge performance. Usually analysed within a structured decision making tool.
Normal compression line	The relationship between void ratio and the normal effective stress for soil loaded beyond the current yield stress in an isotropic compression.
Normal flow	Steady, uniform flow in an open channel where the hydraulic and energy grade lines are parallel and Manning's equation applies. See also *Uniform flow*
Normally consolidated soil	Soil having a current state that lies on the normal compression line.
Numerical model	Mathematical equations that attempt to describe reality and permit prediction of the behaviour of phenomenon such as flow, sediment transport, shoreline evolution etc. See also *Coastal area models (2D and 3D), Empirical modelling, Hybrid model, Physical model, Two/three-dimensional (2D or 3D) model, Shoreline evolution models*
One dimensional compression	Compression taking place with zero radial and horizontal strain.
One dimensional (1D) model	A numerical model in which all the flow parameters are assumed to be constant over the cross-section normal to the flow. There is only a velocity gradient in the flow direction.
Open layer	A layer or stratum of soil from which porewater may drain both upward and downward into overlying and underlying permeable layers, thus enabling two-way drainage. See also *Filter, Drain*

Operation	The day-to-day activities, associated with the flood defence management, exclusive of the maintenance itself.
Operational inspection	A regular inspection of an asset to check it is in working order and in a safe condition. See also *Engineering inspection, Visual asset inspection, Reach inspection*
Optimum moisture content	The water content at which the maximum dry density of a soil is obtained using a specific effort of compaction.
Organic soil	Earth containing a significant proportion of organic material or peat.
Outflanking	Erosion or scour behind or around the land-based end of a structure that may threaten to compromise the stability or integrity of the structure and its function. See also *Scour*
Over-consolidated soil	A clayey soil carrying a higher load in the past. Soil having a current state that lies inside the normal compression line.
Over-consolidation ratio	The ratio of maximum past pressure (pre-consolidation pressure) to the current effective stress.
Overflowing	Passing of water over the top of a structure as a result of a water level higher than the crest of the structure. See also *Overtopping*
Overflow	See *Spillway, Weir* Synonym: Levee overflow, Safety spillway
Overtopping	Passing of water over the top of a structure as a result of wave action, surge or wind. The water level in front of the structure is lower than the crest level of the structure. See also *Wave overtopping, Overflowing*
Overturning	A result of excessive lateral earth pressures with relation to retaining wall resistance thereby causing the retaining wall system to topple or rotate (overturn).
Parapet	See *Crown wall*
Particle size distribution	Soil particle sizes that are determined from a representative sample of soil, which is passed through a set of sieves of consecutively smaller openings.
Passive earth pressure	The maximum horizontal stress exerted by a mass of soil on a retaining surface as the surface moves toward the soil.
Peak	The top of a wave. Antonym: Trough
Peak period	The wave period determined by the inverse of the frequency at which the wave energy spectrum reaches a maximum.
Performance	The degree to which a system (eg a flood defence system), a structure (eg a levee) or a component succeeds when evaluated against some stated aim or objective. See also *Hydraulic performance*
Performance assessment	A comparison of present performance against performance requirements. The assessment considers the effect of condition on each performance requirement and the effect of each performance requirement on the performance of the subsystem or system. The key to performance assessment is an understanding of the link between asset (or system) condition and its response under a range of loading conditions. Outputs from this stage are the probability of failure and residual life.
Performance indicator	Specific, measurable and time-related output of a particular asset management policy or project. May be technical such as acceptable wave overtopping rates or

	conveyance capacity, or more generic such as public satisfaction. Performance indicators are designed to address the quality of the execution of a project or initiative and the degree to which the initiative meets the requirements of funders. They compare actual conditions with a specific set of reference conditions and they measure the distance(s) between the current environmental situation and the desired situation (target), ie distance to target assessment. Synonym: Performance measure
Performance monitoring	Continuous or periodic quantitative and qualitative assessments of the actual performance compared with specific objectives, targets or standards. See also *Condition monitoring*
Performance requirement	The hydraulic, structural, environmental or other criteria to which an asset or system is built and maintained.
Permeability	The property of a soil that controls the rate of flow of water through that soil. It depends on the physical properties of the medium, for example grain size, porosity, and pore shape. See also *Porosity, Porous*
Persistence of storms	The duration of sea states above some severity threshold (eg wave height). See also *Flood duration*
Phreatic surface	See *Piezometric surface*
Physical model	Simulation of a structure and/or its environment, usually in much smaller dimensions, to enable the consequences of future changes to be predicted. Synonym: Scale model See also *Prototype*
Piezometric level	An imaginary line representing the total head in an aquifer, ie it represents the height above a datum plane at which the water level stands in boreholes penetrating the aquifer. See also *Saturation line*
Piezometric surface	An imaginary or hypothetical surface of the piezometric pressure or hydraulic head throughout all or part of a confined or semi-confined aquifer; analogous to the water table of an unconfined aquifer. The piezometric surface provides an indication of the direction of groundwater flow and is used to determine hydraulic gradients. Synonym: Phreatic surface
Piping	The creation of flow channels within a levee or the underlying ground as a result of seepage and continuing internal erosion. Piping can lead to the development of boils or breaches. See also *Internal erosion, Retrogressive erosion*
Pitched stone	Squared masonry, pre-cast blocks or embedded stones laid in regular fashion with dry or filled joints (to increase friction forces). It is often placed on the waterside slope of levees as a protection against wave and ice action.
Placed rockfill, stone packing	Erosion protection surface layer of stones (set by hand or carefully set with a loader). See also *Pitched stone*
Planform	The form of a river or stream when viewed from above, for example, the term 'meandering' is a description of a sinuous planform. See also *Cross-section*
Plastic deformation	The distortion of soil resulting in a permanent and irrecoverable change in shape or volume.
Plastic limit	The moisture content in which a soil will have a plastic consistency.

Plasticity index The difference between the liquid limit and plastic limit of a soil mass.

Ponding
1 A plugging of the filter media to restrict downward movement of water causing surface accumulation.
2 A body of water that is impounded on the landside of a levee when natural drainage is severed or temporarily interrupted by the levee or operation of structures associated with the levee.

Pore pressure The interstitial pressure of water within a mass of soil or rock.

Porosity The ratio of the volume of voids to the total volume of soil, generally expressed as a percentage.

Porous
1 Having many pores or other small spaces that can hold or transport a fluid.
2 For revetments and armour layers, the permitting of rapid through movement of water, such as during wave action.
See also *Permeability*

Pre-consolidation pressure The maximum past pressure of a soil.

