Application of Eurocode 7 to the design of flood embankments

UK/Ireland

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Where we are

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Post	Griffin Court, 15 Long Lane, London, EC1A 9PN, UK
Telephone	+44 (0)20 7549 3300
Fax	+44 (0)20 7549 3349
Email	enquiries@ciria.org
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Application of Eurocode 7 to the design of flood embankments

Principal Authors: Adam Pickles, Rachel Sandham, Arup

Special Advisors: Brian Simpson, Arup, Andrew Bond, Geocentrix

ciria

Griffin Court, 15 Long Lane London, EC1A 9PN
Tel: 020 7549 3300 Fax: 020 7549 3349
Email: enquiries@ciria.org Website: www.ciria.org

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Pickles, A, Sandham, S, Simpson, B, Bond, A

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Summary

This guide provides national guidance for practitioners in the UK and Ireland on the design of flood embankments to EN 1997 *Eurocode 7: Geotechnical design*. The aim is to improve clarity on key issues relating to the design of flood embankments, which are not addressed in detail by the current EN 1997. These include:

- risk categories for flood embankments
- modification of partial factors on the basis of consequence
- distinction between different design situations
- design water levels and pore water pressures
- applicability of ULSs.

The need for this guide was identified by the CIRIA-led UK and Ireland Backing Group for the International Levee Handbook (ILH) project. Where appropriate this guide provides cross references to the extensive guidance included within the ILH.

The guide covers the design of new flood embankments and significant modification of existing embankments in the UK and Ireland. It does not cover assessment of existing embankments or the design of new flood structures, though brief discussion is provided on the importance of detailing the interface between flood embankments and associated structures.

Section 2.3 of the guide presents a risk classification for embankments, which may be used to determine the level of design and construction supervision applied to a scheme, and the associated adjusted partial factors that may be used in design. Differentiation is also made between risk categories and EN 1997 Geotechnical Categories, which can be used to determine the level of ground investigation and analysis undertaken on a scheme.

The guide presents a summary of the typical embankment design process and at each stage identifies the relevant requirements of EN 1997 and provides guidance on how these requirements may be implemented. Particular detail is provided in Section 3 in relation to determining the appropriate design situation for flood embankment and in Section 4 the guide outlines the approach to establishing design values of water levels of water pressures, which are a critical input for flood embankment design. Section 5 outlines the approach to assessing each of the critical ultimate limit states applicable to flood embankments and includes tables of partial factors to be used based on the UK and Irish National Annexes to EN 1997.

During preparation of this guide a number of opportunities for further research development were identified and the guide concludes with recommendations for further work that could be undertaken to improve the design of flood embankments to EN 1997.

This guide reflects thinking at the time of writing (September 2014) from a range of practitioners including members of Evolution Groups working towards the revision of EN 1997 planned for 2020. Subject to feedback from users in the UK and Ireland, this guide is likely to be updated. CIRIA is establishing a Community of Practice through which such feedback will be sought. The guide will, ultimately, be superseded by the publication of the revised EN 1997. Further details can be found on the CIRIA website at **www.ciria.org/ILH**

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Principal authors

Adam Pickles

Adam is a senior geotechnical engineer at Arup with design experience on a wide range of civil and geotechnical engineering projects, both in the UK and overseas. These include high speed rail, highway and railway civil engineering schemes, deep excavations and flood alleviation schemes. Adam was a technical contributor for a UK state of the art retaining wall guidance document, and authored the retaining wall design chapter for the ICE *Manual of Geotechnical Engineering*.

Rachel Sandham

Rachel is an associate director of Arup Water. Her background is in engineering geology and her experience encompasses asset evaluation and risk assessment of a variety of infrastructure, including dams, reservoirs, flood alleviation schemes, aqueducts, rail and highways. She was a member of the National Backing Group of the *International Levee Handbook* (the ILH) and continues to engage in the emerging UK and Ireland Community of Practice.

The authors were supported by a technical working group including:

Mervyn Bramley	Independent engineer
Andrew Bond	Geocentrix
Chris Hartley	Royal HaskoningDHV
Jonathan Simm	HR Wallingford
Philip Smith	Royal HaskoningDHV
Shaun Wersching	CH2MHILL

Project steering group

Following CIRIA's usual practice, the research project was guided by a project steering group (PSG), which included:

Mervyn Bramley (chair)	Independent Engineer
Jackie Banks	Environment Agency
Andrew Bond	Geocentrix
Alan Brown	Stillwater Associates
Michael Collins	Office of Public Works, Ireland
Malcolm Corlett	BAM Nuttall
Brian Doyle	Northern Ireland Rivers Agency
Jaap Flikweert	Royal HaskoningDHV
Chris Hartley	Royal HaskoningDHV
James Morris*	Welsh Government

Mark Morris	Samui
Jonathan Simm	HR Wallingford
Philip Smith*	Royal HaskoningDHV
Andy Tan	Environment Agency
Judith Tracey*	Scottish Government
Shaun Wersching	CH2MHILL

* corresponding members

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- **UK:** Environment Agency/Defra, Scottish Government/SEPA, Department of Agriculture and Rural Development, Northern Ireland, Welsh Government/Natural Resources Wales
- **Ireland:** Office of Public Works.

CIRIA project managers

Owen Jenkins, Louise Clarke

In this guide the following abbreviations have been used to differentiate between in-document references, EN 1997 references and ILH references, so assume:

This document = Section X.X

Clause in EN 1997-1 = EN 1997-1 2.1.1(1)

Clause in UK National Annex to BS EN 1997-1 = NA to BS EN 1997-1 A2.2(1)

Section in the International Levee Handbook = ILH 2.3

The notation used in this guide follows the system listed in EN 1997-1 1.6.

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Abbreviations and acronyms

BIM	Building Information Modelling
CC	Consequences Classes
DA	Design Approach
DMRB	Design Manual for Roads and Bridges
DSL	Design supervision level
EG	Evolution Group
GC	Geotechnical Category
FOS	Factor of safety
GEO	Geotechnical limit states
IL	(Construction) inspection level
ILH	International Levee Handbook
NCCI	Non Contradictory Complementary Information
NDP	Nationally Determined Parameter
PSG	Project steering group
RC	Reliability Classes
SLS	Serviceability limit states
STR	Structural limit states
ULS	Ultimate limit states

1 Introduction

1.1 BACKGROUND – THE INTERNATIONAL LEVEE HANDBOOK

In 2013, the *International Levee Handbook* (the ILH) was published by CIRIA as a result of a joint research project from CIRIA (UK), the French Ministry of Ecology (France) and the USACE (USA). The ILH is an extensive handbook on the safety assessment, management, design and construction of flood embankments, which incorporates all the main elements of good practice.

During the production of the ILH it became apparent that water retaining structures are not covered well by the current version of EN 1997, which deals with geotechnical design and has two parts:

- EN 1997-1 (Part 1): general rules for geotechnical design
- EN 1997-2 (Part 2): requirements for ground investigations and the performance and evaluation of field and laboratory testing.

Some nations have developed Non Contradictory Complementary Information (NCCI) for flood embankments to support the use of Eurocodes, but no such information exists for the UK. Also, EN 1997 includes three different Design Approaches to applying partial factors for structural and geotechnical limits states. None of the countries that have produced NCCI use Design Approach 1 for flood embankments, the approach currently used in the UK and one of the options permitted in Ireland.

The CIRIA-led UK and Ireland Backing Group for the ILH project recognised the need for UK and Ireland national guidance to support EN 1997. This representative industry-wide group subsequently oversaw the production of this guide adopting a consensus-building approach. The Backing Group had representation from designers, contractors, research organisations and the flood risk management authorities in the UK and Ireland.

1.2 OBJECTIVES OF THIS GUIDE

This guide provides national guidance for practitioners in the UK and Ireland on the design of flood embankments to EN 1997. This is an interim guide, which will eventually be superseded by the next revision of EN 1997 (due to be published in 2020). It takes into consideration thinking from relevant EN 1997 Evolution Groups (EG) at the time of writing.

The aim of this guide is to improve clarity on key issues relating to the design of flood embankments, which are not addressed in detail by the current EN 1997. These are:

- risk categories for flood embankments (Section 2.3)
- modification of partial factors on the basis of consequence (Section 2.3)
- applicability of ultimate limit states (ULS) (Section 2.5 and Section 5)
- distinction between different design situations (Section 3)
- design pore water pressures in marine or fluvial environments (Section 4).

In addition, the guide will indicate where information can be found on other issues, including:

- site characterisation
- verification of serviceability limit states (SLS)
- design of structures (flood embankments) resistant to hydraulic loading
- transient and steady state seepage analysis
- design of crest structures and transitions in accordance with EN 1997.

1.3 APPLICABILITY OF THIS GUIDE

The guide covers the design of new flood embankments and significant modification of existing embankments in the UK and Ireland. It does not cover the assessment of existing embankments or the design of new flood walls, pumping stations, gates, closure structures, natural features or other associated structures. The interfaces between flood embankments and associated structures are, however, an important consideration in the design process and are discussed briefly in Section 5. The approach to inspection and assessment of existing embankments is covered in detail in ILH 5.

This guide is intended to be read in conjunction with the ILH, which provides further information on many of the topics covered, and the relevant Eurocodes.

This guide does not address any issues in relation to seismic design of flood embankments.

In this guide 'design' refers primarily to the embankment cross-section, the materials used to form the embankment and foundation soils, as these are the aspects specifically related to EN 1997. For guidance on other aspects of design, such as embankment alignment, crest level and overflow structures, refer to the ILH.

This guide reflects thinking at the time of writing (September 2014) from a range of practitioners including members of Evolution Groups (EG) working towards the revision of EN 1997 planned for 2020. Following feedback from users in the UK and Ireland, it is likely that this guide will be updated. CIRIA is establishing a Community of Practice through which such feedback will be sought. The guide will, ultimately, be superseded by the publication of the revised EN 1997. Further details can be found on the CIRIA website at: **www.ciria.org/ILH**.

1.4 DEFINITIONS

Eurocode terminology is explained in Section 2.

A glossary is provided in the ILH and should be read in conjunction with this guide. Where necessary, further clarification is provided here to facilitate applicability in the UK and Ireland context (eg design flood, extreme flood).

Note that the term 'levee' can be used interchangeably with 'flood embankment' or 'flood defence embankment'.

2 Embankment design process

2.1 INTRODUCTION

The overall process for designing a flood defence comprises multiple stages, starting with identifying the need for an embankment and going through to operation and maintenance. ILH Figure 9.3 presents a flow chart of this process and ILH Figure 9.4 lists the typical activities that may be undertaken at each design stage. EN 1997-1 and this guide are focused on the engineering design of the embankment structure and foundation soils to ensure stability and acceptable long-term performance. This is represented in Stage 3 (detailed design) of ILH Figure 9.4.

Table 2.1, based on ILH Table 9.1, identifies typical engineering considerations for embankment design, along with references to the relevant sections of the ILH where these issues are discussed. Although important, not all these aspects are covered by the requirements of EN 1997-1 so a description of EN 1997-1 coverage, and the applicability of this guide for each design consideration is given in Table 2.1.

Table 2.1 Engineering considerations for embankment design (adapted from ILH Table 9.1)

Design consideration	Relevant ILH section	Relevant EN 1997-1 sections and clauses	Design consideration in relation to EN 1997
Embankment alignment	9.4	–	Determination of embankment alignment is not addressed by EN 1997 and is therefore outside the scope of this guide.
Crest level	9.5	–	Determination of crest level is based on the asset owner/manager requirements and the hydrogeological modelling. It is not addressed by EN 1997 and is therefore outside the scope of this guide.
Cross-section	9.5	10, 11, 12	The cross-section geometry will determine the overall stability and performance of the embankment, and this includes both the proposed embankment and the morphology of the existing foreshore or river channel. A range of factors will influence the possible embankment cross-section, as outlined in ILH 9.5. Once a preferred geometry has been established the EN 1997 design process outlined in this guide is used to verify that the embankment performance will be acceptable for each design situation.
Ground conditions	7.1, 7.7, 7.9	3	Ground conditions, ground water levels, topography and design parameters are all inputs that need to be selected for the EN 1997 design process. These feed into establishing the design situations outlined in Section 3 and the subsequent stability and settlement analyses discussed in Sections 5 and 6.
Materials	7.7.3.5, 9.13	2.4.5	Establishing the properties of natural ground and fill material previously placed on site, or selecting the required properties for any imported material are part of the EN 1997 design process. Details on site characterisation are given in Section 2.4.
Durability and serviceability	7.2.3, 7.9.9, 9.12	2.4.8, 11.6, 12.6	Verifying the long-term serviceability of an embankment is a requirement of EN 1997. Specific details of how to undertake such analyses are not included in EN 1997 and this guide has identified it as an area for further consideration. Depending on

Design consideration	Relevant ILH section	Relevant EN 1997-1 sections and clauses	Design consideration in relation to EN 1997
			the classification of the flood defence embankment, the designer may consider it necessary to include a requirement for long-term instrumentation to monitor and verify serviceability, as discussed in Section 6.
Transitions	9.11	–	Transitions are crucial aspects of embankment design. Typically it is the detailing of the transitions that is the cause of problems, rather than the general arrangement. Although the importance of transitions is highlighted in this guide, they are not specifically addressed in EN 1997 and are therefore not covered in detail, but references to other guidance are given in Section 5.5.
Human impacts	7.6	11.3	Human effects can influence the stability and performance of an embankment. Designs to EN 1997 seek to identify those that are reasonably foreseeable and verify that the design can accommodate them. For unforeseen events the aim is to design the embankment with a reasonable margin of safety and overall robustness to prevent failure. Accidental design situations are defined in Section 2.2.1.
Reliability of existing embankments	5	–	EN 1997 is primarily concerned with the design of new structures, though the approach in EN 1997 and the recommendations of this guide are also considered appropriate for significant modification of existing embankments. EN 1997 is not applicable to the assessment of existing embankments so assessment is outside the scope of this guide. Reference should be made to ILH 5 for guidance.
Construction	7.7.4, 7.9.9, 9.16, 10	4	Consideration of how an embankment will be constructed is an important design aspect. EN 1997 requires clear communication of design assumptions between the designer and the constructor. In addition, for flood embankments the construction phase can be critical for overall stability. Any limitations that are needed to maintain stability during construction must be clearly communicated along with the need for pre-construction trials or long-term monitoring and instrumentation (if required).

The following flow chart has been developed from Stage 3 of ILH Figure 9.4 to identify the process for designing embankments in accordance with EN 1997 and provides references to where the process is discussed within this guide.

