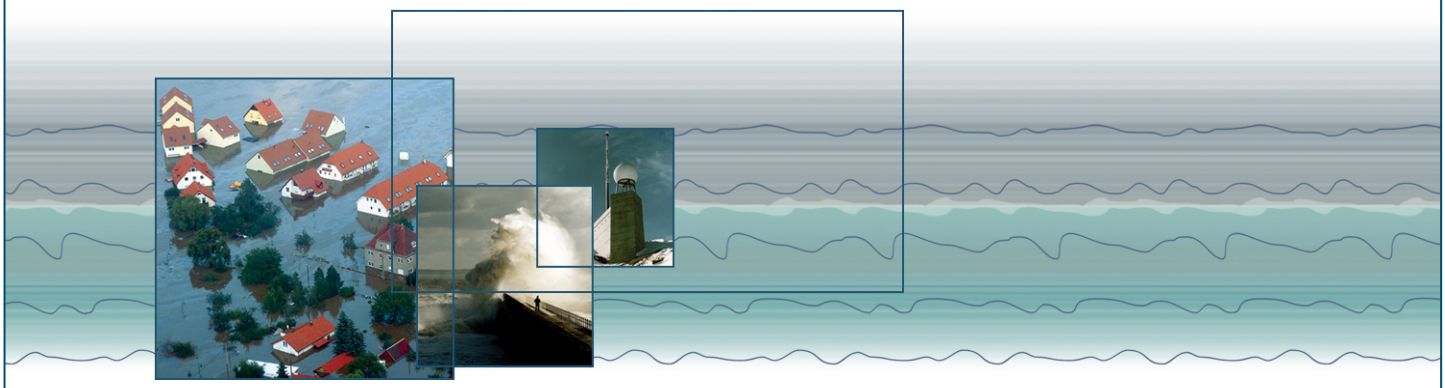
Integrated Flood Risk Analysis
and Management Methodologies



Failure Mechanisms for Flood Defence Structures

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DISCLAIMER

SUMMARY

This report describes failure mechanisms for generic flood defence structures or assets. The objective of this report is to provide a definitive listing of reliability equations for failure mechanisms of flood defence assets for use in flow system modelling.

Three principal load types are considered:

A. Water level difference across a structure
B. Wave loading
C. Lateral flow velocity

Flood defence assets are categorised under four main headings:

1 Foreshores, dunes and banks;
2 Embankments and revetments;
3 Walls;
4 Point structures.

The most useful description of a failure mechanism is a failure or reliability equation representing the strength of the defence, R, and the loading of the defence structure, S, in the form of a limit state equation:

$$Z = R - S$$

A 'signposting' matrix (Table 3.1) presents load type and asset type to reference summary templates within which details of the particular asset, failure mechanism and reliability equation(s) are provided. These templates can be found in Section 4 of the report.

The information presented within this report is intended for use in system modelling of flood risk. The information presented within the failure mode templates offers potential solutions for modelling defence structure behaviour under various load conditions. Since, in reality, many flood defence structure types vary in design, construction and condition, users should recognise the importance of understanding how the structure being assessed may behave in relation to the 'generic' solutions to failure modes presented here. In order to model system risk, the user will need to appreciate the failure process, the applicability to field conditions, how to integrate the failure mode within a fault tree structure for modelling and how to deal with any uncertainties within parameters used to model the failure process. Guidance on fault tree structures for different defences and uncertainties within modelling parameters may be found under FLOODsite Task 7 reports (FLOODsite Report refs T07-x-x).

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1. Introduction

1.1 Background

Countries bordering the North Sea like the Netherlands, Germany, Belgium, Denmark, and UK share a long history in fighting against flooding threats from the sea. The need toprotect these flooded-vulnerable areas which cover 40.000 km2 and home of 16 millions people has been rising since the tendency of increasing natural catastrophe threats and the important role of the threatened areas among those countries. The South Holland and the North Holland provinces which are also the most populous province in the Netherlands, the engine of country's economy, and home of important cities are in risk of flooding. The north coast of Lower Saxony State, the west coast of Schleswig-Holstein State, and the biggest seaport in Germany, Hamburg are potentially flooded during storm seasons. The south east coast of UK, the Flanders coastline, and the west coast of Denmark are also potentially affected by flooding.

A project called FLOODsite has been delivered by the European Commission to improvethe understanding of the causes and their complex interactions involving physical, environmental, ecological and socio-economic aspects of floods. Damage mitigation by applying necessary measures is one the project themes that needs integrated approaches in all aspects of application. Several measures have been implemented to mitigate the damage caused by severe storms and to protect the potentially flooded areas. Coastal defence, either natural or artificial, is one of the measures to deal with flood threat. Natural coastal defences in the form of natural beaches or dunes provide sufficient protection against flood. But, since the increasingly human interferences in the coastal area that largely influence the balance of these natural coastal defences, the safety is no longer guaranteed. An artificial coastal protection is in great need to assure the flooded-free areas in a developed environment. The Netherlands and Germany are two examples where besides the natural protection systems are in place, the artificial coastal protections are also widely implemented.

There are several types of artificial coastal protections ranging from the simple mound of stones or sand bags to the most complicated ones like storm surge barriers. Among all those, dikes have been widely used as flood protection to avoid inundation, particularly in the low lying areas like the Netherlands and the North coast of Germany. To meet its function, a dike should meet certain design conditions. The design conditions are derived from both hydraulic and geotechnical characteristics and their interactions. Water levels and waves are two main hydraulic loads that are very important in dike design while geotechnical stability is contributed to the strength of the dike body. The failure in identifying these loads and the geotechnical strengths can lead to failures or even disastrous situations (breaching).

Main materials of a dike consist of sand and clay. Based on their behaviour and natural characteristics, sand is used mainly for the core of the dike and clay for the cover (revetments) of the dike. Other materials such as artificial revetments (concretes, asphalts, stones, etc), stones (for the toe protection), filter materials (geotextiles, aggregates, etc), and even grass (to prevent surface erosion) are also largely used for dike constructions.

Every type of flood defence structure reacts differently under load. The factors that affect structure performance are equally varied. In order to undertake an effective assessment of the overall reliability of a defence it is essential to have a thorough knowledge and understanding of all key potential failure modes. Over recent years considerable effort has been devoted to improving our knowledge of how defences fail. However, gaps in knowledge still remain and differing approaches and levels of detail may be found in different national approaches.

This work forms part of FLOODsite Task 4 and links with FLOODsite Tasks 2, 5, 6 and 7.

1.2 Aims and objectives

The objective of this report is to provide a definitive listing of reliability equations for failure mechanisms of flood defence assets for use in flow system modelling. Modelling of flow systems is required as part of an effective approach for flood risk management. This requires that the behaviour of the flood defence structures is understood for different load conditions and flood defence asset types.

This document provides a definitive source of information upon which risk management tools and analyses may be based. Such a document may be updated and extended in the future as knowledge of structure performance and types increases. Within the lifetime of the FLOODsite Project (Feb 2009) corrections and additions will be recorded, with the view of releasing a revised edition. All comments and suggestions regarding content, whether corrections or proposing additional information, are welcomed and should be sent to either Andreas Kortenhaus (a.kortenhaus@tu-bs.de) or Mark Morris (m.morris@hrwallingford.co.uk).

1.3 Using this document

The information presented within this report is intended for use in system modelling of flood risk. The information presented within the failure mode templates offers potential solutions for modelling defence structure behaviour under various load conditions. Since, in reality, many flood defence structure types vary in design, construction and condition, users should recognise the importance of understanding how the structure being assessed may behave in relation to the 'generic' solutions to failure modes presented here. In order to model system risk, the user will need to appreciate the failure process, the applicability to field conditions, how to integrate the failure mode within a fault tree structure for modelling and how to deal with any uncertainties within parameters used to model the failure process. Guidance on fault tree structures for different defences and uncertainties within modelling parameters may be found under FLOODsite Task 7 reports (FLOODsite Report Numbers T07-x-x – accessible via the project website document system at www.floodsite.net).

1.4 Report Structure

Chapter 2 provides a brief introduction to the classification of flood defence asset type and hydraulic loading considered. These form the basis of the 'signposting' matrix presented in Chapter 3. Failure mechanisms for various asset type – hydraulic load combinations are presented in Chapter 4. A standard template has been used to present all information and to allow easy updating and extension in the future. A key function of this report has been to highlight what we do not know, as well as to collate details of what we do know. Key gaps in knowledge and methods are highlighted in Chapter 5.

1.5 Some general rules and definitions

The appropriate characterisation of failure processes of flood defences is a key component in effective flood risk management. The quantification of these failure processes is facilitated by a number of methods ranging from indicative equations to more rigorous process-based models, e.g. finite element methods. When set against performance targets such process-based models provide an indication of the structural performance of a flood defence structure. Examples of conventional methods for

analysing the reliability of defence structures are Owen's or Van der Meer's overtopping models or determining factors of safety for geotechnical slips using Bishop's slope stability methods.

Currently, there is an increasing interest to quantify the reliability of coastal and flood defences using probabilistic approaches, i.e. in terms of the complement of the probability of failure. The conventional reliability methods rely heavily on conservative, often expert judgement based estimates. The use of distribution functions for water levels and waves was the first step towards acknowledging a range of model outcomes and associated likelihood. In extension, reliability-based methods recognise a range of values for all the (partly still expert judgement based) parameter estimates involved in such models. An additional benefit is that it provides a measure that maps defence performance to a common dimension. Those results support comparisons highlighting influential failure modes and flood defence sections.

A central concept in reliability-based assessment of flood defences is a limit state equation. This equation links the performance target to the processes that lead to failure to fulfil that target. The limit state equation relates to the traditionally used Ultimate Limit State (ULS) and Serviceability Limit State (SLS) performance target approaches. Starting with a proper definition of the flood defence function and flood defence failure is therefore essential for meaningful results. The defence can fail in different ways, referred to as failure modes. The reliability of the defence is in this approach represented by a combination between the strength of the defence and the loading of the defence structure in the form of the following limit state equation:

$$Z = R - S$$

In which S expresses the loading and can for example be a function of the hydraulic loading conditions or the ground pressures behind a vertical wall. R represents the strength the flood defence structure and can be a function of e.g. the thickness of the revetment blocks or the crest level.

The concept of how to determine the overall probability of failure for a system of flood defences is illustrated by the nine-step-procedure in Figure 1.1. The first column shows the information input, the middle column the action that is carried out and the last column describes the result of the action. The contributions of FLOODsite Task 4 and 7 works within the procedure are marked with the red box.

This procedure can be approached from different user's perspectives. A designer may be more interested to tackle the weak links in the defence system with the implementation of an improvement scheme. That requires a performance assessment of the defence system in place, followed by optimisation of several improvement options. The choice of failure mode equations can be used to accommodate the stage of the design procedure, e.g. feasibility or detailed, and the level of data availability. Decisions for operational purposes also require a performance assessment of the defence system in place. The operational manager may have additional interest in the defence performance for other functions or for instance in the serviceability limit state.

Some additional explanation with the steps in Figure 1.1 is provides below:

Ad 1) The floodplain is defined based on elevations and determines the extent of the system of flood defences as well as the protected assets. Complex topography can obstruct the straightforward definition of system boundaries.

Ad 2) The flood defence types are the main components in the system for which the failure modes are separately analysed. The definition of the main flood defence types is such that all defence sections can be categorised according to these types.

Ad 3) For each defence type an analysis of failure modes is made. Investigation of historical failures, damage events or evidence of frequent repairs support the selection of the relevant failure modes. The failure modes and their mutual relations are then organised according to a fault tree. The fault tree serves to structure the probabilistic calculations in steps 7-9.

Ad 4) The division into stretches with very similar characteristics is the first step towards a more detailed schematisation of the defence system. Similar characteristics are e.g. its orientation to the wind directions, a particular kind of defence type or the use of a certain type of revetment.

Ad 5) The more detailed stretches can be as small as required for the detail of the calculations. Order of magnitude of the lengths can vary from 50 - 300 meter. The characteristics of one cross section are taken to be representative for one stretch.

Ad 6) The effort involved with the data collection depends on the detail of the required performance assessment. Feasibility design stages require indicative data for simplified failure mode equations and hence a relatively limited data collection effort. Detailed design stages on the other hand, build on detailed modelling and proportional data collection efforts.

Ad 7) Several calculations methods are available to calculate the probability of failure. Examples of level II calculations are e.g. FORM (First Order Reliability Method) or SORM (Second Order Reliability Method). Level III calculations are a Monte Carlo simulation or importance sampling. The result is a probability of failure for each failure mode for one cross section. It is also possible to consider the probabilities of failure of different failure modes for different source variables as e.g. as is done with fragility.

Ad 8) Some failure modes have a likelihood to occur simultaneously, this can be modelled with a correlation rather than treating the failure modes as independent events. The probabilities of failure of the separate failure modes can then be combined to one overall probability of failure.

Ad 9) Neighbouring cross sections often share similar characteristics and are therefore more likely to fail simultaneously – this issue also links with the expected breach width.

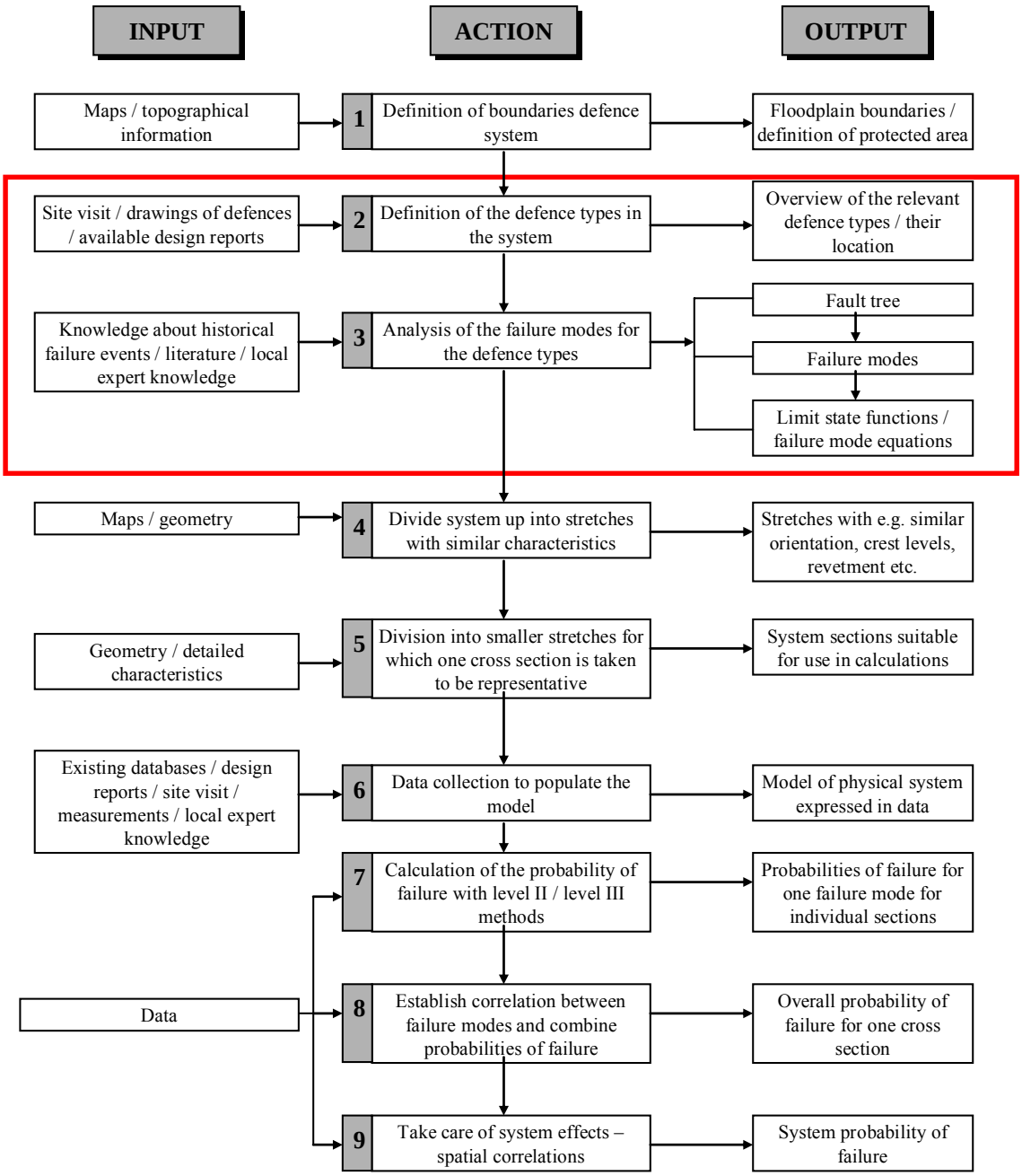


Figure 1.1 Simplified flow chart il`lustrating the determination of a failure probability for a flood defence asset

2. Asset Types, Hydraulic Loading and Related Issues

2.1 Asset Types and Hydraulic Loading

For the purpose of this report, flood defence assets have been categorised under four principal headings:

Asset types:

1. **Foreshores, dunes and banks;**
2. **Embankments and revetments;**
3. **Walls;**
4. **Point structures.**

In reality, many defences are composite, so will include elements from more than one (perhaps all) headings above. Nonetheless, it will generally be convenient to describe failure mechanisms under categories relating to these different structure types.

Within each of these types, there are many potential sub-divisions, often by principal material.

The simplest defence structure is a simple bank or wall composed essentially of main defence element, and foundation. Such simple structures are however very rare, as most structures use multiple layers / materials, each adapted to suit a particular purpose. Many existing defences, particularly in developed areas, will have been modified and adapted in time, so will feature multiple layers / elements.

In considering many defences, it will be convenient to distinguish between inner layers, required principally to provide mass and hence resistance against sliding; and outer layers required to resist direct or indirect flows / pressures. The most common outer layer on any bank / embankment is the revetment. It is important to note that a revetment is not a flood defence structure on its own, solely an adjunct to a bank or other man-made earthwork which can be attacked by water.

Three principal loading types are considered:

Hydraulic Loading:

1. **Water level difference across a structure;**
2. **Wave loading;**
3. **Lateral flow velocities.**

The actual hydraulic loading on any specific defence asset is likely to comprise one or more of these categories hence multiple failure modes (and methods of analysis) will be relevant to any given defence asset.

A summary matrix referencing the failure mode templates in relation to asset type and hydraulic load categories listed above may be found in Section 3 of this report. Section 4 then contains each of the failure mode templates with technical content. Use the summary matrix to identify which templates are of interest and then access Section 4 using the appropriate template reference given in the matrix.

The remaining content of this section offers additional comment on factors affecting hydraulic loading and asset performance.

2.2 Additional Factors Affecting Hydraulic Loading or Asset Performance

In reality, a flood defence system is subject to a complex mix of loading arising from a variety of sources. The condition and performance of the asset is also subject to change. The following sections provide a brief introduction to factors that <u>should also be considered</u> when modelling a flood defence

system. These factors have not been included directly within the *Failure Mechanism Descriptions (Chapter 4)*. However, some of these factors may be taken into account through variation in loading or other parameters included within the performance equations provided.

2.2.1 Differential loading

Rapid changes in water level may occur during floods or as a result of releases from reservoirs. This may cause adverse conditions that reduce the resistance (strength) of the system. By affecting the pore water pressure within the soil and by increasing the weight of the soil such events may precipitate geotechnical instability of the soil underlying protection. Problems can arise if there is a substantial difference between the permeability of an embankment and the underlying material. If, for example, an impermeable embankment sits on much more permeable soils and there is not provision for the adequate dissipation of pore pressures then failure may occur. Steep hydraulic gradients within embankments as a result of rapid changes in the external water level may lead to failure. It is common to find that many river banks fail not at the peak of a flood but during the recession when pore pressures do not dissipate quickly enough.

2.2.2 Impact of channel bends

The presence of a bend modifies the flow distribution in a channel. Bends induce secondary flows normal to the channel centreline and also modify the distribution of velocities in the downstream direction. This may lead to increased flow velocities adjacent to the bank of the channel and so increase the hydraulic loading on any protection work. It is common for the flow around bends to result in scour of the bed of a channel adjacent to the bank. By lowering the bed level at the toe of any protection this may increase the likelihood of failure.

The potential impact of a bend depends upon severity of the bend. This is frequently assessed using the parameter R/W, where R is the radius of curvature of the centreline of the bend and W is the channel width. For the purposes of revetment design it is often assumed that only bends with values of R/W less than 26 need to be considered. In the US Army Corps of Engineers Design Procedure for rip-rap the equation for stone size includes an empirical coefficient Cv to take account of the velocity distribution in bends. Cv takes the value 1 for strainght channels and the inside of bends. The value of Cv for bends is (Maynord, 1993):

$$Cv = 1.283 - 0.2 \log10(R/W) \text{ for } R/W<26$$

$$= 1 \text{ for } R/W>26$$

where R is the radius of curvature of the centreline of the bend
and W is the water surface width at the upstream entry to the bend

There are a number of different equations to predict scour at bends, see Melville and Coleman (2000). See Failure Mode Bc3.1 for details of a method to predict bend scour.

2.2.3 Vortex shedding

Vertical core vortices have been observed in physical models of coastal protection schemes where there is a sharp corner. The vortices are formed at some area of large curvature and are then shed and may track across the revetment. The vortices may impose a significant upward velocity component within the flow that may destabilise elements of the cover layer or may remove underlying filter material.

There is anecdotal evidence to suggest that similar, vertical-core vortices may be formed in the context of river training structures.

There are no known methods to predict whether such vortices will form or the magnitude of the vertical velocities if they do form.

2.2.4 Ship-induced currents

When considering the loadings imposed by flow velocities it may be necessary to take into account ship induced currents that can arise in navigable rivers and canals. The water motion produced by boats is complex, involving changes in water level, waves and currents to various degrees of magnitude and it can have a severe impact on the stability of bed and banks. In most cases the wave loading is the most critical of these loadings.

Two different types of current are produced by ship movement:

Return currents – these are parallel to the channel banks but in the opposite direction to the motion of the vessel,

Propulsion induced currents – these are the high velocity jets of water generated by the ship's propeller and can have a range of directions from parallel to normal to the bank.

Return currents: the loading imposed by return currents only occur during the time that the ship is passing that part of the bank but the current can impose a high shear stress to the bed or bank. The magnitude of the loading depends upon:

a) the speed of the ship,
b) the water levels generated by the motion of the ship,
c) the relative magnitudes of the cross-sectional wetted areas of the ship and the channel.

Methods to determine the speed of the return current are in PIANC (1987). Hemphill and Bramley (1989) provide the following indicative values for UK rivers.

Type of watercourse	Return current speed (m/s)
Smaller canal	<1
Larger canal	<1.5
Navigable river	2 to 3

Propulsion-induced currents: the loading imposed from propulsion-induced currents may be significant when they arise from a vessel starting to move or during manoeuvring. Damage to bed or banks, therefore, tends to occur in front of locks and mooring posts and near banks. The magnitude of the loading depends upon:

a) the strength of the propulsion system
b) the duration that the jet impinges on the bed or bank

This means that the lower the ship's speed the greater is the loading. Depending upon the orientation of the propeller relative to the bank, the loading can be due to:

a) shear stress – when the axis of the propeller is parallel to the boundary
b) hydrodynamic pressures – when the axis is normal to the boundary
c) a combination of both.

PIANC (1987) proposes an expression for the bottom velocity due to propeller jets starting from rest

$$U_b = \alpha\, 1.15 \left(\frac{P_d}{D_o^2} \right)^{0.33} \frac{D_o}{Z_b}$$

where α takes a value between 0.25 and 0.75, depending upon ship and rudder types

P_d is the engine power in kW

D_o is given by

$D_o = D_p$ for ships with propeller in a nozzle

$D_o = 0.7\, D_p$ for ships without nozzle

where D_p is the propeller diameter
Z_b is the vertical height from the boundary to the propeller axis.