Pressure flow Flow within a closed conduit that is confined by and exerts hydraulic pressure on the conduit walls and soffit.
Synonym: Surcharged flow
See also *Free surface flow, Full flow*

Pressure head Height of a column of water required to develop a given pressure at a given point.
See also *Energy head, Hydrodynamic (Hydraulic) head*

Primary consolidation The long-term consolidation of clay or an organic soil from the loss of water in the voids due to high pressure.

Principal strains The strains occurring in the directions of the principal axes of strain.

Principal stresses Normal stresses acting in the direction of principal axes of stress.

Probabilistic Descriptor of method or process in which the variability of input values (eg asset loading and strength) and their sensitivity are taken into account to give results in the form of a range of probabilities for different outcomes (eg failure).
See also *Deterministic*

Probabilistic design This deals primarily with the consideration of the effects of random variability upon the performance of an engineering system during the design phase. Each variable is viewed as a probability distribution rather than a single value or number.

Probability Measure of the chance that an event will occur. Typically defined as the relative frequency of occurrence of that event out of all possible events and expressed as a percentage with reference to a time period eg one per cent annual exceedance probability.

Process A systematic series of actions directed to some end, eg a breaching process is composed of a succession of failure mechanisms.

Progressive failure Failure process where, once a threshold is exceeded, some residual strength enables the asset to maintain restricted performance while further progressive loss of strength takes place.
See also *Failure, Sudden failure*

Protection objective Level of protection related to an event with a certain recurrence period that shall be achieved by the protection measures.
See also *Design criteria*

Prototype The actual structure or condition being simulated in a model.
See also *Physical model*

Pumping station	A structure used to evacuate water from interior drainage in a flood defence system. See also *Flood defence asset*
Quarry run	Materials with no fines control and including all granular material found in the quarry blast pile that can be picked up in a typical loading shovel, ie only blocks too large for easy digging and loading are left behind.
Quasi-three-dimensional (3D) model	A numerical model in which the flow parameters vary in two dimensions, but which allows determination of the flow parameter in the third dimension. See also *Two/three-dimensional (2D or 3D) model*
Raised defence	Any raised structure that protects an area from flooding. See also *Flood protection structure*
Random waves	The laboratory simulation of irregular sea-states that occur in nature.
Rating curve	A relationship between discharge/ flow and depth or water elevation at a given point.
Reach	Watercourses are divided up into measurable lengths called reaches for ease of management. See also *Frontage*
Reach inspection	An inspection measuring the probability and consequences of failure of a particular reach. This information can be used to determine the frequency of asset visual inspection.
Reflection	The process by which (part of) the energy of the wave is returned seaward.
Refraction	The process by which the direction of a wave moving in shallow water at an angle to the seabed contours is changed so that the wave crests tend to become more aligned with those contours.
Refurbishment	The process of returning an asset to its original as-designed performance. Synonym: Renovation See also *Rehabilitation*
Regular waves	Waves with a single height, period and direction. Synonym: Monochromatic waves
Rehabilitation	The process of restoring an asset for the purpose of returning that asset to design performance. See also *Refurbishment, Reinforcement*
Reinforcement	The process of improving the performance of an asset (or one of its components) against an event or a degradation mechanism. See also *Rehabilitation*
Relative compaction	A minimum density specification usually designated as a percentage of the maximum dry density.
Relative density	The density of a granular soil relative to the minimum and maximum densities achieved for that particular soil.
Relief well	A vertically installed well consisting of a well screen surrounded by a filter material designed to prevent in-wash of foundation materials into the well. Relief wells are used extensively to relieve excess hydrostatic pressures in pervious foundation strata overlain by more impervious top strata, conditions that often exist landside of levees and downstream of dams and various hydraulic structures.
Renovation	See *Refurbishment*
Repair	Restoring to operating condition after damage has occurred and a structure's functionality has been reduced. Repair can also be thought of as corrective maintenance.
Reservoir	An artificial lake, basin or tank in which a large quantity of water can be stored.

Residual risk, remaining risk The remaining level of risk at any time before, during and after a program of risk mitigation measures has been taken.

Residual water level The components of water level not attributable to astronomical effects.
See also *Surge, Still water level, Tidal range*

Resilience The ability to adapt to changing conditions and prepare for, withstand, and rapidly recover from disruption.

Resurgence
1. Any natural situation where water flows to the surface of the earth from underground (ie the aquifer surface meets the ground surface).
2. More specifically, in this handbook, flow of water at the surface of the landside of a levee or in the nearby natural ground.

See also *Seepage, Leakage*

Retaining wall Walls, usually constructed of concrete, rock, or sheet piles, which provide lateral stability of the earth, preventing the soil from sloughing or slope failure. Different types of retaining walls exist, eg gravity wall, counterfort wall, I-wall, T-wall.

Retrogressive erosion Internal erosion starting on the landside progressing towards the waterside of the levee.
Synonym: Backward erosion
See also *Internal erosion, Piping*

Return period For a given parameter (eg water level), the mean duration between two events where this parameter was observed. Inverse of the probability that a given event will occur in any one year.
Annual exceedance probability (AEP) is the preferred term for flood risk management, one per cent AEP being equivalent to a 100-year return period.
Synonym: Recurrence period, Recurrence interval

Return seepage Seepage water on the landside of the levee that is captured by some type of collection system and evacuated.
See also *Seepage*

Return seepage channel Channel made for the purpose of collecting and evacuating seepage water. This channel can be made by digging a ditch or by building a small levee known as 'return seepage levee'.

Return seepage levee See *Seep water levee*

Revetment Works to protect the slopes of a levee against erosion, typically constructed from armourstone, masonry, asphalt or concrete blocks.
See also *Facing*

Riparian Of, pertaining to, situated, or dwelling on the bank of a river or other body of water. A riparian zone or riparian area is the interface between land and a river or stream. Plant habitats and communities along the river margins and banks are called riparian vegetation, characterised by hydrophilic plants.

Risk Risk is defined as being a function of the probability that an event will occur and the consequence associated with that event. Risk = f (probability x consequence).
A measure of the probability and severity of undesirable consequences or outcomes.

Risk analysis Risk analysis is a decision-making framework that comprises three tasks – risk assessment, risk management, and risk communication.

Risk assessment The process of identifying hazards and potential consequences, estimating the magnitude and probability of consequences, and assessing the significance of the risk(s). A tiered approach can be used with the effort in assessing each risk proportionate to its importance in relation to other risks and likely consequences.