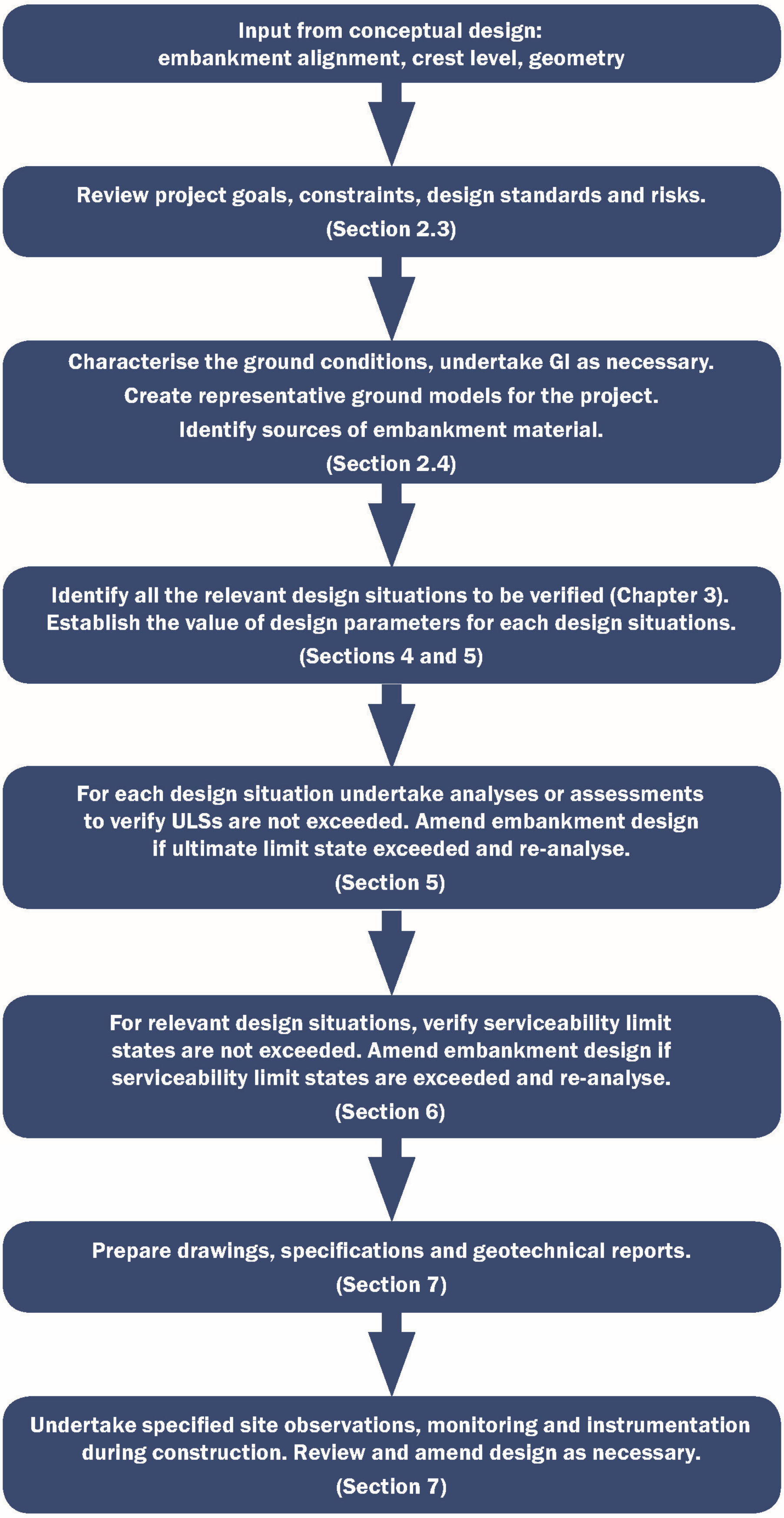


Figure 2.1 Flow chart of EN 1997 design process (developed from Stage 3 of ILH Figure 9.4)

2.2 OVERVIEW OF EUROCODE BASIS OF DESIGN

The following documents in the Eurocode series are of particular relevance to flood embankment design:

- EN 1990: *Eurocode – Basis of structural design*
- EN 1997-1: *Eurocode 7: Geotechnical design – Part 1: General rules*
- NA to BS EN 1997-1: *UK National Annex to Eurocode 7: Geotechnical design – Part 1: General rules*
- NA to IS EN 1997-1: *Irish National Annex to Eurocode 7: Geotechnical design – Part 1: General rules*
- EN 1997-2: *Eurocode 7: Geotechnical design – Part 2: Ground investigation and testing*
- NA to BS EN 1997-2: *UK National Annex to Eurocode 7 – Geotechnical design – Part 2: Ground investigation and testing*.

Note that a National Annex has not been published for IS EN 1997-2.

Country specific requirements and Nationally Determined Parameters (NDPs) are provided in the National Annexes. This guide relates to the UK and Ireland, so the appropriate National Annex must be used depending upon where the project is located. The designer must always consult both the main standard and the relevant National Annex when undertaking design in accordance with EN 1997.

When designing a flood embankment, the aim is to determine an embankment geometry that is stable, provides the required standard of protection against flooding and, if required, is resilient to withstand overtopping. In order to achieve these objectives the designer should consider the various situations that could occur during the lifetime of the structure and verify that there is a sufficient margin of safety against failure occurring for each situation. To meet this requirement the designer checks that the design resistances are greater than the design actions for each situation.

The term 'resistance' is defined in EN 1990 1.5.2.15 to mean the *"capacity of a member or component, or a cross-section of a member or component of a structure, to withstand actions without mechanical failure"*. For example, in the context of embankments, this could refer to sliding resistance on a slip surface.

'Action' is defined in EN 1990 1.5.3.1 as *"(a) Set of forces (loads) applied to the structure (direct action); (b) Set of imposed deformations or accelerations caused for example, by temperature changes, moisture variation, uneven settlement or earthquakes (indirect action)"*. Examples of actions for flood embankments include the mass of the embankment, traffic loads, external water pressures and pore water pressures. Actions are divided into two categories – permanent and variable, which in EN 1990 1.5.3, are as follows:

- **permanent action (G)** that is likely to act throughout a given reference period and for which the variation in magnitude with time is negligible, or the variation is always in the same direction (monotonic) until the action attains a certain limit value
- **variable action (Q)** whereby the variation in magnitude with time is neither negligible nor monotonic

This generic process is applicable to all geotechnical structures and is outlined in EN 1997. The following sections introduce these principles and concepts, and relate them specifically to flood embankments.

2.2.1 Concept of design situations

Design situations are particular sets of circumstances that the design should accommodate in order to fulfil its function. EN 1997-1 2.2 identifies a range of typical considerations that may need to be addressed when determining the relevant design situations for geotechnical structures. These include variability of the ground, variability of the structure geometry and variations in actions (such as imposed loads or water levels). Proximity to adjacent structures or infrastructure is also considered.

Design situations are classified as one of four types in EN 1990, based on the duration and likelihood of the situation occurring:

- **persistent:** refers to conditions of normal use (ie water level within normal operational range)
- **transient:** temporary conditions (such as during construction or during repair or the design flood event)
- **accidental:** exceptional conditions (such as extreme high water levels resulting from a failure of part of the system, eg blockage of an outlet, failure of an embankment elsewhere)
- **seismic:** applicable during a seismic event (not addressed in this guide).

The Eurocodes require that the assumptions for creating design situations should be varied and sufficiently severe to address circumstances that can reasonably be expected during the lifetime of the structure. The current proposals from Evolution Group 9 (EG9) of the Eurocode committee is that accidental situations relate to non-natural situations such as loading resulting from failure of another part of the system as previously identified. Extreme natural events are not considered accidental situations.

A detailed presentation of design situations relevant to flood embankment design is included in Section 3.

2.2.2 Principles of limit states

Limit states are states beyond which the behaviour of the structure would be unacceptable. For each design situation, EN 1997 requires that the designer verifies limit states are not exceeded. Limit states are divided into two types:

- ultimate limit states (ULS) (see Section 2.2.2.1)
- serviceability limit states (SLS) (see Section 2.2.2.2).

Verification that limit states will not be exceeded may be achieved through calculation, prescriptive measures, load tests and experimental models, or the Observational Method. In this guide it is generally assumed that verification of limits states will be undertaken by calculation.

2.2.2.1 Ultimate limit states

Ultimate limit state (ULS) failures of a structure will often threaten the safety of people and property. For flood embankments, mass instability through slope failure or translational sliding, loss of stability through hydraulic heave and soil erosion (external or internal) are all examples of ULS failures. Examples of instability mechanisms are illustrated in ILH 3.5.2.3, and ILH Tables 7.44 and 7.45.

EN 1997-1 2.4.7 identifies five types of ULS. For different types of structure the code then provides examples of the types of mechanisms that may need to be verified to ensure these limits states are not exceeded. Table 2.2 summarises the five limit states and gives example mechanisms applicable to flood embankment design.

Table 2.2 ULS for flood embankments

ULS type	Description	Flood embankment mechanism
EQU	Loss of equilibrium of the structure or the ground, considered as a rigid body, in which the strengths of structural materials and the ground are insignificant in providing resistance.	Not applicable to flood embankments. Ground strength is always significant in providing resistance.
STR	Internal failure or excessive deformation of the structure or structural elements including footings, piles or basement walls, in which the strength of structural materials is significant in	Not applicable to flood embankments. This guide does not cover structural flood defences such as sheet pile walls. A brief discussion of associated structures is given

ULS type	Description	Flood embankment mechanism
	providing resistance.	in Section 5.5 and in ILH 9.15.
GEO	Failure or excessive deformation of the ground, in which the strength of soil or rock is significant in providing resistance.	Applicable to flood embankments. Involves mass instability of slopes along slip planes and block translation. See EN 1997-1 Sections 11 and 12, and ILH 9.9 and ILH 9.10.
UPL	Loss of equilibrium of the structure or the ground due to uplift by water pressure (buoyancy) or other vertical actions.	Applicable to flood embankments. Uplift forces from ground water exceed overburden pressure. See ILH Box 9.30 and ILH Figure 3.191.
HYD	Hydraulic heave, internal erosion and piping in the ground caused by hydraulic gradients.	Applicable to flood embankments. Sand 'boiling', internal erosion. See ILH 9.10.3.4 and ILH Figure 3.190

The calculation methods for verifying these limits states are presented in Section 5.

2.2.2.2 Serviceability limit states

Serviceability limit states relate to the ongoing performance of a structure, the comfort of users and the appearance of the works. Typically SLS refers to deformations, vibrations or damage that may affect durability. For a flood embankment this may mean excessive settlement reducing the level of flood protection provided, differential settlement leading to damage at transitions between different design solutions (such as between earthworks and structures), desiccation that could reduce resilience to overtopping flow or excessive seepage affecting the amenity on the landward side.

It should be appreciated that the failure of a flood embankment to meet the SLS performance requirements may lead to a ULS failure during a flood event. Section 6 outlines the requirements for assessing SLSs.

2.2.3 Design values and partial factors

In order to verify that limit states are not exceeded, calculations are usually undertaken to demonstrate that design values of the effects of actions do not exceed design resistances or design values of serviceability criteria. Actions may include the mass of the ground, earth pressures, external water pressures, pore water pressures, imposed traffic or impact loading. For flood embankments, resistance is typically provided by the strength of the material forming the embankment and the foundation soils. In some situations structural elements (such as sheet piles) may be introduced to enhance stability but detailed consideration of such structural elements is outside the scope of this guide.

Within the Eurocode framework, 'design' values of actions, material properties or resistances have a specific definition, which means that an appropriate margin of safety or partial factor has been applied. These margins or factors are normally applied to 'representative' values of actions, 'characteristic' values of material properties, and 'nominal' values of dimensions. EN 1990 1.5 provides a summary of the main definitions used in the Eurocode documents. Issues to be considered when deriving characteristic values of material parameters are presented in ILH 7.8.4.

In many circumstances the design value of a parameter is determined from its characteristic value through application of a partial factor. The following factors are used in EN 1997-1:

- factor on material strength, γ_M
- factor on actions (or effect of actions), γ_F
- factor on resistance, γ_R

The numerical value of these factors depends on the design situation, the limit state under consideration and, for structural (STR) and geotechnical (GEO) limit states, the Design Approach. Three different Design Approaches are set out in EN 1997-1 for STR and GEO limit states and further details on the Design Approaches are given in EN 1997-1 Annex B. The selection of which Design Approach to use is a national choice and country specific requirements can be found in the appropriate National Annex.

The numerical values of partial factors are NDPs and the values to be used in the UK (which adopts Design Approach 1) are given in the UK National Annex to EN 1997-1. Partial factors for use in Ireland are included in the Irish National Annex. All three Design Approaches are permitted in the Irish National Annex, but this guide relates only to Design Approach 1. The values of partial factors for Design Approach 1 are the same in the UK and Irish National Annexes. Selection and application of partial factors in flood embankment design is addressed in detail in later sections of this guide.

For some parameters, in particular for flood embankments the external water levels and pore water pressures, it may not be appropriate to apply partial factors to characteristic values to obtain design values. In such cases design values may be directly assessed or obtained by adding a margin onto the nominal or characteristic value. Establishing the design values of water levels and pore water pressures is discussed in more detail in Section 4.

2.3 RISK CATEGORISATION

The role of flood embankments in the wider context of flood risk management is addressed in ILH 2 and more detail about risk analysis and attribution is provided in ILH 5.2. The definition of risk as outlined in ILH Box 2.2 is that it is a function of the probability of both the flood and an associated failure occurring, and the consequences of such a failure on affected people, property or environments. Flood embankments are used in a range of situations and the level of risk associated with the failure or poor performance of an embankment will vary depending upon factors such as the ground conditions, size of the embankment, the volume of water impounded and the number of people, type of infrastructure, property and land protected by the embankment. Assessment of the risk category of a flood embankment early in the project is crucial as this may have an impact on the level of investigation, design and checking that is undertaken. Two classifications are used in the Eurocodes for categorising the risk to a structure, EN 1990 provides Consequence Classes (CC) and Reliability Classes (RC), while EN 1997 introduces Geotechnical Categories (GC). These classifications (CC/RC and GC) are described in Sections 2.3.1 and 2.3.2.

2.3.1 Consequences Classes and Reliability Classes

EN 1990 2.2 introduces the concept of design reliability. The level of design reliability required for a structure may vary depending upon the consequence of a particular failure mechanism, public perception of risk and the likelihood of occurrence of the particular design situation under consideration.

EN 1990 Annex B provides an Informative Annex for the management of structural reliability for construction works. This annex introduces Consequences Classes (CC1 to CC3), which classify a structure on the basis of the consequences of failure, in terms of loss of life, social and environmental considerations. Detailed classification criteria are not provided in EN 1990 but ILH Box 9.3 presents an example method of risk categorisation for flood embankments with risk categories I to IV. These categories are determined on the basis of flood duration, embankment height, the number of people at risk and the potential damage to buildings and infrastructure. These categories are based on the effect of a failure being a breach of the embankment and a release of water. These ILH risk categories can be considered equivalent to EN 1990 Consequences Classes.

Table 2.3 shows a proposed relationship between the two systems. The use of four risk categories in the ILH prevents a one-to-one relationship so the table relates ILH Risk Category II and III to the same Consequences Class. Differentiation between ILH Risk Category II and III is made in Table 2.5.

EN 1990 Annex C associates Consequences Classes (CC1 to CC3) with Reliability Classes (RC1 to RC3), each with a different recommended reliability index. The associated Reliability Classes are shown in Table 2.3. Reliability index is a measure of the probability of failure and for categories with higher consequences of failure it will generally be desirable to have a lower probability of failure through an increased reliability index. In the UK and Ireland direct assessment of the reliability index of a flood embankment (for example through probabilistic methods) is not commonly undertaken in design, but EN 1990 allows for reliability differentiation through modification of partial factors (EN 1990 B3.3 and B6). Alternatively, differentiation between design supervision levels (EN 1990 B4) and inspection levels during execution (EN 1990 B5) may also be made on the basis of Consequences Class and hence the level of reliability required.

Table 2.3 Relationship between ILH Risk Category and EN 1990 Consequences Class and Reliability Class

ILH Risk Category	EN 1990 Consequences Class	EN 1990 Reliability Class
I	CC1	RC1
II	CC2	RC2
III		
IV	CC3	RC3

Note

These classes assume failure, ie breach of the embankment occurs.