In the absence of specific data a first approximation value of 2.5 m/s can be used.

In many cases field measurements will be required to assess the loadings with confidence.

2.2.5 Ship induced waves

The movement of ships produces a complex pattern of waves and currents which may impose a loading on any bed or bank protection.

As a vessel moves along a watercourse, it displaces water at the bow. In water bodies with confined widths two main water patterns occur:

a) primary system of return flow around the vessel,
b) secondary system consisting of oblique waves generated at the bow and stern of the vessel.

Primary system: The primary system consists of different elements:

a) super-elevation of water levels ahead of the vessel
b) depression of water associated with return flow past the vessel
c) recovery in water levels at the stern of the vessel

Secondary system: The secondary system consists of V-shaped patterns of waves generated at the bow and stern and their interference patterns. For most situations the angle of the individual wave crests to the bank can be taken as approximately 35^0. The amplitude of the waves depends upon:

a) the speed of the vessel
b) the size and shape of the vessel

The most reliable information on wave amplitudes is obtained from field measurement. Hemphill and Bramley (1989) provide the following typical values for UK conditions:

Channel type	Boat size (t)	Wave height (m)
Smaller canals	<80	<0.3
Larger canals	<400	<0.5
Navigable rivers	<40	<0.4

Hemphill and Bramley present the following design equations for the stability of rip-rap in boat generated waves:

Primary waves (transverse stem waves)

$$D_{n50} = 0.67 \frac{H_i}{\Delta (\cot \alpha)^{1/3}}$$

Secondary waves

$$D_{n50} = 0.56 \frac{H_i}{\Delta} (\cos \beta)^{0.5}$$

where D_{n50} is stone size
H_i is the height of the highest wave expected
Δ is the relative density of stone
α is angle of bank slope to the horizontal
β is the angle of the individual wave crests to the line of the bank, usually taken as 35°.

For concrete blocks the following equation provides a first estimate of the block thickness:

$$D = 0.43\ H_i\ (\cos\alpha)\ (\cos\beta)^{0.5}/\Delta$$

PIANC (1987) provides more detailed design equations.

2.2.6 Tension cracks

Geo-technical failure of the soil may occur if the resistance (strength) of the soil is reduced. The development of tension cracks within the soil may reduce the stability of the soil and may affect the critical failure geometry.

2.2.7 Surcharge loadings

The loading on bank protection may be increased by the placing of additional material on or near the protection. The weight of this additional material may contribute to the loading on the protection. An example of this is the storage of materials beside a sheet-pile river bank. By increasing the loading this may lead to failure of the sheet-piling.

2.2.8 Presence of trees or shrubs

The presence of trees or shrubs on an embankment may affect the stability of the structure, though their potential impact can be complex. The presence of root systems may increase the strength of the soil and hence increase soil stability. The presence of shrubs and particularly trees may, however, provide a surcharge loading to the system and hence reduce stability.

2.2.9 Vandalism

Where isolated protection units, such as riprap, are used, which are sufficiently light to be carried by hand, they may be vulnerable to vandalism. Individual stones from the cover layer may be removed, exposing the under-layers and hence leading to failure of the protection during subsequent floods. In places where this may be a problem it may be necessary to increase the weight of the stones, not on hydraulic grounds but to prevent removal.

2.2.10 Burrowing animals

Burrowing animals may precipitate a piping failure of an embankment. This is can present particular problems if the species concerned is protected in any way.

2.2.11 Debris / Ice impact

The effects of debris or ice impact on a structure can be significant. The summary matrix (Section 3) references hydraulic loading on the horizontal axis. In addition to water level difference, wave loading and lateral flow velocities, two further 'loading' categories are referenced, namely ice, debris and ship impact and operational failure. Hence, some guidance on potential failure modes due to debris / ice impact may be found within the failure mode templates of this report.

2.2.12 Transition

Where a revetment is protected then the limiting flow velocity for that protection can be established using the appropriate equations for the protection, for example, stone protection. If the part of the revetment is unprotected cohesive soil then it is impractical to design for no erosion under any flow condition and limited erosion during significant floods is likely to be acceptable. At the present there are no known reliable methods to associated flow conditions with erosion rates.

Erosion is frequently exacerbated at the transition between a protected and an unprotected length of bank. The mechanism appears to be increased turbulence in the area where the flow adjusts to the differences in hydraulic roughness and possibly bank alignment. There are no known methods to

predict this increased loading due to the transition. In the absence of such methods it is recommended that the design of bank protection should take account of the potential for erosion of the unprotected bank upstream and downstream of any protection.

For a discussion of how to design against erosion at transitions see Hemphill et al, 1989 and Przedwojski et al, 1995.

2.3 Time dependent change issues

2.3.1 Breach Formation

The failure mechanisms described within this report relate to the failure (structural or performance based) of specific components of a flood defence asset. For example, in relation to a flood defence embankment, these components might comprise the grass cover, toe stability or rip rap protection.

The performance of each of these components may be integrated within a risk based system models in different ways. Assumptions of complete failure (i.e. open breach) may be based purely on the failure of any component (i.e. grass or rip rap protection failed) or on a series or combination of component failures. Linking of these components is demonstrated through fault trees and is reported under Task 7 of FLOODsite. It is important that the assumptions made in assuming failure and also for the calculation of potential inundation as a result of failure are clearly understood. Within (current) system models, these assumptions are likely to simplify the science and calculation process considerably in order to allow realistic run times for modelling the performance of large catchments.

Work under FLOODsite Task 6 addresses the development of predictive breach models. These models simulate part or all of the embankment failure process, from initial wave overtopping or overflow, through to final breach dimensions. A flood hydrograph resulting from the breach is the most common output from such models. These models are linked with the failure mode information reported here in as much as different stages of the predictive modelling may draw on a number of the various component failure processes, either in limit state format or as a time varying process.

A review of current capabilities for breach modelling is reported under Task 6, M6.1, Report T06-06-01.

2.3.2 Deterioration Processes

Deterioration processes have not really been an integral part of conventional deterministic reliability analysis (todate). Process-based models for deterioration processes are less developed and organised than those describing reliability as a snapshot in time. Deterioration processes can be incorporated in the probability of failure by analysing which failure modes they affect. Deterioration can trigger seemingly irrelevant failure modes and are therefore sometimes confused with the failure mode.

Examples of the challenges that time-dependency introduces are:

- The representation in the probability of failure of failure processes that are dependent upon the history of loading. Such processes are: erosion and scouring issues, cracking and fissuring

- Process-based or statistical models that take dominant factors into account in the deterioration of structures, such as third party use, animal burrowing, and tree rooting, are relatively poorly developed. The statistical occurrence of the ‘loading’ by an animal population, third party use or a tree blowing over is one aspect of the problem. Another aspect is the physical quantification of the damage to the structure, and which failure modes that damage affects.

- Some variables in the failure mode models may not have a direct influence on the reliability in the light of the flooding problem. However, they can still have quite an important role in the overall performance of the structure. An example is vegetation, which, besides its protective function

during a storm also offers daily protection against third party use and rainfall. In that case, whole life cycle costing can offer a solution.

- The extent to which time-dependent models can be extrapolated to longer time horizons based on the limited time scope of laboratory experiments poses a problem.

The significance of deterioration / time dependent processes combined with the relatively poor level of understanding and modelling ability means that this is an area where research needs to be focussed during the coming years.

3. Failure Mechanism Matrix

3.1 The Matrix

'The Matrix' (Table 3.1) has been developed as a means of referencing information on each failure mechanism. The Matrix has been structured according to asset type and hydraulic load. Consideration has also been given to impact loading and operational failure of assets. Any particular asset type might be subject to more than one hydraulic load type. Equally, real flood defence assets often comprise hybrid structures that might fall into more than one of the Matrix structure categories.

The function of the Matrix is to act as a signposting system for the user to reference appropriate failure mechanism information.

3.2 System modelling – dealing with fault trees and uncertainties

This report presents the underpinning science for simulating defence structure failure modes within the context of system modelling of flood risk. The science presented here will aid modellers to assess system risk, but also requires the use of approporiate fault trees and consideration of parameter uncertainties in order to build an appropriate risk model. The development of fault trees and dealing with uncertainty is addressed under Task 7 of the FLOODsite project. Task 7 reports (Report reference format T07-x-x) may be accessed via the project website at www.floodsite.net

Flood Defence Failure Mechanisms Version 18_5		FAILURE MODES INDUCED BY HYDRAULIC LOAD CONDITIONS																													
		1	1.1	1.2	1.3	1.4	1.5	1.6	2	2.1	2.2	2.3	2.4	2.5	3	3.1	4	4.1	4.2	4.3	5	5.1
		Water level difference across structure	Erosion of surface by overflow	Bulk sliding or overturning of structure or element	Deep slip / slide	Shallow slip / slide	Piping and/or internal erosion	Crest level too low - overflow (functional failure)	Wave loading	Erosion of seaward face / slope	Bulk displacement (slip / overturning) of defence / seaward element	Local surface failure or element displacement	Erosion of landward face / slope	Crest level too low - overtopping	Lateral Flow Velocity	Erosion (scour) of bed or bank	Structure Impact	Ship or similar impact	Ice effects	Debris	Operational Failure	Operation or closing failure
FLOOD DEFENCE ASSETS	A Foreshores, dunes and banks																					
	Aa Sand beach and dune (fine granular material)		**Aa 1.1**					Ba1.6 (a/b)		**Aa 2.1a/b**			**Aa 2.4**	Ba 2.5		Ba 3.1		**Aa 4.1**	**Ab 4.2**	Aa 2.1a/b		
	Ab Shingle / gravel / rock beach or ridge (coarse granular material)		Aa 1.1				**Ab 1.5**	Ba1.6 (a/b)		**Ab 2.1a/b**			Ab 2.1a/b	Ba 2.5		Ba 3.1		**Ab 4.1**	**Ab 4.2**	Ab 2.1a/b		
	B Embankments and revetments																					
	Ba Homogeneous embankments (primarily cohesive materials, may include grass cover, and/or variable foundation)		**Ba 1.1**	Bb 1.2	**Ba 1.3a/b/c**	**Ba 1.4**	**Ba 1.5a/ai/aii/b/c/d**	**Ba 1.6 (a/b)**		**Ba 2.1a/b**	Bb 1.2	**Ba 2.3**	**Ba 2.4i/ii/iii/b/c/d**	**Ba 2.5**		**Ba 3.1**				Ba 2.1a/b		
	Bb Composite embankments, multiple layers etc (some cohesive materials)		Ba 1.1	**Bb 1.2**	**Bb 1.3a/b**	**Bb 1.4**	Ba 1.5ai/aii/b/c/d	Ba1.6 (a/b)		Ba 2.1a/b	Bb 1.2	Ba 2.3	Ba 2.4i/ii/iii/b/c/d	Ba 2.5		Ba 3.1				Ba 2.1a/b		
	Bc Revetment protection to embankments		**Bc 1.1**			Bc 1.4	**Bc 1.5**	Ba1.6 (a/b)		**Bc 2.1 a/b/c/d/f/g/h/j/k/m/n**		**Bc 2.3 a/b**	**Ba 2.4 iii**	Ba 2.5		**Bc 3.1 a/b/c/d**		**Bc 4.1**	**Bc 4.2**	Bc 2.1 a/b/c/d/f/g/h/j/k/m/n		
	C Walls																					
	Ca Mass concrete vertical or battered walls		Ba 1.1	**Ca 1.2 a/b/c/d**	Bb 1.3a/b		Ba 1.5ai/aii/b/c/d	Ba1.6 (a/b)		**Ca 2.1a/b**	**Ca 2.2 a/b**	**Ca 2.3**	Ba 2.4i/ii/iii/b/c/d	Ba 2.5		Ba 3.1		**Ca 4.1**	**Ca 4.2**	**Ca 4.3**		
	Cb Sheet pile, cantilever or tied back		Ba 1.1	**Cb 1.2a/b/c/d**	Bb 1.3a/b		Ba 1.5ai/aii/b/c/d	Ba1.6 (a/b)		Ca 2.1a/b	Ca 2.2 a/b	Cb 2.3?	Ba 2.4i/ii/iii/b/c/d	Ba 2.5		Ba 3.1		Ca 4.1		Ca 4.3		
	Cc Crown or parapet wall on structure		Bc 1.1	**Cc 1.2a/ai/b/c/d**	Bb 1.3a/b		**Cc 1.5** (and see Ba 1.5)	Ba1.6 (a/b)		Bc 2.1 a/b/c/d/f/g/h/j/k/m/n	**Cc 2.2a/b**	Cc 2.3	Ba 2.4 iii	Ba 2.5		Bc 3.1 a/b/c/d		Ca 4.1	**Bc 4.1**	Ca 4.3		
	D Point Structures																					
	Da Barriers			**Da 1.2**	Bb 1.3 (MD / DL / FB)		Ba 1.5ai/aii/b/c/d	Ba1.6 (a/b)			**Da 2.2**	Da 2.3		**Da 2.5**		Ba 3.1		**Da 4.1**	**Da 4.2 a/b/c**	**Da 4.3**		**Da 5.1 a/b**
	Db Sluices, gates			**Db 1.2**	Bb 1.3 (MD / DL / FB)		Ba 1.5ai/aii/b/c/d	Ba1.6 (a/b)			**Db 2.2**	Db 2.3		Da 2.5		Ba 3.1		**Db 4.1**	**Db 4.2**	**Db 4.3**		**Da 5.2**

Key

The entries in this table identify individual (or generic) families of failure mechanisms for each combination of structure and load type.

B Entries in bold are labled as generic, and may apply to more than one combination of structure and load type.

A shaded entry suggests that the failure mechanism is impossible or of such low probability as not to signify in realistic fault trees. Failure mechanisms may occur in series and/or parallel in any particular fault tree.

Table 3.1 Matrix of Flood Defence Asset Failure Mechanisms (version 18_6)

4. Failure Mechanism Descriptions

This section contains failure mode information for various combinations of hydraulic loading and flood defence asset type. Information for each combination is presented within a standard template comprising:

- Summary
- Diagram / Photo
- Reliability equation
- Loading and Resistance Equations
- Parameter Definitions
- Sources for failure mechanism equations / analysis methods
- Sources of uncertainty
- Remarks / Comments

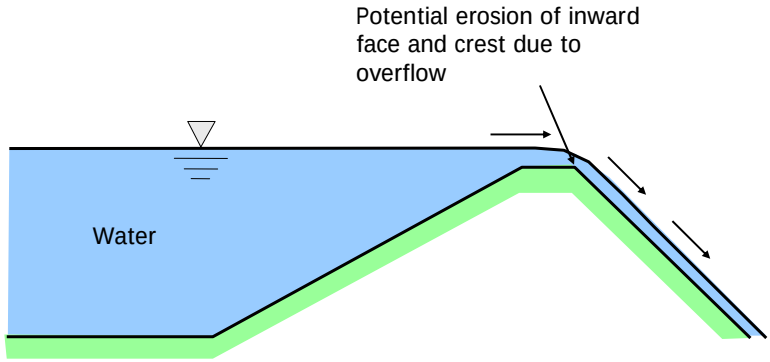
- Status of draft (history of development for individual templates)

The appropriate template for a given hydraulic load / asset type may be referenced through the 'signposting' Matrix (Table 3.1).

Readers are advised to consider the points raised in Chapter 2 before using template information for system modelling.

Aa 1.1 Erosion of cover of inner slope by overflow

Summary: Damage to inner (dry) slope when overflow discharge (or velocity) exceed limit for type / condition of grass cover on crest and inner slope. The "load" is the actual overflowing discharge, q_0. The "strength" is a critical discharge, q_c, that exceeds the resistance of the cover.



Reliability equation:
The reliability function can be expressed in terms of the computed overflow velocity or discharge and a critical or allowable velocity / discharge:

$$z = m_{qc} \cdot q_c - m_{q0} \cdot q_0$$

where:

q_0, q_c = the actual and critical overtopping discharges [m^3/sm] or [l/sm]
m_{q0}, m_{qc} = model factors for actual and critical discharges [-]

Loading equations:
Overflow given by broad crest weir equation:

$$q_0 = c_D \cdot c_V \ (h_{over}) \sqrt{2 \cdot g \cdot h_{over}}$$

Resistance (strength) equations:
Critical overflow, q_c, calculated from:

$$q_c = v_c^3 / (\tan \alpha_i \cdot C^2)$$

Roughness on inner slope may be estimated using critical velocity, v_c, and roughness length, k, by:

$$C = 25 \cdot (q_c / k \cdot v_c)^{1/6}$$

The critical failure velocity, v_c, at time, t_e, which causes failure of the grass surface is given by:

$$v_c = \frac{3.8 \cdot f_g}{(1 + 0.8^{10} \cdot \log t_e)}$$

Parameter definitions:

c_D = coefficient for weir shape, crest width [-]
c_V = dissipation coefficient [-]
h_{over} = depth of flow over (local) crest [m]
v_c = critical flow velocity [m/s]
α_i = angle of the inner slope [°]
C = roughness factor according to De Chézy [$m^{0.5}$ / s]
f_g = condition quality of grass [-], varying between: f_g = 0.7 for bad turf; or f_g = 1.4 for good turf
t_e = overflow duration [h]

Sources of failure mechanism equations / methods:
Hewlett et al (1985);

Sources of uncertainties in failure equations / input parameters: Guidance on model uncertainties; Identify data on parameter uncertainties, s.d., distribution types;
Remarks: See also Ba 1.6 and Bc 1.1

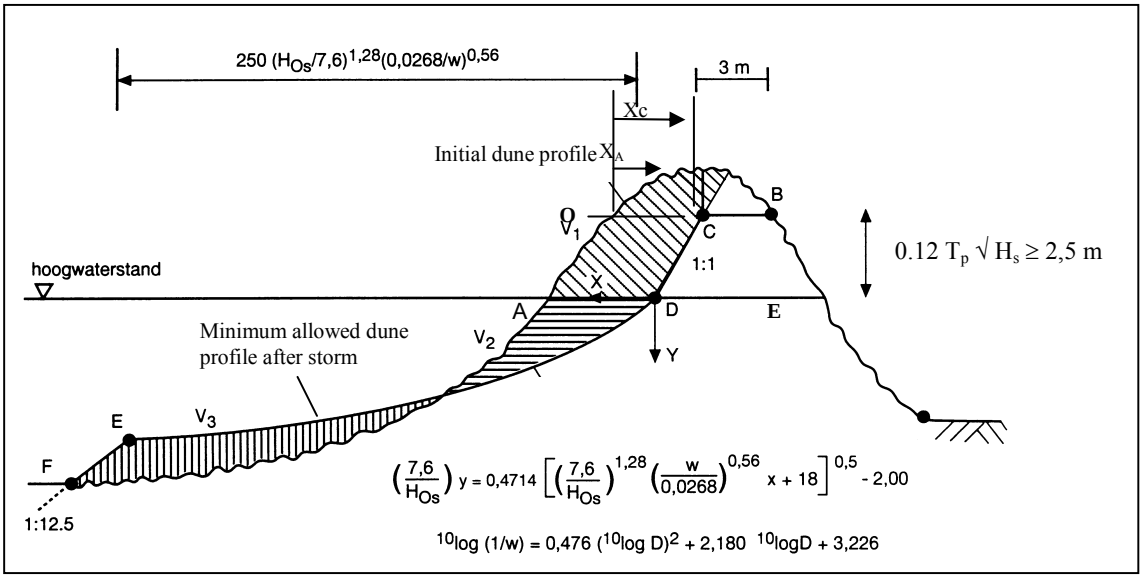
Status of Draft

Date	Revision	Prepared by	Organisation	Approved by	Notes
		Randa Hassan	PC RING		
25/01/ 07	v3_4_p03		LWI		edited

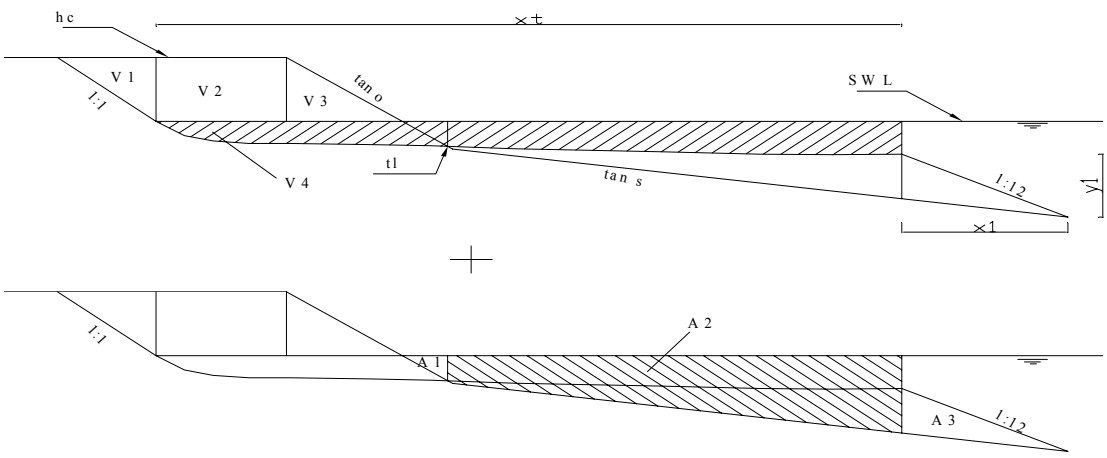
Aa 2.1a Erosion of seaward face of sand by waves

Summary: Direct wave-driven erosion of sand dunes should be approached similarly as shingle beaches (Ab 2.1): estimating the crest level and crest retreat and then calculating the probability of breaching. Condition grades may be included by a factor in the crest retreat. Breach occurs when post-eroded profile is insufficient to withstand storm conditions. The profile of the dune as a function of the loading conditions during storm is predicted with the model according to Vellinga (1986).