Risk attribution	The contribution of specified assets or groups of assets to the overall risk associated with a leveed area. This helps interventions to be targeted on managing the greatest risks.
Risk control	The deliberate action taken to reduce the potential for harm or maintain it at an acceptable level.
Risk management	The systematic process of risk assessment, options appraisal and implementation of any measures to control or mitigate risk.
Risk monitoring	The definition of the measures necessary to control the risk, coupled with their use – the management of the risk. The risk management process should include the arrangements for monitoring the effectiveness of the control measures together with their review to ensure continuing relevance.
Runoff	Overland flow produced by rainfall.
Run-up, run-down	1 The rush of water up a structure or beach as a result of wave action. 2 The upper and lower levels reached by a wave on a beach or coastal structure, relative to still water level, measured vertically. See also *Swash zone*
Rutting	A long stretch of depressions in the levee crown or levee toe caused by vehicular traffic wearing away a longitudinal or vertical portion of the levee roadway. See also *Depression*
Sand	Sediment particles, mainly of quartz, with a diameter of between 0.062 mm and 2 mm, generally classified as fine, medium, coarse or very coarse. See also *Clay, Silt*
Sandbag	A sack made of hessian/burlap, polypropylene or other materials that is filled with sand or soil. Synonym: Floodbag
Saturation line	Representation of the piezometric levels on a cross-section. See also *Piezometric surface*
Scale model	See *Physical model*
Scenario	Account or synopsis of a possible course of action or events.
Scour	In a stream: erosion of the bed or banks of a watercourse by the action of moving water typically associated with channel contraction or local feature such as bridge pier. On the coast: erosion resulting from shear forces associated with flowing water and wave actions. See also *Caving, Outflanking*
Sea defences	Works to prevent or alleviate flooding by the sea. See also *Coastal defences*
Sea state	Description of the sea surface with regard to wave action.
Seepage	In soil engineering, the movement of water in soils. Seepage depends on several factors, including permeability of the soil and the pressure gradient. See also *Leakage, Resurgence*
Seepage berm	Construction of additional weight at the landside toe of the levee to counteract upward seepage forces and/or additional length required to reduce uplift pressures at the toe of the levee to tolerable values.
Seepage force	The force transmitted to a mass of soil due to the seepage of groundwater.
Seepage pressure	The seepage force per unit volume.
Seepage velocity	The average velocity at which groundwater flows through the pores of a soil. The ratio of the volume flow rate to the average area of voids in a soil cross-section.

Seep water levee	Small levee on the landside of the main levee made for the purpose of collecting and evacuating seepage water in a channel. It can also impound seeping water to reduce the hydraulic gradient. Synonym: Return seepage levee
Sensitivity analysis	Testing the potential variations in the outcome of an evaluation by altering the values of important factors that have uncertainty.
Settlement	The downward movement of the ground surface or a structure on or in the ground as a result of external stresses. See also *Differential settlement*
Settling	The process by which particulates settle to the bottom of a liquid and form sediment.
Shallow water	Commonly, water of such depth that surface waves are noticeably affected by bottom topography. It is customary to consider water of depths less than half the surface wavelength as shallow water. Antonym: Deep water See also *Shoaling*
Shear strength	The maximum shear stress that a soil can sustain under a given set of conditions.
Shear stress	The force per unit area acting tangentially to a given plane or surface.
Sheet pile	Interlocking panels of steel that are driven into the ground to provide lateral support. See also *I-Wall, Cut-off wall*
Shoaling	Decrease in water depth resulting in the transformation of wave profile as they propagate inshore or, more specifically, a change in wave height related to the changing speed of propagation of wave energy.
Shoaling coefficient	Ratio of shoaled wave height to deep water wave height.
Shoulder	Horizontal section between levee crest and slope. See also *Berm*
Significant wave height	Average height of the highest one-third of the waves in a given sea state.
Significant wave period	Average of the periods associated with the highest one-third of wave heights in a given sea state.
Sill	1 A submerged structure across a river to control the water level upstream. See also *Weir* 2 The crest of a spillway.
Silt	A sedimentary material consisting of grains or particles of disintegrated rock, smaller than sand and larger than clay. The diameter of the particles ranges from 0.0039 mm to 0.0625 mm. Silt is often found at the bottom of bodies of water where it accumulates slowly by settling through the water. See also *Clay, Sand*
Site investigation	The process of methodically observing, sampling and testing for the purpose of characterising the ground and investigating potential hazards. Site investigation techniques can be intrusive or non-intrusive. See also *Geophysical survey, Ground investigation*
Sliding	1 Movement of a layer of materials along a slope or on a horizontal plane. Synonym: Sloughing 2 A result of excessive lateral earth pressures with relation to retaining wall resistance thereby causing the retaining wall system to move away (slide) from the soil it retains.
Slope	1 Inclined face of a cutting, bank or levee.

	2 Amount of inclination of a surface or a line to the horizontal. It is a special case of the gradient in calculus where zero indicates gravitational level. A larger number indicates higher or steeper degree of 'tilt'. Often slope is calculated as a ratio of 'rise' to 'run', or as a fraction ('rise" over "run') in which run is the horizontal distance and rise is the vertical distance.
Slope drain	Layer of coarse soil that can be put on potential water sources on the landside slope to stabilise and dewater. See also *Drain, Filter*
Slope protection	A structure (eg rock or concrete) on the slope intended to protect the underlying material against erosion by current and/or wave action. See also *Bed protection, Revetment*
Sloughing	Movement of a mass of soil down a bank or slope into the channel usually occurring when the bank or underlying stratum is saturated. See *Sliding*
Sluice	A water channel that is controlled at its head by a gate.
Sod	1 Section cut or torn from the surface of grassland, containing the matted roots of grass. 2 Surface of the ground, especially when covered with grass or turf.
Soil classification	A standardised classification system for quantifying certain soil characteristics that is important for determining soil behaviour.
Soil suction	See *Capillary action (or capillarity)*
Specification	1 An explicit set of requirements to be satisfied by a material, product, or service. 2 Document detailing the materials, construction and/or measurement requirements for a contract, agreed by the contracted parties before they undertake the contract.
Specific energy	The energy of a fluid relative to bed level, given by the sum of pressure and velocity heads. See also *Energy head*
Spillway	A designed section of a levee with lower crest elevation, protected crest and slope, through which flow can be discharged in order to protect the levee system against failure by overflowing non-protected sections. Synonym: Safety spillway, Overflow, Levee overflow See also *Weir*
Stable	1 *Physics:* having the ability to react to a disturbing force by maintaining or re-establishing position, form, or function. A structure can be statically stable or dynamically stable. 2 *Chemistry:* not readily decomposing, as a compound; resisting molecular or chemical change.
Stake	1 A monetary or commercial interest, investment, share, or involvement in something, as in hope of gain. 2 A personal or emotional concern, interest, involvement, or share. See also *Damage potential, Properties at risk, Vulnerability*
Stakeholder	An individual or group with an interest in, or having an influence over, the success of a proposed project or other course of action.
Standard of protection (SoP)	Criteria to be achieved during analysis and design.
Standard of service (SoS)	The performance of an asset at a specific point in time expressed in terms of a physical attribute(s) of the asset or system (eg crest level, pump capacity).