In Section 3, typical design situations and associated failure mechanisms for flood embankments are identified. In some cases, these mechanisms may lead to failure of part of the embankment which would not directly cause a breach. An example would be a shallow slip of material that did not intersect the crest. Provided such a slip was repaired before further slips occurred the consequences of the failure are less severe than for a larger slip that resulted in a breach. It is therefore considered unreasonable to apply the same level of reliability to all failure mechanisms associated with an embankment when the consequence of a mechanism may vary significantly. EN 1990 B3.1(3) acknowledges this in the following statement: "*Depending on the structural form and decisions made during design, particular members of the structure may be designated in the same, higher or lower consequences class than for the entire structure*."

As part of the design process the designer should assess the overall Consequences Class of a flood embankment based on a breach occurring, but in assessing individual mechanisms there is an opportunity to apply a revised Consequences Class and Reliability Class to optimise the design. An example application of this may be a rapid drawdown situation. Traditionally a lower global factor of safety has been accepted for this mechanism as the associated failures tend to be shallow and so will not lead to a breach of the embankment. Provided the designer assesses the consequence of the analysed mechanism, then modified partial factors in accordance with the revised Reliability Class may be applied as outlined in Section 2.3.1.1.

2.3.1.1 Reliability differentiation by partial factors

EN 1990 permits the modification of partial factors in order to achieve a level of reliability compatible with Consequences Classes. A 'reliability multiplier' may be applied to action or resistance partial factors. EN 1990 B3.3 suggests that typically the partial factor on actions, γ_F, would be modified and defines K_{FI}, a multiplier applicable to partial factors on actions. In this guide a multiplier, K_{MI}, is proposed to be applied to the partial factor on material strength, γ_M, for those limit states where soil strength is critical to design.

At present there is little data on which to base the derivation of the reliability multiplier for different flood embankment Reliability Classes. However in preparing this guide the project steering group (PSG) was keen that the principle of modifying partial factors on the basis of risk be incorporated.

Table 2.4 summarises a proposed scheme of modifying partial factors. EN 1990 suggests a 10% variation in factors between Reliability Classes, while other countries have adopted variations in parameters of between 5% and 10%. Given the limited work undertaken to justify modification of partial factors, a variation of 5% is proposed. In Section 8 further work is recommended to encourage more detailed analysis to establish a relationship between Consequences Class, reliability index and reliability multiplier for flood embankments.

Table 2.4 Partial factor multipliers for reliability differentiation

ULS	**Reliability Class 1**		**Reliability Class 2**		**Reliability Class 3**	
	Factor on material strength multiplier, K_{MI}	**Factor on actions multiplier, K_{FI}**	**Factor on material strength multiplier, K_{MI}**	**Factor on actions multiplier, K_{FI}**	**Factor on material strength multiplier, K_{MI}**	**Factor on actions multiplier, K_{FI}**
GEO DA1-1	1	0.95	1	1	1	1.05
GEO DA1-2	0.95	1	1	1	1.05	1
UPL	1	0.95	1	1	1	1.05
HYD	1	0.95	1	1	1	1.05

The partial factor multipliers in Table 2.4 are used to modify the partial factors provided in the UK and Irish National Annexes. Summary tables of partial factors, taking into account these Reliability Class multipliers, are included in Section 5.

2.3.1.2 Design supervision and construction inspection level differentiation

In addition to using Consequences Classes to determine different levels of design reliability, EN 1990 also identifies accompanying measures that may be associated with each Reliability Class relating to design supervision level (DSL) and construction inspection level (IL). EN 1990 2.2(5) also identifies inspection and monitoring as a method of achieving reliability.

The intention is that for higher Consequences Class structures, there is greater degree of checking and supervision of both the design and construction processes, thereby providing greater reliability. As identified in ILH Box 9.2 this tiered approach is comparable to the certification process adopted in HA (1997a), which incorporates the checking procedures defined in HA (1997b).

The adoption of four categories of risk level in the ILH prevents direct application of the DSL and construction ILs in EN 1990. So Table 2.5 presents a proposed relationship between the EN 1990 Reliability Classes, the ILH Risk Categories, DSL and construction IL in EN 1990. Construction IL3 is proposed for both ILH category III and IV structures. This 'third party' inspection is recommended to be undertaken by the designer. The extent of such site inspection and attendance (eg full time or part time attendance) may vary depending upon the risk category.

Table 2.5 Reliability Classes and accompanying control measures

EN 1990 Reliability Class	ILH Risk Category	Design supervision level (DSL)	Construction inspection level (IL)
RC1	I	DSL1: normal supervision – self checking	IL1: self-inspection
RC2	II	DSL2: normal supervision – checking by a different person/team within organisation	IL2: inspection in accordance with the procedures of the organisation, eg contractor self-certification
	III	DSL2: normal supervision – checking by a different person/team within organisation	IL3: third party inspection. For embankments this is recommended to be undertaken by the designer
RC3	IV	DSL3: extended supervision – third party checking by a different team/organisation	IL3: third party inspection. For embankments this is recommended to be undertaken by the designer

In the absence of a national framework for a tiered system of quality assurance of flood embankments, the specific requirements for DSL and construction IL should be agreed at a project level with reference to the general requirements in EN 1990 Table B4 and Table B5.

2.3.2 Geotechnical Categories

For geotechnical structures EN 1997-1 defines three Geotechnical Categories (EN 1997-1 2.1). These categories have been created to establish a minimum level of investigation, analysis, design and construction control that is proportionate to the complexity of the problem. Geotechnical categories are primarily concerned with the complexity of the ground conditions or the proposed structure and can be considered to influence the likelihood of a failure occurring. It is noted that in some instances the definition of Geotechnical Categories used in EN 1997 includes the term 'risk', which implies that the consequence of failure is also relevant.

Geotechnical Category (GC) 1 relates to "*small and relatively simple structures with negligible risk*" (EN 1997 2.1(14)). GC2 applies to "*conventional structures with no exceptional risk or difficult ground conditions*" (EN 1997-1 2.1(17)). GC3 covers structures that fall outside the other two categories. The Geotechnical Category definitions in EN 1997 are general and so the applicability to flood embankment design is not clear. It is likely that many flood embankments will fall within GC2, though larger embankments or those on particularly challenging ground may be GC3. Relatively few flood embankments are expected to be included in GC1. The applicability is summarised in Table 2.6.

Table 2.6 Geotechnical Categories applied to flood embankments

EN 1997 Geotechnical category	Description from EN 1997	Minimum level of investigation, analysis, design and construction control	Applicability to flood embankments
GC1	Negligible risk. Small and relatively simple structures where fundamental requirements will be satisfied on the basis of experience and qualitative geotechnical investigations – calculations usually not necessary. GC1 procedures should be used only if there is no excavation below the water table or if comparable local experience indicates that a proposed excavation below the water table will be straightforward. EN 1997-1 2.1 (14, 16)	These procedures should be used only where there is negligible risk in terms of overall stability or ground movements and in ground conditions that are known from comparable local experience to be sufficiently straightforward. In these cases the procedures may consist of routine methods for foundation design and construction. EN 1997-1 2.1 (15)	Only likely to be applicable to the design of low height flood embankments with good ground conditions that are identified as CC1.
GC2	GC2 includes conventional types of structure and foundation with no exceptional risk or difficult ground or loading conditions EN 1997-1 2.1 (17)	Designs for structures in GC2 should normally include quantitative geotechnical data and analysis to ensure that the fundamental requirements are satisfied. Routine procedures for field and laboratory testing and for design and execution may be used for GC2 designs. EN 1997-1 2.1 (18,19)	The majority of flood embankments are expected to be classified as GC2. Ground investigation to be undertaken in accordance with EN 1997-2. Design supervision and construction inspection levels to be based on RC2 requirements. Inspection and maintenance may be specified.
GC3	GC3 should include structures or parts of structures, which fall outside the limits of GC1 and GC2. GC3 includes the following examples: • very large or unusual structures • structures involving abnormal risks, or unusual or exceptionally difficult ground or loading conditions • structures in highly seismic areas • structures in areas of probable site instability or persistent ground movements that require separate investigation or special measures. EN 1997-1 2.(20, 21)	Risk mitigation for GC3 structures will require project specific consideration but in general may require a higher level of site characterisation, a greater level of design supervision, possibly through independent checking of the design and rigorous inspections and controls during construction. It is possible that increased partial factors or model factors may also be specified in order to provide the required level of design reliability. EN 1997-1 2.4.7.1 (4))	Large flood embankments (greater than 6 m height), or those on unusual ground identified as the highest Consequences Class should be classified as GC3. Ground investigation to exceed the requirements of EN 1997-2. Design supervision and construction inspection levels to be based on RC3 requirements. Enhanced level of inspection and maintenance may be specified.

There is some overlap between the design requirements for Consequences Classes and Geotechnical Categories. Further details of the requirements for Geotechnical Categories are given in the following sections of EN 1997-1:

Table 2.7 Requirements for Geotechnical Categories in EN 1997-1

Section number	Section title
2.1	Design requirements
3.2.1	Geotechnical investigations
4.2.2	Inspection and control
4.3.1	Checking ground conditions
4.4	Checking construction
4.5	Monitoring

EN 1997-1 notes that a GC3 classification may lead to the adoption of higher partial factors, though no recommendations for modified factors are given in EN 1997-1. In many cases GC3 classified structures will also be CC3 and so the modified factors in Table 2.4 may be used in addition to the higher classes of design and construction control.

The categorisation of a structure should be undertaken at an early stage in the project, and then be reviewed and changed as necessary at each stage of the design process. As noted in EN 1997-1 2.1(13) "*The various design aspects of a project can require treatment in different Geotechnical Categories. It is not required to treat the whole of the project according to the highest of these categories*".

While EN 1997-1 does not explicitly make the link between Geotechnical Categories, Consequences Classes and Reliability Classes there are many areas of overlap. In many cases it may be reasonable to link Consequences Class (and hence Reliability Class) with Geotechnical Categories, though there will always be exceptions to this simplification. For example the design of a new flood embankment of CC1 may be classified as a GC2 structure where it is constructed on soft clays. In these more complex situations discussions should be held with the asset owner/manager and sensible engineering judgement applied to appropriately classify the structure given the level of reliability required.

2.4 SITE CHARACTERISATION

Site characterisation is a crucial part of any flood embankment design. Site characterisation includes the following processes:

- desk study
- walkover survey
- geological assessment
- ground investigation
- evaluation of geotechnical parameters
- geotechnical reporting.

EN 1997-1 3 relates to geotechnical data and EN 1997-2 addresses ground investigation and testing. EN 1997-1 provides useful check lists for the issues to be considered when characterising a site and should be referenced by the designer.

Ground investigations form part of the site characterisation process. Although EN 1997-2 goes some way to providing guidance on undertaking investigations, this is non-specific and may be more applicable to raised earthworks for highways rather than water retaining structures, which have special issues. EN 1997-1 3.2.1, however, is specific in requiring investigations to provide sufficient data to allow reliable assessment of parameters to be used in design.

The activities required to characterise a site in accordance with EN 1997 are detailed in Table 2.8. References to key EN 1997 clauses are included in the table and cross references to relevant sections of the ILH are also included for a more detailed discussion.

Table 2.8 Summary of site characterisation process

Stage	Activity	EN 1997 clauses	ILH references
Desk study	The desk study is the initial phase of data gathering using existing sources of information. Desk studies provide an insight into the geotechnical nature of the site, historical influences on the site, and the form of any existing flood embankments on site. A site walkover should be undertaken as part of the desk study to provide context and verify the information obtained.	EN 1997-1 3.4	Section 7.1.4 *The desk study* provides guidance on: ● undertaking a desk study (7.1.4.1) ● types of information to be considered and its uses (7.1.4.2) ● site walkover survey (7.1.4.3).
Failure and deterioration modes of existing embankments	Investigations need to answer specific questions. In the case of flood embankments this relates to stability during construction or improvement works, and when under hydraulic loading, which could include overtopping. For the investigations to establish the appropriate information, all credible failure and deterioration modes need to be identified using, as a starting point, the geotechnical information collated and interpreted in the desk study, ground conditions, internal structure of the existing flood embankment, and the properties of any borrow material to be used.	EN 1997-2 2	Section 7.7.2 *Ground investigation relative to failure and deterioration modes*, provides guidance on: ● failure and deterioration modes both within the foundation soils and levee structure ● investigation and monitoring appropriate to each mode. Section 3.5 *Understanding failure modes* provides guidance on: ● defining failure ● main process of deterioration, damage and breach.
Phasing of investigations	If necessary, the project programme should allow for the phasing of investigations. Phasing of investigations allow the progressive gathering of information. Data gathered during earlier phases will allow more effective targeting of subsequent phases to address specific issues. The phased approach to investigations could be integrated with investigations needed for archaeology, contamination and unexploded ordnance if required.	EN 1997-2 2.2 EN 1997-2 2.3 EN 1997-2 2.4 EN 1997-2 2.5	Section 7.1.1 *Site characterisation* process provides guidance on: ● tiered approach to characterisation (7.1.1.1) ● phased approach to characterisation (7.1.1.2) ● planning the process (7.1.1.3).

Spacing of investigation points	All intrusive investigations should be located to address specific issues using the information presented in the desk study and, if available, earlier phases of the investigation, including geophysics, to target them. Intrusive investigation should not be undertaken at a defined fixed interval along a flood embankment. The spacing guidance offered by EN 1997 should be taken as a guide to the quantum of intrusive investigations. For flood embankments it is important to understand the variation in ground conditions (properties and profile) across the full width of the embankment and beyond the embankment toe. So, off centre-line investigations should also be undertaken.	EN 1997-2 2.4.1.3 EN 1997-2 B.3	Section 7.9.7.2 *Spatial distribution of intrusive investigations,* including Table 7.140 and Section 7.9.7.4.
Depth of investigations	The depth of ground affected by a flood embankment should be investigated. This is not just the zone affected by the self-weight of the embankment (settlement, overall stability) but also that resulting from the interaction of the embankment with the hydraulic loads (uplift, seepage, internal erosion). It is also good practice to include at least one borehole that is deeper than might be considered necessary to prove the underlying geology.	EN 1997-2 B.3	Section 7.9.7.3 *Depth of exploration holes,* including Table 7.141.
Sampling	Cohesive soils are more suited to being sampled by methods that result in limited sample disturbance and so produce higher quality samples. Samples need to be of sufficient quality and quantity to be representative of the *in situ* soil and to allow the geotechnical properties to be assessed. Consideration should be given to the sample type and quality, and size and sampling frequency. The potential for sample disturbance during handling, transportation and storage should also be addressed as these can affect the quality of the test specimen and test result.	EN 1997-2 2.4.1.4 EN 1997-2 3 BS EN ISO 22475-1:2006 BS 5930:1999 +A2:2010	Section 7.9.8 *Sampling methods* includes guidance on: • selection of techniques (7.9.8.1) • quality and size (7.9.8.2) • frequency/spacing (7.9.8.3) • labelling, handling, transportation and storage (7.9.8.4).