Sketch of dune profile



Simplified sketch



Reliability equation:

Limit state equation establishes whether the initial profile is sufficient to provide the minimum allowed profile. General form of the LSE is:

$$z = m_R \cdot d_w - m_S \cdot c_r$$

where :

d_w	=	the width of the dune [m]
c_r	=	the crest retreat caused by the storm [m]
m_R, m_S	=	model factors, taking account of the uncertainty involved with the simplified models [-]

Loading equations:	**Resistance (strength):**

The following expressions are used to calculate V_1, V_2, V_3, V_4, A_1, A_2 and A_3.

$$V_1 = \tfrac{1}{2}\left(h_c - h\right)^2$$

$$V_2 = \left(h_c - h\right)\ d$$

$$V_3 = \frac{\tfrac{1}{2}\left(h_c - h\right)^2}{\tan o}$$

$$V_4 = \left[\frac{H_s}{7.6}\cdot 0.47\cdot\frac{1}{C_1\cdot 1.5}\left\{C_1\cdot x_t + 18\right\}^{1.5} - 2x_t - hx_t\right] - \left[\frac{H_s}{7.6}\cdot 0.47\cdot\frac{1}{C_1\cdot 1.5}\cdot 18^{1.5}\right]$$

$$C_1 = \left(\frac{7.6}{H_s}\right)^{1.28}\left(\frac{w}{0.0268}\right)^{0.56}$$

$$A_a = \frac{\tfrac{1}{2}\left(h - tl\right)^2}{\tan o}$$

$$A_2 = \left(x_t - \left(\frac{h - tl}{\tan o} + d\right)\right)\left(h - tl\right) + \tfrac{1}{2}\left(x_t - \left(\frac{h - tl}{\tan o} + d\right)\right)^2\cdot\tan s$$

$$A_3 = \tfrac{1}{2}x_1 y_1 - \tfrac{1}{2}x_1\left\{y_1 - \left(x_t - \left(\frac{h - tl}{\tan o} + d\right)\right)\cdot\tan s\right\}$$

$$x_1 = \frac{\left(x_t - \left(\frac{h - tl}{\tan o} + d\right)\right)\cdot\tan s}{\tfrac{1}{12} - \tan s}$$

$$y_1 = \tfrac{1}{12}x_1$$

A minimum width of the dune needs to be prescribed as resistance, e.g. d_w = 3.0 m

Parameter definitions:

h_c	=	crest level of the dune [m]
t_l	=	toe level of initial dune profile [m]
h	=	storm water level [m]
tan o	=	slope of the initial dune profile (simplified) [-]
tan s	=	slope of the initial dune profile (simplified) [-]
H_s	=	significant wave height [m]
w	=	fall velocity of the sand particles [m/s]
V_1	=	eroded volume of sand defined above high water level [m^3/m]
V_2	=	eroded volume of sand defined below high water level [m^3/m]
V_3	=	accreted volume of sand defined [m^3/m]
V_4	=	volume between storm water level and response profile of the dune [m^3/m]

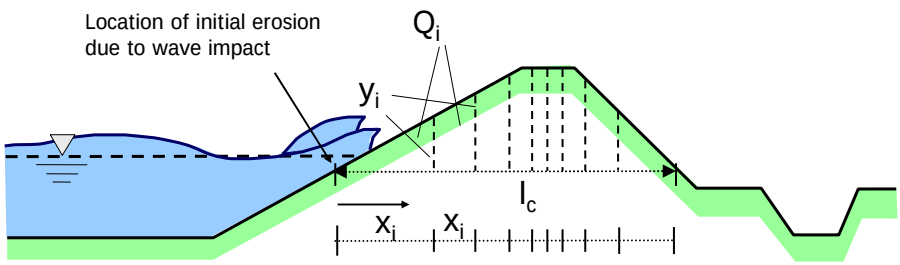
Sources of failure mechanism equations / methods:

Buijs F, Simm J, Wallis M, & Sayers P (2005); Steenbergen, H.M.G.M. & Vrouwenvelder, A.C.W.M (2003); Vellinga, P., (1986)
Sources of uncertainties in failure equations / input parameters:
Remarks:

Date	Revision	Prepared by	Organisation	Approved by	Notes
			PC RING		
25/01/ 07	v3_4_p03		LWI		edited

Aa 2.1b Erosion of sand core

Summary:
Erosion of embankment sand core under wave action.



Reliability equation:
The reliability function is expressed by:

$$z = t_e - t_s$$

where:

t_e = time for erosion of sand core over the erosion length l_c [s]
t_s = duration of storm [s]

Loading equations:
Duration of storm t_s

Resistance (strength) equations:
Time for erosion t_e

$$t_e = N_w \cdot T_p$$

with N_w = number of waves (erosion events) until erosion of core is completed so that

$$l_c = \sum_{i=1}^{N_w} x_i$$

and: x_i calculated from y_i and Q_i for each erosion event (wave) by means of the dike geometry:

$$x_i = \frac{2 \cdot Q_i}{y_i + y_{i-1}}$$

with:

$$Q_i = \text{const} = C_s \cdot \frac{u_0^{\,4}}{g^2 \cdot T_p}$$

$$C_s = A \cdot e^{-b\frac{H_s}{D_{50}}}$$

$$u_0 = k^* \cdot \sqrt{2 \cdot g \cdot A_{98}}$$

$$A_{98} = 3 \cdot H_s \cdot \tanh\left(0.65 \cdot \xi\right)$$

and l_c = horizontal length of dike core with respect to the height of initial erosion event:

$$l_c = (h_k - y_{imp}) \cdot (m + n) + B_k$$

with y_{imp} = vertical position of initial impact event with respect to dike toe [m]:

$$y_{imp} = h_w - \left(0.8 + 0.6 \cdot \tanh\left(\xi - 2.1\right)\right) \cdot H_s$$

where:

$$\xi = \frac{\tan\alpha}{\sqrt{H_s / L_0}}$$

$$L_0 = \frac{g \cdot T_p^{\,2}}{2 \cdot \pi}$$

Parameter definitions:

h_W	=	still water level at toe of dike [m]
h_k	=	height of dike crest [m]
B_k	=	width of crest [m]
m	=	outer slope [-]
n	=	inner slope [-]
α	=	angle of outer slope [-]
H_s	=	significant wave height [-]
T_p	=	peak wave period [s]
u_0	=	wave run-up velocity at still water level [m/s]
A_{98}	=	wave run-up height [m]
C_s	=	empirical coefficient according to Larson et al. (2004) [-]
x_{imp}	=	x-coordinate of initial impact point at outer slope with regard to dike toe [m]
y_{imp}	=	y-coordinate of initial impact point at outer slope with regard to dike toe [m]
i	=	single erosion segments with length x_i [-]
n_w	=	number of waves until erosion of dike core is completed [-]
x_i	=	horizontal length of single erosion segment [m]
y_i	=	vertical length of single erosion segment [m]
y_{i-1}	=	vertical length of previous erosion segment [m]
Q_i	=	volume of single erosion segment [m^3/m]
A	=	empirical factor according to Larson et al. (2004), A = $1.34 \cdot 10^{-2}$ [-]
b	=	empirical factor according to Larson et al. (2004), b = $3.19 \cdot 10^{-4}$ [-]
k*	=	empirical factor , e.g. k* = 1.0 [-], see Schüttrumpf (2001)
D_{50}	=	diameter of sand particle [m]
g	=	gravitational constant [m/s^2]

Sources of failure mechanism equations / methods:

Larson, M.; Erikson, L.; Hanson, H. (2004); Schüttrumpf, H. (2001)

Sources of uncertainties in failure equations / input parameters:

Remarks:

Status of Draft

Date	Revision	Prepared by	Organisation	Approved by	Notes
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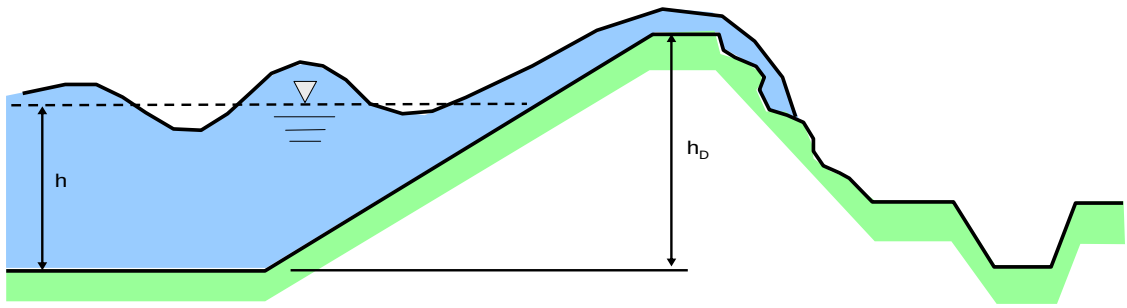
Aa 2.4 Erosion of core by wave overtopping

Summary: Erosion of inner slope material (sand) by wave overtopping velocities. Material properties given bank geometry.



Reliability equation:
The reliability function is expressed by:

$$z = t_3 - t_s$$

where:

t_3 = total duration of breach [h]
t_s = storm duration [h]

Loading equations:
Duration of storm t_s, e.g.: t_s= 6,5 h

Resistance (strength) equations:
Time for total breach of dike
Overflow:

$$t_3 = t_2 + \frac{2}{f_2 \cdot k_2} \cdot \left(\sqrt{h_w} - \sqrt{h_w - h_{k,0}}\right)$$

Overtopping and overflow ($h_w \leq h_k + h_{ok}$):

$$t_{3a} = t_2 + \frac{z_{98} \cdot \left[1 - \exp\left(1{,}833 \cdot \left(\frac{h_{k,o} - h_w}{z_{98}}\right)\right)\right]}{1{,}833 \cdot f_2 \cdot c_2 \cdot q_0^{1/3}}$$

$$t_3 - t_{3a} = \frac{2 \cdot c_2\, q_0}{f_2 \cdot k_2^2}\left(\frac{k_2}{c_2 \cdot q_0} \cdot h_w^2 - \ln\left[1 + \frac{k_2}{c_2 \cdot q_0} \cdot \sqrt{h_w}\right]\right)$$

$$t_2 = t_1 + \frac{B_k}{c_1 \cdot q^{1/3}}$$

$$t_1 = t_0 + \frac{l_t}{c_0 \cdot q^{1/3}}$$

$$c_0 = \frac{0{,}01}{(1-p)\Delta} \cdot \left(\frac{g}{C_f}\right)^{1/3} \cdot \frac{(\sin\beta)^{1/3}}{\cos\beta}$$

$$c_1 = \frac{0{,}01}{(1-p)\Delta} \cdot \left(\frac{g}{C_f}\right)^{1/3} \cdot \frac{(\sin\beta_1)^{1/3}}{\cos\beta_1}$$

Parameter definitions:

h_W	=	still water level at toe of dike [m]
h_k	=	height dike crown [m]
$h_{o,k}$	=	critical water level above crown height [m]
t_3	=	time of complete erosion of dike in phase III (total breach) after Visser (2000), modified after Kortenhaus (2003) [h]
z_{98}	=	wave run-up height according to Schüttrumpf (2001) or others [m]
t_2	=	time till the end of phase II (erosion of inner slope, see Figure 80 in report) [h]
f_2	=	coefficient for deceleration of erosion process [-]
$h_{k,0}$	=	original crown height of dike [-]
q_0	=	wave overtopping rate [l/s·m] for crest freeboard $R_c = 0$
t_0	=	start time of erosion if inner slope [h]
l_t	=	partial length of the dike at the inner toe [m]
q	=	mean overtopping rate [l/(s·m)]
p	=	porosity of sand bed [-]
Δ	=	relative density of sand [-]
C_f	=	(Chezy) friction of sand bed [-]
β	=	outer slope [°]
β_1	=	internal friction angle of sand [°]
g	=	gravitational constant [m/s^2]

Sources of failure mechanism equations / methods:
Kortenhaus, A.; Oumeraci, H. (2002); Kortenhaus, A. (2003), Visser, P.J. (2000):

Sources of uncertainties in failure equations / input parameters:

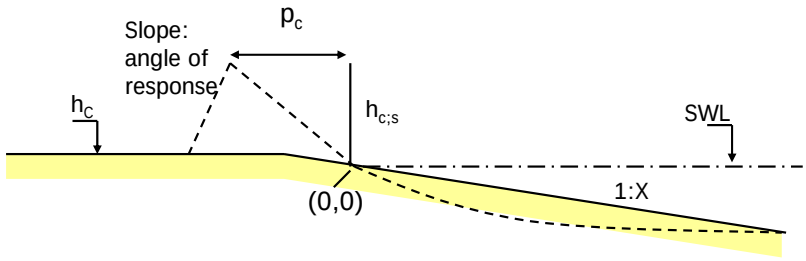
Remarks:
Procedure by Visser (2000) was used until phase III only but extended by Kortenhaus (2003) to include waves and overflow. WARNING! LSE is time dependent and needs to be combined with other time-dependent processes before calculating failure probabilities. Results if calculated individually can differ significantly.

Status of Draft

Date	Revision	Prepared by	Organisation	Approved by	Notes
			LWI		
25/01/ 07	v3_4_p03		LWI		edited

Ab 2.1a Erosion of shingle beach

Summary: Erosion of outer face of shingle bank or ridge under direct wave attack. Material properties given by particle size and ridge crest level.



Reliability equation:
The reliability function is expressed by crest retreat given the hydraulic loading conditions relative to the original width of the shingle beach, expressed as the probability of crest retreat exceeding the shingle bank width:

$$z = w - p_c$$

Loading equations:
Profile of shingle bank beach can be calculated with the parametric model according to Powell (1990). Simplified representation in Figure above showing initial profile defined by h_c, initial crest level and slope of 1:7.

The response 'hinges' around the intersection between the storm water level and the slope of the original beach profile, indicated with (0,0). $h_{c;s}$ and p_c are respectively the vertical and horizontal position of the crest level of the response profile.

$$h_{c;s} / H_s = 2.86 - 62.69(H_s / L_{om}) + 443.29(H_s / L_{om})^2$$

$$\frac{p_c D_{50}}{H_s L_{om}} = -0.23\left(\frac{H_s T_m g^{1/2}}{D_{50}^{3/2}}\right)^{-0.588}$$

Range of validity:
H_s/L_{om} – 0.01-0.06
$H_s T_m g^{1/2}/D_{50}^{3/2}$ = 3000-55000

Resistance (strength) equations:

Beach (D_{50})	**Slope**	**Material size**
Fine	1:12	10 mm
Medium	1:9	20 mm
Coarse	1:7	40 mm

Parameter definitions:

p_c	=	retreat of the shingle beach crest [m]
w	=	width of shingle beach, determined as narrow / wide and condition grade [m]
h_c	=	crest height [m]
$h_{c,s\,(m)}$	=	crest height after the storm, with reference to the intersection between the water level and the beach slope, point (0,0) [m]
L_{om}	=	offshore wave length [m]
H_s	=	significant height [m]
T_m	=	wave period [s]

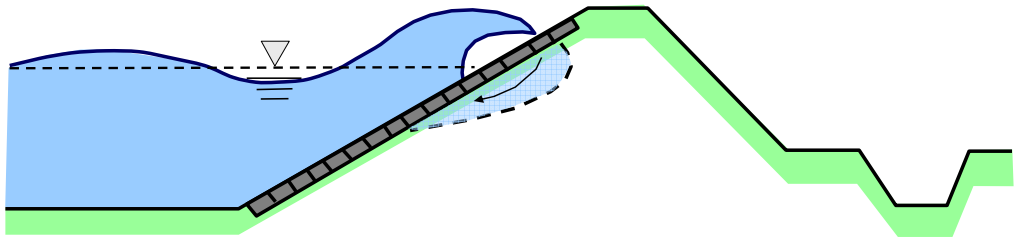
D_{50} = material mean size [m]
Sources of failure mechanism equations / methods: Powell, K.A., (1990) *Predicting short term profile response for shingle beaches*, Report SR219, HR Wallingford.
Sources of uncertainties in failure equations / input parameters:
Remarks:

Status of Draft

Date	Revision	Prepared by	Organisation	Approved by	Notes
		FB	HRW	WA	
25/01/ 07	v3_4_p03		LWI		edited

Ab 2.1b Movement of cover layer elements under wave action

Summary: Onset of failure of granular mound / slope under direct wave attack. This method is based on simple analysis of onset of motion, as then qualified by empirical data. Three alternative methods might be used.



Limit State Equation:
Damage when loose material (such as sand and rock) starts to move. Waves and currents determine the lift and drag forces acting on the stones of the cover layer (load). The inertial forces and forces due to friction and interlocking with are stones are the stabilizing forces (strength).

The limit state equation is expressed as the difference between the actual ($D_{n50,actual}$) and the required stone diameter (D_{n50}):

$$z = D_{n50,actual} - D_{n50}$$

Many methods have been developed for the prediction of the required rock or grain size of top layer elements designed for wave attack. Three possibilities are highlighted here.

Loading equations:
The **Hudson** formula (1953, 1959), originally developed based on tests with regular waves, can be re-written for applications with irregular waves into:

$$D_{n50} = \frac{1.27 \cdot H_s}{\Delta \left(K_D \cot\alpha\right)^{1/3}}$$

Van der Meer's (1988) formula reads for "plunging conditions" ($\xi_m \leq \xi_c$):

$$D_{n50} = \frac{H_s}{6,2 \cdot P^{0,18} \cdot \left(S_d / \sqrt{N_w}\right)^{0,2} \cdot \xi_m^{-0,5} \cdot \Delta}$$

and for "surging conditions" ($\xi_m > \xi_c$)::

$$D_{n50} = \frac{H_s}{\Delta \cdot 1,0 \cdot P^{-0,13} \cdot \left(S_d / \sqrt{N_w}\right)^{0,2} \cdot \sqrt{\cot\alpha} \cdot \xi_m^{P}}$$

with the damage level S_d defined as:

$$S_d = \frac{A_e}{D_{n50}^2}$$

and

$$\xi_c = \left[6.2\, P^{0.31} \sqrt{\tan\alpha}\right]^{\frac{1}{P+0.5}}$$

Resistance:
Actual stone diameter $D_{n50,\ actual}$

An alternative formula is derived by **Van Gent *et al*.** (2003). It reads:

$$D_{n50} = \frac{H_s}{\Delta \cdot \left(S_d/\sqrt{N}\right)^{0.2} \cdot 1.75\cot\alpha^{0.5} \cdot \left(1 + D_{n50\text{-core}}/D_{n50}\right)}$$

Note that the formula derived by Van Gent *et al*. is calibrated for a selected range of structure geometries (non-homogeneous structures with 1:4 < α < 1:2). Using tests with other structure geometries (*e.g.*, more gentle slopes than 1:4, slopes steeper than 1:2, or homogeneous structures) might lead to different conclusions.

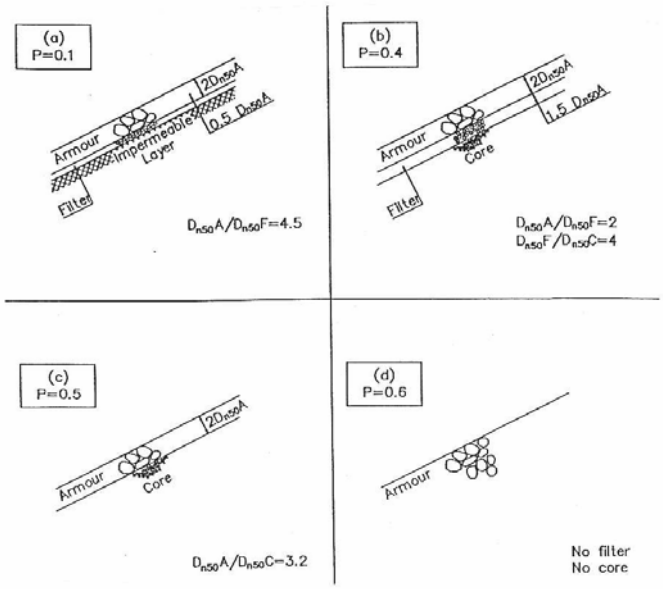
Strictly speaking, the acceptable damage level S_d and the relative density of the revetment elements are indicators of the strength. Therefore, the recommended values of S_d are put in the table on the righthandside.

In the Shore Protection Manual of 1984 (CERC, 1984) the following values of K_D were suggested for the Hudson formula:
- for breaking waves: $K_D = 2.0$
- for non-breaking waves: $K_D = 4.0$

The following table shows damage levels S_d corresponding to start of damage, intermediate damage and failure for five different slope angles given by cot α.

cot α	1.5	2	3	4	6
start of damage	2	2	2	2	2
inter-mediate damage	3-5	4-6	6-9	8-12	8-12
failure	8-12	8	12	17	17

The following graph shows values of the permeability, depending on the stone diameter in the different armour and filter layers.



Parameter definitions:

α	=	slope angle [-]
N	=	number of incident waves at toe [-]
S_d	=	damage level [-]
ξ_m	=	$\tan\alpha/(2\pi/g \cdot H_s/T_m^{\,2})^{0.5}$ [-]
P	=	permeability parameter (0.1<P<0.6) [-]
$D_{n50\text{-core}}$	=	the nominal stone diameter of core material [m]

Sources of failure mechanism equations / methods:
Hudson R.Y. (1953); Hudson R.Y. (1959); Coastal Engineering Research Center (1984); Van der Meer J. W. (1988); Van Gent M.R.A., Smale A. & Kuiper C. (2003)

Sources of uncertainties in failure equations / input parameters:

Remarks:
For loose materials the main input parameters are the hydraulic loading parameters (H_s, T_p) and parameters describing the structure, such as the slope angle, specific weight of individual stones, pore pressures and the internal friction and cohesion (interlocking). One of the things that should be taken care of during construction and maintenance of a revetment consisting of loose rock is that the stones do not break because than the weight of the individual stones can no longer be guaranteed. Hence, the design formulae should then be applied with a smaller stone diameter.

also see Ba 2.1 and Bc 2.1

Status of Draft

Date	Revision	Prepared by	Organisation	Approved by	Notes
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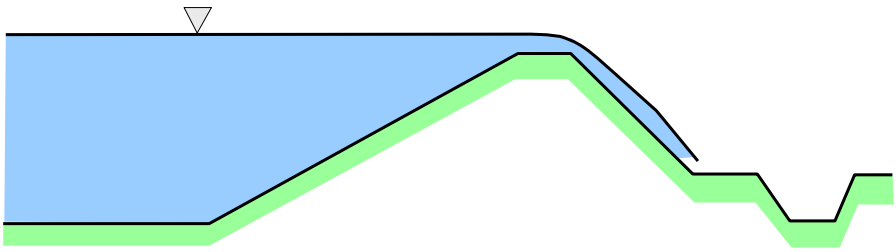
Ba 1.1 Erosion of embankment surface by overflow

Summary: Failures of the crest or rear face of the embankment have often been caused by flow of water over the crest and down over the rear slope. If the flow velocities are high, grass cover may be damaged then eroded, leading to direct erosion of embankment materials. This mechanism may dominate where the flood water level exceeds the embankment crest level and waves are small.

Damage is assumed to occur when the overflow discharge (or velocity) exceeds a limit given for the type and condition of grass cover on the crest and/or inner slope.