Standard Penetration Resistance The number of blows required to drive a split-spoon sampler during a standard penetration test a distance of 0.305 m after the initial penetration of 0.15 m.

Standard Penetration Test (SPT) A field test that measures resistance of the soil to the penetration of a standard split-spoon sampler that is driven 300 mm into the ground at the base of a borehole with a 63.5 kg hammer dropped from a height of 0.76 m. The standard penetration resistance is derived from this test.

Stationary process A process in which the mean statistical properties do not vary with time.

Steady state pore pressure The pore water pressure at equilibrium when all excess pore pressures within a soil mass have fully dissipated.

Still water level Average water surface elevation at any instant, excluding local variation due to waves and wave set-up, but including the effects of tides, surges and long period seiches.
See also *Residual water level, Still water level*

Stochastic Having random variation in statistics.

Stoplogs Timber or metal beams spanning horizontally between grooves in piers or abutments of a control structure, used to isolate part of the structure or related reach for maintenance, or to raise the elevation of water retained.
Synonym: Stop planks

Storm event A storm event can be described by several sea-states, eg the increasing phase, the maximum phase and the decreasing phase. At locations under tidal influence the typical sea-state is often only two to three hours, but without tidal effects it may last six hours or longer depending on the evolution in time of wind conditions (typical timescale in the order of 12 hours to one day).
See also *Event, Flood*

Storm surge A rise of sea elevation caused by water piling up against a coast under the force of strong onshore winds such as those accompanying a hurricane or other intense storm. Reduced atmospheric pressure may contribute to rise.
See also *Surge*

Strain Deformation of a body or structure as a result of an applied force.

Strand line An accumulation of debris (eg seaweed, driftwood and litter) cast up onto a beach, and lying along the limit of wave uprush.

Stratum A layer of rock or soil with internally consistent characteristics that distinguish it from other layers.

Stream regime Combinations of river discharge and water levels characteristic for a prescribed period (usually a year or a season). The stream regime determines the overall morphology of the stream.
See also *Regime theory, Flow regime, River training structure*

Stress Physical pressure, pull or other force exerted on one thing by another.

Stress history The past history of loading and unloading of a soil mass.

Structure A constructed component using processed materials, such as concrete, masonry, armourstone, and steel, which is part of a flood protection system.

Subcritical Flow condition where the Froude number is less than unity. Subcritical flow describes the flow condition where upstream water level is influenced by conditions that exist downstream.
See also *Submerged weir flow, Critical flow, Supercritical flow*

Submerged weir flow Flow over crest of weir or other hydraulic structure that does not pass through critical flow, where the upstream water level depends on the water level downstream of the structure. The downstream water depth above crest level exceeds critical flow depth above crest level.

	Synonym: Drowned weir flow and Sub-modular flow See also *Subcritical*
Submergence ratio	The ratio between downstream water depth above weir crest level and the upstream water depth above weir crest level.
Subsidence	Subsidence is movement of the ground (mostly vertical) that is not caused by the application of an external load. Examples of subsidence include karst, internal erosion, the collapse of mine workings, settlement due to animal burrowing and desiccation shrinkage caused by seasonal moisture take by trees and other large vegetation.
Substrate	Material underlying or supporting a structure or another layer of material. See also *Foundation*
Sudden failure	Failure where the break process is fast. It can lead to more severe consequence than a progressive failure. See also *Failure*
Suffusion	The migration of soil particles through the soil matrix driven by flow through the soil. Suffusion is a contributor to the manifestation of internal erosion.
Supercritical	Flow condition where Froude number is greater than unity. Supercritical flow describes the flow condition where upstream water level is not influenced by conditions that exist downstream. See also *Modular flow, Critical flow, Subcritical flow*
Surcharged flow	See *Pressure flow*
Surf zone	The zone of wave action extending from the water-line (which varies with tide, surge, set-up etc) out to the most seaward point of the zone (breaker zone) at which waves approaching the coastline start breaking, typically in water depths of between 5 m and 10 m.
Surge	Changes in water level because of meteorological forcing (wind, high or low barometric pressure) causing a difference between the recorded water level and that predicted using harmonic analysis, and may be positive or negative. See also *Storm surge, Tidal range, Still water level, Residual water level*
Suspended load	The material moving in suspension in a fluid, kept up by the upward components of turbulent currents or by colloidal suspension. See also *Bed load, Sediment load, Total load*
Sustainability (sustainable development)	The concept of development that meets the needs of the present without compromising the ability to meet future needs.
Swamp	An area of low-lying wet or seasonally flooded land, often having trees and dense shrubs or thickets.
Swash zone	The zone of wave action on the beach, which moves as water levels vary extending from the limit of run-down to the limit of run-up. See also *Run-up, run-down*
Swell (waves)	Wind-generated waves that have travelled out of their generating area. Swell characteristically exhibits a more regular and longer period and has flatter crests than waves within their fetch. See also *Wind sea*
System	Assembly of elements, and the interconnections between them, constituting a whole and generally characterised by its behaviour (eg elements in a structure, or assets in an asset system).
Tailwater level	The water level downstream of a weir or other water regulating structure.

Tension crack	Crack appearing at the surface of a soil mass, often adjacent to a retaining wall or top of a failing slope.
Threshold of motion	The point at which the forces imposed on a sediment particle overcome its inertia and it starts to move.
Tidal cycle	Elapsed time between successive high and low waters.
Tidal range	Vertical difference in high and low water level once decoupled from the water level residuals. See also *Residual water level*
Tidal window	The window of time within a tidal cycle that permits construction or other work.
Tide	Water movements that essentially are generated by the global response of oceans to astronomic effects. On the continental shelves and in coastal waters, particularly bays and estuaries, the effect is amplified by shallow water and coastal platforms. See also *Astronomical tide*
Toe	The intersection of the landside or waterside slope of a levee with the ground surface.
Toe blanket	A revetment of resistant material placed to protect the surface of a structure (eg levee, dam, bottom of a spillway, chute) from erosion engendered by falling water, turbulent flow, or other factors. Synonym: Apron
Toe drain	A drain, ditch, or pervious pipe which has been engineered to carry seepage water away from the levee toe to control through seepage or under seepage. See also *Drainage ditch*
Tolerance	Engineering tolerance is the permissible limit of variation in a measured value or physical property of a material, manufactured object, system, or service. The tolerance may be specified as a factor or percentage of the nominal value, a maximum deviation from a nominal value, an explicit range of allowed values, be specified by a note or published standard with this information, or be implied by the numeric accuracy of the nominal value.
Top soil	The surface covering of soil which contains humus and can support vegetation.
Total load	The sum of bed load and suspended load in the river.
Total stress	Usually refers to the vertical stress, which at any point is the weight of everything above that point per unit area.
Transition points/ lines/surfaces	Locations along a flood protection system where there is a change in material (ie soil to concrete) or a change in type of structure (ie levee to gate or railroad crossing).
Trash rack	A structure built on the waterside of a structure, often a culvert, pumping station or weir, to prevent material entering the structure and causing blockages. Synonym: Trash screen See also *Boulder trap*
Trough	The part of a wave with the least magnitude; the lowest part of a wave. Antonym: Peak
Tsunami	Water waves caused by the displacement of a large body of water (ocean or large lake) with wavelengths in the order of minutes rather than seconds.
T-wall	A cantilever reinforced concrete wall consisting of a vertical concrete stem and flat or sloped base slab that form an inverted 'T'. The structural members are fully reinforced to resist applied moments and shears. See also *Retaining wall*