In situ testing	*In situ* testing is usually employed where: • a fast result is required • mass soil characteristics differ from those derived on small laboratory test specimens • it is difficult to obtain representative samples • it is required to complement laboratory tests. The frequency of *in situ* testing should address the same factors considered in the selection of the sampling frequency and the determination of geotechnical parameters (see *Determination of characteristic parameters*), which includes laboratory testing. However, the frequency will also reflect the cost and the volume of *in situ* soil affected by the test.	EN 1997-2 4	Section 7.8.1.4 *Measurement in situ* includes guidance on: • background information on the *in situ* measurement of soil properties • frequency of testing. Section 7.8.3 *Determination of geotechnical parameters and methods* includes guidance on the determination of geotechnical parameters through *in situ* tests, along with other methods. Section 7.9.9 *Field instrumentation and monitoring*, Box 7.35 and Section 7.7.4 *Ground investigation validation through pre-construction trials* includes guidance on the use of field instrumentation in combination with a trial section of levee to assess the undrained shear strength and consolidation characteristics of 'soft' clay foundation soils.
Determination of geotechnical parameters	Soils are natural materials that will be laterally and vertically variable, even within a defined horizon. Measured geotechnical parameters will also reflect this natural variation. The test results will be affected by sample and specimen disturbance, and the testing methods. Geotechnical parameters can be assessed using a number of techniques: • typical values from published information or local knowledge • empirical correlations with index properties • direct measurement in the laboratory or *in situ* • indirect measurement using geophysical techniques. Parameters assessed using typical values or from empirical correlations should be validated by comparing them with directly measured parameters. Laboratory testing schedules should follow a hierarchical structure with many index tests and fewer tests aimed at measuring mechanical or hydraulic properties. All mechanical or hydraulic tests should include an associated index test so that the results can be compared with empirical correlations and to allow the measured engineering property to be inferred to other soils with similar index properties.	EN 1997-1: 2.4.5.2 and 2.4.5.3	Section 7.8.4 *Determination of characteristic values* includes guidance on: • considerations when assessing a characteristic value • quantifying characteristic values. Section 7.8.1.3 *Measurement in the laboratory* includes guidance on scheduling laboratory tests.

Instrumentation and monitoring	Instrumentation can be used to monitor the response of the ground and flood embankment during construction. It can aid in the assessment of the current condition or monitor long-term performance. There are many requirements that should be considered when designing an instrumentation programme. Some of these are: ● potential failure and deterioration modes should be understood to identify the critical responses to be monitored ● instrumentation needs to be capable of accommodating the anticipated responses ● some redundancy/duplication should be included in the system ● backfill grouts should have the same strength as the surrounding soils ● different forms of instrumentation should be combined at one location to provide an integrated understanding of the soil response ● baseline/datum readings should take account of variables in boundary conditions that could affect the reading. Manual readings should initially be undertaken more frequently to allow reading errors to be assessed early on in the programme. Remote monitoring by telemetry is available and is relatively common. Advanced technologies using discrete multiple sensors to form an instrumentation string and distributed monitoring systems based on fibre optics are available.	EN 1997-1: 2.7 EN 1997-2: 2.5	Section 7.7.2 *Ground investigation relative to failure and deterioration modes*, provides guidance on: ● failure and deterioration modes within both the foundations soils and levee structure. ● processes, investigation and monitoring appropriate to each mode. Section 7.7.4 *Ground investigation validation through pre-construction trials*, includes guidance on field monitoring as a construction control. Section 7.9.9 *Field instrumentation and monitoring* provides guidance on: ● considerations in the selection of instrumentation (7.9.9.1) ● installation records (7.9.9.2) ● obtaining baseline and monitoring readings, and what to record (7.9.9.3) ● presentation of the data (7.9.9.5) ● types of instrumentation, including traditional, and new and evolving technologies (7.9.9.6 and 7.9.9.7)).

Characteristics values	The determination of the ground model and selection of characteristic parameter values are the most important geotechnical design tasks. Safety factors cannot compensate for gross errors of judgement (Bond and Harris, 2008). There are many factors that should be considered when assessing characteristic values. Some of these are: ● adopted design codes and standards ● degree of conservatism ● applicability of the data to the limit state being considered, ie test method, upper or lower characteristic value ● behaviour of the soil under load, ie strain softening or hardening ● method by which empirical factors were derived ● quantity and quality of data ● comparable experience Fundamentally, consideration should be given to how much ground is involved in the development of the limit state. The characteristic value should be 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. Characteristic values may also vary depending upon the design situation and limit state under consideration. While it is generally lower bound estimates of material properties that are critical to the design, there are occasions where an upper bound property is more onerous.	EN 1997-1: 2.4.5.2 and 2.4.5.3	Section 7.8.4 *Determination of characteristic values* includes guidance on: ● considerations when assessing a characteristic value ● quantifying characteristic values.

2.5 RESILIENCE AND SURFACE EROSION

ILH 9.1.1 states *"Levees should have resilience"* and the following definition is quoted: *"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). Differentiation is then made in the ILH between the capacity of the structure to accommodate the loadings for which it is being designed and the robustness of the structure to accommodate situations in which the levee is overtopped. In this guide resilience is used to signify the level of robustness of the design to accommodate these overtopping events. While the level of resilience required is subject to agreement with the asset owner/manager, in many cases it is likely to be desirable that the embankment does not fail as a result of erosion or instability during an overtopping event.

Another aspect of resilience is climate change. In consultation with the asset owner/manager consideration should be given to the extent to which climate change is accommodated in the design. For example, this may include incorporating a wider footprint or stronger foundations to allow for incremental raising during the design life of an embankment.

The ULSs defined in EN 1997 and discussed in Section 2.2.2.1 are primarily concerned with ensuring the stability of the embankment and avoiding processes of internal erosion. A failure mechanism that has not been discussed so far in this guide is external erosion as a result of overtopping from waves or overtopping flow when flood levels are above crest level. This is an important design aspect in the resilience of an embankment. EN 1997-1 2.4.2(9) requires that "*Actions in which ground- and free-water forces predominate shall be identified for special consideration with regard to deformations, fissuring, variable permeability and erosion*". However there are no specific requirements listed in EN 1997 with regard to the method of verifying an external erosion limit state mechanism. Further discussion of surface erosion and resilience to overtopping is not included in this guide and it is recommended that the approaches outlined in the ILH are adopted.

Table 2.9 References to relevant ILH sections discussing external erosion

ILH section number	ILH section title
ILH 3.5.2	Main processes of deterioration, damage and breach
ILH 4.5	Vegetation management
ILH 7.8.3.6	Erodibility
ILH 8.4	External erosion
ILH 9.1	Principles of levee design
ILH 9.6	Surface protection measures
ILH 9.11	Transitions
ILH 9.13.3.6	Resilience to external erosion and seepage

3 Design situations

3.1 INTRODUCTION

As outlined in Section 2.2.1, EN 1997-1 requires the design process to identify design situations that can reasonably be expected to occur during the lifetime of the structure. There are a number of design situations that may occur during the life of a flood embankment and these should be assessed individually to verify the design is adequate. The following sections and Table 3.2 summarise the main design situations that are likely to be applicable to most flood embankments. However this is not an exhaustive list and the designer must always consider whether any particular circumstances of a design mean that other design situations may exist that could be more critical.

Given the variable nature of ground conditions and topography, it is likely that a number of representative geometries and ground models will need to be selected to cover the range of conditions encountered across a particular project. The identified design situations would need to be considered for each of these geometries and ground models.

3.2 ASSESSMENT OF DESIGN SITUATIONS

The considerations in Table 3.1 may be used to help define a design situation. Although these will identify most common design situations, the designer must always check if there are any local conditions that need to be taken into account. Examples of typical design situations for flood embankments are listed in Table 3.2.

Table 3.1 Considerations used to define design situations

Consideration	Description
Design situation type	Is it persistent, transient or accidental? Seismic design situations are not covered by this guide.
Limit states	Identify which limit states are to be verified. For ULS this includes GEO, UPL and HYD (and STR when composite structures are required). The application of SLS criteria should also be considered.
Geometry	The geometry assumed should be the most onerous that could apply for a given design situation. For example, this may include over-steepened slopes during construction, excavation at the embankment toe for future maintenance, or loss of ground due to scour.
Soil model	Consider whether analyses are to be undertaken using effective stress or total stress parameters or both. Verify relevance of soil parameter measurement to limit state being considered. Account should be taken for changes in soil properties over time (eg softening, consolidation, fissuring, desiccation, animal burrowing).
Water levels	These are the external water levels acting on either side of the embankment and need to take into account variations due to waves, tides and flooding. This is one of the main differences between each design situation and is discussed in more detail in Section 4. Considerations should be given to situations of high water (eg flooding), low water (eg rapid drawdown or negative surges) and floodwater becoming trapped on the landside of the embankment following flooding.
Seepage model and pore water pressures	This refers to the magnitude and distribution of pore pressures in the embankment and underlying foundation material. While this will be linked in some way to water levels it will depend upon the duration of the event and the nature of the soil. Typically a seepage assessment will be needed to determine the pore water pressure distribution when designing with effective stress parameters. Consideration should be given as to whether a steady state or transient seepage analysis would be applicable. Further discussion of this is given in Section 4.4.

Consideration	Description
Permanent actions	Identify all the permanent actions that need to be considered for the design situation. Typically these include the mass of the embankment and any associated structures. For flood embankment water is often considered to be a permanent action. This is discussed in more detail in Section 4.3.
Variable actions	Identify varying actions that may be imposed on the embankment. These are often construction or operational traffic, but effects from nearby structures may also need to be taken into account. In particular, there may be a need to consider the presence of emergency or maintenance plant operating during a flood situation. Other examples are given in ILH 7.6.
Accidental actions	Only applicable to accidental situations. Identify those actions that only occur during the accidental situation. This may include water levels that exceed the design flood due to failure of another part of the system leading to exceptionally high water levels, eg blockage of a spillway or failure of a flood defence upstream. Naturally occurring extreme water levels should not be considered accidental. The asset owner/manager should be involved in discussions to agree the extent to which resilience to such actions is incorporated into the design.

3.3 OVERVIEW OF TYPICAL DESIGN SITUATIONS

Four design situations have been identified that commonly occur in flood embankment design:

- construction
- normal operating conditions
- flood event
- rapid drawdown following flood.

Table 3.2 outlines the assumptions behind each of these design situations. Water levels and pressures are a critical aspect of these situations and more discussion of the selection of water levels is given in Section 4.

While generally relevant, these design situations may not be applicable to all flood embankments and there are cases where additional design situations may need to be considered. For example, it is likely that for most embankments the level of freeboard means that in the flood situation the design water level will be taken at the crest of the embankment (or just above if an allowance is made for overtopping flow). In the case where the embankment is designed to frequently overtop (ie low design return period and small freeboard) the resilience of the levee will potentially be critical. In this situation the degree of resilience should be discussed with the asset owner/manager and considerations given to the degree of resilience provided or mitigation measures implemented (eg spillways) to minimise risk. Reference should be made to Section 2.5 and the ILH for detailed guidance on erosion protection and resilience.

However, in situations where a particularly large freeboard is provided, the design water level may be below the top of the embankment. In such cases an additional situation may be considered where the water rises to the top of the embankment and overtops as a result of failure of part of the system (eg a blocked outlet or failure of another part of the flood defence). This would be an accidental situation. The designer should consult with the asset owner/manner to agree the extent to which the design will accommodate such accidental events.

Other examples of situations for which embankment stability may need to be verified include water becoming trapped on the landside of the embankment following a flood or the rapid drawdown of the water level following a flood or during a negative surge on the waterside of the embankment.

A separate set of seismic design situations may also need to be considered.

Table 3.2 Design situations

	Construction	Normal conditions	Flood	Rapid drawdown after flood
Situation type	**Transient**	**Persistent**	**Transient**	**Transient**
Relevant ultimate limit states (ULSs)	GEO (mass stability) SLS check not usually critical	GEO (mass stability) UPL (buoyancy) HYD (heave, internal erosion, piping) SLS to be verified, eg settlement, seepage rates	GEO (mass stability) UPL (buoyancy) HYD (heave, internal erosion, piping) SLS (seepage)	GEO (mass stability) UPL (buoyancy) HYD (heave, internal erosion, piping) SLS check not usually critical
Geometry	Potential variations in geometry during construction need to be considered, for example embankments may be higher than in the permanent situation due to placement of surcharge, slopes may be steeper due to zoned construction, or trenches may have been excavated to allow placement of drainage or seepage cut-off barriers. Slopes may lack protection from weather that they would have in the permanent state.	The geometry will likely be based on the specified design geometry but consideration should be given to feasible permanent variations that could occur during the life of the embankment, such as scour and desiccation cracking. It is important that the designer confirm with the owner/operator if there are any anticipated future developments that may influence the geometry or use of the embankment.	The geometry will be based on the specified design geometry (see *Normal conditions*).	The geometry will be based on the specified design geometry (see *Normal conditions*).
Soil model	Need to consider both effective stress and total stress analysis, especially on soft alluvial soils.	Both effective stress and total stress analyses should be undertaken and should consider design situations that may occur over the full design life of the embankment. If necessary, changes in shear strength of the embankment fill or the foundation soils over the design life should be considered.	Both effective stress and total stress analyses should be undertaken. This may require assessment of changes in undrained strength of the foundation soils post construction and should consider the possibility of a flood occurring early in the design life of the structure while elevated pore pressures caused by embankment construction are still present.	Effective stress analysis is likely to be critical but both drained and undrained assessments should be undertaken.
Water levels (see Section 4)	Design water level should be most onerous that could occur during construction, with a low (1%) probability of being exceeded. This may be assessed using a detailed hydraulic modelling or by adding a margin to the normal river level. See notes to	Characteristic water level based on mean river level. Design water level to be characteristic plus a margin of safety appropriate to general variability of river level.	Characteristic water level based on water level likely to occur during life of structure, ie a 50% probability of exceedence. Design water level to include a margin over the characteristic level. Should be worst that could occur during life of structure, ie 1% probability of	Characteristic water level on the waterside based on lowest water level that is likely to occur following a flood event (eg mean river level). Design water level to be characteristic less a margin of safety appropriate to general

	Construction	Normal conditions	Flood	Rapid drawdown after flood
	Table 2.4 for more details.		exceedance. For most embankments this will be limited by the crest level and overtopping.	variability of river level.
Seepage conditions and pore pressure distribution (see Section 4)	Maximum water level is likely to be a short-term event so may undertake transient seepage assessment to obtain pore pressure distribution (where effective stress analysis is critical). Starting point for a seepage analysis would be current *in situ* ground water pressure profile.	Water pressures based on steady state seepage profiles with design water levels as boundary conditions	The potential duration of a flood event should be considered in order to evaluate the pore pressure profile that may generate within the ground and embankment fill material. This should be related to the flood hydrograph. While it may be simplest to assume steady state seepage based on the design water levels, this may be an overly cautious approach and a transient seepage analysis may lead to more economic design	The initial pore pressure distribution of the seepage analysis should be taken from the flood event situation. The rate of drawdown then needs to be considered in a transient seepage analysis to obtain the pore pressure profile following rapid drawdown based on the design water levels
Permanent actions	Embankment self-weight. Water pressures.	Embankment self-weight. Water pressures.	Embankment self-weight. Water pressures.	Embankment self-weight. Water pressures.
Variable actions	Construction traffic.	Operational traffic.	Emergency/maintenance traffic.	Operational traffic.
Accidental actions	See definition in Section 2.2.1.			
Comments	This situation may be critical for overall embankment geometry. May need to consider staged construction to enable a more economic geometry to be constructed on softer ground.	This situation is rarely critical for ULS, but settlement and seepage limits may mean that SLS criteria control the design.	May affect stability of both waterside and landside slopes and therefore geometry. Seepage could also be a controlling factor on the design.	Only likely to affect waterside slope stability. The relevance of this situation will depend on the anticipated flood hydrograph. To be a critical case it typically requires water to be held at a high level for a long period, before dropping rapidly (more typical of larger rivers in the US than in the UK/Ireland) or on dams that permanently impound water but may be drained rapidly (eg for maintenance). However there are cases of rapid drawdown problems on UK/Irish flood defences and it is the relative rate of water level changes that matter.