Reliability equation:
The "load" in the failure model taken as the actual overtopping discharge, q_0. The "strength" is taken as a critical discharge, q_c, that exceeds the resistance of the cover. The reliability function for this mechanism can therefore be expressed by:

$$z = m_{qc} \cdot q_c - m_{q0} \cdot q$$

where:

q_c = critical overtopping discharge [l/s·m]
q_0 = overtopping discharge [l/s·m]

Loading equations:
Overflow given by broad crest weir equation:

$$q_0 = c_D \cdot c_V \cdot h_{over} \sqrt{2 \cdot g \cdot h_{over}}$$

Resistance (strength) equations:
CIRIA TN71 , see Hewlett et al (1985), suggest that the critical overtopping discharge, q_c, can be calculated from:

$$q_c = v_c^3 / (\tan \alpha_i \cdot C^2)$$

The roughness factor on the inner slope may be estimated by the method of Strickler using the critical velocity, v_c, and a roughness length, k, by:

$$C = 25 \left(\frac{q_c}{k} v_c\right)^{1/6}$$

The critical failure velocity, v_c, at time, t_e, which causes failure of the grass surface is given by:

$$v_c = 3.8 \cdot \frac{f_g}{1 + 0.8\, {}^{10}\log t_e}$$

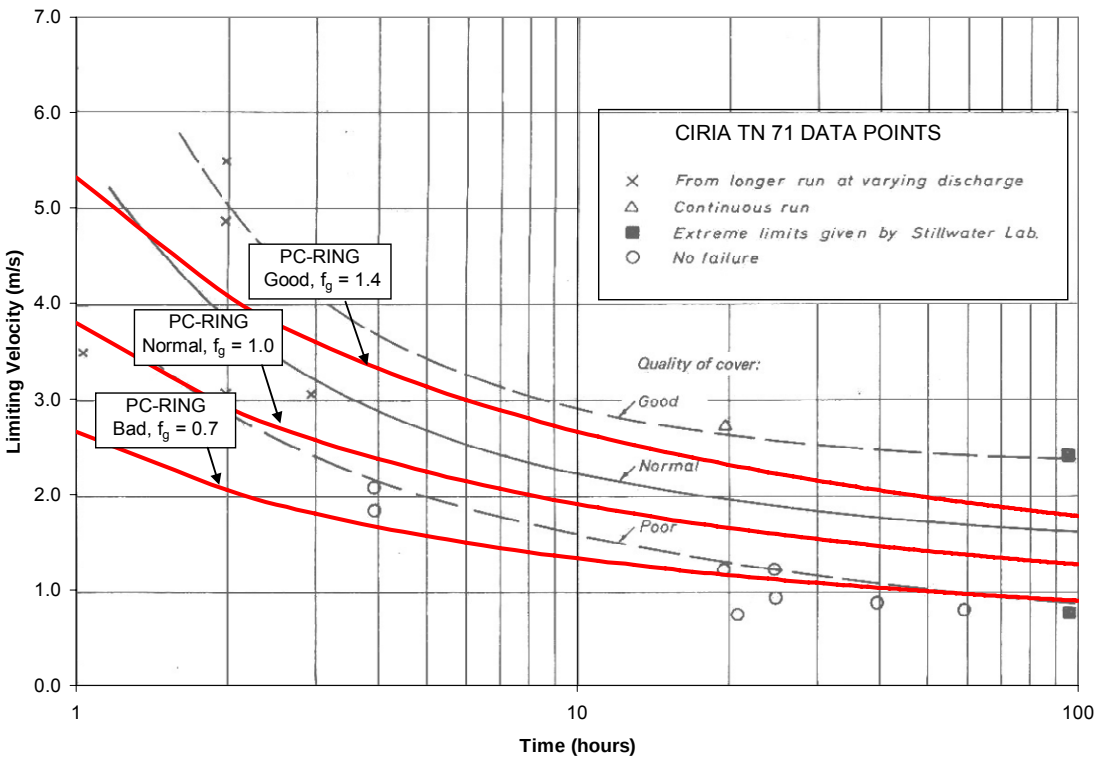
Parameter definitions:

q_0, q_c = actual and critical overtopping discharges [m^3/s·m]or [l/s·m]
m_{q0}, m_{qc} = are model factors for the actual and critical discharges [-]
c_D = coefficient for weir shape, crest width [-]
c_V = dissipation coefficient [-]

h_{over}	=	depth of flow over (local) crest [m]
v_c	=	critical flow velocity [m/s]
α_I	=	angle of the inner slope [°]
C	=	roughness factor according to De Chézy [$m^{0.5}$ / s]
f_g	=	condition quality of grass [-], varying between: f_g = 0.7 for bad turf; and f_g = 1.4 for good turf
t_e	=	overflow duration [h]

Sources of failure mechanism equations / methods:
Hewlett H W M, Boorman L A, Bramley M E (1987); Whitehead and Nickersons, (1976); Young, M; Hassan R. (2006)

Sources of uncertainties in failure equations / input parameters:
This format is drawn from PC RING which in turn draws its source from CIRIA TN71. Slightly more conservative results are obtained when applying values of f_g. recommended by PC-RING



Resistance of grass – PC-RING compared with CIRIA TN 71 (Young and Hassan (2006))

Remarks:
IMPORTANT! Current version is identical to Aa 1.1

Status of Draft

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		MWM		RH	
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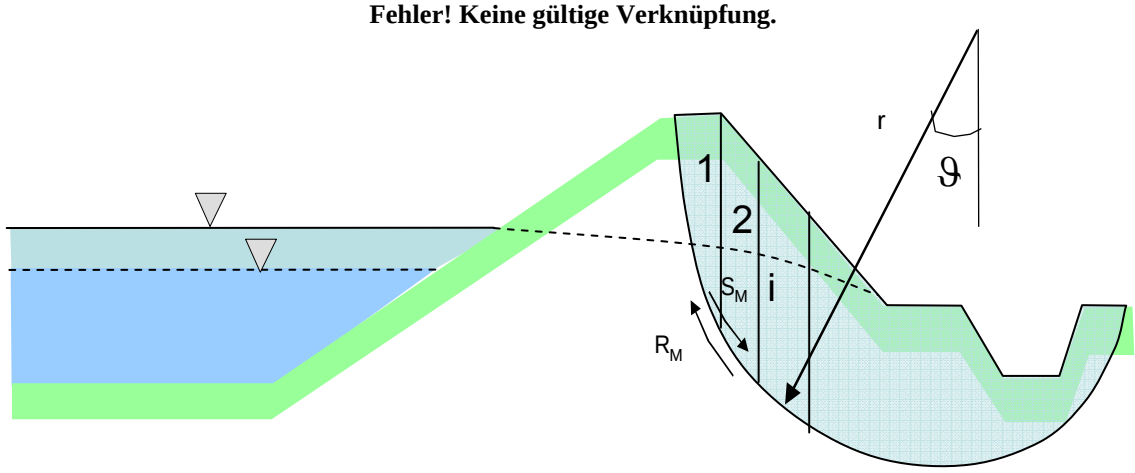
Ba 1.3a Deep slip in dike slope (inner or outer)

Summary: Deep slip in outer face of dyke. May be initiated by (rapid) draw-down of water level on outer face of the embankment, when material properties may altered in time / space.

Fehler! Keine gültige Verknüpfung.



Reliability equation for Bishop simplified method:
The reliability function is expressed by:

$$z = Fs - 1$$

with:

$$Fs = \frac{\sum R_M}{\sum S_M}$$

where:

ΣR_M = sum of resisting moments of single slices [kNm]
ΣS_M = sum of driving moments of single slices [kNm]

Loading equations:

$$\sum S_M = r \sum G_i \cdot \sin \vartheta_i$$

with weight of single slice:

$$G_i = \gamma_i \cdot A$$

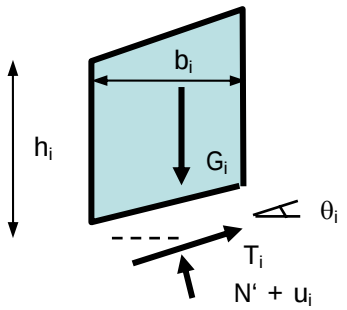
$$A_i = b_i \cdot h$$

Resistance (strength) equations:

$$\sum R_M = r \sum T_i = r \sum \frac{(G_i - u_i b_i) \cdot \tan \varphi_i + c_i \cdot b_i}{\cos \vartheta_i + \frac{1}{Fs} \cdot \tan \varphi_i \cdot \sin \vartheta_i}$$

In order to calculate Fs an interative procedure is needed since $R_M = R_M(Fs)$.

Detail of slice



Parameter definitions:

G_i	=	mass force of slice [kN/m^2]
T	=	shear resistance in cracking gap [kN/m^2]
Fs	=	safety factor [-]
b_i	=	width of slice [m]
h_i	=	height of slice [m]
A_i	=	area of slice [m^2]
u_i	=	pore water pressure at slice [kN/m^2]
c_i	=	cohesion at slice [kN/m^2]
r	=	radius of slip circle [m]
γ_i	=	volume weight of single soil slice [kN/m^3]
φ	=	internal friction angle [°]
θ_i	=	direction angle of slices [°]

Sources of failure mechanism equations / methods:
DIN (1983); Bishop, A.W. (1955); Abramson, L.W., Lee, T.S., Sharma, S., Boyce, G.M. (2002); Janbu, N., (1954); Janbu, N., (1973), Utili (2004)

Sources of uncertainties in failure equations / input parameters:
Primary sources of uncertainties are in the applicable soil parameters and lack of detailed information on internal structure of the embankment (unknown discontinuities, tension cracks, pre-existing slip surfaces, zones of different materials), nature of foundations. Other uncertainties are the probable shape of the slip surface, and the 2-dimensional assumptions.

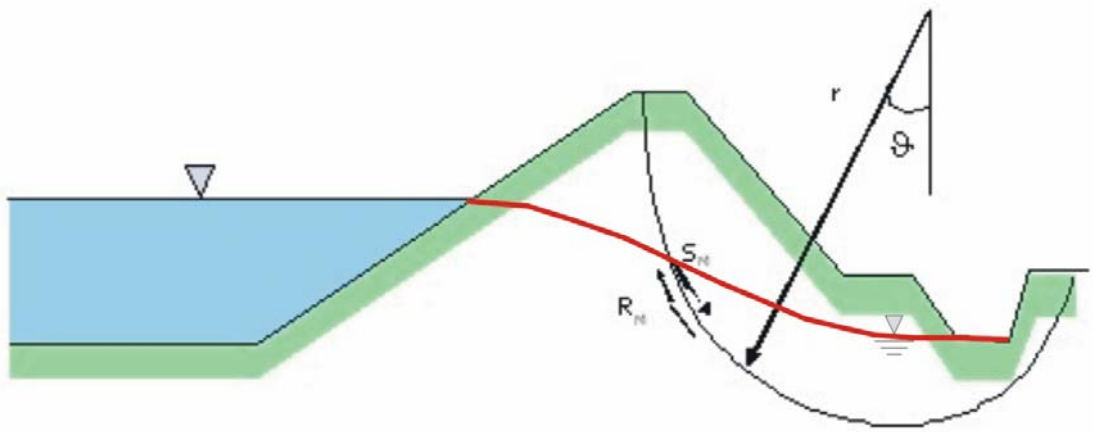
Remarks:
From a reliability analysis viewpoint, calculation of a margin of safety is more desirable than a factor of safety, but all the LEM give factors of safety. Therefore a performance function in terms of safety margin cannot be defined. To calculate the most critical surface with associated minimum factor of safety, iterations need to be done. To perform probabilistic calculations, factors of safety for different values of the parameters must be determined through a deterministic approach beforehand. The simplified Bishop method is suitable when the slip surface can be assumed circular. Circular failure surface cannot be assumed, for instance when pre-existing planar slip surfaces are present. In such a case, rigorous methods such as Spencer, Morgensten & Price, Janbu, Sarma methods are suggested. All these methods require considering two equilibrium equations: one for moment equilibrium and another one for horizontal forces. The factor of safety is found iteratively and satisfies both equations.

Status of Draft

Date	Revision	Prepared by	Organisation	Approved by	Notes
			LWI		
		MWM		RH	
25/01/ 07	v3_4_p03		LWI		edited
25/03/ 07	v4_4_p03	Stefano Utili	UStrath		edited

Ba 1.3b Cap or slip failure in dikes / embankments

Summary: Local slip / slide or instability in crest / inner face of clay bank. May be driven by high water level, wave overtopping or rainfall infiltration over a long period, perhaps water penetration into tension cracks at crest (see Ba 1.5e).



Reliability equation for Bishop simplified method:
The reliability function is expressed by:

$$z = Fs - 1$$

with:

$$Fs = \frac{\sum R_M}{\sum S_M}$$

where:

ΣR_M = sum of resisting moments of single segments [kNm]
ΣS_M = sum of driving moments of single segments [kNm]

Loading equations:
Driving mass forces calculated from:

$$\sum S_M = r \sum G_i \cdot \sin \vartheta_i$$

with:

weight of single segment:

$$G_i = \gamma_i \cdot A_i$$

length of single segment:

$$l_i \approx \frac{b_i}{\cos \vartheta_i}$$

Resistance (strength) equations:
Shear resistance calculated from:

$$\sum R_M = r \sum T_i = r \sum \frac{(G_i - u_i b_i) \cdot \tan \varphi_i + c_i \cdot b_i}{\cos \vartheta_i + \frac{1}{Fs} \cdot \tan \varphi_i \cdot \sin \vartheta_i}$$

In order to calculate Fs an interative procedure is needed since $R_M = R_M(Fs)$.

Parameter definitions:

G_i = mass force of segment [kN]
T = shear resistance in gap [kN/m^2]
Fs = safety factor [-]
b_i = width of segment [m]
h_i = height of segment [m]
A_i = aera of segment[m^2]
u_i = water pore pressure at segment [kN/m^2]
c_i = cohesion at segment [kN/m^2]
r = radius of crack circle [m]
φ = internal friction angle [°]
θ_i = direction angle of segment [°]

Sources of failure mechanism equations / methods:
DIN (1983); Bishop, A.W. (1955); Abramson, L.W., Lee, T.S., Sharma, S., Boyce, G.M., (2002), Utili (2004).

Sources of uncertainties in failure equations / input parameters:
Primary sources of uncertainties are in the applicable soil parameters and lack of detailed information on internal structure of the embankment (unknown discontinuities, tension cracks, pre-existing slip surfaces, zones of different materials), nature of foundations. Other uncertainties are the probable shape of the slip surface, and the 2-dimensional assumptions.

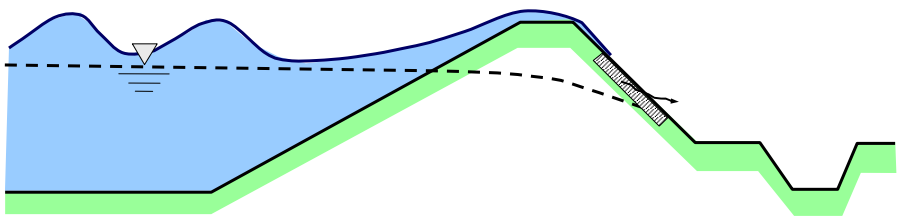
Remarks:
From a reliability analysis viewpoint, calculation of a margin of safety is more desirable than a factor of safety, but all the LEM give factors of safety. Therefore a performance function in terms of safety margin cannot be defined. To calculate the most critical surface with associated minimum factor of safety, iterations need to be done. To perform probabilistic calculations, factors of safety for different values of the parameters must be determined through a deterministic approach beforehand. The simplified Bishop method is suitable when the slip surface can be assumed circular. Circular failure surface cannot be assumed, for instance when pre-existing planar slip surfaces are present. In such a case, rigorous methods such as Spencer, Morgensten & Price, Janbu, Sarma methods are suggested. All these methods require considering two equilibrium equations: one for moment equilibrium and another one for horizontal forces. The factor of safety is found iteratively and satisfies both equations.

Status of Draft

Date	Revision	Prepared by	Organisation	Approved by	Notes
			LWI		
25/01/ 07	v3_4_p03		LWI		edited
25/03/ 07	v4_4_p03	Stefano Utili	UStrath		edited

Ba 1.4 Sliding of clay cover on inner slope of dike

Summary: Slide of upper layer on inner face driven by overflow velocities and infiltration of overflowing water into the inner slope. Soil resistance may be modified by infiltration into the upper surface of the dyke.



Reliability equation:
The reliability function is expressed by:

$$z = R - S$$

where:

R = force due to cohesion and soil reaction [kN]
S = force due to current velocity and weight [kN]

Loading equations:

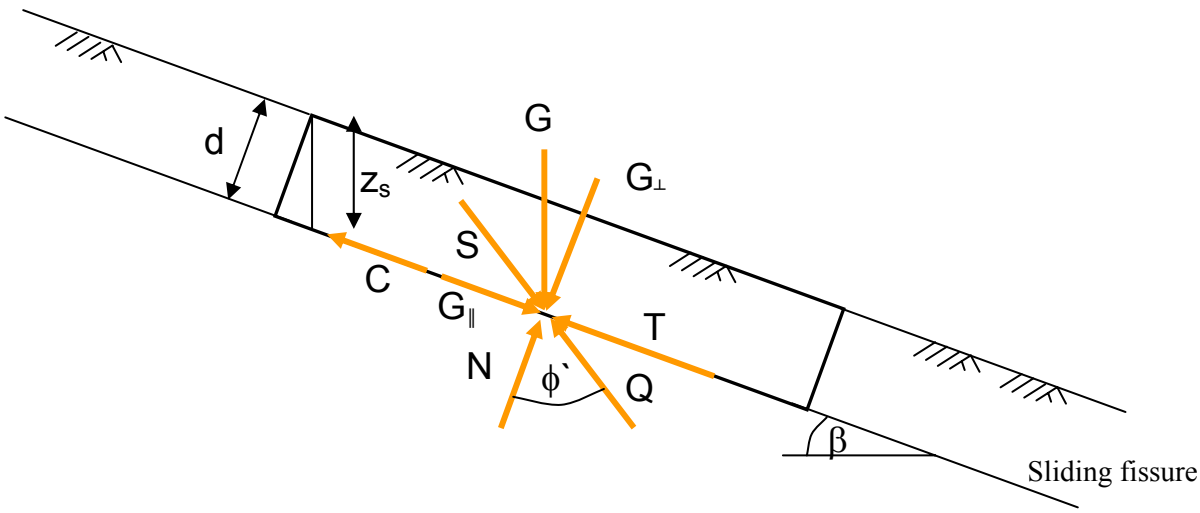
$$S = \sin\beta + i \cdot \frac{\gamma_w}{\gamma'} \cdot \cos(\alpha_S - \beta)$$

Resistance (strength) equations:

$$R = \frac{c'}{\cos\beta \cdot z_s \cdot \gamma'} + \tan\varphi' \cdot \cos\beta + i \cdot \frac{\gamma_w}{\gamma'} \cdot \sin(\alpha_S - \beta) \cdot \tan\varphi'$$

with:

$$z_s = \frac{d}{\cos\beta}$$



Parameter definitions:

z_s = depth of slope area effected by flow [m]
G = mass force of soil element
α_s = flow direction inside of the soil element [°]
β = slope [°]

c‘	=	effective cohesion [kN/m^2]
φ´	=	effective angle of friction [°]
γ´	=	saturated volume weight of soil [kN/m^3]
γ_w	=	volume weight of water [kN/m^3]
i	=	hydraulic gradient [°]

Sources of failure mechanism equations / methods:
Richwien, W.; Weißmann, R. (1999)

Sources of uncertainties in failure equations / input parameters:

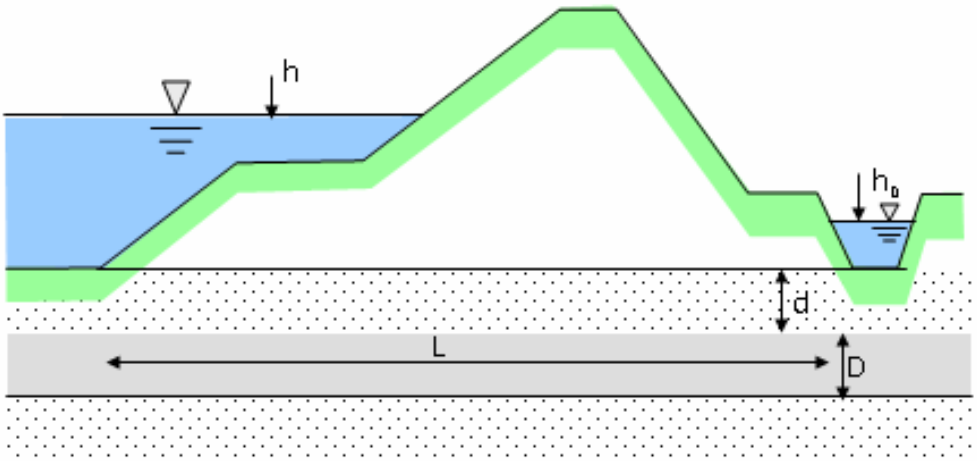
Remarks:
LSE compares flow forces on the inner slope. If flow is strong enough to overcome friction forces of the clay element induced by cohesion and friction angle failure will occur.

Status of Draft

Date	Revision	Prepared by	Organisation	Approved by	Notes
			LWI		
25/01/ 07	v3_4_p03		LWI		edited

Ba 1.5ai Piping under dikes

Summary: Primarily flow through and / or under embankment. May lead in turn to internal erosion



Reliability equation:
The reliability function is expressed by:

$$z = h_{crit} - \Delta h$$

where:

h_{crit}	=	critical pressure head gradient for piping [-]
Δh	=	existing pressure head gradient [-]

Loading equations:

$$\Delta h = h_w - h_h$$

Resistance (strength) equations:

$$h_{crit} = \alpha\, c\; l_D \left(\frac{\rho_s}{\rho_w} - 1\right) \tan\theta \left(0{,}68 - 0{,}10\ \ln c\right)$$

with:

$$\alpha = \left(\frac{D}{l_D}\right)^{\left(\frac{0.28}{\left(\frac{D}{l_D}\right)^{2.8} - 1}\right)} \qquad c = \eta\, d_{70} \left(\frac{1}{\kappa\, l_D}\right)^{1/3}$$

Parameter definitions:

D	=	thickness of sand layer [m]
l_D	=	length of seepage line [m]
h_{crit}	=	critical water level difference between water level in front and behind dike
d70	=	diameter of sand particle [mm]
ρ_s	=	density of sand particle [t/m^3]
ρ_w	=	density of water [t/m^3]
κ	=	internal permeability [-]
θ_B	=	bedding layer [°]
η_S	=	drag coefficient [-]
ν	=	kinematic viscosity of water [Pa·s]
g	=	gravitational constant [m/s^2]
h_W	=	water depth at dike foot [m]

H_s	=	significant wave height [m]
ΔH	=	overall head difference [m]
L_k	=	seepage length [m]
c_k	=	creep factor

Sources of failure mechanism equations / methods:

Bligh, W.G. (1910): Dams, barrages and weirs on porous foundations. Engineering News, vol. 63/64, pp. 708.

Sellmeijer, J.B. (1988): On the mechanics of piping under impervious structures. Ph.D. thesis, LGM mededelingen / Laboratorium voor Grondmechanica, Stichting Waterbouwskundig Laboratorium, LGM, Delft, The Netherlands, 43 pp.

Weijers, J.B.A.; Sellmeijer, J.B. (1993): A new model to deal with the piping mechanism. *Filters in Geotechnical and Hydraulic Engineering, Proceedings of the First International Conference (Geo-Filters)*, Brauns, Heibaum & Schuler (eds.), Rotterdam, Holland: Balkema, Karlsruhe 1992, S. 349-355.

TAW (1999) "Technical Report on Sand Boils (Piping)"

Sources of uncertainties in failure equations / input parameters:

Remarks:

Status of Draft

Date	Revision	Prepared by	Organisation	Approved by	Notes
		???	PC RING	WA / MM	
25/01/ 07	v3_4_p03		LWI		edited

Ba 1.5aii Piping directly underneath sheet pile cut-off

Summary: Failure due to piping directly underneath the sheet pile cut-off is taken into account if the water level exceeds the ground water level in the earth bank behind the wall. This ensures a positive water head over the concrete structure, which drives the piping process. One of the requirements is that the water level persists long enough for the piping process to initiate.