Two/three-dimensional (2D or 3D) model	A mathematical model in which the parameters vary in two or three dimensions. See also *Quasi three dimensional (2D or 3D) model, Coastal area model (2D and 3D)*
Ultimate bearing capacity	The bearing stress which would cause shear failure in the soil below a foundation; dependent upon the shear strength of the soil, applied loads and on the shape and depth of the foundation.
Uncertainty	Lack of sureness about someone or something ranging from almost complete sureness to almost complete lack of conviction about an outcome. Caused by (a) natural variability (inherent uncertainty) or (b) incomplete knowledge (epistemic uncertainty).
Underlayer	Granular or armourstone layer beneath an armour layer that serves either as a filter or to provide a consistent elevation.
Undrained shear strength	The shear strength of a saturated soil at a given water content (or voids ratio, or specific volume) under loading conditions where no drainage of pore water can take place.
Uniform flow	Flow with water surface slope parallel to the bed slope and constant depth from section to section. See also *Normal flow*
Unit weight	The ratio of the total weight of soil to the total volume of a unit of soil.
Upgrading	Improved performance against a particular criterion.
Uplift	1 Upward pressure in the pores of a material (interstitial pressure) or on the base of a structure. 2 The situation in which pore water pressure within a confined or semi-confined aquifer can exceed the total weight of the overlying soil or structure and lead to a failure caused by upward movement.
Up-rush	1 The landside return of water following the back-rush of a wave. 2 The flow of water up or down (down-rush) the face of a structure following wave breaking. See also *Run-up, run-down*
Upstream	In the direction opposite to the flow of a stream.
Velocity head	Kinetic energy of flowing water, represented as the vertical height to which water would rise in a pitot tube. See also *Energy head*
Vertical stress	The total or effective stress acting vertically in a soil mass at a given depth caused by the soil's own weight and possible surcharge and overlying weight.
Visual inspection	A visual inspection of a flood defence asset to assess its condition in line with a fixed risk-based programme. The result of this inspection is used to report both externally and internally on the condition of the asset.
Void ratio	The ratio of the volume of voids to the volume of solids (soil grains).
Vulnerability	The susceptibility of people and assets in the leveed area to physical or emotional injury or damage during an event. See also *Damage potential, Stake*
Water content	The ratio between the mass of water and the mass of soil solids.
Watercourse	All rivers, streams, burns, ditches, drains, cuts, culverts, dikes, sluices, sewers and passages carrying or designed to carry water, excluding pipes or other works for the sole purpose of supplying water to a premise.
Water level	Elevation of still water level relative to a datum. See also *Still water level*
Waterside	Refers to the side of the flood protection structure towards the water. Antonym: Landside

Water table	The surface where the water pressure head is equal to the atmospheric pressure. See also *Piezometric surface*
Waterway	A navigable channel.
Wave breaking	Reduction in wave energy and height in the surf zone due to limited water depth.
Wave climate	The seasonal and annual distribution of wave height, period and direction.
Wave directional spectrum	Distribution of wave energy as a function of wave frequency and direction.
Wave field	Values of wave height, period and direction defined over a specified area at a given time.
Wave frequency	The inverse of wave period.
Wave generation	Growth of wave energy by wind.
Wave height	The vertical distance between a crest and the preceding trough.
Wavelength	The horizontal distance between two successive crests or troughs in a wave record.
Wave period	The time for a wave crest to traverse a distance equal to one wavelength.
Wave rose	Diagram showing the long-term distribution of wave height and direction.
Wave set-up	Superelevation of the water surface over the normal surge elevation attributable to onshore mass transport of the water by wave action alone.
Wave spectrum	A function that describes the distribution of wave energy over wave frequency.
Wave steepness	The ratio of wave height to wavelength.
Wear	The erosion of material from a solid surface by the action of another substance or surface or process. This superficial degradation may be induced by weathering or attrition.
Weathering	Physical, chemical and biological action that leads to deterioration in strength of the rock mass or deterioration in strength of the pieces of produced armourstone. See also *Degradation, External erosion, Internal erosion*
Weir	Low dam that is built across a river to raise the water level. divert the water, or control its flow See also *Hydraulic control structure, Spillway, Sill*
Whole life cycle	The total working life of an asset including planning, design, construction, use, operation, inspection, maintenance and refurbishment, replacement or decommission. See also *Life cycle cost*
Wind field	Values of wind speed and direction defined over a specified area at a given time.
Wind rose	Diagram showing the long-term distribution of wind speed and direction.
Wind sea	Wave conditions directly attributable to recent/local winds, as opposed to swell. Antonym: Swell (waves)
Wind set-up	Elevation of the water level over an area directly caused by wind stress on the water surface.
Wind stress	The way in which wind transfers energy to the sea surface.
Winnowing	The process of separating fine sediment from coarser sediment by fluid flow.
Works	The end products of construction as a whole.
Yield point	The point at which the soil loading behaviour changes from elastic to inelastic.

Yield stress	The stress at which yielding takes place in soils. The stress at which the swelling-recompression line joins the normal compression line.
Zero air voids curve	The curve created by plotting dry densities of soils corresponding to saturation versus water content.
Zone	Part of the levee cross-section consisting of the same soil.
Zoned levee	Levee with different soil material over the cross-section.