4 Water levels, pore water pressure and seepage profiles

4.1 INTRODUCTION

Water pressures are often the critical action when assessing the stability of a flood embankment. This includes external water pressures acting on the embankment surface and pore water pressures within the embankment and foundation soil.

It is important to note that the water levels used when assessing the stability of the embankment in accordance with EN 1997 are not necessarily the same as those used to determine the required crest level of the embankment. Crest levels will be determined by the standard of protection to be provided by the flood defence. Determination of this level is not covered by EN 1997 and is not covered in this guide. For details of setting crest levels see ILH 9.5, and Kirby and Ash (2000).

It is noted that as part of the joint Environment Agency/Defra Research and Development Programme, the freeboard guidance note (Kirby and Ash, 2000) is currently being updated to provide a consistent, scientifically robust and evidence-based methodology for the development of residual uncertainty allowances in flood and coastal erosion risk management. Publication of the revised guide (SC120014) is anticipated in 2015.

Having established the required crest level, the assessment of water levels and water pressures to be used in the stability analyses is governed by the requirements of EN 1997.

The guidance presented in the following sections has been influenced by the proposals of the EG9, which has been looking at the issue of water pressures in design. However it should be noted that at the time of writing the conclusions of the Group had not been finalised so changes to their recommendations may occur.

4.2 EXTERNAL WATER LEVELS

For flood embankments, water levels will control the forces applied to the embankment by external water, which is impounded by the embankment. They will also act as boundary conditions to the assessment of pore water pressure profiles within the embankment and foundation soils.

Changes in water levels define many of the design situations for flood embankments. For a given design situation, the characteristic and design water levels should be determined based on the EN 1997-1 requirements outlined below.

Characteristic water level

- EN 1997-1 2.4.5.3(1) requires that *"Characteristic values of the levels of ground and ground-water or free water shall be measured, nominal or estimated upper or lower levels"*. Assessment of the water level may be undertaken through consideration of hydrological, hydrogeological and environmental information together with statistical analysis, if suitable data are available.
- EN 1997-1 2.4.6.1(6) requires that for limit states with less severe consequences (generally serviceability limit states) design pore water pressures *"shall be the most unfavourable values which could occur in normal circumstances"*. This definition is also considered to be applicable to characteristic water levels.

In terms of flood embankments, characteristic water levels are therefore proposed to be the most onerous that are likely to occur for a given design situation. EG9 defines this as a water level that has a return period at least equal to the duration of the design situation. It is proposed that this is taken to mean the water level that has a 50% probability of exceedance over the duration of the design life. ILH Box 2.8 provides a summary of how the return period and duration may be combined to determine the likelihood of exceedance or ‘encounter probability’. Table 4.1 presents the equivalent characteristic water level return period for a selection of design life durations based on the guidance in ILH Box 2.8.

Design water level

Design water levels for ULSs require a margin of safety over the characteristic level and may be derived by either direct assessment or by adding a margin to the characteristic water level. EN 1997-1 2.4.6.1(6) requires that for limit states with severe consequences (generally ULS) design ground water pressures (or levels) *“shall represent the most unfavourable values that could occur during the design lifetime of the structure*”. EN 1997-1 2.4.4(1) states that “*water levels … shall be treated as geometrical data*” and therefore factoring characteristic water levels to obtain design water levels is not appropriate.

For flood embankments ULS design water levels are proposed to be the most onerous that could occur for a given design situation. EG9 recommends this to be defined as a water level that has a 1% probability of being exceeded in the design life of the structure.

Table 4.1 presents both the equivalent characteristic water level and equivalent design water level return period for a selection of design life durations, based on the guidance in ILH Box 2.8.

Table 4.1 Return periods for characteristic and design water levels for different duration design situations

Design life (years)	**Characteristic water level:** Return period for 50% probability of exceedance	**Design water level:** Return period for 1% probability of exceedance
10	15	1000
50	75	5000
100	150	10000
120	180	12000

Where possible, design water levels should be directly assessed using appropriate analysis and modelling to determine a water level with the required probability of exceedance. However in practice this may be difficult to undertake. An alternative approach is to add a margin to the characteristic water level.

There is no guidance in EN 1997 on the magnitude of margin that should be applied to achieve the required probability of occurrence as there are so many factors to take into account. However ILH 9.10.3.5 outlines a method by which the margin may be assessed. This method considers the ‘permanent’ water level and the potential ‘variable’ height of water above this permanent level (for example due to tidal effects or flooding). The corresponding ‘permanent’ and ‘variable’ water depths are then factored by the appropriate factor on actions and combined to calculate a design depth of water and consequently a design water level. While this is not strictly in accordance with EN 1997 as it applies multiplier factors rather than additive margins to the geometrical data it may give a reasonable indication of the margin to apply to obtain design water levels. See ILH 9.10.3.5 for more details.

For the majority of embankments, it is likely that the waterside design water level in the flood situation will be taken at, or just above, the crest of the embankment. This is because typical crest levels represent a lower return period than the proposed 1% probability of exceedance. In the design flood situation overtopping will then occur and the external water level will only exceed the embankment crest level by the depth of any assumed overtopping flow. This overtopping water level should then be taken as the design water level.

For many embankments with a low standard of protection that have a small freeboard, it is also possible that the characteristic water level may be at, or close to, the crest of the embankment. Again, this level should be taken as the characteristic water level. In such situations there may only be a small margin (or no margin at all) between the characteristic and design water levels. This may require special consideration when assessing design water pressures and is discussed in more detail in Section 4.3.

Characteristic and design water levels for the design situations

Examples of the characteristic and design water levels for the design situations previously identified are presented in Table 4.2. These correspond to water levels on the waterside of the embankment. Consideration should also be given to selecting an appropriate design water level on the landside so that the combined effect is the most unfavourable for the design situation.

The water levels for each design situation identified in Table 4.2 are illustrated in Figure 4.1.

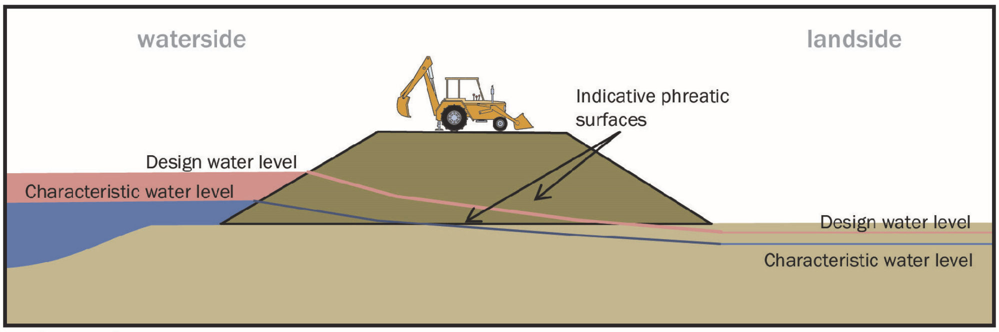
Table 4.2 Example definition of water side water levels for design situations in Table 4.1

Design situation	Situation type	Characteristic waterside water level (ie level likely to occur, 50% probability of exceedance)	Design waterside water level (ie most onerous that could occur, 1% probability of exceedance)
Construction (assume 1-year duration) (see Figure 4.1a)	Transient	2-year return period water level	100-year return period water level[2]
Normal operating conditions (see Figure 4.1b)	Persistent	Quasi-permanent water level[1], eg mean water level	Add margin to characteristic value (magnitude of margin based on general water level variability)
Flood event (assume 100-year design life) (see Figure 4.1c or 4.1d)	Transient	Top of embankment or 150-year return period water level, whichever is lower.	Top of embankment (plus a depth of overtopping flow) or 10 000-year return period water level, whichever is lower.
Rapid drawdown following flood (see Figure 4.1e)	Transient	Quasi-permanent water level[1], eg mean water level	Subtract margin from characteristic value (based on general water level variability or negative tidal surge)

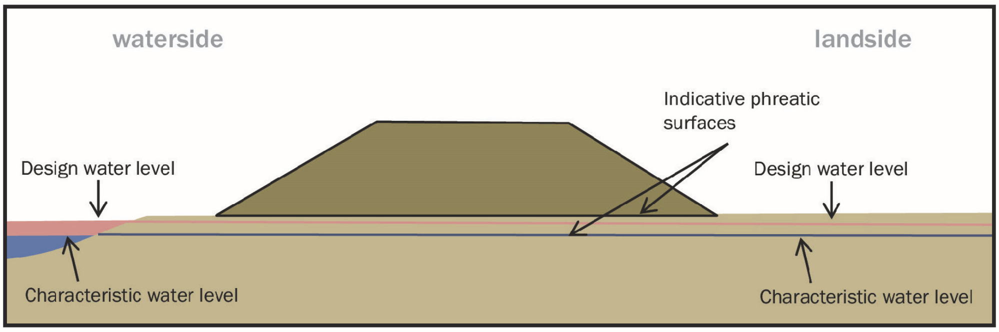
Notes

1 Quasi-permanent values defined in EN 1990 1.5.3.18 as *"value determined so that the total period of time for which it will be exceeded is a large fraction of the reference period"*.

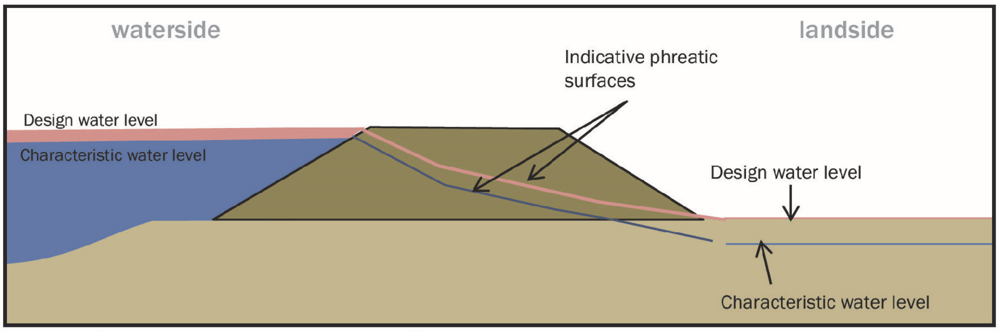
2 The construction situation water levels should be evaluated for the site conditions at the time of construction. The 100-year return period water level for an embankment that is not complete may be different to that for the completed embankment (eg an incomplete embankment may impound less water, the probability of a flood may be affected by the time of year construction takes place). Where the derived water level is considered unduly onerous for the construction stage, consideration may be given to adopting a lower water level provided this is achieved through specific provisions on site to control water levels (eg use of a temporary spillway). Selection of the construction stage design water level should form part of the overall risk management strategy. The cost and risk implications should be discussed and agreed with the asset owner.



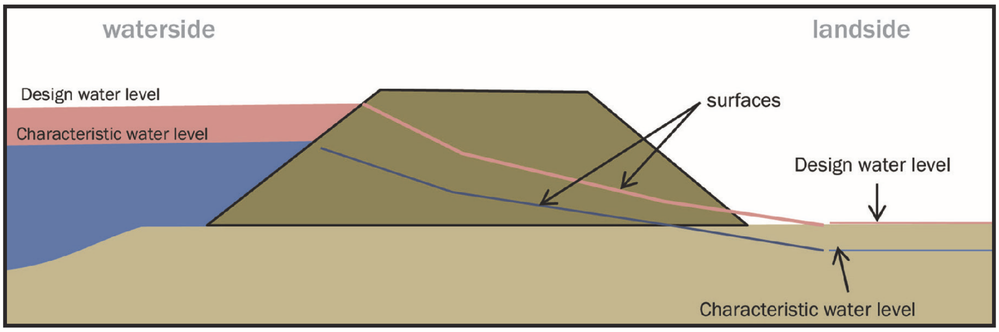
a Construction



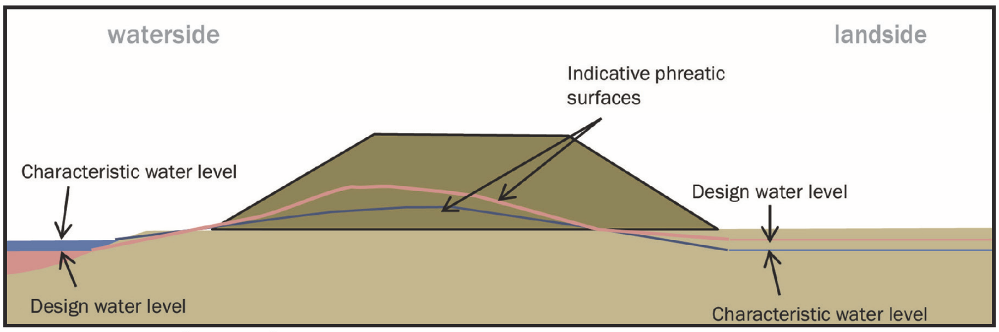
b Normal operating conditions



c Design flood event – embankment with typical freeboard (design water level at embankment crest)



d Design flood event – embankment with large freeboard (design water level determined is below than the embankment crest)



e Rapid drawdown after design flood event

Figure 4.1 Illustrations of design situation water levels

4.3 WATER PRESSURES

4.3.1 Overview

External water pressures acting on the surface of an embankment and the pore water pressure distribution within the embankment and foundations soils are critical to the assessment of limit states. Although linked to variations in external water level, they will also be influenced by:

- the pore pressure distribution present in the ground before any changes during the design situation
- variation in permeability within the embankment and foundation soils
- the duration of any design situation
- the reaction of the soil to changes of stress state.

Estimation of the pore water pressure distribution should be undertaken using a seepage assessment (see Section 4.4). Determination of whether the pore pressures derived from the seepage model are to be considered characteristic or design values will depend on which water levels were used as inputs to the seepage assessment and whether any further partial factors have been applied.

4.3.2 Characteristic water pressures

Characteristic pore water pressures are those derived from a seepage assessment based on characteristic water levels, with no partial factors being applied.

4.3.3 Design water pressures

There are two options for determining design water pressures:

1. **Direct assessment of design water pressures from design water levels:** where a design water level has been derived either by direct assessment or by adding a margin to the characteristic water level (see Section 4.2) the design pore water pressures may be derived from the design water level through an appropriate seepage model.
2. **Factoring of characteristic water pressures:** characteristic pore water pressures are those corresponding to the equivalent characteristic water level through an appropriate seepage model. Design water pressures may be calculated by applying a partial factor to characteristic water pressures.

While both methods for determining design water pressures are permitted by EN 1997, there are situations when one approach may be preferred to the other.

In general, the guidance from EG9 is to avoid factoring water pressures, so the recommendation is to determine design water pressures directly from design water levels.

4.3.4 Discussion of direct assessment and applying partial factors to obtain design pore water pressures

Where there is a reasonable margin between the characteristic and design water levels used to assess the corresponding characteristic and design water pressures, the recommendations of EG9 are considered applicable. However there is a question over assessing design water pressures direct from design water levels through a seepage model in a situation where the embankment geometry means that there is only a small margin (or no margin at all) between the characteristic and design water levels (see Figure 4.2c). In such situations, depending on what other assumptions are changed in the seepage assessments, there may only be a small difference in the characteristic water pressures and the directly assessed design water pressures. It would be physically impossible for the water level to rise significantly higher once overtopping of the embankment occurs, so provided the other inputs to the seepage model are appropriate the design value of water pressures will comply with the definition of a 1% probability of exceedance.