Reliability equation:
The reliability ís expressed by:

$$z = m_{c,p,R} \cdot \Delta h_c - \Delta h_a$$

where:

D_{hc}	=	critical head difference [m]
D_{ha}	=	actual head difference [m]
$m_{,c,p,R}$	=	modal factor [-]

Loading equations:
The head over the concrete structure:

$$\Delta_{ha} = h - g_w$$

Resistance (strength) equations:
The critical head associated with the piping process:

$$\Delta h_c = \left(L_v + 1/3 \cdot L_h\right)/c$$

Parameter definitions:

h	=	the river water level [mOD]
g_w	=	the groundwater level behind the concrete structure [mOD]
L_v	=	the vertical seepage length [m]
L_h	=	the horizontal seepage length [m]
c	=	the creep ratio [-]

Sources of failure mechanism equations / methods:
Terzaghi, K, and Peck, R.B. (1967)

Sources of uncertainties in failure equations / input parameters:

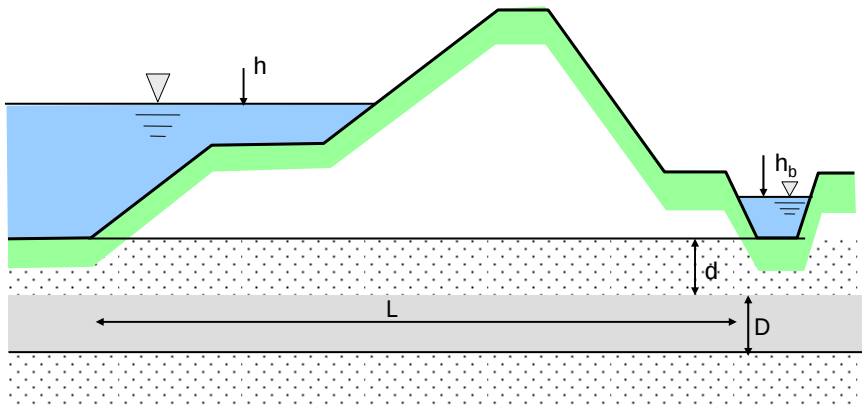
Remarks:

Status of Draft

Date	Revision	Prepared by	Organisation	Approved by	Notes
			PC RING		
25/01/ 07	v3_4_p03		LWI		edited

Ba 1.5aiii Uplifting of impermeable layers behind earth embankment

Summary: Uplifting behind embankments occurs if the difference between the local water level h, and the water level "inside", h_b is larger than the critical water level h_c



Reliability equation:
The reliability function is expressed by:

$$z = m_0 \cdot h_c - m_h \cdot \Delta h$$

where:

h_c	=	critical water level [m]
Δh	=	difference between local water depth in front of dike and water level in the floodplain [m]
m_o	=	model uncertainty factor [-]
m_h	=	model uncertainty factor for damping[-]

Loading equations:

$$\Delta h = h - h_b$$

Resistance (strength) equations:

$$h_c = \frac{\gamma_{wet} - \gamma_w}{\gamma_w} d$$

Parameter definitions:

γ_{wet}	=	saturated volumetric weight of the impermeable soil layers
γ_w	=	volumetric weight of the water
d	=	thickness of the impermeable layers
h	=	water level on the river [m]
h_b	=	water level in the floodplain [m]

Sources of failure mechanism equations / methods:
Vrouwenvelder et al. (2001)

Sources of uncertainties in failure equations / input parameters:
Vrouwenvelder et al. (2001)

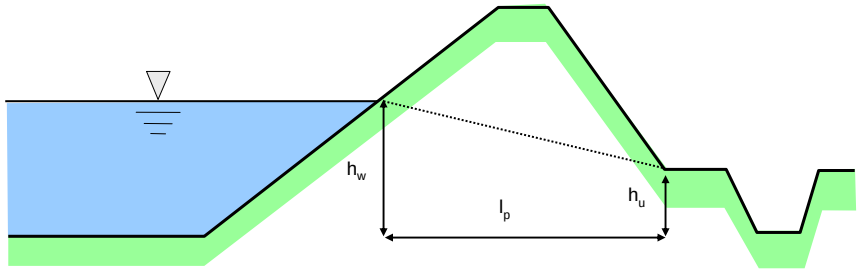
Remarks:

Status of Draft

Date	Revision	Prepared by	Organisation	Approved by	Notes
			PC RING		
25/01/ 07	v3_4_p03		LWI		edited

Ba 1.5b Seepage through sand core

Summary: Sub-mechanism covering flow through general material or through vermin holes. Predominantly sandy or silty fluvial / estuarial dyke, or pre-damaged clay surface layer. Flows / pressures lead to surface outflow or internal erosion and bursting.



Note: Outflow could occur above toe of rear slope.

Reliability equation:
The reliability function is expressed by:

$$z = t_{PhL} - t_s$$

where:

t_{PHL}	=	time for complete seepage through the dike [h]
t_s	=	duration of elevated water level [h]

Loading equations:
t_s = duration of storm

Resistance (strength) equations:
Duration for complete seepage through dike:

$$t_{Phl} = \frac{l_p}{k \cdot m_u}$$

with:

$$l_p = \sqrt{(x_u - x_w)^2 + (h_w - h_u)^2}$$

$$m_u = \frac{h_w - h_u}{x_u - x_p}$$

Parameter definitions:

h_w	=	height of still water level [m]
h_u	=	height of inner berm [m]
x_w	=	x- coordinate of intersection point of still water level and outer slope [m]
x_u	=	x- coordinate of leaking point at the inner berrm [m]
l_p	=	length of seepage line [m]
k	=	Permeability of (core) material [m/s]

Sources of failure mechanism equations / methods:

Sources of uncertainties in failure equations / input parameters:
The use of a simple Darcy law for seepage assumes that flow rates are relatively slow, that the core material can be treated as giving a uniform permeability, so any clay cover has been eroded, perforated or is no longer intact.

Remarks:
The LSE equation assumes that the clay is either eroded or not present any more. Seepage through the sand core is then calculated. Wave action is not considered here. WARNING! LSE is time dependent

and needs to be combined with other time-dependent processes before calculating failure probabilities. Results if calculated individually can differ significantly.

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Date	Revision	Prepared by	Organisation	Approved by	Notes
			LWI		
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Ba 1.5c Internal erosion or suffusion and/or filter stability under steady flows

Summary: Two primary types of filter instability are covered:

- **Internal erosion or suffusion**: the finer particles are conveyed through the voids associated with the coarse particles within the same layer:
- **Interfacial or filter instability**: the particles of one layer are conveyed through the pores between the particles of another (usually overlying) filter layer

Filters must allow for transport of water to prevent pressure build up, referred to as filter permeability. Suffusion may occur if the coarser particles in a gap-graded material are greater than 5 times the size of the finer particles. The fine fraction may then not be contained under significant hydraulic gradients. Suffusion failure may be predicted by simple methods by Kenney et al, Kenney & Lau, Lubochov etc

Reliability equation:

$$z = f\left(\frac{D_{s,f}}{D_{s,b}}; c_1; c_2\right)$$

For steady flows, Terzaghi (see e.g. CIRIA / CUR 1991) gives criteria to prevent migration

for uniformly graded materials: $D_{50f}/D_{50b} < c_1$ with $c_1 = 5$

for wide graded materials both: $D_{15f}/D_{85b} < c_1$ with $c_1 = 5$

and: $c_1 < D_{50f}/D_{50b} < c_2$ with $c_1 = 5$; $c_2 = 20\text{-}60$

and to ensure adequate permeability: $D_{20f}/D_{20b} > c_1$ with $c_1 = 5$

Loading equations:
The loading follows from the hydrostatic pressure, which is not included in the equations.

Resistance (strength) equations:
The resistance depends on the grading characteristics of considered stone size.

Parameter definitions:
D_s = particle size corresponding to sieve size, index "f" refers to filter, Index "b" refers to base layer

Sources of failure mechanism equations / methods:
CIRIA/CUR (1991); Pilarczy K ed (1998); Kenney T.C. Chahal R., Chiu E., Ofoegbu G.I., Omange G.N. & Ume C.A. (1985); Kenney T.C. & Lau D (1985)

Sources of uncertainties in failure equations / input parameters:

Remarks:
see also Bc 2.1n for filter stability under reversing flows

Status of Draft

Date	Revision	Prepared by	Organisation	Approved by	Notes
			INFRAM		
		ND???		WA / MM	
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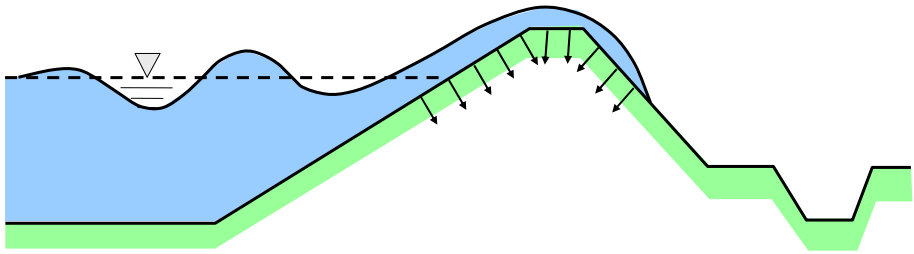
Ba 1.5d Infiltration into a dike

Summary: A contributing mechanism where wave action (or heavy rainfall?) increases the moisture content of the embankment or clay cover, thus changing the strength of the surface material.



Reliability equation:
The reliability function is expressed by:

$$z = t_{sat} - t_s$$

where:

t_s = duration of storm [h]
t_{sar} = time for saturation of cover layer [h]

Loading equations:
t_s= duration of storm

Resistance (strength) equations:
Time for infiltration of cover layer:

$$t_{sat} = \frac{e_s \cdot d}{v}$$

Parameter definitions:

d = thickness of cover layer [m]
e_s = fraction of air pore [-]
v = velocity of infiltration [m/s]
t_s = duration of strom [h]
t_{sat} = time of saturation [h]

Sources of failure mechanism equations / methods:
Weißmann, R. (2002)

Sources of uncertainties in failure equations / input parameters:

Remarks:
Failure is defined when water infiltrates through the cover layer of the dike. This is taken as one essential condition for further failure modes in a fault tree and is therefore defined as LSE here. WARNING! LSE is time dependent and needs to be combined with other time-dependent processes before calculating failure probabilities. Results if calculated individually can differ significantly.
Generally linked to Ba 1.3c or Ba 1.4

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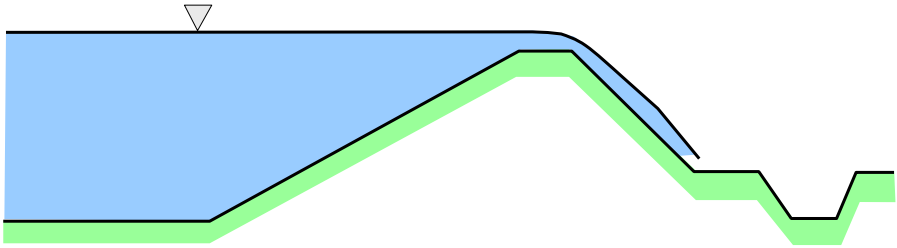
Ba 1.6 Overflow of dike (functional failure)

Summary: Overflow discharge rate / volume exceeds admissible volume of water behind the sea defence. The load S is the actual energy height of water overflowing the flood defence h_E, the strength R is an admissible energy height calculated from a critical volume of water that exceeds the limit for inundation. May often be re-cast in terms of volumes / discharges.



Reliability equation:
The reliability function is expressed by:

$$z = h_{E,\,adm} - h_E$$

where:

h_E = actual energy height of water overflowing the flood defence [m]
$h_{E,\,adm}$ = admissible energy height of water calculated from critical overflow discharge [m]

Loading equations:
Energy height given by broad crest weir equation:

$$h_E = h_W + h_k + \frac{v_0^{\,2}}{2g}$$

Resistance (strength) equations:
Energy height due to critical overflow calculated from:

$$h_{E,adm} = \left(\frac{q_{adm}}{A}\right)^{\frac{2}{3}}$$

The critical discharge depends on whether the stability of the structure is normative or whether the storage capacity is normative. The critical discharge due to the storage capacity can be calculated with the following formula:

$$q_{adm} = V_B / t$$

$$V_B = K / B$$

$$K = A_f \cdot h_{wlr}$$

The admissible wave overflow rate for stability is not given here. A logical value has to be determined by the manager of the flood defence (TAW).

Parameter definitions:

h_k = crest height of the dike [m]
h_W = water level above crest [m]
A = C = 1,86*k_d*k_f (broad crest weir) for v_0=0.0 m/s [-]
A = 1,444*(1+Cw)*C_L*C_R*C_m*C_n (wide crest weir) for v_0 > 0.0 m/s [-]
kd, kf = coefficients for consideration of the crest width B_k, and sharpcrestedness of the weir R_k [-]
Cw, CL, CR, Cm, Cn
= coefficients for consideration of the crest width B_k, and sharpcrestedness of the weir R_k, seaward slope 1:m, and landward slope 1:n for a broad crested weir [-]

Sources of failure mechanism equations / methods: Hewlett, H.W.M.; Boorman, L.A.; Bramley, M.E.; Whitehead, E. (1985), Oumeraci, H.; Schüttrumpf, H.; Bleck, M. (1999)
Sources of uncertainties in failure equations / input parameters:
Remarks: Due to numerical stability of solving the FORM iterations the limit state equation was modified to heights rather than discharges.

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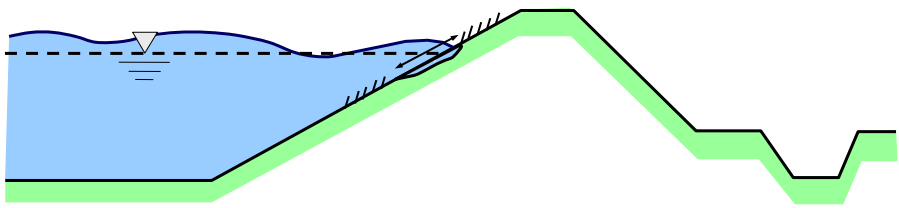
Ba 2.1a Erosion of grass cover by wave attack

Summary: Initiation of erosion by wave action, but only valid whilst grass cover is intact (at least partially).



Reliability equation:
The reliability function is expressed by:

$$z = t_{RG} - t_s$$

where:

t_{RG} = duration of grass erosion [h]
t_s = duration of storm [h]

Loading equations:

t_s= duration of storm, e.g.: t_s= 5.6 h

Resistance (strength) equations:
Duration of grass erosion

$$t_{RG} = \frac{d_G}{\gamma_G \, c_E \, H_s^2}$$

Parameter definitions:

d_G = thickness of grass cover [m]
c_E = coefficient for erosion resistance of grass cover [$(m \cdot s)^{-1}$]
H_s = significant wave height [m]
γ_G = velocity coefficient [-]

Sources of failure mechanism equations / methods:
INFRAM (2000); PC-Ring (2003); Smith, G.M.; Seijffert, J.W.W.; Van der Meer, J.W. (1994)

Sources of uncertainties in failure equations / input parameters:

Remarks:
The duration of grass erosion is based upon model tests in the large wave flume of Delft Hydraulics (Delta flume) but there is very limited tests only, see Smith et al. (1994). WARNING! LSE is time dependent and needs to be combined with other time-dependent processes before calculating failure probabilities. Results if calculated individually can differ significantly.

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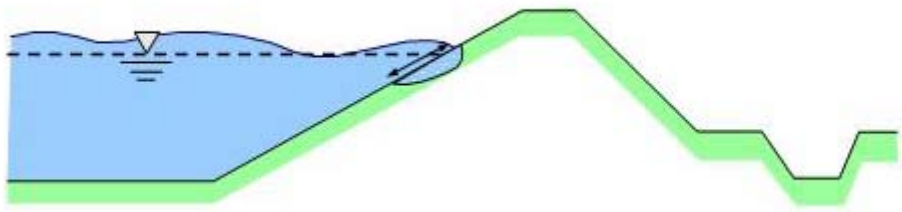
Ba 2.1b Wave driven erosion of clay layer of dikes

Summary: Direct wave-driven erosion of clay layer (without significant grass cover).



Reliability equation:
. The reliability function is expressed by:

$$z = t_{RK} - t_s$$

where:

t_{RK} = duration of clay erosion [h]
t_s = duration of storm [h]

Loading equations:

t_s= duration of storm

Resistance (strength) equations:
Duration of clay erosion:

$$t_{RK} = 0{,}4 \frac{L_K \, c_{RK}}{r^2 \, H_s^2}$$

where:

$$L_K = \frac{d_K}{\sin \alpha}$$

$$\beta \le \beta_r \rightarrow r = 1$$

$$\beta > \beta_r \rightarrow r = \frac{(110° - \beta)}{(110° - \beta_r)} \ge 0$$

c_{RK} = 7.10^3 ms for bad clay and 54.10^3 ms for good clay. Sand c_{RK} = 0.

Parameter definitions:

d = thickness of clay layer [m]
d_K = thickness of remaining clay layer [m]
c_{RK} = coefficient for erosion resistance of clay [(m·s)]
H_s = significant wave height [m]
r = reduction factor for oblique wave attack [-]
α = slope of dike [°]
β = angle of wave attack [°]

Sources of failure mechanism equations / methods:
INFRAM (2000); PC-Ring, (2003)

Sources of uncertainties in failure equations / input parameters:

Remarks:
See also comments on grass erosion. WARNING! LSE is time dependent and needs to be combined with other time-dependent processes before calculating failure probabilities. Results if calculated

individually can differ significantly.

Note: Erosion of sand (core) may be covered by Aa 2.1b.

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				WA / MM	
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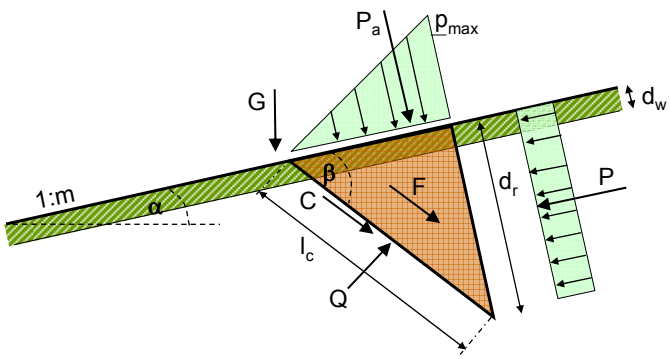
Ba 2.3 Wave impact

Summary: Local failure of clay cover under conditions of wave impacts onto outer face of the dike.

Definition sketch:



Reliability equation:
The reliability function is expressed by:

$$z = C_{u,adm} - C_u$$

where:

$C_{u,adm}$ = admissible cohesion [kPa]
C_u = cohesion [kPa]

Loading equations:
Admissable cohesion $C_{u,adm}$

e.g.: $C_{u,adm}$= 25.0 kPa (from German design guidance)

Resistance (strength) equations:

$$C_u = \frac{P \cdot a + G + P_a \cdot d}{e \cdot b} - \frac{c_w \cdot l_r \cdot d_w}{\sin\beta \cdot e}$$

where:

$$P = f_p \cdot p_{max} \cdot d_r \cdot l_r$$

$$p_{max} = \frac{f_{p,max}}{m} \cdot H_s \cdot \gamma_w$$

$$P_a = 0.5 \cdot p_{max} \cdot l_c \cdot \cos\beta \cdot f_A \cdot l_r$$

$$G = 0.5 \cdot d_r \cdot l_c \cdot \cos\beta \cdot \delta_k \cdot g \cdot f_G \cdot l_r$$

$$l_c = \frac{d_r}{\sin\beta}$$

$$a = \sin\alpha - \cos\beta \cdot \tan(90 - \beta + \alpha)$$

$$b = -\sin(\beta - \alpha) - \cos(\beta - \alpha) \cdot \tan(90 - \beta + \alpha)$$

$$d = \cos\alpha + \sin\alpha \cdot \tan(90 - \beta + \alpha)$$

$$e = \frac{l_r \cdot d_r}{\sin\beta} + f_F \cdot d_r^{\,2} \cdot \cot\beta$$

Parameter definitions:

P	=	mean force within gap [kN]
P_A	=	mean force due to impact [kN]
p_{max}	=	maximum pressure due to impact [kPa]
G	=	mass force of soil element [kN]
H_s	=	significant wave height [m]
α	=	angle of outer slope [°]
β	=	angle of shear gap [°]
m	=	outer slope[-]
d_r	=	depth of gap [m]
l_r	=	width of gap [m]
l_c	=	length of gap [m]
d_w	=	effective depth of root penetration[m]
c_w	=	cohesion due to root penetration [kPa]
γ_w	=	volume weight of water [kN/m^3]
γ_k	=	volume weight of soil [kN/m^3]
g	=	gravitational constant [m/s^2]
f_A	=	factor for mean force due to wave impact [-]
f_p	=	factor for P [-]
f_{pmax}	=	factor for p_{max} [-]
f_G	=	factor for force due to mass of soil [-]
a,b,d,e	=	substitutes

Sources of failure mechanism equations / methods:

Sources of uncertainties in failure equations / input parameters:

There are a number of unknowns in the equations, amongst which are the maximum pressure due to wave impact, the factor for maximum pressure in the crack f_p, the depth of the crack d_r, and the factor for the impact load itself and its relation to the pressure in the crack. Further uncertainties result from unknown soil parameters although some field experience is available. The area of root cohesion and its magnitude is due to some variations as well.

Remarks:

The limit state equation results from an equilibrium of forces acting on a triangle piece of soil which is eroded by wave impacts. The main resistant force is the shear resistance due to the undrained shear strength c_u. The main stress forces result from the wave impact on the soil.

Further reading:

Bezuijen, A.; Müller, G.; Wolters, G. (2005); Buß, J.; Kortenhaus, A. (2006), Führböter, A. (1966), Führböter, A. (1994); Pohl, C. (2005); Richwien, W. (2002); Wolters, G.; Müller, G.U. (2002)

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			LWI		
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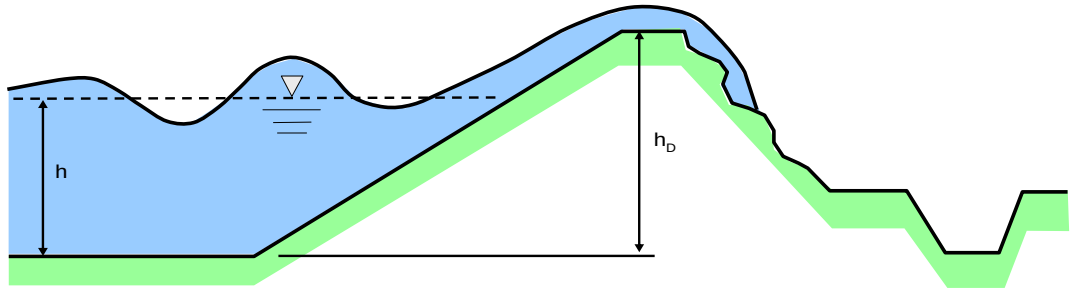
Ba 2.4ai Erosion of grass cover on inner slope due to wave overtopping

Summary: Failure function based on (peak) overtopping velocities compared with a limiting peak velocity (or discharge rate) for the onset of damage to the given (condition of) grass cover. The method may also be re-cast in terms of shear stress



Reliability equation:
The limit state equation may be expressed in terms of discharge, velocity or shear stress:

$$z = v_c - v_A$$

where:

v_c = critical velocity [m/s]
v_A = actual velocity [m/s]

Loading equations:
Velocities – representative overtopping velocities may be calculated for simple dike slopes from:

$$v_A = \frac{v_{B,0} + \frac{k_1 h_B}{f} \tanh\left(\frac{k_1 t}{2}\right)}{1 + \frac{f\, v_{B,0}}{h_B\, k_1} \tanh\left(\frac{k_1 t}{2}\right)}$$

with:

$$k_1 = \sqrt{\frac{2 f g \sin\beta}{h_B}}$$

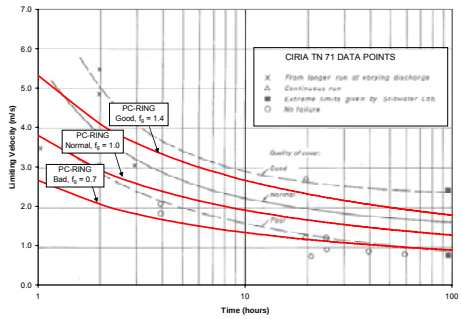
$$h_B = \frac{v_{B,0} \cdot h_{B,0}}{v_B}$$

$$t \approx -\frac{v_{B,0}}{g \sin\beta} + \sqrt{\frac{v_{B,0}^2}{g^2 \sin^2\beta} + \frac{2 s_B}{g \sin\beta}}$$

$$v_{B,0} = \left[0,75 \cdot \left(\frac{\pi \cdot H_s}{T_m}\right) \cdot n \cdot \xi_d \cdot \sqrt{\frac{z_{98} - R_c}{H_s}}\right] \cdot \exp\left(-\frac{f \cdot B_K}{2\, h_{B,k}}\right)$$

$$h_{B,0} = 0,0793 \cdot (z_{98} - R_c) \cdot \frac{\cos\alpha}{\cos\beta}$$

Resistance (strength) equations:
The critical velocity can be calculated using the following figure