Abbreviations

ACM	Articulated concrete mattresses
AD	Alpine Department
ADSC	Analogue to digital signal converter
ASCE	American Society of Civil Engineers
ASTER	Advanced Spaceborne Thermal Emission And Reflection Radiometer
ASTM	American Society for Testing and Materials
AWWA	American Water Works Association
BS	British Standards
CBA	Cost-benefit analysis
CETMEF	Centre d'études techniques maritimes et fluviales.
CH	High plastic clays
CL	Lean clay
CMP	Corrugated metal pipe
CRR	Cyclic resistance ration
CSM	Conceptual site model
CSR	Cyclic stress ratio
DGPS	Differential global positioning system
DT	Destructive testing
DTM	Digital terrain model
EA	Environment Agency
EAD	Expected annual damage
EIA	Environmental impact assessment
EMS	Environmental Management System
EOC	Emergency operations centre
EST	Equilibrium sediment transport
EWS	Early warning system
FEM	Finite element method
FEMA	Federal Emergeny Management Agency
FRMS	Flood risk management structure
GEOTECH	Geotechnical
GFR	Glass-fibre reinforced
GIS	Geographical information system
GPS	Global positioning system
HAT	Highest astronomical tide
HDPE	High density polyethylene
HSE	Health and Safety Executive
ICE	Institute of Civil Engineers
ICOLD	International Commission on Large Dams
ILH	International Levee Handbook
IRSTEA	Institut national de recherche en sciences et technologies pour l'environnement et l'agriculture

ISO	International Organization for Standardization
LAT	Lowest astronomical tide
LiDAR	Light detection and ranging
LMS	Levee management system
LSAC	Levee safety action classification
LSM	Life safety model
MCA	Multi-criteria analysis
MEMS	Mechanical-Electro-Mechanical
MHHW	Mean higher high water
ML–CL	Low compressible silt with some low compressible clay
MLLW	Mean lower low water
MSL	Mean sea level
MWL	Mean water level
OHSAS	Occupational Health and Safety Management System
O&M	Operation and maintenance
PDCA	Plan-do-check-act
PHMSA	Pipeline and Hazardous Materials Safety Administration
PIANC	Permanent International Association of Navigation Congresses (now called International Navigation Association)
POC	Point of contact
POT	Peak over threshold
PVC	Polyvnyl chloride
QMS	Quality management system
SAA	ShapeAccelArray
SAR	Synthetic Aperture Radar
SBCA	Societal benefit cost analysis
SLS	Service limit state – related to the failure of levees
SPR	Source-pathway-receptor
SSI	Soil-structure interaction
STOWA	Foundation for Applied Water Research
SWL	Still water level
SYMADREM	Syndicat mixte interrégional d'aménagement des digues du Rhône et de la mer
TAW	Technical Advisory committee on Water defence
ULS	Ultimate limit state – related to the failure of levees
USACE	US Army Corps of Engineers

Notation

$\overline{\eta}$	Still water line mean water surface elevation or wave set-up	(m)
$\overline{\eta}_{max}$	Maximum mean wave set-up	(m)
$\overline{\eta}_w$	Wind set-up	(m)
$\overline{\eta}_b$	Wave set-up	(m)
A	Catchment area	(m^2)
A	Cross-sectional flow area, or sub-section of cross-sectional flow area if subscripted.	(m^2)
a	Coriolis coefficient	(-)
a	Slope angle of structure (coastal revetment or breakwater)	(°)
A_c	Cross-sectional area of waterway	(m^2)
A_m	Submerged cross section of vessel	(m^2)
A_n	Sub-element of cross-sectional area of flow in channel. Suscript n denotes sub-element number within cross-section divided into n elements	(m^2)
B	Channel width	(m)
b	Width of vertical slice (in slope stability calculation)	(m)
b	Weir width in direction of flow	(m)
b_i	Width of vertical slice i (in slope stability calculation)	(m)
B_J	Width of overtopping jet at impact with armour protection	(m)
C	Chezy coefficient	($m^{1/2}/s$)
C	Expansion or contraction coefficient	(-)
C	Weir flow coefficient	(-)
C	Wave velocity	(m/s)
c	Propagation celerity of waves	(m/s)
c'	Effective cohesion of soil	(kN/m^2)
C_0	Weir discharge coefficient, function of weir shape	(-)
C_c	Compression index	(-)
C_d	Discharge coefficient	(-)
c'_d	Design value of effective cohesion of soil	(kN/m^2)
c_g or C_g	Group velocity	(m/s)
c_h	Coefficient of horizontal consolidation	(m^2/s)
c'_k	Characteristic value of effective cohesion of soil	(kN/m^2)
C_r	Coefficient of wave reflection	(-)
CR	Compression ratio	(%)
C_u	Undrained shear strength of soil	(kN/m^2)
C_U	Coefficient of uniformity	(-)
c_u	Undrained cohesion	(kN/m^2)
C_{ud}	Design value of undrained shear strength of soil	(kN/m^2)
C_{uk}	Characteristic value of undrained shear strength of soil	(kN/m^2)
c_{ur}	Undrained residual cohesion	(kN/m^2)
c_v	Coefficient of vertical consolidation	(cm^2/s)

C_α	Coefficient of secondary compression	(cm^2/s)
$C_{\alpha\varepsilon}$	Modified secondary compression index	(-)
d	Structure (crest) height relative to bed level (breakwaters, dams)	(m)
D_{10}	Effective grain size for 10 per cent passing	(m)
D_{50}	Sieve diameter, diameter of stone that exceeds the 50 per cent value of sieve curve	(m)
D_{60}	Effective grain size for 60 per cent passing	(m)
D_{90}	Grains size not exceeded by 90 per cent value of mass of the sieve curve	(m)
D_b	Average bedform height	(m)
d_b	Wave break point water depth	(m)
d_c	Critical depth of flow	(m)
d_n	Normal depth of flow	(m)
E	Total energy of flow at a cross-section	(m)
e	Voids ratio	(-)
E_d	Design value of the effect of actions	Unit of the parameter
e_o	Initial voids ratio	(-)
e_p	Voids ratio at the end of primary consolidation	(-)
E_u	Undrained elastic modulus	(kN/m^2)
F	Factor of safety (geotechnical), defined as ultimate resistance/required resistance	(-)
f	Laceys silt factor	(-)
F_J	Total force of overtopping jet on scour protection per unit length of wall	(N/m)
f_p	'Peak' frequency of wave spectrum	(1/s)
Fr	Froude number, $Fr = U/(gh)^{1/2}$	(-)
g	(Submerged weight)	(m/s^2)
$G'_{stb;d}$	Design value of the stabilising permanent vertical actions for heave verification	Unit of the parameter
G_s	Specific gravity/particle density	(-)
H	Wave height, from trough to crest	(m)
H	Water level upstream of lateral diversion weir or sill; head differential between upstream and downstream water levels at a weir	(m)
H	Energy grade line elevation	(m)
h	Water depth; height of floodwall; depth of water above lateral weir crest	(m)
H^*	Equivalent head above lateral diversion weir crest	(m)
h_1	Height of average wave surge level above or below floodwall crest; height of water level above floodwall crest	(m) + or -
$H_{1/3}$	Significant wave height based on time domain analysis, average of highest 1/3 of all wave heights	(m)
H_a	Velocity head on approach to weir	(m)
H_b	Wave height	(m)
h_D	Hydraulic depth of flow in river	(m)
H_E	Mean energy wave height	(m)
h_f	Energy loss term between two cross-section locations	(m)
H_i	Incident wave height	(m)
H_{m0}	Significant wave height calculated from the spectrum, $H_{m0} = 4\sqrt{m_0}$	(m)
H_o	Offshore or deep water wave height	(m)
h_p	Water depth perpendicular to river bottom	(m)