However the designer may also wish to consider whether the directly assessed design values will lead to a sufficient margin of safety for each of the relevant limit states under consideration compared with previous design practice. For limit states where the effect of water is the primary action (ie UPL and HYD) this may lead the designer to consider applying partial factors to characteristic water pressures in order to obtain more onerous design water pressures.

Where design water pressures are to be calculated by factoring characteristic water pressures the factor on actions, γ_F, should be applied. However the designer must select whether the water pressures are permanent or variable, favourable or unfavourable as this will affect the value of the partial factor used. In general the 'single source principle' (EN 1997-1 2.4.2(9)) may be used so that all effects of water pressure are treated the same. It is likely that water pressures will be unfavourable, though consideration should be given to water pressures being favourable (see Box 4.1).

Box 4.1 Are water pressures permanent or variable?

One of the benefits of directly assessing design values of water pressure is that it avoids the need to answer this question. However where the designer considers that the directly assessed design values of pore water pressure are not sufficiently onerous this question must be addressed so the appropriate partial factors can be selected. When selecting whether the action of water is permanent or variable it is possible to consider the definitions from EN 1990 (see Section 2.2). In addition, it is also important to assess what effect it has on the factors that will actually be applied and how the resulting solutions will compare to previously methods of design based on overall factors of safety.

It is changes in water level that define many of the flood embankment design situations. So, it would be simple to say that water is a variable action. However, if the 'reference period' quoted in the definition of a permanent action is taken as the duration of the design situation (for example the duration of a flood) then the water levels and pressures will generally rise to a maximum and then reduce. This is consistent with monotonic variation as used in the definition of a permanent action, and it may be reasonable to take water as a permanent action.

It may be helpful to consider the effect the partial factors would have on the various limit state analyses.

For the GEO limit state, traditional design methods typically aim to achieve an overall factor of safety on slope stability of 1.3 to 1.4. In Design Approach 1 combination 2 the strengths of materials are reduced by a material partial factor (between 1.25 and 1.4). The factor on permanent actions is 1, and the factor on variable actions is 1.3. Applying a partial factor of 1.3 to water pressures is therefore generally considered too onerous as it will lead to higher equivalent factors of safety. Treating water as a permanent action would be preferred in this case. While Design Approach 1 combination 1 (DA1-1) is rarely critical for flood embankments, when relevant it is considered that applying the DA1-1 partial factor on permanent actions to water pressures is also reasonable.

For UPL and HYD limit states, water pressure tend to be the dominant destabilising action, with soil weight being the stabilising action. The partial factor on stabilising and destabilising actions is as follows:

Factor on:	**UPL***	**HYD**
Permanent favourable action $\gamma_{Q;st}$	0.9	0.9
Permanent unfavourable action $\gamma_{Q;dst}$	1.1	1.35
Variable unfavourable action $\gamma_{G;dst}$	1.5	1.5
Equivalent factor if water is treated as permanent	1.1/0.9 = 1.22	1.35/0.9 = 1.5
Equivalent factor if water is treated as variable	1.5/0.9 = 1.67	1.5/0.9 = 1.67
Traditional target factors of safety	1.4–1.5	None

Note:

* UPL may also include resistances from the ground or structural elements (eg piles or anchors) with alternative partial factors.

Taking a simplistic view that the equivalent overall factor of safety is the ratio of the factor applied to unfavourable actions (water) and the factor applied to favourable actions (soil weight), treating water as variable gives an overall factor of 1.67 for both limit states. This is higher than has traditionally been used. Treating the water as a permanent action gives an equivalent overall factor of 1.5 for HYD, which is considered reasonable, and 1.22 for UPL. For UPL this value is considered to be lower than previous design practice.

It is therefore recommended that where characteristic water actions are to be factored, they are treated as permanent actions. In the case of UPL, an additional model factor of 1.1 is proposed to bring the equivalent factor of safety in-line with previous practice.

4.3.5 Summary of proposed approach to determining design pore water pressures

It is recommended that where possible design water pressures are directly assessed on the basis of design water levels. However, where the designer is concerned that there is only a small difference between the characteristic water pressures and directly assessed design water pressures assessed from seepage analysis, alternative design pore pressures may be calculated by applying partial factors to the characteristic pore pressure. Based on partial factors in the current version of NA to EN 1997 it is recommended that the factor on permanent actions be used, though in the UPL ultimate limit state an additional model factor of 1.1 should also be applied (see Section 5.3).

This process is summarised in the following flow chart (Figure 4.2).

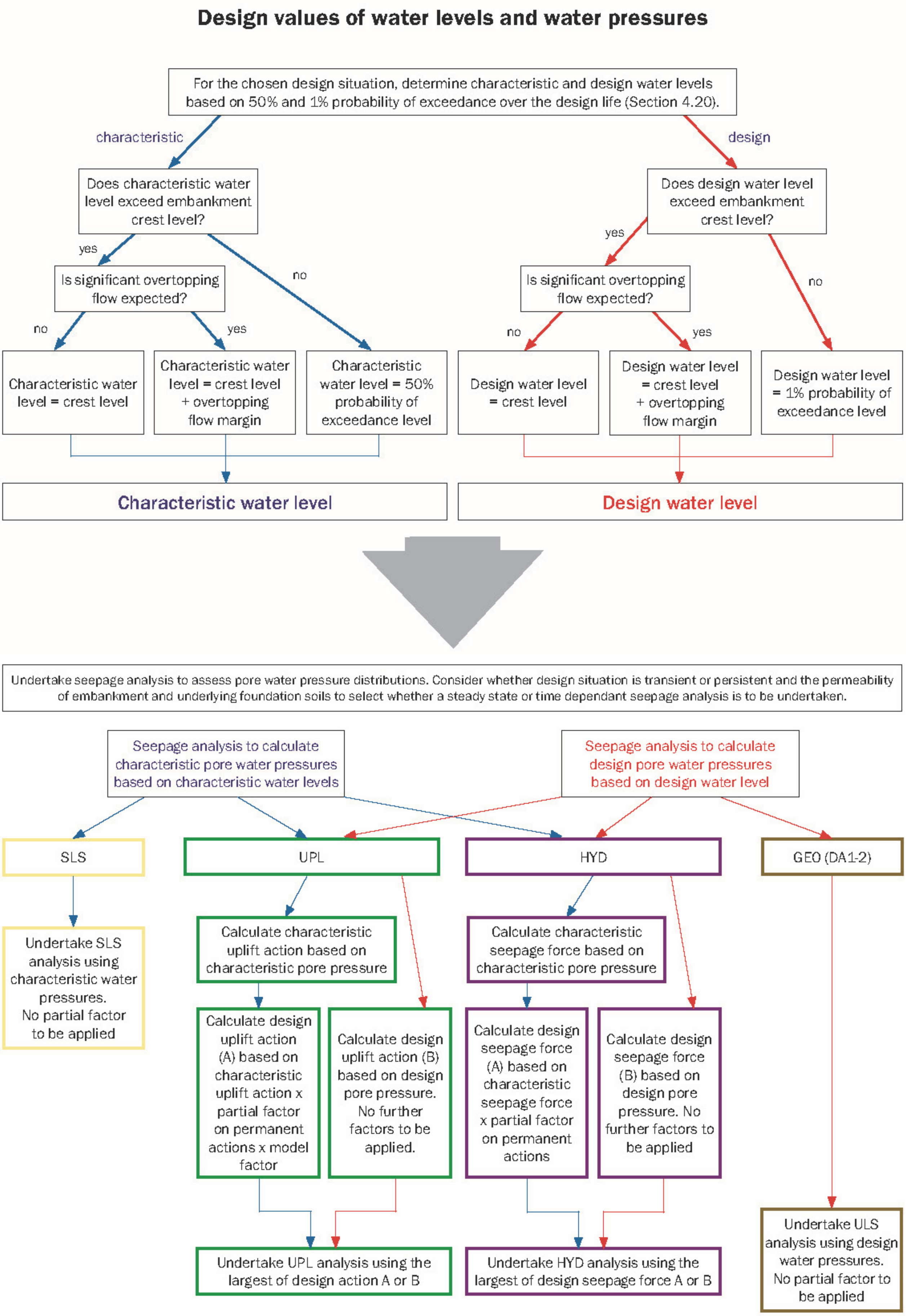


Figure 4.2 Process for determining design values of pore water pressure

In all cases the designer should review the design pore water pressures and verify that, based on confidence in the seepage prediction, complexity of ground conditions and embankment risk category, there is a sufficient margin of safety in the assumed values but that the design pressures are not unreasonably onerous. The overriding principle is that the design pore pressure distribution for a particular design situation should be the most onerous that could occur in the design life of the structure, with a probability of exceedance of 1%

It should also be noted that the application of partial factors is not a substitute for a suitable assessment of the ground conditions or geotechnical parameters (particularly permeability) or for an appropriate seepage assessment. Changes in assumptions relating to permeability or seepage are likely to cause a much greater magnitude of change in the calculated piezometric pressures, hydraulic gradients and flow rates than would be the case if partial factors were applied to the actions.

4.4 SEEPAGE ASSESSMENTS

Seepage assessments are the method by which pore pressures in the ground are estimated. Understanding the likely seepage scenario for a design situation can be complex and guidance on this is given in ILH 8.3.1 and ILH 9.7.2. Pore pressures distributions will depend on a variety of factors including:

- external water levels
- permeability and stratification of the embankment and foundation soils (in particular any layering or variations in permeability or anisotropy)
- duration of hydraulic loading
- precipitation infiltration.

In addition to the internal pore pressure distribution, seepage assessments may also provide exit gradients and seepage flow rates that are important for hydraulic limit states and drainage design.

The assumption of steady state seepage conditions is only likely to be necessary for persistent design situations. In most transient situations likely to be encountered in the UK or Ireland the duration of any flood is unlikely to be sufficient to create steady state conditions. In these situations a transient seepage assessment may provide a more realistic estimate of the pore pressures within the embankment. However, such calculations are more complex and require a more detailed understanding of the soil properties. For this reason, a steady state seepage assessment may be preferred in other design situations as it will generally give a safe solution.

Seepage analyses are discussed in detail in ILH 8.3.1. The distribution of pore-pressures may be determined based on past experience, site specific monitoring or calculation. Calculation may be by the graphical flow net method or by the use of numerical analysis. As with all complex analyses, the designer must be satisfied that the assumptions and limitations of the software are understood and that the method is applicable to the situation being modelled.

A benefit of more advanced numerical methods is that consolidation-induced pore pressures can be combined with seepage pore pressures. In such situations the results can be coupled to a traditional slope stability program to evaluate mass stability.

When undertaking a seepage analysis for a design situation which incorporates a flood event, it is crucial that, in addition to the design water levels, consideration is given to the variation in water level over time. This variation is typically represented by a hydrograph.

For a given design situation involving flooding, a hydrograph with a maximum water level corresponding to the characteristic water level of that design situation would equate to a ‘characteristic’ flood hydrograph. As set out in Section 4.1, in order to provide a margin of safety on the effects of water the design water level should be more onerous than the characteristic water levels. The 'characteristic' flood hydrograph is not applicable in the ULS design situation. If possible a hydrograph with a maximum water level equal to the design water should be calculated from hydraulic modelling. However in other circumstances calculation of the design hydrograph may not be appropriate so a 'design' hydrograph may be generated by transforming the characteristic hydrograph (by 'stretching') so that the maximum water level on the hydrograph equates to the required design water level. Given the importance of flood duration, particularly in a transient seepage analysis, it may also be appropriate to stretch the duration of the hydrograph, particularly the duration of the peak water level, to ensure that a suitably cautious pore pressure profile is calculated.

This approach to transforming hydrographs is illustrated in Figure 4.3.

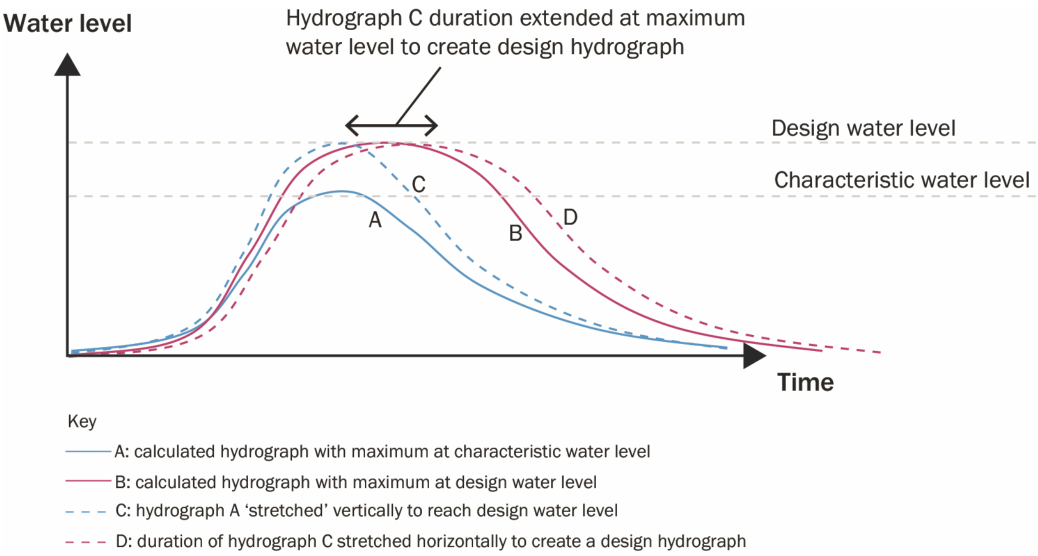


Figure 4.3 Determination of design hydrograph

No guidance is available on applying margins to the duration of maximum water level. The importance and effect of this will depend upon the permeability of the soils in the embankment and foundation soils. The designer should use experience and judgement to determine when the situation may be sensitive to the duration of loading. Again, the overriding criteria is to determine a design pore pressure distribution, which is the most onerous that could occur in the lifetime of the structure, with a probability of exceedance of 1%.

An important material property in seepage analysis is permeability. This is a parameter that is difficult to assess and to do so requires that relatively specialist testing be undertaken (ILH 7.8.3.5). Both the relative difference in permeability between soil zones and the absolute values of permeability will influence the pore pressure distribution. The designer must take great care to select a suitably cautious distribution of permeability throughout the embankment and foundation soil. In particular it is possible that zones of differing permeability could lead to high localised rates of seepage and hydraulic gradients that may cause internal erosion or hydraulic heave, or low permeability layers could constrain flow and lead to zones of higher pore pressure elsewhere, which cause uplift instability. So, it is appropriate to consider either upper or lower characteristic values of permeability, dependent upon the limit state being considered.

5 Ultimate limit states for flood embankments

5.1 INTRODUCTION

This section presents the ULSs that need to be considered when designing a flood embankment. For each limit state the following is included:

- a summary of the failure mechanism to be considered (ILH 3.5.2, and ILH Tables 7.44 and 7.45)
- the fundamental principle that must be satisfied in accordance with EN 1997
- details of the application of partial factors
- a summary table of current partial factors, including an allowance for Reliability Class
- typical actions to be considered.

5.2 OVERALL STABILITY (GEO)

For flood embankments, the potential GEO ultimate limit state mechanism that should be verified is overall stability (EN 1997-1 11). This may comprise analysis of circular or non-circular slips or the use of finite element analysis. ILH 8.6 provides details on the options available for undertaking slope stability analysis. ILH 9.10 provides a useful outline of the process involved in analysing embankment failure mechanisms. This section provides guidance on how the design input parameters to a stability analysis should be determined based on the Eurocode design approach.