Resistance of grass – PC-RING compared with CIRIA TN 71 (Young and Hassan (2006))

Parameter definitions:

v_A = velocity on inner slope [m/s]

$v_{B,0}$	=	velocity at beginning of inner slope [m/s]
v_c	=	critical velocity [m/s]
h_B	=	layer thickness on inner slope [m]
$h_{B,0}$	=	layer thickness at beginning of inner slope [m]
$h_{k,B}$	=	layer thickness at crest [m]
f	=	friction factor [-]
H_s	=	significant wave height [m]
f_G	=	parameter for grass quality [-]
t_s	=	duration of storm [h]
β	=	inner slope [°]
α	=	outer slope [°]
B_k	=	crest width [m]
R_c	=	crest freeboard [m]
s_B	=	control variable inner slope [-]
z_{98}	=	wave run-down [m]
q_M	=	material quality (1,0 for Sand) [-]
q_G	=	grass quality (between 0,0 and.1,0) [-]

Sources of failure mechanism equations / methods:
Schüttrumpf H (2001); Schüttrumpf, H. (2003)

Sources of uncertainties in failure equations / input parameters:

Remarks:

Status of Draft

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				RMH	
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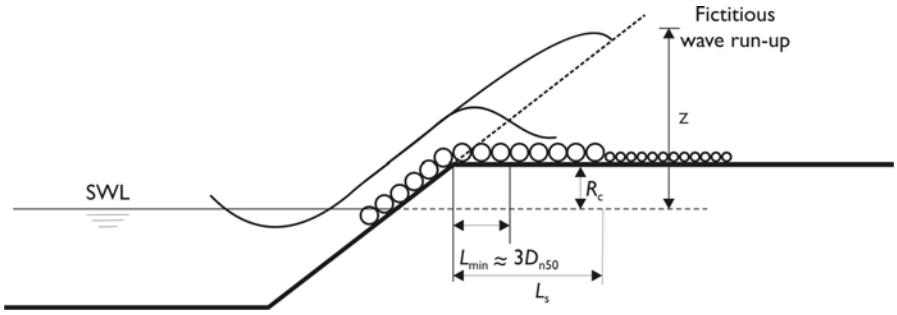
Ba 2.4aiii Erosion of crest (rubble mound structures)

Summary: Erosion of (un-bound rubble) crest material by wave overtopping velocities



Reliability equation:
The reliability equation can be expressed as follows:

$$z = u - u_{1\%}$$

where:

u	=	maimum bearable overtopping velocity for existing rubble mound structure at crest [m/s]
$u_{1\%}$	=	existing average wave overtopping velocity at crest [m/s]

Loading equations:
The loading for calculating the finer material behind the protected area L_s is the depth averaged velocity on the crest, which can be calculated with:

$$u_{1\%} = 1.7\left(g\gamma_{f-c}\right)^{0.5}\left(\left(R_{u1\%} - R_c\right)/\gamma_f\right)^{0.5} \Big/ \left(1 + 0.1 L_s/H_s\right)$$

with:

$$L_s = 0.2\psi T\sqrt{g\left(R_u - R_c\right)}$$

$$R_{u1\%}/\left(\gamma H_s\right) = c_0\xi_{s-1,0} \qquad \text{for } \xi_{s-1,0} \le p$$

$$R_{u1\%}/\left(\gamma H_s\right) = c_1 - c_2/\xi_{s-1,0} \qquad \text{for } \xi_{s-1,0} \ge p$$

Resistance (strength) equations:
The value 2 to 2.7 for dimensionless flow velocity used in the expressions to calculate the finer material outside the protected area L_s, can be considered as a stability number, indicating the strength of the structure:

$$\frac{u^2}{g \cdot \Delta D_{n50}} = 2 \text{ to } 2.7$$

Parameter definitions:

L_s	=	length of 'splash-area' (m)
ψ	=	'importance-of-structure' factor (>1): engineering judgement factor (-)
T	=	wave period (s) for which $T_{m-1,0}$ can be used
R_u	=	fictitious wave run-up level (m)
R_c	=	crest level relative to still water (m)
$u_{1\%}$	=	maximum velocity (depth-averaged) at the rear side of the crest [m/s] during a wave overtopping event, exceeded by 1% of the incident waves
B	=	crest width [m]
R_c	=	rest level relative to still water at the seaward side of the crest [m]
γ_f	=	roughness of the seaward slope [-]; γ_f = 0.55 for rock slopes and γ_f = 1 for impermeable slopes
γ_{f-c}	=	roughness at the crest [-]; γ_{f-c} = 0.55 for rock crests and γ_{f-c} = 1 for impermeable crests
$R_{u1\%}$	=	fictitious run-up level [m]
c_0	=	1.45, c_1 = 5.1, $c_2 = 0.25\ c_1^2 / c_0$ and $p = 0.5\ c_1 / c_0$
γ		the reduction factor [-]; $\gamma = \gamma_f \gamma_\beta$, taking into account the effects of angular wave attack (γ_β , which can be approximated by: $\gamma_\beta = 1 - 0.0022 \cdot \beta$, where $\beta \le 80°$) and roughness (γ_f)
$\xi_{s-1,0}$	=	the surf-similarity parameter [-], defined as $\xi_{s-1,0} = \tan\alpha / (2\pi / g \cdot H_s / T_{m-1,0}^{2)0.5}$

Sources of failure mechanism equations / methods: USACE (2003); Pilarczyk (1998); TAW (2002a); Van Gent, M. and Pozueta, B. (2004)
Sources of uncertainties in failure equations / input parameters:
Remarks: For rubble mound structures, the same material used on the seaward slope is usually applied at the structure crest to prevent erosion of the crest. The loading for calculating the protected area L_s is the fictitious wave run-up for which several prediction methods are available, see for example TAW (2002), CEM (2003) and above. prviously this was Bc 2.4a (also Bc 2.1)

Status of Draft

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		P. van Gelder	TUD		
		ND		WA / MM	
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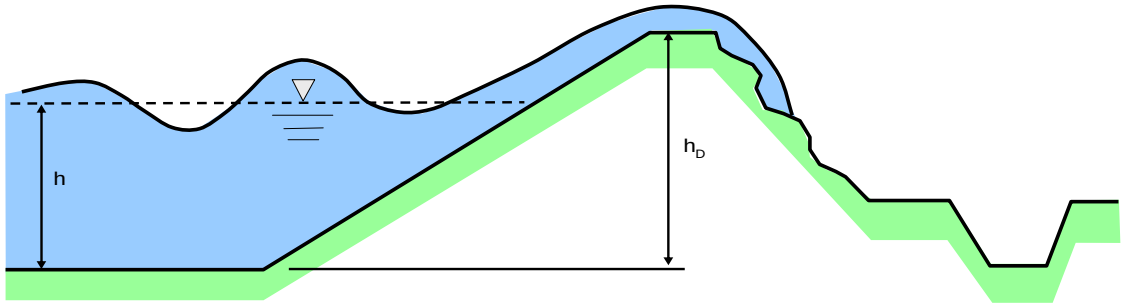
Ba 2.4b Erosion of clay inner slope by wave overtopping (Turf set off)

Summary:
Superficial slip of the turf layer, forming a large horizontal fissure along the dike crest, is referred to as "turf set-off and has been observed as a first stage of failure on the inner slope of dikes with heavy wave overtopping. Erosion starts near the dike crest.



Reliability equation:
The limit state equation is expressed as:

$$z = R - S$$

Where:

R	=	maximum bearable shear force of clay layer [kN/m^2]
S	=	actual shear force due to wave overtopping [kN/m^2]

Loading equations:

$$S = \left(\left(\gamma_s \cdot d \cdot \sin\alpha\right) + \left(\gamma_w \cdot u_{B,2\%}{}^2 / C^2\right)\right)$$

Resistance (strength) equations:

$$R = \sin\theta \cdot T_r \cdot A_r / A + \left(\cos\theta \cdot T_r \cdot A_r / A + \gamma' \cdot d\cos\alpha\right)\tan\varphi'$$

Parameter definitions:

d	=	depth to failure surface [m]
α	=	dike inner slope angle [°]
φ'	=	effective soil angle of friction [°]
γ'	=	submerged soil unit weight [kN/m^3]
γ_s	=	saturated soil unit weight [kN/m^3]
γ_w	=	water unit weight [kN/m^3]
T_r	=	root tensile strength [kN/m^2]
A_r / A	=	root area ratio [-]
Θ	=	root angle of shear rotation [°]
$u_{B,2\%}$	=	overtopping velocity (Schüttrumpf and Van Gent, 2003) [m/s]
C	=	Chezy [m$^{0.5}$/s]

Sources of failure mechanism equations / methods:
Hewlett H W M, Boorman L A, Bramley M E (1987); Marsland A (1966); Möller J, Weißmann R, Schüttrumpf H, Kudella M, Oumeraci H, Richwien W, Grüne J (2002); Oumeraci H & Schüttrumpf H (2004); Young, M and Hassan R. (2006)

Sources of uncertainties in failure equations / input parameters:
Turf set-off is pictured as superficial sliding to occur just under the turf. This sliding mechanism presumes a condition of full saturation, and seepage parallel to the surface. These conditions will only be met if the soil response is dominated by the soil structure, and after an initial period to allow saturation.

Remarks:
The dominant effect of soil structure, present in the top part of the cover layer, makes it feasible for superficial sliding to occur just under the turf. The depth considered is of the order of 30cm below the surface, which is significantly shallower than normally contemplated for geotechnical sliding failure.
The sliding model developed assumes full saturation, and parallel seepage flow within the turf layer. As such, no soil cohesion is considered and the structured soil gains its strength from composite action with the grass roots.
At this shallow depth, shear stress applied by the overtopping flow is significant (additional 25% load), and has been included in the limit state equation for sliding

Reliability or factor of safety, assessed with this LSE becomes marginal for values of root area ratio less than 0.005%, below 30cm depth.

Status of Draft

Date	Revision	Prepared by	Organisation	Approved by	Notes
		R. Hassan	IHE		
				RMH	
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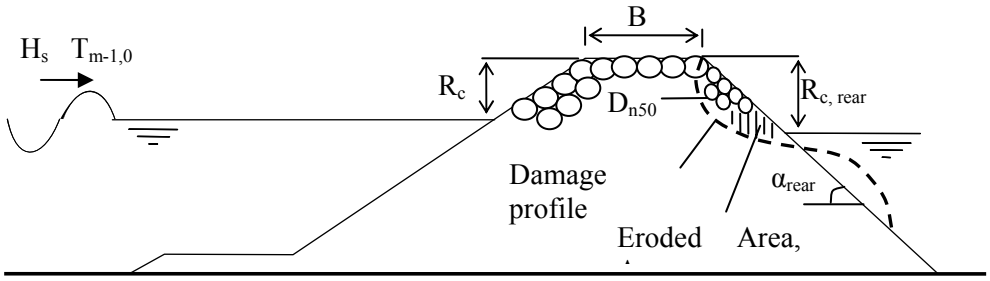
Ba 2.4c Erosion of downstream face / slope (rubble mound)

Summary: Erosion of rubble materials on crest / rear slope of rubble mound or revetment by wave overtopping.



Limit State Equation:
The limit state function is described as:

$$z = D_{n50,act} - D_{n50}$$

with:

$D_{n50,act}$ = actual average stone diameter of rubble mound structure [m]
D_{n50} = required stone diameter for a given amount of acceptable damage [m]

Loading equations:

$$D_{n50} = \left(\frac{S_d}{\sqrt{N}}\right)^{-1/6}\left(\frac{u_{1\%}T_{m-1,0}}{\sqrt{\Delta}}\right)(\cot\alpha_{rear})^{-2.5/6}\left(1+10\exp\left(-R_{c,rear}/H_s\right)\right)^{-1/6}$$

with:

$$u_{1\%} = 1.7\left(g\gamma_{f-c}\right)^{0.5}\left(\left(R_{u1\%}-R_c\right)/\gamma_f\right)^{0.5}/\left(1+0.1B/H_s\right)$$

where

$$R_{u1\%}/(\gamma H_s) = c_0\xi_{s-1,0} \quad \text{for } \xi_{s-1,0} \le p$$

$$R_{u1\%}/(\gamma H_s) = c_1 - c_2/\xi_{s-1,0} \quad \text{for } \xi_{s-1,0} \ge p$$

The table below shows damage levels S_d corresponding to start of damage, intermediate damage and failure for five different slope angles given by cot α.
Van Gent and Pozueta (2004) suggest to use the damage levels corresponding to 'intermediate damage' as acceptable damage levels.

X	1.5	2	3	4	6
start of damage	2	2	2	2	2
inter-mediate damage	3-5	4-6	6-9	8-12	8-12
failure	8-12	8	12	17	17

Resistance (strength) equations:
$D_{N50,act}$ = actual average stone diameter

Parameter definitions:

H_s = significant wave height of the incident waves at the toe of the structure [m]
$T_{m-1,0}$ = spectral wave period, also called the energetic wave period [s]
Δ = the relative density of cover layer, $\Delta = (\rho_r-\rho_w)/\rho_w$ [-]
D_{n50} = nominal mean diameter of the material on the rear side slope [m]
S_d = damage parameter [-]; $S_d = A_e / D_{n50}^2$, with A_e = eroded area [m^2]
N = number of waves [-]
α_{rear} = angle of the rear side slope [°]

$R_{c,rear}$	=	crest freeboard relative to the water level at rear side of the crest [m]
$u_{1\%}$	=	maximum velocity (depth-averaged) at the rear side of the crest [m/s] during a wave overtopping event, exceeded by 1% of the incident waves
B	=	crest width [m]
R_c	=	crest level relative to still water at the seaward side of the crest [m]
γ_f	=	roughness of the seaward slope [-]; $\gamma_f = 0.55$ for rock slopes, $\gamma_f = 1$ for impermeable slopes
γ_{f-c}	=	roughness at the crest [-]; $\gamma_{f-c} = 0.55$ for rock crests, $\gamma_{f-c} = 1$ for impermeable crests
$R_{u1\%}$	=	fictitious run-up level [m], exceeded by 1% of the incident waves
c_0	=	1.45, $c_1 = 5.1$, $c_2 = 0.25\ c_1^2 / c_0$ and $p = 0.5\ c_1 / c_0$
γ	=	the reduction factor [-]; $\gamma = \gamma_f \gamma_\beta$, taking into account the effects of angular wave attack (γ_β , which can be approximated by: $\gamma_\beta = 1 - 0.0022 \cdot \beta$, where $\beta \leq 80°$) and roughness (γ_f)
$\xi_{s-1,0}$	=	the surf-similarity parameter [-], defined as $\xi_{s-1,0} = \tan\alpha / \sqrt{(2\pi / g \cdot H_s / T_{m-1,0}^2)}$

Sources of failure mechanism equations / methods:
Van Gent, M. and Pozueta, B. (2004)

Sources of uncertainties in failure equations / input parameters:

Remarks:
see also Ab 2.1 and Ab 2.4

Status of Draft

Date	Revision	Prepared by	Organisation	Approved by	Notes
			WL /Delft		
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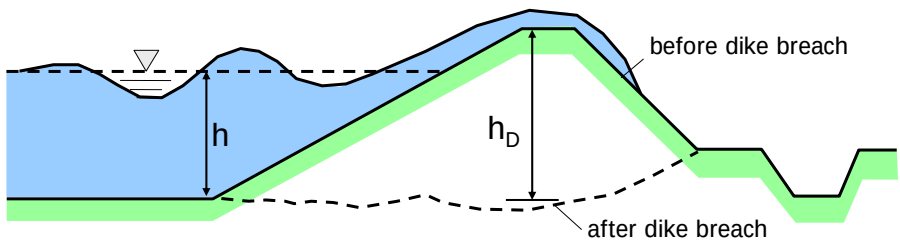
Ba 2.4d Erosion of core by wave overtopping

Summary: Erosion of core materials by wave overtopping.



Reliability equation:
The reliability function is expressed by:

$$z = t_3 - t_s$$

where:

t_3 = total duration of breach [h];
t_s = storm duration [h]

Loading equations:
Duration of storm t_s, e.g.: t_s= 6,5 h

Resistance (strength) equations:
Time for total breach of dike
Overflow:

$$t_3 = t_2 + \frac{2}{f_2 \cdot k_2} \cdot \left(\sqrt{h_w} - \sqrt{h_w - h_{k,0}}\right)$$

Overtopping and overflow ($h_w \leq h_k + h_{ok}$):

$$t_{3a} = t_2 + \frac{z_{98} \cdot \left[1 - \exp\left(1{,}833 \cdot \left(\frac{h_{k,o} - h_w}{z_{98}}\right)\right)\right]}{1{,}833 \cdot f_2 \cdot c_2 \cdot q_0^{1/3}}$$

$$t_3 - t_{3a} = \frac{2 \cdot c_2\, q_0}{f_2 \cdot k_2^2} \left(\frac{k_2}{c_2 \cdot q_0} \cdot h_w^2 - \ln\left[1 + \frac{k_2}{c_2 \cdot q_0} \cdot \sqrt{h_w}\right]\right)$$

Erosion of inner slope:

$$t_2 = t_1 + \frac{B_k}{c_1 \cdot q^{1/3}}$$

$$t_1 = t_0 + \frac{l_t}{c_0 \cdot q^{1/3}}$$

$$c_0 = \frac{0{,}01}{(1-p)\Delta} \cdot \left(\frac{g}{C_f}\right)^{1/3} \cdot \frac{(\sin\beta)^{1/3}}{\cos\beta}$$

$$c_1 = \frac{0{,}01}{(1-p)\Delta} \cdot \left(\frac{g}{C_f}\right)^{1/3} \cdot \frac{(\sin\beta_1)^{1/3}}{\cos\beta_1}$$

Parameter definitions:

h_W	=	still water level at toe of dike [m]
h_k	=	height dike crown [m]
$h_{o,k}$	=	critical water level above crown height [m]
t_3	=	time of complete erosion of dike in phase III (total breach) after Visser (1999), modified after Kortenhaus (2003) [h]
z_{98}	=	wave run-up height according to Schüttrumpf (2001) or others [m]
t_2	=	time till the end of phase II (erosion of inner slope) [h]
f_2	=	coefficient for deceleration of erosion process [-]
$h_{k,0}$	=	original crown height of dike [-]
q_0	=	wave overtopping rate [l/s·m] for crest freeboard $R_c = 0$
t_1	=	time till the end of phase I (erosion of cover layers) [h]
t_0	=	start time of erosion if inner slope [h]
l_t	=	partial length of the dike at the inner toe [m]
q	=	mean overtopping rate [l/(s·m)]
p	=	porosity of sand bed [-]
Δ	=	relative density of sand [-]
C_f	=	friction of sand bed [-]
β	=	outer slope [°]
β_1	=	internal friction angle of sand [°]
g	=	gravitational constant [m/s^2]

Sources of failure mechanism equations / methods:
Visser, P.J. (1995); Kortenhaus, A. (2003)

Sources of uncertainties in failure equations / input parameters:

Remarks:
The LSE describes the first phases of breach in a sand core / bank as given by the model of Visser (1995). It assumes that all cover layers (grass, clay) have been eroded before and calculates the time for failure of the three first phases after Visser.

WARNING! LSE is time dependent and needs to be combined with other time-dependent processes before calculating failure probabilities. Results can differ significantly if calculated individually.

Status of Draft

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			LWI		
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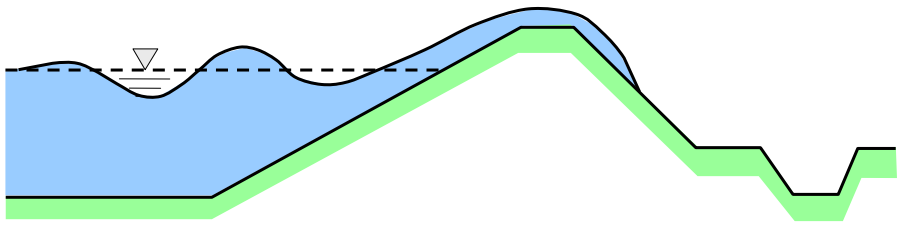
Ba 2.5 Excessive wave overtopping of dike (functional failure)

Summary: Functional failure of flood dyke / embankment by excessive wave overtopping, either by reference to the receiving area (Task 8) or by virtue of failing to deliver adequate resitance to hazards. (Will need to note that a range of calculation methods are available)



Reliability equation:
Wave overtopping rate exceeds admissible rate of water behind the sea defence. The "load" is the actual wave overtopping rate; the "strength" is a critical rate which is higher than the limit for inundation. The reliability function is expressed by:

$$z = q_{adm} - q$$

where:

q_{adm}	=	admissible wave overtopping rate [l/(s·m)]
q	=	actual wave overtopping rate [l/(s·m)]

Loading equations:
Overtopping :

$$\frac{q}{\sqrt{2gH_s^3}} = Q_0 \cdot \exp\left(-5{,}5\frac{R_c}{z_{98}}\right)$$

where:

$$Q_0 = \begin{cases} 0{,}038\,\xi_d & \text{for } \xi_d < 2{,}0 \\ \left(0{,}096 - \dfrac{0{,}160}{\xi_d^3}\right) & \text{for } \xi_d \geq 2{,}0 \end{cases}$$

$$\xi_d = \frac{\tan\alpha}{\sqrt{H_s / L_{0m}}}$$

$$L_{0m} = \frac{gT_m^2}{2\pi}$$

Resistance (strength) equations:
Admissible wave overtopping rate e.g. q_{adm}= 10 l/sm:

These overtopping limits need to be associated with given service states, see CLASH WP6 etc

Parameter definitions:

z_{98}	=	wave run-up height at slope (2% probability of exceedence) [m]
H_s	=	significant wave height at toe of dike [m]
T_m	=	mean wave period of incident waves [s]
α	=	outer slope [°]
L_{0m}	=	wave length at toe of dike relating to T_m [m]
R_c	=	crest freeboard [m]

Sources of failure mechanism equations / methods:

Allsop N.W.H., Bruce T., Pearson J., Franco L., Burgon J. & Ecob C. (2004) *Safety under wave overtopping – how overtopping processes and hazards are viewed by the public* Proc. 29th ICCE, Lisbon, publn. World Scientific, pp 4263-4274, ISBN 981-256-298-2.

Allsop N.W.H., Bruce T., Pearson J. Alderson J.S. & Pullen T. (2003) *Violent wave overtopping at the coast, when are we safe?* Int. Conf. on Coastal Management 2003, pp 54-69, ISBN 0 7277 3255 2, publn. Thomas Telford, London.

Besley P. (1999) *Overtopping of seawalls – design and assessment manual* R & D Technical Report W 178, ISBN 1 85705 069 X, Environment Agency, Bristol,

Schüttrumpf H (2001), *Wellenüberlaufströmung bei Seedeichen – Experimentelle und theoretische Untersuchungen*, Ph.D.-Thesis, Leichtweiss Institut für Wasserbau, Braunschweig.

TAW (2002): Wave run-up and overtopping at dikes. Technische Adviescommissie voor de Waterkeringen (TAW), Den Haag, The Netherlands, 63 pp.

Sources of uncertainties in failure equations / input parameters:
Model uncertainty on discharge for simpler sections is around x / ÷ 3x, may be x / ÷ 10-50x for complex structures or less-frequently tested cases.

Remarks:
Different formulae for wave overtopping rates are available for sloping structures. All can be used to calculate overtopping probabilities. Sometimes more stable to use crest heights in LSE when calculating P_f by FORM rather than overtopping rate.

Needs to identify the structure type, linking appropriate overtopping prediction method to the particular structure type.

Also see Ba 2.4.