H_r	Reflected wave height	(m)
H_{rms}	Root mean square wave height	(m)
H_s	Significant wave height, Breaking wave height	(m)
h_s	Water depth at a distance of l/2L or 5H_{max} seaward of structure toe	(m)
H_{so} or H'_{os}	Deep water significant wave height	(m)
h_t	Depth of tailwater; Water depth downstream of weir discharge	(m)
h_w	Height of wave crest above toe of floodwall	(m)
i	Hydraulic gradient of (phreatic) water level	(-)
i_b	Gradient of river bed	(-)
I_l	Liquidity index	(%)
I_p	Plasticity index	(-%)
I_s	Channel sinuosity (L_s/L_v)	(m/m)
K	Conveyance	(m^3/s)
k_h	Coefficient of horizontal permeability	(m/s)
k_s	Bed roughness, hydraulic roughness	(m)
k_{sg}	Grain roughness	(m)
k_{sD}	Bedform roughness	(m)
L	Wave length, in the direction of propagation	(m)
L	Length of lateral diversion weir crest	(m)
L	Length along channel between two cross-sections, weighted reach length	(m)
L	Length of levee ring	(m)
l	Length of slip failure	(m)
L_b	Average bedform length	(m)
L_{ch}	Length between adjacent cross-sections for channel	(m)
L_{jump}	Length of hydraulic jump	(m)
L_{lob}	Length between adjacent cross-sections for left overbank	(m)
L_o	Offshore or deep water wave length, $L_o = gT^2/2\pi$	(m)
L_{rob}	Length between adjacent cross-sections for right overbank	(m)
L_s	Ship length	(m)
L_s	Channel length	(m)
L_v	Valley length	(m)
M	Total soil mass	Mg
m_0	Zeroth moment of wave spectrum	(m^2s)
m_5	Coefficient for degree of sinuosity in Cowan's method	(-)
MDD	Maximum dry density	(Mg/m^3)
M_{ed}	The design overturning moment (in slope stability analysis)	Unit of the parameter
M_{Rd}	The design restoring moment (in slope stability analysis)	Unit of the parameter
m_v	Coefficient of volume compressibility	(m^2/N)
n	Manning's coefficient of bed roughness	(s/$m^{1/3}$)
N	Standard penetration test blow count	blows/ 0.3048 m
n_{irr}	Irregularity component of unit roughness for Manning's coefficient	(s/$m^{1/3}$)

n_t	Unit roughness for Manning's coefficient, comprised of three components	($s/m^{1/3}$)
n_{sur}	Surface material component of unit roughness for Manning's coefficient	($s/m^{1/3}$)
n_{veg}	Vegetation component of unit roughness for Manning's coefficient	($s/m^{1/3}$)
n_x	Component of Manning's coefficient for Cowan's method	($s/m^{1/3}$)
OMC	Optimim moisture content	(%)
P	Wetted perimeter, height of lateral weir crest above toe of levee	(m)
p	Probability	(%)
p'	Effective pressure	(kN/m^2)
$P_{f,loc,req}$	Local probability of macro-instability	(1/year)
$P_{f\|inst}$	Probability of breaching as a result of slope instability of the inner slope	(1/year)
p'_o	*In situ* effective pressure	(kN/m^2)
Q	Imposed vertical surface load per metre run (in slope stability calculation)	(kN/m)
q	Specific discharge; unit discharge per meter length of weir crest	($m^3/s/m$)
Q'	Spatially varied discharge over lateral weir	(m^3/s)
Q , Q_w	Water discharge	(m^3/s)
Q_{am}	Stream discharge upstream of lateral diversion	(m^3/s)
Q_{av}	Stream discharge downstream of lateral diversion	(m^3/s)
Q_{ch}	Water discharge within channel only	(m^3/s)
Q_d	Design value of imposed vertical surface load per metre run (in slope stability calculation)	(kN/m)
Q_i	Imposed vertical surface load on slice i per metre run (in slope stability calculation)	(kN/m)
Q_k	Characteristic value of imposed vertical surface load per metre run (in slope stability calculation)	(kN/m)
Q_{lat}	Total discharge over lateral weir	(m^3/s)
Q_{lob}	Water discharge within left overbank of cross-section	(m^3/s)
Q_{rob}	Water discharge with right overban of cross-section	(m^3/s)
Q_s	Sediment discharge	(m^3/s or T/ day)
R	Hydraulic radius	(m)
r	Relative intensity of turbulence	(-)
r	Weir crest radius, ogee crest	(m)
r, r_c	Centre-line radius of river bend	(m)
R_c	Crest freeboard, level of crest relative to still water level	(m)
R_d	Run-down level, relative to still water level	(m)
R_d	Design value of the resistance to an action	Unit of the parameter
R_u	Run-up level, relative to still water level	(m)
$R_{u2\%}$	Run-up level exceed by only two per cent of run-up tongues	(m)
$R_{up\%}$	Run-up level exceed by only p of run-up tongues; p is a probability of occurrence in (%)	(m)
S	Energy slope, slope of energy gradeline	(m/m)
s, s_0	Wave steepness, $s = H/L_o$	(-)
S_0	Gradient of river bed	(Radians)
$S_{dst;d}$	Design value of the destabilising seepage force in the ground	Unit of the parameter
S_f	Friction slope in open channels	(m/m)
s_m	Wave steepness for mean period wave, $s_m = 2\pi H_s/(gT_m{}^2)$	(-)