The fundamental principle that must be satisfied using the Eurocode design approach for GEO ultimate limit states is:

$$Ed \leq Rd \qquad 1$$

(from EN 1997-1 Equation 2.5)

E_d = design value of the effect of actions

R_d = design value of the resistance to an action

For the limit equilibrium slope stability calculations typically used for embankment analysis, the overturning moment is the action effect and the restoring moment is the resistance.

As noted in Section 2.2.3, in order to determine the design values of action effects and resistances there are three sets of partial factors to consider:

- 'A' factors on actions (or on the effects of actions), γ_F
- 'M' factors on material strength, γ_M
- 'R' factors on resistance, γ_R.

EN 1997-1 allows partial factors to be applied using three different Design Approaches for GEO ultimate limit states. In Design Approach 1 (which has been adopted by the UK and Irish National Annexes), there are two combinations of factors to be checked (EN 1997-1 2.4.7.3.4.2(1)):

- Combination 1 (DA1-1): 'A1' + 'M1' + 'R1'
- Combination 2 (DA1-2): 'A2' + 'M2' + 'R1'.

A1+M1+R1 and A2+M2+R1 refer to sets of partial factors to be used in combination. The UK NA to EN 1997-1 tabulates the specific values of these partial factors (NA to BS EN 1997-1 Tables A.NA.3, A.NA.4 and A.NA.5) however the values given are identical to the recommended values in EN 1997-1. In general the Irish NA adopts the partial factor values recommended in EN 1997-1 and refers to the tables provided in EN 1997-1 rather than reproducing the tables of partial factors.

The NA to BS EN 1990 provides values for partial factors on actions for buildings (Tables NA.A1.2(A, B, and C)) and for bridges (Tables NA.A2.4(A, B, and C)). Although embankments do not readily fit into either of these categories ILH Tables 9.17 and 9.18 indicate that the factors for buildings are reasonable for adoption in flood embankment design. There is currently no differentiation in the factor on actions between persistent and transient design situations. Table 5.1 summarises the partial factors to be used based on the UK and Irish National Annexes to EN 1997-1, incorporating the proposed Reliability Class multiplier discussed in Section 2.3.1.1.

Table 5.1 Partial factors for GEO ultimate limit state verification

Parameter		DA1-1 Persistent and transient design situations			DA1-2 Persistent and transient design situations			Accidental design situations[1]		
Reliability Class		**RC1**	**RC2**	**RC3**	**RC1**	**RC2**	**RC3**	**RC1**	**RC2**	**RC3**
Factors on actions (or on the effects of actions), $\gamma_F \times K_{FI}$	**Permanent unfavourable**	1.28	1.35	1.42	1	1	1	1	1	1
	Permanent favourable[2]	1.05	1	0.95	1	1	1	1	1	1
	Variable unfavourable	1.43	1.5	1.58	1.24	1.3	1.37	1	1	1
	Variable favourable	0	0	0	0	0	0	0	0	0
Factors on material strength, $\gamma_M \times K_{MI}$	**Angle of shearing resistance**	1	1	1	1.19	1.25	1.31	1.06	1.12	1.18
	Effective cohesion	1	1	1	1.19	1.25	1.31	1.06	1.12	1.18
	Undrained shear strength	1	1	1	1.33	1.4	1.47	1.12	1.18	1.24
	Unconfined strength	1	1	1	1.33	1.4	1.47	1.12	1.18	1.24
	Weight density	1	1	1	1	1	1	1	1	1
Factors on resistance, γ_R		1	1	1	1	1	1	1	1	1

Notes

1 EN 1997 does not currently provide factors on material strength for accidental situations. However a proposal for the future revision of EN 1997 is that the factors on material strength for accidental situations will be the square root of the value adopted for persistent/transient design situations (before any Reliability Class multiplier is applied). This is the basis of the values in the table.

2 Partial safety factors based on the inverse of the reliability multiplier used for favourable actions.

In frictional materials, the mass of the soil simultaneously has two effects:

- acting unfavourably – as the driving force for instability
- acting favourably – providing resistance through increased effective stresses in the ground.

In theory this means different factors would need to be applied to each action effect. However, to address situations such as this, within Eurocodes there is a 'single source principle' (EN 1997-1 2.4.2(9)). This allows the designer to apply a single partial factor to all actions arising from a single source. This means that either the factor on favourable actions or the factor on unfavourable actions should be applied to the entire soil mass. The most onerous of the situations should then be considered in design. Where an action is favourable, a lower characteristic value should be used and where an action is unfavourable an upper characteristic value should be used.

In practice, for flood embankments DA1-2 will typically be critical for stability. Exceptions could occur where very high imposed actions (eg building load, or especially high traffic surcharges) are applied.

Table 5.2 identifies the main actions to be considered for a typical flood embankment. The designer should check whether a specific situation requires any additional actions to be considered.

Table 5.2 Typical actions for embankment stability

Action	**Type**	**Favourable or unfavourable?**	**Notes**
Soil mass	Permanent	Both	Analyse entire soil mass as both favourable and unfavourable and take the most onerous case. Use upper characteristic soil weights when unfavourable, and lower characteristic soil weights when favourable.
Imposed load	Permanent or variable	Either	Analyse as either favourable or unfavourable and take the most onerous case. (Note that the partial factor on favourable variable actions is zero).
Water pressures	Permanent	Unfavourable	As outlined in Section 4.3, for embankment stability design (DA1-2) it is recommended that water pressures are assessed directly from design water levels so no direct factoring is required. Where factoring water pressures is considered appropriate by the designer, water pressures would typically be assumed as permanent.

The material strength parameters used in an analysis must be modified by the appropriate partial factor depending upon which Design Approach combination is being considered.

Having established the inputs, the analyses should be carried out and the design actions checked to ensure that they do not exceed the design resistance using one of the methods outlined in ILH 8.6. A number of publications provide detailed guidance on undertaking slope stability calculations to EN 1997, including Bond and Harris (2008) and Frank *et al* (2005).

Where the analysis indicates insufficient resistance is available, the embankment geometry or the specification of the embankment fill material should be amended. ILH 9.9 includes suggested options for enhancing the stability of embankments in different design situations.

5.3 BUOYANCY (UPL)

For flood embankments the UPL ultimate limit state comprises loss of equilibrium of a block of ground or a structure due to uplift by water pressure. This situation is likely to be applicable where ground water flow results in elevated pore pressures beneath an impervious layer (this could be soil or a structural element, such as erosion protection or spillway). The case of a highly permeable layer in hydraulic continuity with the external water level overlain by a less permeable layer is illustrated in Figure 5.1 as an example.

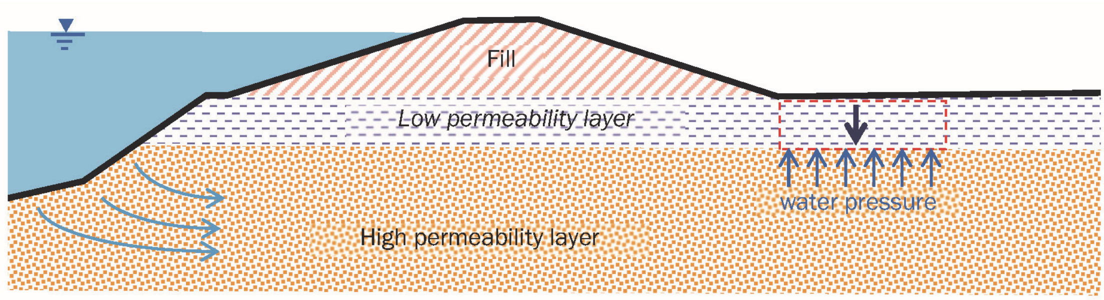


Figure 5.1 Example of UPL ultimate limit state

Verification of UPL ultimate limit state is carried out by checking that:

$$V_{dst,d} \leq G_{stb;d} + R_d \qquad 2$$

(from EN 1997-1 Equation 2.8)

$V_{dst,d}$ = combined design value of destabilising permanent and variable vertical actions

$G_{stb;d}$ = sum of the design value of the stabilising permanent vertical actions

R_d = design value of any additional resistance to uplift

The destabilising action generally comes from water pressure acting on the underside of a structure or low permeability soil layer. As discussed in Section 4.3, design values of water pressure may be directly assessed based on design water levels, in which case it is not necessary to apply any further factors to obtain the design value of destabilising vertical actions due to water. However adding a margin to the waterside water levels may only have a small effect on the water pressures at the toe of the embankment, which is typically where UPL is most critical. UPL is a limit state where it may be appropriate to calculate design values of water pressure based on applying the factor for destabilising permanent actions to characteristic water pressures.

Based on current partial factors, it is also recommended that a model factor of 1.1 is applied to the destabilising water pressure when characteristic values are factored to obtain design water pressures.

The designer must check whether there are any particular circumstances that mean additional destabilising actions are present (eg structural forces). If so, then the appropriate values of partial factors should be used to obtain design actions from the characteristic actions (NA to BS EN 1997-1 Table A.NA 15 and EN 1997-1 Table A15).

Stabilising permanent actions would typically comprise the vertical component of self-weight of the soil or other impervious layer. These values should be factored using the partial factor values recommended in the appropriate NA. Depending upon the geometry of the particular situation, failure may require shearing of the soil. In this case the soil strength provides a resistance to uplift. To determine the design value of the resistance the soil strength provides, the soil strength should be factored based on the values in NA to BS EN 1997-1 Table A.NA 16 or NA to IS EN 1997-1 Table NA.2. If further restraint to uplift is provided by structural elements such as piles or anchors then the value of design resistance that element provides should be assessed using the recommended partial factors in the National Annexes.

Table 5.3 summarises the partial factors to be used from the National Annexes, incorporating the proposed Reliability Class multipliers.

Table 5.3 Partial factors for UPL ultimate limit state verification

Parameter		Persistent and transient design situations			Accidental design situations[1]		
Reliability class		**RC1**	**RC2**	**RC3**	**RC1**	**RC2**	**RC3**
Factors on actions (or on the effects of actions), $\gamma_F \times K_{FI}$	**Permanent unfavourable**	1.05	1.1	1.16	1	1	1
	Permanent favourable[2]	0.95	0.9	0.86	1	1	1
	Variable unfavourable	1.43	1.5	1.58	1	1	1
	Variable favourable	0	0	0	0	0	0
Factors on material strength, $\gamma_M \times K_{MI}$	**Angle of shearing resistance**	1.19	1.25	1.31	1.06	1.12	1.18
	Effective cohesion	1.19	1.25	1.31	1.06	1.12	1.18
	Undrained shear strength	1.33	1.4	1.47	1.12	1.18	1.24
Factors on resistance, γ_R	**Anchorage resistance**	Refer to NA for anchor design factors					
	Tension pile resistance	Refer to NA for pile design factors					

Notes:

1 EN 1997 does not currently provide factors on material strength for accidental situations. However a current proposal for the future revision of EN 1997 is that the factors on material strength for accidental situations will be the square root of the value adopted for persistent/transient design situations (before any Reliability Class multiplier is applied).

2 partial safety factors based on the inverse of the reliability multiplier used for favourable actions.

A summary of typical actions considered in the verification of uplift stability is given in Table 5.4.

Table 5.4 Typical actions for verification of uplift stability

Action	Type	Favourable or unfavourable?	Notes
Soil mass	Permanent	Favourable	Lower characteristic soil weights to be used.
Imposed load	Permanent or variable	Either	The direction of loading should be considered to assess whether it is favourable or unfavourable. The partial factor on variable favourable actions is zero.
Water pressure	Permanent	Unfavourable	As outlined in Section 4.3, water pressures are generally assessed directly from design water levels so no direct factoring required. However where factoring water pressures is considered appropriate by the designer, water pressures should typically be assumed as permanent and a model factor of 1.1 also applied.

Practical options for mitigating the risk of uplift are discussed in ILH 9.7.3.

5.4 HYDRAULIC HEAVE AND INTERNAL EROSION (HYD)

For HYD to be relevant there needs to be a hydraulic gradient (and consequently seepage) within the particular design situation. In hydrostatic conditions there is no need to check HYD. Hydraulic heave (Section 5.4.1) and internal erosion (Section 5.4.2) are two different mechanisms within the HYD limit state.

A crucial aspect to the assessment of HYD ultimate limit states is a reliable assessment of the seepage profile in the embankment and underlying soils. As discussed in ILH 8.3 there are a number of factors that will influence a seepage analysis. Of these the distribution of permeability within the structure is probably the most difficult to assess while having a significant influence on the resulting seepage profile. The designer must ensure that a suitably cautious and safe assessment of the hydraulic conditions has been made and verify that the subsequent pore pressure distribution is appropriate.

5.4.1 Hydraulic heave

Failure by heave occurs when upwards seepage forces act against the weight of the soil, reducing the vertical effective stress to zero. Soil particles are then lifted away by the vertical water flow and failure occurs (boiling).

Verification of hydraulic heave is undertaken by checking that for every relevant soil column either:

$u_{dst;d} \leq \sigma_{stb;d}$ (design using total stresses)/method 1

or

$S_{dst;d} \leq G'_{stb;d}$ (design using submerged weight)/method 2

$u_{dst;d}$ = design value of the destabilising total pore water pressure at the bottom of the column

$\sigma_{stb;d}$ = design value of the total vertical stress at the bottom of the column

$S_{dst;d}$ = design value of the seepage force in the column

$G'_{stb;d}$ = design value of the submerged weight of the column

Partial factors on the permanent and variable favourable and unfavourable actions are given in NA to BS EN 1997-1 Table A.NA.17 and EN 1997-1 Table A17.

The inequalities reproduced here do not give an indication of how partial factors should be incorporated to determine the design actions and resistances. This uncertainty is discussed in more detail by Simpson *et al* (2011), particularly with respect to the design water action.

If it is assumed that factors are applied to the characteristic total pore pressure for method 1 and characteristic seepage force for method 2 to derive the design action then the result is that in all cases the design water pressure corresponding to the potential head loss over the column based on method 1 (total stress method) is smaller than that for the method 2 (submerged weight method), ie method 1 gives a higher equivalent factor of safety. This is the method recommended by Bond and Harris (2008). However it is considered that this approach is overly conservative for flood embankments and so method 2 is recommended. Further discussion of this is given in Bond and Harris (2008) and Frank *et al* (2005).

The uncertainty in how to apply partial factors is avoided if water pressures are directly assessed on the basis of design water levels since factoring of water actions is not required. This is the recommended approach for flood embankments. However, as discussed in Section 4.3, if the designer considers that this approach does not provide a sufficient margin of safety, it is recommended that method 2 is used with the design seepage force calculated by applying the factor on unfavourable actions to the characteristic seepage force.

The benefit of directly assessing design water pressures on the basis of design water levels is that computer analysis can readily make a comparison between contours of design pore pressure or seepage force and contours of stress to verify that the limit state is not exceeded.

The stabilising action at a given position in the embankment comprises the total vertical stress or effective vertical stress, depending upon the analysis method. The design value is calculated by factoring the characteristic stress by the factor on permanent favourable actions given in the National Annexes.

Table 5.5 summarises the partial factors to be used for the HYD ultimate limit state. These are derived from the partial factors given in the UK and Irish National Annexes after including the different Reliability Class multipliers.