Status of Draft

Date	Revision	Prepared by	Organisation	Approved by	Notes
			HRW		
				WA / MM	
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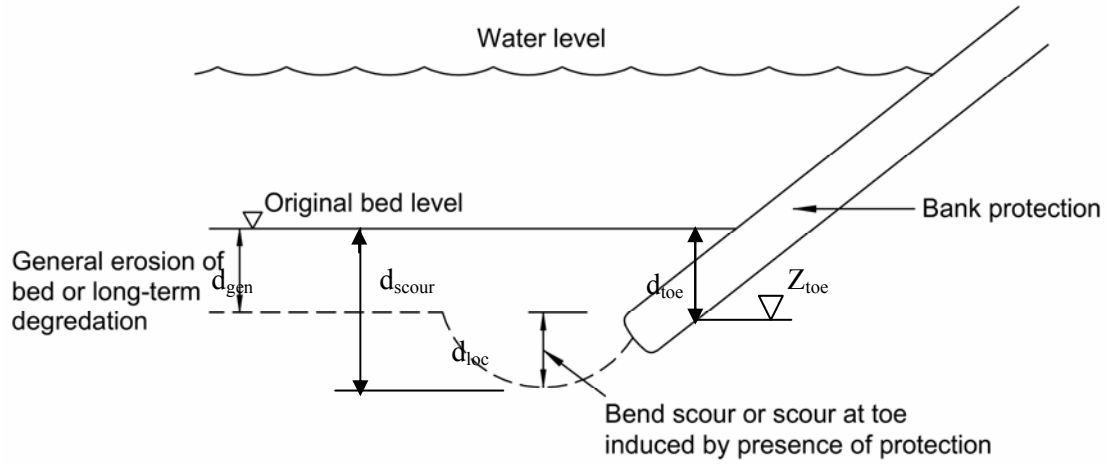
Ba 3.1 Erosion of toe of protection

Summary: Erosion of toe of river bank protection leading to failure which may be due to a) localised scour along the toe of the revetment, b) general erosion of the bed or, c) long-term degradation of the bed.



Reliability equation:
The reliability function is expressed as follows:

$$z = d_{toe} - d_{scour}$$

where:

d_{toe} = Protected depth of toe, determine from original bed level to the lowest point of the toe structure [m]

d_{scour} = total scour depth in front of the toe structure, it includes general erosion, long term degradation of the bed and localised scour along the toe structur [m]

Loading equations:

$$d_{scour} = d_{gen} + d_{loc}$$

where:

$$d_{loc} = y_{bs} - \left(Z_0 - d_{gen}\right)$$

For $1.5<r_c/W<10$ and $20<W/y_u<125$:

$$\frac{y_{bs}}{y_u} = 1.8 - 0.051\left(\frac{r_c}{W}\right) + 0.0084\left(\frac{W}{y_u}\right)$$

for $\frac{r_c}{W} < 1.5$: $\frac{r_c}{W} = 1.5$

for $\frac{W}{y_u} < 20$: $\frac{W}{y_u} = 20$

Resistance (strength) equations:

$$d_{toe} = Z_0 - Z_{toe}$$

where:

Z_o is determined from design profile/ actual measurement;

Z_{toe} is the lowest point of the toe structure

Parameter definitions:

A	=	cross-sectional area [m^2]
d_{scour}	=	depth of scour at toe of protection [m]
d_{loc}	=	localised scour along the toe structure at outer bend [m]
d_{toe}	=	protected depth of toe [m]
d_{gen}	=	general erosion, long term degradation of the bed level [m]
r_c	=	radius of curvature of bend [m]
W	=	water surface width [m]
y_{bs}	=	depth of water at bend [m]

y_u	=	average flow depth (A/W) in the channel upstream of the bend [m]
Z_o	=	initial, unscoured bed level adjacent to toe of protection [m]
Z_{toe}	=	level of toe of protection, taking into account the deployment of any falling or launching apron (the lowest point of the toe structure) [m]

Sources of failure mechanism equations / methods:
Melville B W and Coleman S E (2000); CUR (1995), Maynord (1996)

Sources of uncertainties in failure equations / input parameters:
There is significant uncertainty in the description of the various physical processes that can lead to erosion of the toe.

Remarks:
Failure of protection due to erosion of the toe is one of the commonest causes of failure.
d_{gen}: can be determined by using numerical models, see Floodsite Task 5.

In the first case scour may result from a number of causes. For example, scour may arise as a result of the construction of a hard revetment or scour may arise due to a bend. There are empirical rules of thumb of doubtful origin and validity that the depth of scour caused by the construction of a revetment may be between 30 and 40% of the depth of flow (CUR,1995). There are also equations for predicting scour at bends which have a slightly more credible origin, (Melville and Coleman, 2000).

Status of Draft

Date	Revision	Prepared by	Organisation	Approved by	Notes
		R. Bettess	HRW		
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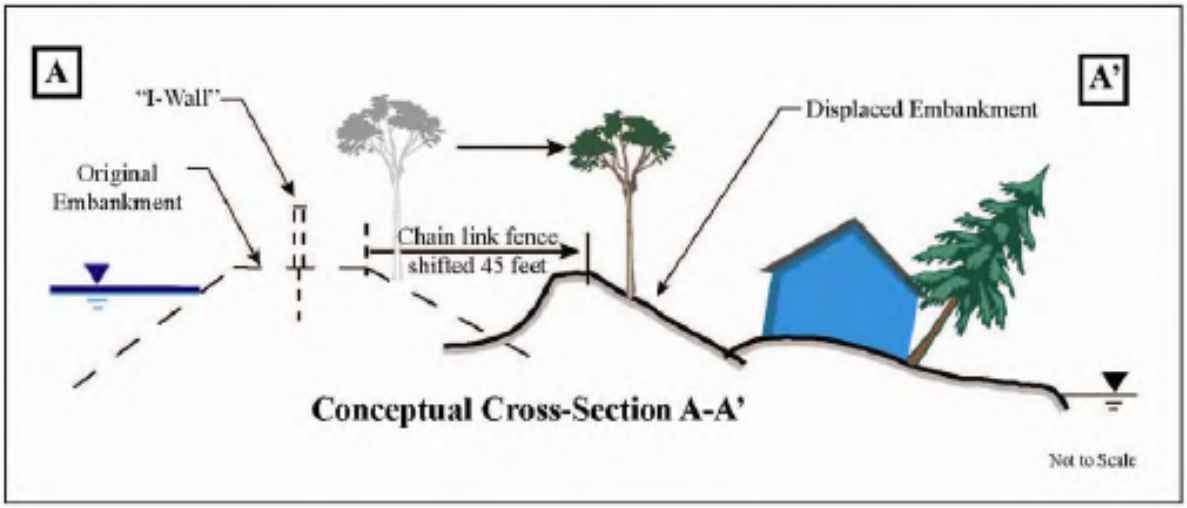
Bb1.2 Sliding of embankment

Summary: The horizontal sliding of an embankment is a complicated process that typically depends on an adverse combination of several rather than a single factor. The factors that could lead to sliding include

- reduction in deadweight of organic fill material in embankment due to seasonal dessication
- increase in hydrostatic horizontal loading due to formation of deep tension cracks
- increase in uplift pressures beneath embankment in confined founding strata that is in hydraulic continuity with flood water.



Reliability equation:

The reliability equation is expressed by:

$$z = R - P_{wh}$$

where:

R = shear strength of entire system [kN/m^2]
P_{wh} = horizontal component tof water pressure (hydrostatic) [kN/m^2]

Loading equations:

$$P_{wh} = \frac{1}{2}\gamma_w h^2$$

Resistance (strength) equations:

$$R = S + P_p$$

Shear strength S:
Drained condition:

$$S = \left(A_{unsat}\gamma_{unsat} + A_{sat}\gamma_{sat} + A_{fg}\gamma_{fg} - \gamma_w hl\right)\tan\phi'$$

Undrained condition:

$$S = S_u l$$

Passive earth resistance P_p:

$$P_P = \frac{1}{2}d^2\gamma_{fg} + 2dS_U$$

Parameter definitions:

A_{unsat} = Area (volume/unit length) of the unsaturated part of the embankment [m^3/m]
A_{sat} = Area of the saturated part of the embankment [m^2]
A_{fg} = Area of the fine grained soil beneath the embankment [m^2]

γ_{unsat}	=	unit weight of the unsaturated part of the embankment [kg/m^3]
γ_{sat}	=	unit weight of the saturated part of the embankment [kg/m^3]
γ_{fg}	=	unit weight of the fine grained natural soil beneath the embankment (saturated) [kg/m^3]
γ_w	=	unit weight of water [kg/m^3]
h	=	maximum hydrostatic head due to water filled tension crack or height of flood water above toe of embankment [m]
l	=	length of horizontal sliding surface beneath embankment [m]
ϕ'	=	angle of internal friction of founding strata (eg coarse grained soil) [°]
d	=	thickness of the fine grained natural soil layer [m]
S	=	shear strength of soil [kN/m^2]
S_u	=	undrained shear strength of the fine grained soil [kN/m^2]

Sources of failure mechanism equations / methods:
Bezuijen A., Kruse G.A.M., Van M.A. (2005)

Sources of uncertainties in failure equations / input parameters:
Rather than uncertainty in parameters the main problem is to recognise on site the conditions triggering this type of sudden failure. A crucial issue is the origin and propagation of the vertical crack.

Remarks: Although horizontal sliding of flood embankments occurs and has been observed, it is nevertheless very difficult to accurately diagnose the actual failure mechanism(s). This is illustrated by the lack of agreement at times in the publish literatre on the interpretation of notable case studies such as embankment failure in the Netherlands in 2003.

The shear resistance along the base of the embankment can be estimated using drained or undrained soil strength parameters depending on the grain size. In the case of coarse grained soils (sands or gravels) the drained shear strength of the founding strata would be reduced due to any increase in pore pressure beanth the embankment, such as the presence of a confined stratum of soil in hydraulic continuity with the flood waters. In contrast the shear strength of a fine grained soil woul din the short temr be undrained and depend on the undrained shear strength. Likewise the horizontal load is given by the horizontal component of the water pressure, which could be present in the form of a water filled tension crack within the main body of the embankment or simply the maximum hydrostatic load imposed on esternally on the embankment body. In any case the performance function can be expressed in both drained and undrained shear strength parameters.

Status of Draft

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		Mark Dyer			
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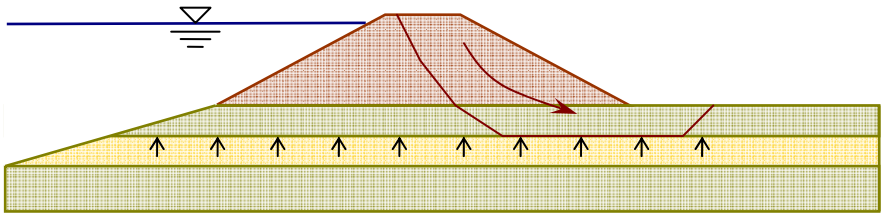
Bb 1.3a Non circular deep slip (uplift pressures from foundation)

Summary: A non circular failure mode can be generated by a significant increase in hydrostatic pressures beneath the embankment in a confined stratum of highly permeable soil in hydraulic continuity with flood water, which leads to uplift of the toe of the embankment on the landward side as illustrated below. The Janbu method is recommended for LEA modeling of a non circular failure mode.



Reliability equation for simplified Janbu's method:

The reliability function is expressed by:

$$z = Fs - 1$$

with: $Fs = \frac{\sum R_M}{\sum S_M}$

where::

$\sum R$ = sum of resisting forces for *n* slices [kN]

$\sum S$ = sum of driving forces for *n* slices [kN]

Loading equations:

$$S = \sum_{i=1}^{n}\left[\left(N'_i + U_{\alpha_i}\right)\sin\alpha_i - U_{\beta_i}\sin\beta_i\right]$$

Resistance (strength) equations:

$$R = \sum_{i=1}^{n}\left[Q_i \sin\delta_i + \frac{c_i + N_i'\tan\phi'_i}{Fs}\cos\alpha_i\right]$$

Parameter definitions:

N'	=	effective normal force [kN]
U_α	=	pore water force [kN]
α	=	inclination of slice base
U_β	=	surface water force [kN]
β	=	inclination of slice top [°]
Q	=	external load (surcharge) [kN]
δ	=	inclination of surcharge [°]
C	=	soil cohesion [kN]
ϕ'	=	internal friction angle [°]
Fs	=	factor of safety [-]

Sources of failure mechanism equations / methods:
Randolph and Marsland (198x, Geotechnique); Padfield and Schofield (198x, Geotechnique); Bromhead, E.N. (1986); Janbu N. (1954)

Sources of uncertainties in failure equations / input parameters:
Remarks: From a reliability analysis viewpoint, calculation of a margin of safety is more desirable than a factor of safety, but all the LEM give factors of safety. Therefore a performance function in terms of safety margin cannot be defined. To calculate the most critical surface with associated minimum factor of safety, iterations need to be done. To perform probabilistic calculations, factors of safety for different values of the parameters must be determined through a deterministic approach beforehand. Preliminary studies may be made using Janbu's simplified procedure (without vertical interslice forces) and correction factors. For more accurate studies, Janbu's Generalized Procedure of Slices (GPS) method, Spencer's procedure, the Morgenstern-Price method, or Sarma's method are available for a more rigorous analysis. All these methods require considering two equilibrium equations: one for moment equilibrium and another one for horizontal forces. The factor of safety is found iteratively and satisfies both equations.

Status of Draft

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		Mark Dyer			
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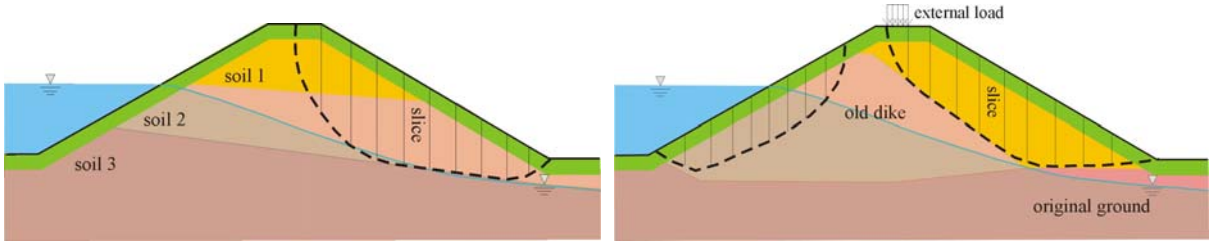
25/03/ 07	v4_4_p03	Stefano Utili	UStrath		edited

Bb 1.3b Non circular deep slip (composite embankments)

Summary: Deep slip in inner or outer face of composite embankment. The existence of discontinuities, stratification, nonhomogeneity, former slides, etc, influences the shape of actual slip surface, which can not be assumed as circular (Bishop's method should not be used – instead of it Janbu,'s, Spencer's, Morgenstern-Price or Sarma's methods are recommended).



Reliability equation for simplified Janbu's method:

The reliability function is expressed by:

$$z = Fs - 1$$

with:

$$Fs = \frac{\sum R_M}{\sum S_M}$$

where:

$\sum R$ = sum of resisting forces for *n* slices

$\sum S$ = sum of driving forces for *n* slices

Loading equations:

$$S = \sum_{i=1}^{n}\left[\left(N'_i + U_{\alpha_i}\right)\sin\alpha_i - U_{\beta_i}\sin\beta_i\right]$$

Resistance (strength) equations:

$$R = \sum_{i=1}^{n}\left[Q_i \sin\delta_i + \frac{c_i + N_i'\tan\phi'_i}{Fs}\cos\alpha_i\right]$$

Parameter definitions:

N'	=	effective normal force [kN]
U_α	=	pore water force [kN]
α	=	inclination of slice base
U_β	=	surface water force [kN]
β	=	inclination of slice top [°]
Q	=	external load (surcharge) [kN]
δ	=	inclination of surcharge [°]
C	=	soil cohesion [kN]
ϕ'	=	internal friction angle [°]
Fs	=	factor of safety [-]

Sources of failure mechanism equations / methods:
Abramson, L.W., Lee, T.S., Sharma, S., Boyce, G.M. (2002); Bromhead, E.N. (1986); Janbu N. (1954); Sarma S.K. (1973)

Sources of uncertainties in failure equations / input parameters:
Soil parameters, lack of detailed information on internal structure of the embankment (unknown discontinuities, tension cracks, pre-existing slip surfaces, zones of different materials), nature of

foundations, probable shape of the slip surface
Remarks: From a reliability analysis viewpoint, calculation of a margin of safety is more desirable than a factor of safety, but all the LEM give factors of safety. Therefore a performance function in terms of safety margin cannot be defined. To calculate the most critical surface with associated minimum factor of safety, iterations need to be done. To perform probabilistic calculations, factors of safety for different values of the parameters must be determined through a deterministic approach beforehand. Preliminary studies may be made using Janbu's simplified procedure (without vertical interslice forces) and correction factors. For more accurate studies, Janbu's Generalized Procedure of Slices (GPS) method, Spencer's procedure, the Morgenstern-Price method, or Sarma's method are available for a more rigorous analysis (Abramson at al., 2002). All these methods require considering two equilibrium equations: one for moment equilibrium and another one for horizontal forces. The factor of safety is found iteratively and satisfies both equations. In case of embankments having composite internal structure and subjected to complex loading conditions, finite element analysis (FEM) could be also used (PLAXIS code for example), giving much more information than slice methods, but not analytical expressions for reliability analysis (DL)

Status of Draft

Date	Revision	Prepared by	Organisation	Approved by	Notes
		Mark Dyer			
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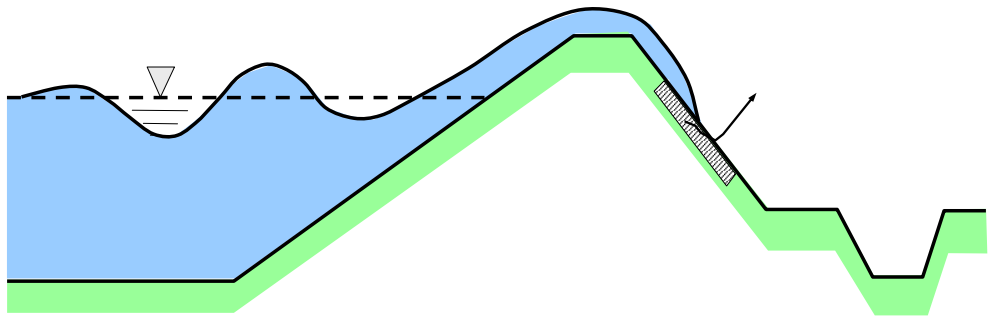
25/03/ 07	v4_4_p03	Stefano Utili	UStrath		edited

Bb 1.4 Clay uplift at inner slope of sea dikes

Summary: Localised failure on inner slope driven by wave overtopping and/or overflow, and change of strength caused by infiltration.



Reliability equation:
The reliability function is expressed by:

$$z = R - S$$

where:

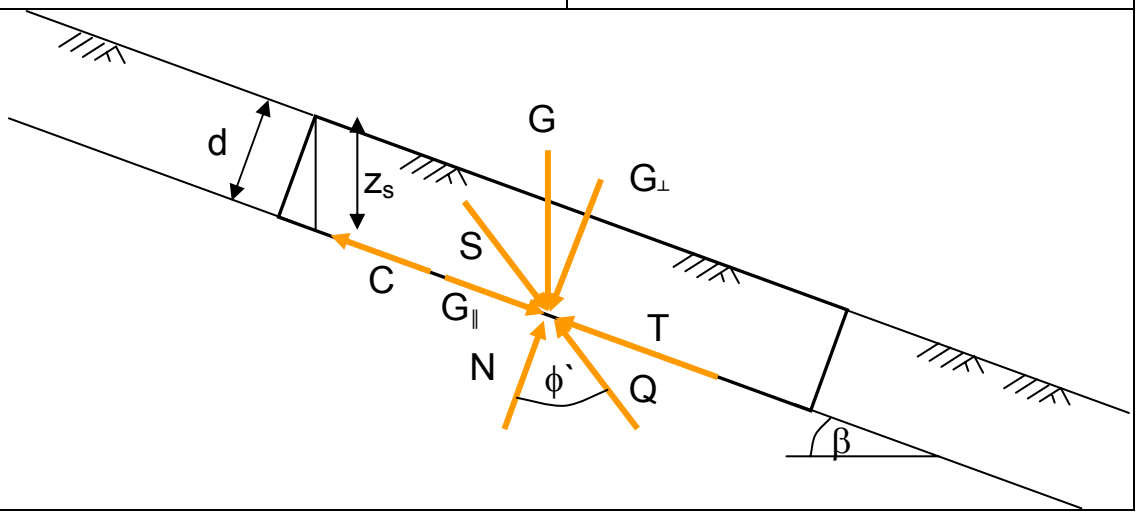
R = mass forces of clay [kN]
S = uplift forces [kN]

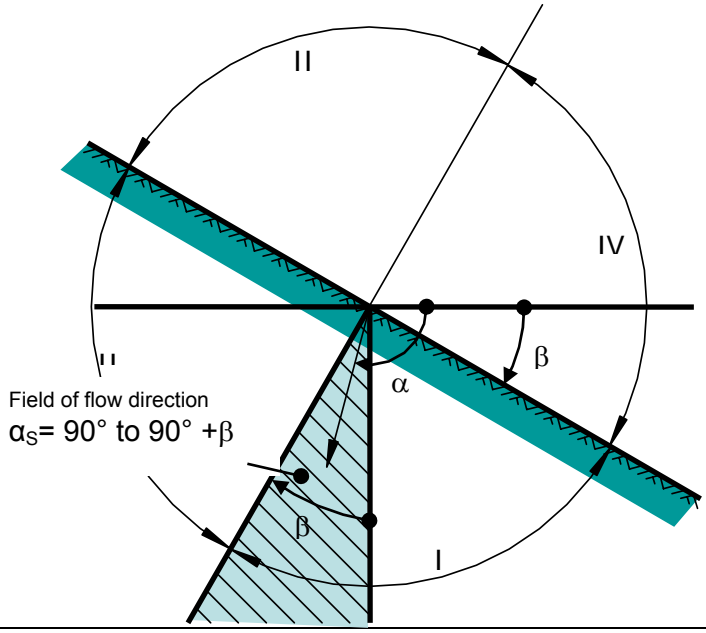
Loading equations:
Uplift forces:

$$S = i \cdot \gamma_w \cdot \sin(\alpha_S - \beta - 180°)$$

Resistance (strength) equations:
Resisting mass forces calculated from:

$$R = \gamma' \cdot \cos\beta$$





Parameter definitions:

z_s	=	depth of slope area effected by flow [m]
d	=	thickness of soil element [m]
α_s	=	flow direction [°].
β	=	slope [°]
γ_w	=	volume weight of water [kN/m^3]
γ‘	=	saturated volume weight of soil [kN/m^3]
φ	=	shear strength of undrained soil [kN/m^2]
i	=	hydraulic gradient [-]
c‘	=	cohesion of undrained soil [kN/m^2]

Sources of failure mechanism equations / methods:

Richwien, W.; Weißmann, R. (1999); Kortenhaus, A.; Oumeraci, H. (2002)

Sources of uncertainties in failure equations / input parameters:

Remarks:

LSE compares flow conditions on the inner slope. Failure is defined when the uplift forces induced by flow is larger than the holding forces by the self weight of the soil element.