s_{om}	Offshore (deep water) wave steepness for mean period wave, $s_{om} = H_{so}/ L_{om} = 2\pi\ H_{so}/(gT_m{}^2)$	(-)
s_{op}	Offshore (deep water) wave steepness for peak period wave, $s_{op} = H_{so}/L_{op}$ Lop $= 2\pi\ H_{so}/(gT_p{}^2)$	(-)
SWL	Still water level	(m)
T	Wave period	(s)
t_f	Fall time for water particle at wave crest to fall to ground for floodwall overtopping	(s)
T_m	Mean wave period	(s)
T_p	Wave period corresponding to the maximum frequency value	(s)
U	Horizontal depth-mean current velocity	(m/s)
u	Pore pressure	kN/m²
u^*	Shear velocity, $u^* = \sqrt{\tau_b/\rho_w}$	(m/s)
u'	Fluctuating velocity component	(m/s)
U_{10}	Wind speed 10 m above sea surface	(m/s)
u_d	Design value of pore pressure	kN/m²
$u_{dst;d}$	Design value of the destabilising total pore water pressure	Unit of the parameter
u_k	Characteristic value of pore pressure	kN/m²
u_{max}	Maximum velocity at the surface for vertical velocity profile	(m/s)
u_o or u_{bmax}	Maximum wave-induced orbital velocity near the bed	(m/s)
U_z	Wind speed at a height of z(m) above sea surface	(m/s)
v	Average flow velocity	(m/s)
V	Total soil volume	(m³)
Va	Approach velocity at weir	(m/s)
V_J	Jet entry velocity for weir overflow	(m/s)
V_{mid}	Mid surf zone longshore current	(m/s)
V_w	Horizontal wave velocity	(m/s)
W	Stream top width	(m)
W	Self weight of slice per meter run (in slope stability calculation)	(kN/m)
w	Moisture content	(%)
W_d	Design value of self weight of slice per metre run (in slope stability calculation)	(kN/m)
W_i	Self weight of slice *i* per metre run (in slope stability calculation)	(kN/m)
W_k	Characteristic value of self weight of slice per metre run (in slope stability calculation)	(kN/m)
w_l	Liquid limit	(%)
w_p	Plastic limit	(%)
x, y, z	Distances along orthogonal axes	(m)
X_b	Horizontal distance from shoreline to breakpoint	(m)
x_c	Horizontal distance for jet trajectory	(m)
x_L	Horizontal distance for jet trajectory	(m)
X_{Ru}	Horizontal distance of wave runup	(m)
x_U	Horizontal distance for jet trajectory	(m)
y	Flow depth	(m)
y_1	Flow depth at location 1 (2, 3, etc as indicated by subscript number)	(m)
y_{bs}	Depth of flow due to bend scour	(m)
y_{cs}	peak depth of flow due to confluence scour	(m)

y_{me}	Depth of scour due to plunging jet	(m)
y_n	Normal flow depth in open channel	(m)
y_s	Scour depth relative to the original bed	(m)
y_{ws}	Peak depth of flow due to sediment wave migration	(m)
Z	Stage or water level relative to a stream gauge	(m)
z	Level of riverbed compared with reference level, distance above channel bed for vertical flow distribution within flow depth; elevation at which wide speed is measured for fetch limited wave growth	(m)
z_0	Reference level of vertical velocity profile, also called: bed roughness length	(m)
α	Strength parameter correlation	(-)
α	Inclination of the base of a slice to the horizontal (in slope stability analysis)	(°)
α_i	Inclination of the base of slice *i* to the horizontal (in slope stability analysis)	(°)
Δu	Increase in pore water pressure	(kN/m²)
$\Delta\sigma_v$	Increase in total vertical stress	(kN/m²)
Λ_{GEO}	Degree of utilisation of the available design resistances by the design actions or the effects of the design actions	(-)
ρ	Bulk density	(Mg/m³)
ρ_d	Dry density	(Mg/m³)
ρ_s	Density of soil particles	(Mg/m³)
ρ_w	Density of water	(KN/m³)
$\sigma_{stb;d}$	Design value of the stabilising total vertical stress	Unit of the parameter
σ'_{vo}	*In situ* vertical effective stress	(KN/m³)
σ'_y or p'_y	Yield stress or preconsolidation pressure	(kN/m²)
φ'	Effective angle of shearing resistance	(°)
φ'_d	Design value of effective angle of shearing resistance	(°)
φ'_k	Characteristic value of effective angle of shearing resistance	(°)
Φ	The standard normal function	(-)
Ω	Stream power index	(m³/s)
Ω_{lim}	Stream power index thrreshold value	(m³/s)
β	Shore slope	(m/m)
β	Main channel contraction angle	(Radians)
β_{req}	Reliability index	(1/year)
ϕ'	Effective friction angle	(°)
ϕ'_{cv}	Effective critical state or constant volume friction angle.	(°)
ϕ'_p	Effective peak (maximum) friction angle	(°)
ϕ'_r	Effective residual friction angle	(°)
γ_u	Undrained friction angle	(°)
γ	Bazin representative bed roughness	(m $^{0.5}$)
γ	Specific weight of water	(N/m³)
γ_b	Wave breaker depth index	(-)
γ_b, γ_f, γ_b	Factors reflecting influence of berms, slope roughness and wave obliquity, respectively	(-)
γ_{bulk}	Bulk unit weight of soil	kN/m³
γ_c	Partial factor applied to effective cohesion	(-)

γ_d	Partial model factor	(-)
γ_G	Partial factor applied to permanent actions (including self-weight)	(-)
$\gamma_{G:dst}$	Partial factor for a permanent destabilising action	(-)
$\gamma_{G:fav}$	Partial factor for a permanent favourable action	(-)
$\gamma_{G:inf}$	Partial factor for permanent action (including self-weight) in calculating lower design values	(-)
$\gamma_{G:stb}$	Partial factor for a permanent stabilising action	(-)
$\gamma_{G:sup}$	Partial factor for permanent action (including self-weight) in calculating upper design values	(-)
γ_n	Consequence factor	(1/year)
γ_Q	Partial factor applied to variable actions (including applied surface loads)	(-)
$\gamma_{Q:dst}$	Partial factor for a destabilising action causing hydraulic failure	(-)
γ_{Re}	Partial factor applied to earth resistances	(-)
γ_ϕ	Partial factor applied to φ	(-)
κ	Von Karman coefficient	(-)
μ	Vane shear strength correction factor	(-)
ν	Kinematic viscosity	(m^2/s)
θ	Angle between the bottom contour and the wave crest	(°)
θ	Angle of jet trajectory to horizontal plan for floodwall overtopping	(°)
θ_b	Breaking wave angle relative to shore normal	(°)
θ_J	Average angle of jet trajectory	(°)
θ_L	Jet trangle of upper nappe	(°)
θ_m	Mean wave direction calculated form directional wave spectrum	(°)
θ_p	Peak wave direction	(°)
θ_U	Jet angle of lower nappe	(°)
τ	Shear stress	(kN/m^2)
τ_0	Shear stress at the bed	(N/m^2)
ξ	Surf similarity parameter	(-)
$\xi_{g\ safety\ standard}$	Correlation factor for safety standard	(-)
ξ_m	Mean surf similarity parameter	(-)
ξ_p	Peak surf similarity parameter	(-)

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The French Ministry of Ecology (MEDDE) is in charge of the policy for flood management, land planning and industrial and natural risks mitigation. Its services have the duty of controlling the safety of hydraulic works (dams and dikes) in the frame of a regulation that has been significantly enhanced in 2007. Along some main rivers, the Ministry services are also directly in charge of managing hundreds of kilometres of state owned levees.

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