Table 5.5 Partial factors for HYD ultimate limit state verification

Parameter		**Persistent and transient design situations**			**Accidental design situations**		
Reliability Class		**RC1**	**RC2**	**RC3**	**RC1**	**RC2**	**RC3**
Factors on actions (or on the effects of actions), $\gamma_F \times K_{FI}$	**Permanent unfavourable**	1.28	1.35	1.42	1	1	1
	Permanent favourable[1]	0.95	0.9	0.86	1	1	1
	Variable unfavourable	1.43	1.5	1.58	1	1	1
	Variable favourable	0	0	0	0	0	0

Note:
1 Partial safety factors based on the inverse of the reliability multiplier used for favourable actions.

A summary of typical actions considered in the verification of hydraulic heave stability is given in Table 5.6.

Table 5.6 Typical actions for verification of hydraulic heave stability

Action	**Type**	**Favourable or unfavourable?**	**Notes**
Soil mass	Permanent	Favourable	Lower characteristic soil weights to be used.
Imposed load	Permanent or variable	Either	The direction of loading should be considered to assess whether it is favourable or unfavourable. The partial factor on variable favourable actions is zero.
Water pressure	Permanent	Unfavourable	As outlined in Section 4.3, water pressures are generally assessed directly from design water levels so no direct factoring required. However where factoring water pressures is considered appropriate by the designer, water pressures should typically be assumed as permanent.

Where verification of HYD indicates a potential instability, the design should be modified. This typically involves implementing measures to decrease the hydraulic gradient. Practical options for mitigating the risk of hydraulic heave are discussed in ILH 9.7.3. As discussed in Section 5.4.2, there are a number of mechanisms for internal erosion. Prevention of heave does not mean that internal erosion will not occur so each of the mechanisms discussed in the next section should also be considered.

5.4.2 Internal erosion

ILH 3.5.2.2 identifies and describes four different internal erosion mechanisms that potentially exist within a flood embankment or at a foundation soil interface. These are identified as follows:

- contact erosion
- suffusion
- backward erosion
- concentrated leak erosion.

ILH Figures 3.176 to 3.180 provide illustrations of how internal erosion may affect a flood embankment.

Two conditions must be met for internal erosion to occur:

1. Particles detach as hydraulic shear stresses exceed resistant contact forces.
2. Once detached, particles must be transported through the soil.

ICOLD (2014) provides extensive discussion of internal erosion mechanisms and methods of evaluation.

Internal erosion is driven by seepage through the soil and is influenced by geometric conditions. Internal erosion can be controlled through suitable application of filter criteria for different materials within an embankment, the foundation soils or at the interface between the two. ILH 8.5 describes in detail the mechanisms by which these methods of internal erosion occur, methods of calculating the critical hydraulic gradient and filter criteria for design.

EN 1997 10.4(1) states that *"Filter criteria shall be used to limit the danger of material transport by internal erosion".*

Where filter criteria are not met, EN 1997-1 10.4(5) requires that the designer must ensure that the design value of hydraulic gradient is **well below** the gradient at which particles begin to move. No partial factors are provided in EN 1997-1 with which to assess this limit state. The design should be verified on the basis of an overall factor of safety (FOS) using current methods. For the purposes of this guide FOS is defined as 'critical hydraulic gradient' divided by 'characteristic hydraulic gradient'.

As reported in USACE (1993) values for the target FOS can range anywhere between 1.5 and 15. However, it also notes that the FOS is generally in the range of 2.5 to 5. In contrast advisory guidance by DWA (2011) provides a set of factors to be applied to the characteristic and critical gradients for different erosion methods for persistent, transient and accidental limit states. The equivalent FOS for these partial factors ranges between 1.5 and 2 for the persistent limit states.

At present there is no clear guidance on the recommended FOS to use for controlling internal erosion. ICOLD (2014) has compiled the latest thinking in terms of assessing internal erosion risk and critical gradients, but further work is needed to provide clarity on target factors of safety. As an initial recommendation, FOS = 5 is proposed as a suitably cautious value to use for most design situations when filter criteria are not met. However, as reported by Skempton and Brogan (1994), a factor of 10 may be appropriate in gap graded soils.

ILH 9.8 outlines a process for assessing and mitigating the risk of internal erosion and seepage.

5.5 ASSOCIATED STRUCTURES

5.5.1 Ultimate limit state

This guide is primarily aimed at the design of flood embankments. However there will be occasions when embankments are constructed in combination with other structures such as crest walls, embedded cut-off walls and spillways. A great benefit of the Eurocode approach is that it gives a consistent method for the design of both earthworks and structures. ILH 9.15 discusses the implications of structures associated with flood embankments. The ULS for structures, in particular retaining walls, are discussed in EN 1997-1 and more detailed guidance on analysis and design is given in Chapman (2000) and Gaba *et al* (2003).

As with flood embankments, clear definition of the relevant design situations is crucial to the design of associated structures. In general the design situation applicable to flood embankments will also be applied to the associated structures. However in addition to verifying overall stability and uplift stability, the STR ultimate limit states must also be verified.

For these structures it is important that both DA1-1 and DA1-2 are considered. The main difference for DA1-1 comes from the need to factor all actions, which will include water pressures. As discussed in Section 4, it is generally desirable to avoid factoring water pressures directly. For wall design this is important since the frictional nature of soils means that as well as increasing the direct action, it has an additional effect on soil shear strength and hence the resulting earth pressures. However it is still important to ensure a margin of safety is applied to water pressures in structural design. For flood embankments the proposed approach is to add a margin of safety to the characteristic water level to determine the design water level. As with embankments, for structures such as crest walls it is feasible that for a given design situation water at, or close to, the top of the wall is the characteristic level. Since water will overflow and will not rise above the top of the wall (unless some allowance is made for overtopping flow) adding a margin to obtain design water levels is not considered appropriate. For DA1-1 it is recommended that the effect of the action (ie the resulting bending moments and shear forces in the structure and the resulting moments and forces acting on the structure) based on the characteristic water level is factored by the partial factor on permanent actions to obtain a design action effect. For DA1-2 design water pressures should be based on design water levels. The overall stability and design resistance of the structural element should then be verified against the design action effects from both DA1-1 and DA1-2.

5.5.2 Transitions

The designer of flood embankments with associated structures should pay particular attention to the detailing of transitions, in addition to overall strength and stability. The risks primarily comprise the creation of localised pathways for internal and external erosion, and ILH 9.11 discusses in detail the requirements for transition design. One potential source for localised pathways is differential settlement between adjacent earthworks or structures. Although assessment of settlement is typically considered to be a SLS, this is an example where exceeding an SLS could lead to ULS conditions if sufficient internal erosion is allowed to occur through gaps created by differential settlement such that the stability of the embankment is compromised.

6 Serviceability limit states for flood embankments

6.1 IDENTIFICATION OF SERVICEABILITY LIMIT STATES

Serviceability relates to ensuring acceptable ongoing performance of a flood embankment rather than overall stability. However, in the case of flood embankments, there are situations where apparent serviceability considerations can adversely affect the performance of the embankment at the ULS.

ILH 9.12 identifies three particular aspects of serviceability:

- settlement and rutting
- desiccation cracking
- animal burrowing.

Settlement and rutting can lead to a reduction in the overall level of protection the flood embankment provides since the crest level will be lower. In addition localised settlement may create a low-point away from a designated spillway that could concentrate flow in a flood event and create external erosion problems, which lead to embankment instability.

Desiccation cracking and animal burrowing both act to increase soil permeability and so allow localised increased in seepage rates that could promote internal erosion or create elevated pore pressures that affect overall stability.

A further SLS not identified in ILH 9.12 is seepage discharge rates. Even if the embankment can be shown to be stable with high rates of flow through the embankment, the visual appearance of water seeping through during a flood may create concern to local residents and excessive discharge rates may also affect the amenity of areas on the landward side of the embankment. Acceptable seepage rates, potentially linked to the capacity of landward drainage to deal with the water flow, should be agreed on a project basis.

6.2 ASSESSING SLS CRITERIA

Identifying acceptable limits on the various SLS mechanisms should be undertaken on a project specific basis to ensure the design and construction effect to mitigate SLS risks is appropriate to the overall scheme requirements and level of project risks. Unduly tight SLS criteria may result in a significantly more expensive initial design, when a regime of ongoing inspection and maintenance may prove more cost effective. In other situations exceeding a particular SLS criteria may require disproportionate levels of remedial works (or present the risk of a ULS failure), in which case designing to avoid the limit state in the first place may be the client's preferred approach.

ILH 9.12.1 discusses the design of embankments to manage settlements and rutting. Analytical methods of calculating potential settlement are discussed in ILH 8.7.

Limit state criteria relating to desiccation or animal burrowing cannot be readily predicted, so in these situations the aim of the design is to reduce the risk of them occurring. For more details see ILH 9.12.2, ILH 9.12.3 and Frith *et al* (1997).

7 Reporting and design input to construction and operation stages

An important part of designing in accordance with Eurocodes is the need for clear communication of design decisions between project members and the need to monitor and review construction. This is one aspect of the design process where the level of input will vary depending upon the risk categorisation of the flood embankment.

7.1 REPORTING

EN 1997-1 introduces the requirement for preparation of two specific reports, the ground investigation report (GIR) and the geotechnical design report (GDR). The requirements for the content of these reports are provided in EN 1997-1 3.4 and EN 1997-1 2.8 respectively. Although not specifically aligned to the Eurocode requirements, ILH Table 7.13 and ILH Table 9.7 gives useful checklists of information to be included in geotechnical reports at different project stages.

The GIR comprises a collation of all factual and derived data (including derived soil parameters) available for the site. The intention of the GDR is to communicate the assumptions, data, method of analysis, design situations, and verification of the relevant limit states.

The level of input required to prepare these reports should be proportional to the size and risk classification of the project. For higher risk structures a comprehensive presentation of the design situations and associated design actions considered in the design should be provided, along with critical assumptions that may affect construction and any requirements for construction supervision and monitoring.

ILH Table 9.6 lists the overall reporting required for flood embankment design by stage. Geotechnical reports are typically produced at detailed design stage. However, interim reports may be prepared at the conceptual design stage and some revision of the reports may be required during construction to reflect any changes made. This is so that the final documentation incorporated into the operation and maintenance stage is consistent with the construction works actually undertaken.

The increasing use of Building Information Modelling (BIM) is likely to facilitate improved communication of critical information between the various parties involved in a project.

7.2 CONSTRUCTION STAGE DESIGN INPUT

As stated in ILH 9.16, the requirement for design does not finish when construction documentation is complete. Designers should remain involved for the construction and operation stages of a flood embankment. The GDR should be provided before construction starts to give information and understanding about the key design assumptions and critical aspects of the design, and to define the extent to which site conditions and construction quality will be verified by site inspections, testing and monitoring. Guidance on these areas can be found in ILH 7.7.4, ILH 7.7.5 and ILH 7.9.9.

Where possible, input from the constructor should be sought at the concept and design stages of a project to ensure that construction issues are fully considered during the design process.

The designer should have an active role during construction. Activities may include:

- Reviewing the constructor's planned method of work to ensure the proposal does not conflict with the designer's assumptions, eg oversteepening/overwidening embankment for optimum compaction causing overload, rate of construction greater than rate of dissipation of pore pressures; excavating landside of an embankment that is subject to uplift pressures.

- Monitoring the quality of the construction operations in-line with the recommendations of the GDR. This may focus particularly on inspecting and reacting to the ground conditions encountered on site and verifying compatibility with design assumptions.

A more detailed list of the typical construction stage activities for a designer is listed in ILH 9.16.2.

As with reporting, the level of site monitoring and verification of site conditions will vary with the risk category of the structure. EN 1997-1 4.2.2 details a potential approach to site inspection and control for each geotechnical category.

7.3 OPERATION

Within the GDR, the designer should detail the maintenance required to ensure the safety and serviceability of the flood embankment. In particular any critical parts of the structure that require regular inspection should be identified, along with any potential serviceability risks, such as desiccation or animal burrowing, which may require ongoing maintenance to avoid exceeding SLS criteria.

Adequate communication of the design assumptions and construction records is important during the operation phase for assessing the impact of changes such as increased loading (due to climate change), deterioration of embankment condition, or changes in the design situation (eg due to construction of new houses in a protected area).

The designer's role may also continue beyond the construction stage to review the embankment performance, recommend remedial solution to any problems that occur or to evaluate the impact of any potential improvements proposed during the operation of the embankment.

8 Areas of further work

In preparing this guide, a number of issues have been identified with respect to designing flood embankments to EN 1997 that would benefit from further consideration and research. This section summarises these issues and considers whether they would be addressed by:

- UK/Ireland Community of Practice work on identifying good practice
- UK/Ireland research
- pan-European work on identifying good practice
- pan-European research.

8.1 RISK CATEGORISATION

This guide proposes a framework of risk categorisation for flood embankments and includes proposed Reliability Classes and partial factor multipliers in Section 2.3. Cautious multiplier values have been adopted in this guide due to an absence of data on which to base the proposals. Further research into the reliability index of flood embankments designed in accordance with EN 1997, and an assessment of the sensitivity of the reliability index to the various assumptions and partial factors would be valuable to inform the determination of revised multipliers in the future. This is considered appropriate as UK/Ireland research though it would draw on the research work undertaken in other European countries

Further work considering the ILH risk categories and reviewing the range of real embankments that are assigned to each category would be beneficial to verify that the proposed classification is achieving its intended purpose. Given the applicability of the ILH this would be pan-European work identifying good practice.

8.2 WATER PRESSURES

As identified in Section 4, determining appropriate design water levels and water pressures can be complex. Further research to understand the relationship between the magnitude of the margins applied to obtain design water levels and the resulting reliability of the embankment would be beneficial to validate the EG9 recommendation to avoid factoring water pressures.

In particular this work may inform selection of revised partial factors on actions to ensure adequate reliability of flood embankments is achieved when directly assessing design water levels and pressures. Given the differing design approaches adopted across Europe this is considered appropriate as UK/Ireland research.

8.3 INTERNAL EROSION

Section 5.4.2 discusses internal erosion. While ICOLD (2014) compiles the current methods of analysing internal erosion, there is a lack of clear guidance on the FOS to be used.

Research into the application of partial factors to be used to design against internal erosion and calibration of these values with approaches adopted in other countries would be beneficial. This may include consideration of varying the partial factors to be adopted, based on the grading of the embankment fill and foundation soil, and the duration and frequency of the design situation under consideration.

This research is expected to have broad appeal and would be appropriate as pan-European.

8.4 WORKED EXAMPLES

During the preparation of this guide it was identified by the PSG that publication of worked examples following the recommendations made here would be of great benefit to illustrate the EN 1997 process to flood embankment designers. Initially it is anticipated that this would be developed as UK/Ireland Community of Practice work on identifying good practice, but this could then be provided to the Eurocode EG dealing with worked examples for review and potential wider use.

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IS EN 1997-2:2007 *Eurocode 7: Geotechnical design – Part 2: Ground investigation and testing*

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This guide provides national guidance for practitioners in the UK and Ireland on the design of flood embankments to Eurocode 7: *Geotechnical design* (EN 1997). The need for this guide was identified during the production of the CIRIA *International Levee Handbook* (ILH) published in 2013.

This guide, which has been written to be used alongside EN 1997, and the ILH cover the design of new flood embankments and significant modification of existing embankments. It presents a summary of the typical embankment design process and at each stage identifies the relevant requirements of EN 1997, and information on how these requirements may be implemented. Appropriate signposting to the ILH is also made throughout.

Topics covered include a risk classification for embankments used to determine design and construction supervision approaches, the determination of appropriate design situations particularly in terms of design water levels, and an approach to assessing critical ultimate limit states.

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