See also also Ba 1.4

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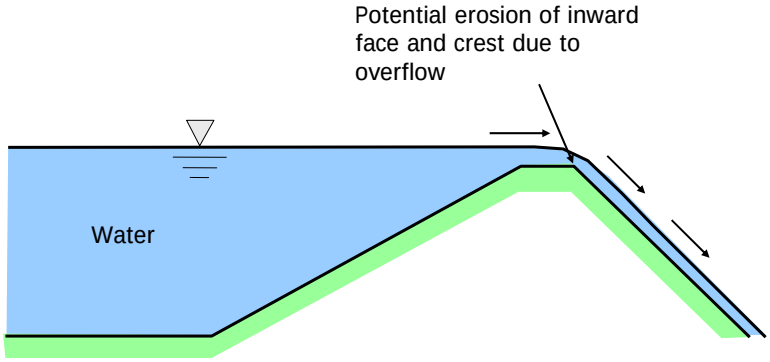
Bc 1.1 Erosion of cover of inner slope by overflow

Summary: Damage to surface when overflow velocity exceed limit for type / condition of revetment cover on crest and inner slope. The "load" is the actual overflowing velocity, u_k. The "strength" is a critical velocity, u_c that exceeds the resistance of the cover.



Reliability equation:

$$z = m_{uc} \cdot u_{cr} - u_k$$

where:

u_{cr}	=	critical velocity of protect material used [m/s]
u_k	=	average velocity on the inner slope [m/s]
m_{uc}	=	model factor for determination of resistance [-]

Loading equations:

Free-discharge weir:

$$u_k = \sqrt{g \cdot d_k}$$

Submerged weir:

$$u_k = \sqrt{2g \cdot (d_0 - d_1)}$$

Resistance (strength) equations:

Critical velocity, u_c, calculated from:

$$u_c = 1.2 \cdot \sqrt{(g \cdot \Delta D)}$$

Parameter definitions:

d_k	=	the water depth above the crest [m]
Δ	=	the relative density of cover layer [-]
D	=	characteristic thickness [m]
g	=	gravity acceleration = 9.81 [m/s^2]
d_o	=	the water level in front of (or at upstream of) the dyke [m]
d_1	=	the water level at downstream of the dike [m]

Sources of failure mechanism equations / methods:

CUR/TAW (1992)

Sources of uncertainties in failure equations / input parameters:

Guidance on model uncertainties;
Identify data on parameter uncertainties, s.d., distribution types;

Remarks:

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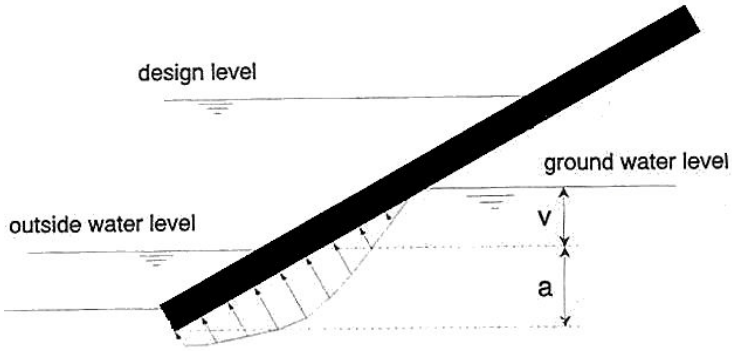
Bc 1.4 Cover layer uplift (falling water level), asphalt revetments

Summary: When the outside water level decreases, but the revetment has low permeability, a perched water surface (elevated phreatic pressures) act on the underside of the revetment. Failure of the cover layer will occur if the the hydraulic gradient is not adequately balanced by the weight of revetment layer, assisted by the soil and friction forces in the subsoil.



Reliability equation:
The reliability equation is expressed as follows:
For cohesionless soil the equilibrium reads:

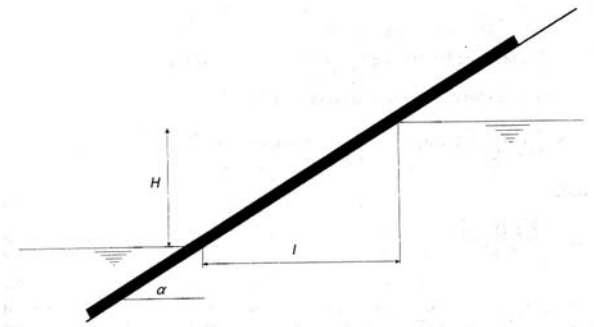
$$z = \tan\varphi - \tan\alpha \cdot \left[1 + \frac{\rho_w}{\rho_g - \rho_a} \cdot \frac{i}{\sin\alpha} \right]$$

where
ρ_q = density of the subsoil [kg/m^3]

Loading equations:
The gradient i can be derived with:

$$i = \sin\alpha - \frac{H \cdot \cos\alpha}{l}$$

See figure below for an explanation of the variables H and l.



Source: Van Herpen, 1998

Resistance (strength) equations:
The strength is expressed in terms of the angle of internal friction of the subsoil (tan φ).

Parameter definitions:

α	= slope angle	[-]
ρ_a	= density of the revetment	[kg/m^3]
ρ_g	= density of the subsoil	[kg/m^3]

ρ_w = density of the water [kg/m^3] φ = angle of internal friction of the subsoil [º]
Sources of failure mechanism equations / methods: TAW, 2004
Sources of uncertainties in failure equations / input parameters: The empirical coefficients and the quality of the cover material form the most important source of uncertainty. The fact that an asphalt revetment is used for several functions at the same time (traffic, water defence, recreation) might also lead to unusual loadings (and hence uncertainties) which have not been taken into account during the design.
Remarks:

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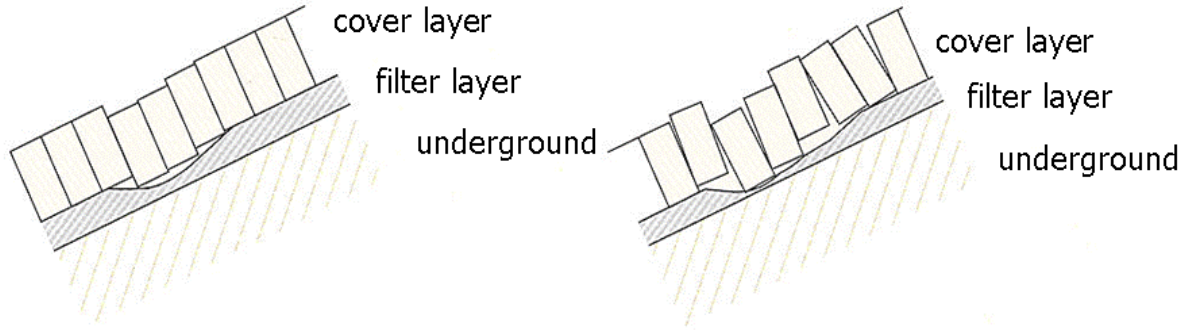
Bc 1.5 Erosion of subsoil through filter or cover layer (block revetments, block mats and concrete mattresses, gabions, geosystems)

Summary: Erosion of finer materials through coarse layers by wave (or steady) hydraulic gradients



Limit State Equation:
Migration of subsoil particles through the filter layer or through the cover layer leads to local erosion of the subsoil near the water level and will result in a local settlement of the filter and cover layer. During wave rundown there is a large piezometric head difference on top of the revetment, which can be quantified in terms of a hydraulic gradient in the filter layer.
Damage occurs when the maximum gradient in the filter layer (*i*) exceeds the limit value (i_{cr}), which is dependent on the filer properties. Hence, the limit state equation can be expressed as:

$$z = i_{cr} - i$$

Loading equations:
The maximum gradient in the filter layer can be computed with:
1. maximum downward gradient: $i = \sin\alpha$
2. maximum upward gradient:

$$i = \cos\alpha \tan\phi \left(1 - \exp\left[\frac{-\phi_b}{2\Lambda \cdot \cos^2\alpha \cdot \tan\phi}\right]\right) - \frac{\sin\alpha}{2} \exp\left[\frac{-\phi_b}{2\Lambda \cdot \cos^2\alpha \cdot \tan\phi}\right]$$

Resistance (strength) equations:
The "strength" is the ratio of the factor $c_{geotextile}$, the (square of the) porosity and the grain size:

The following criteria for geotextiles with O_{90} between 100 and 300 µm on clay or sand can be used (Klein Breteler *et al.*, 1994):

$$i_{cr} = \frac{c_{geotextile}}{n^2 D_{15}}$$

The following values for $c_{geotextile}$ can be used:

- good clay (colloid content = 39%; d_{50} = 9 µm; d_{90} = 80 µm): $c_{geotextile} = 0.1$
- medium and poor clay (colloid content = 20%; 42 µm < d_{50} < 130 µm; 100 µm < d_{90} < 400 µm): $c_{geotextile} = 0.01$;
- fine sand (d_{50} = 90 µm; d_{90} = 130 µm): $c_{geotextile} = 0.001$.

For granular filters a number of filter layers may be required. In each case the relationship between one layer and the next should satisfy the Terzaghi rules:
$D_{85f} > 0.25 D_{15c}$
$D_{50f} > 0.14 D_{50c}$

	$D_{15f}>0.14D_{15c}$ $D_{10f}>0.1D_{60f}$ where f and c denote filter and cover, respectively.

Parameter definitions:

n	=	the porosity of the filter layer (usually $0.3 < n < 0.4$) [-],
D_{n15}	=	the grain size of the granular material on the geotextile [m].
α	=	slope angle [-]
Λ	=	leakage length [m]
φ_b	=	potential head induced in the filter or a gabion [m]
φ	=	angle of internal friction of the revetment material [°]

Sources of failure mechanism equations / methods:
Klein Breteler, M. and A. Bezuijen (1998); TAW (2004)

Sources of uncertainties in failure equations / input parameters:
The empirical coefficients and the quality of clay or s and (content) form the most important source of uncertainty.

Remarks:

Status of Draft

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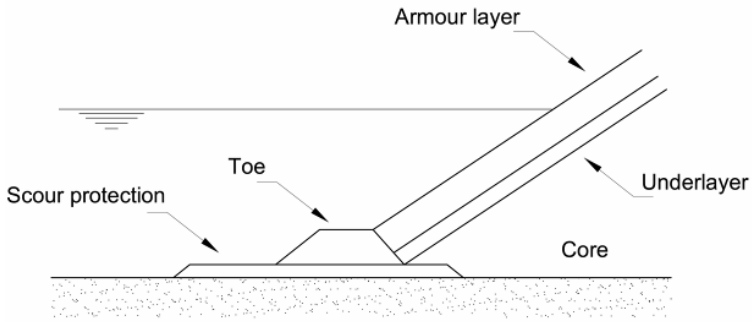
Bc 2.1a Toe erosion to rubble mound slopes

Summary: The most frequent initiating failure mechanism for seawalls is bed scour leading to undermining, loss of fill, local or global slips, armour slide, etc. Unprotected natural beds may scour substantially in current flows and/or waves. This mechanism checks the stability of granular protection berm against damage by waves.



Reliability equation:
The reliabilty equation is written as follows:

$$z = H_{s,max} - H_s$$

where:

$H_{s,max}$ = maximum allowable wave height for given stone diameter [-]
H_s = actual significant wave height Hs in front of structure [-]

Loading equations:
Significant wave height H_s

Resistance (strength) equations:

Maximum bearable wave height $H_{s,max}$:

$$H_{s,max} = (\Delta D_{n50} N_{od}^{0.15})(2 + 6.2h_t / h)^{2.7}$$

with range of validity:

$0.4 < h_t/h < 0.9$
$3 < h_t/D_{n50} < 25$

The formula uses the damage level N_{od} (actual number of displaced stones related to a width, along the longitudinal axis of the structure, of one nominal diameter) as an indicator of the strength. For "standard" toe size of about 3-5 stones wide and 2-3 stones high:
- N_{od} = 0.5, start of damage
- N_{od} = 2.0, some flattening out
- N_{od} = 4.0, damage

Parameter definitions:

H_s = significant wave height [m]
Δ = the relative density of cover layer, $\Delta = (\rho_r - \rho_w)/\rho_w$ [-]
D_{n50} = nominal mean diameter [m]
N_{od} = damage level [-]
h = water depth [m]
h_t = water depth at the toe [m]

Sources of failure mechanism equations / methods:
Van der Meer et al (1998), Van der Meer J., d' Angremond K. and Gerding E (1995) "Toe structure stability of rubble mound breakwaters". ICE Proc. Coastal Structures and Breakwaters '95.

CIRIA / CUR / CETMEF Rock Manual (2006)
Sources of uncertainties in failure equations / input parameters:
Remarks: See also Ab 2.1 and Ca 2.1

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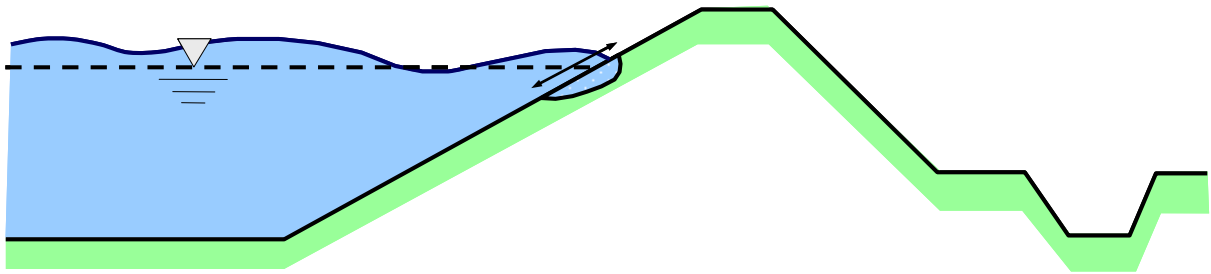
Bc 2.1b Erosion of revetment (grass cover) seaward face by up-rush velocity; and/or by ship waves

Summary: Direct attack on grass revetment cover by wave velocities



Reliability equation:
The reliability function is expressed by:

$$z = v_c - v_A$$

where:

vc = critical velocity [m/s]
v_A = actual velocity [m/s]

Loading equations:
Actual velocity of wave run-up:

$$v_A = 0,75 \cdot \pi \cdot \frac{H_i}{T_i} \cdot k_{si} \cdot m \cdot \sqrt{z_{98} - z_a}$$

with:

$T_i \approx 0.82 \bullet v_s \bullet 2\pi/g$

$$z_{98} = 3,0 \cdot H_i \cdot \tanh\left(0,65 \cdot \xi_{op}\right)$$

<u>Primary waves</u> :

$\xi_{op} = 1$

and

$H = \hat{h}_f$ (front wave)
$H = z_{max}$ (stern wave)

Average water level depression can be determined from:

$$\frac{v_s^2}{gh} = \frac{2\hat{h}/h}{(1 - A_s/A_c - \hat{h}/h)^{-2} - 1}$$

Water level depression $\hat{h}$ at the bank:

$$\hat{h} = \bar{h} \cdot \left(1 + \frac{2yd}{A_c}\right) \quad \text{for } b_w / L_s < 1.5$$

$$\hat{h} = \bar{h} \cdot \left(1 + \frac{4yd}{A_c}\right) \quad \text{for } b_w / L_s \geq 1.5$$

Stern wave: $z_{max} = 1.5 \bullet \hat{h}$

Resistance (strength) equations:
Critical velocity for outer slope:

$$v_c = f_G \cdot q_M \cdot \frac{3,8}{\left(1 + 0,8 \cdot \lg t_s\right)}$$

with:

$$f_G = \begin{cases} 0,7 + 0,7 \cdot q_G & \text{for } 0,0 \leq q_G \leq 1,0 \\ & \text{and } q_M = 1,0 \\ 1,0 & \text{for } q_M \neq 1,0 \end{cases}$$

Secondary waves :

$T_i \approx 0.82 \bullet v_s \bullet 2\pi/g$

$$\xi_{op} = \frac{\tan\alpha}{\sqrt{H_i \cdot g \cdot \left(2\pi \cdot T_i^2\right)}}$$

and

$H = H_i \bullet \cos\beta$

with:

$$\frac{H_i}{h} = \zeta \cdot \left(\frac{s}{h}\right)^{-1/3} \cdot Fr^4$$

ζ is the coefficient of proportionality, representing the ship's geometry. $\zeta = 1.2$ is a reasonable upper limit.

Parameter definitions:

v_A	=	velocity of wave run-up [m/s]
v_c	=	critical velocity [m/s]
t_s	=	duration of storm [h]
f_G	=	parameter for quality of grass [-]
z_{98}	=	wave run-up being exceeded by 2% of all wave run-ups [-]
H_s	=	significant wave height [m]
T_m	=	mean wave period [s]
T_p	=	peak wave period [s]
k_{si}	=	breaker index [-]
m	=	mean outer slope [-]
q_M	=	material quality (1,0 for sand) [-]
q_G	=	quality of grass (between 0,0 und 1,0) [-]
v_s	=	ship's speed (m/s)
A_s	=	area of ship's cross section (m2)
A_c	=	area of canal's cross section (m2)
h	=	water depth (m)
y	=	the eccentricity ship in canal (m)
b_w	=	width of the waterway (m)
L_s	=	the length of the ship (m)
ζ	=	the coefficient of proportionality, representing the ship's geometry.
s	=	distance from the ship's sailing line (m)

Sources of failure mechanism equations / methods:

CUR/TAW (1992); Laustrup, C.; Toxvig, H.; Poulsen, L.; Jensen, J. (1990); Schiereck, G.J. (2001); Schüttrumpf H (2001)

Sources of uncertainties in failure equations / input parameters:

Remarks:

The calculation of the critical velocity is taken from field tests described and analysed by Laustrup (1990). The formula for calculating velocities is based on model tests both under small scale and large scale conditions.

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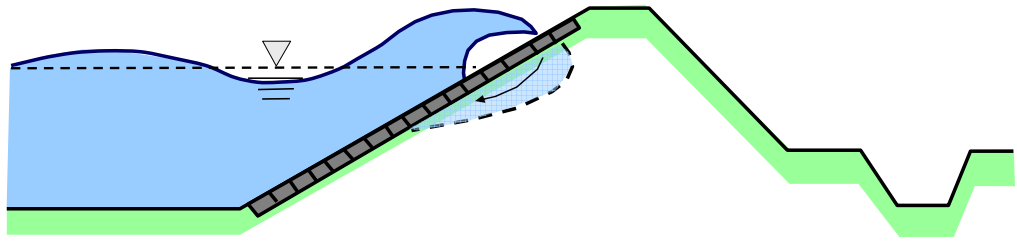
Bc 2.1c Erosion of revetment armour (rock) on seaward face

Summary: Onset of failure of granular mound / slope under direct wave attack. This method is based on simple analysis of onset of motion, as then qualified by empirical data. Three alternative methods might be used.



Limit State Equation:
The reliability function is expressed by:

$$z = D_{n50,actual} - D_{n50}$$

where:

$D_{n50,actual}$ = actual stone diameter of revetment elements [m]
D_{n50} = required stone diameter of revetment elements [m]

Loading equations:
The **Hudson** formula (1953, 1959), originally developed based on tests with regular waves, can be re-written for applications with irregular waves into:

$$D_{n50} = \frac{1.27 H_s}{\Delta \left(K_D \cot \alpha \right)^{1/3}}$$

Van der Meer's (1988) formula reads for "plunging conditions" ($\xi_m \leq \xi_c$):

$$D_{n50} = \frac{H_s}{6{,}2 \cdot P^{0{,}18} \cdot \left(S_d / \sqrt{N_w} \right)^{0{,}2} \cdot \xi_m^{-0{,}5} \cdot \Delta}$$

and for "surging conditions" ($\xi_m > \xi_c$)::

$$D_{n50} = \frac{H_s}{\Delta \cdot 1{,}0 \cdot P^{-0{,}13} \cdot \left(S_d / \sqrt{N_w} \right)^{0{,}2} \cdot \sqrt{\cot \alpha} \cdot \xi_m^{P}}$$

with the damage level S_d defined as:

$$S_d = \frac{A_e}{D_{n50}^2}$$

and $\xi_c = \left[6.2\, P^{0.31} \sqrt{\tan \alpha} \right]^{\frac{1}{P+0.5}}$

An alternative formula is derived by Van Gent *et al.* (2003). It reads:

$$D_{n50} = \frac{H_s}{\Delta \cdot \left(S_d / \sqrt{N} \right)^{0.2} \cdot 1.75 \cot \alpha^{0.5} \cdot \left(1 + D_{n50-core} / D_{n50} \right)}$$

Resistance (strength) equations:
Actual stone diameter $D_{n50,actual}$

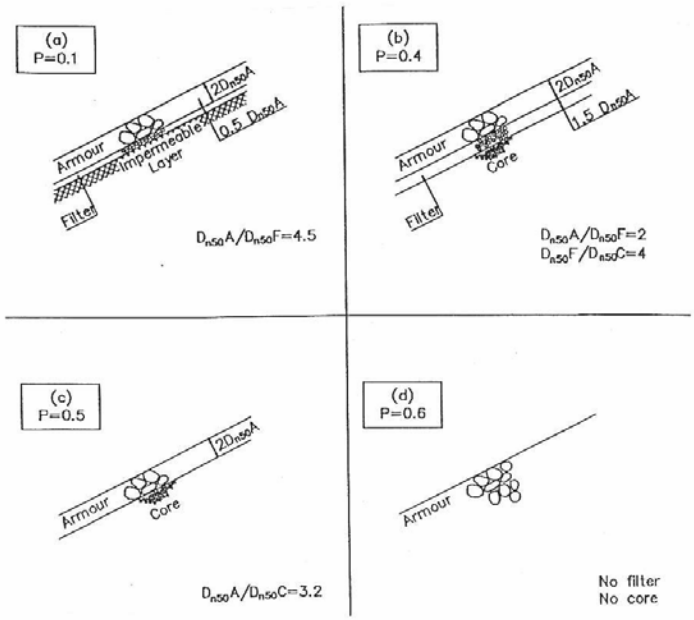
Note that the formula derived by Van Gent *et al*. is calibrated for a selected range of structure geometries (non-homogeneous structures with 1:4 < α < 1:2). Using tests with other structure geometries (*e.g.*, more gentle slopes than 1:4, slopes steeper than 1:2, or homogeneous structures) might lead to different conclusions.

In the Shore Protection Manual of 1984 (CERC, 1984) the following values of K_D were suggested for the Hudson formula:
- for breaking waves : $K_D = 2.0$
- for non-breaking waves : $K_D = 4.0$

The following table shows damage levels S_d corresponding to start of damage, intermediate damage and failure for five different slope angles given by cot α.

cot α	1.5	2	3	4	6
start of damage	2	2	2	2	2
inter-mediate damage	3-5	4-6	6-9	8-12	8-12
failure	8-12	8	12	17	17

The following graph shows values of the permeability, depending on the stone diameter in the different armour and filter layers.



Parameter definitions:

α	=	slope angle [-]
N	=	number of incident waves at toe [-]
S_d	=	damage level [-]
ξ_m	=	$\tan\alpha / \sqrt{\cdot H_s / \left(g \cdot T_m^{\,2} / (2\pi)\right)}$ [-]

P	=	permeability parameter (0.1<P<0.6) [-]
$D_{n50\text{-}core}$	=	the nominal stone diameter of core material [m]

Sources of failure mechanism equations / methods:
Hudson R.Y. (1953); Hudson R.Y. (1959); Coastal Engineering Rresearch Center (1984); Van der Meer J. W. (1988); Van Gent M.R.A., Smale A. & Kuiper C. (2003)

Sources of uncertainties in failure equations / input parameters:
For loose materials the main input parameters are the hydraulic loading parameters (H_s, T_p) and parameters describing the structure, such as the slope angle, specific weight of individual stones, pore pressures and the internal friction and cohesion (interlocking). One of the things that should be taken care of during construction and maintenance of a revetment consisting of loose rock is that the stones do not break because than the weight of the individual stones can no longer be guaranteed. Hence, the design formulae should then be applied with a smaller stone diameter.

Remarks:
See also Ab 2.1b

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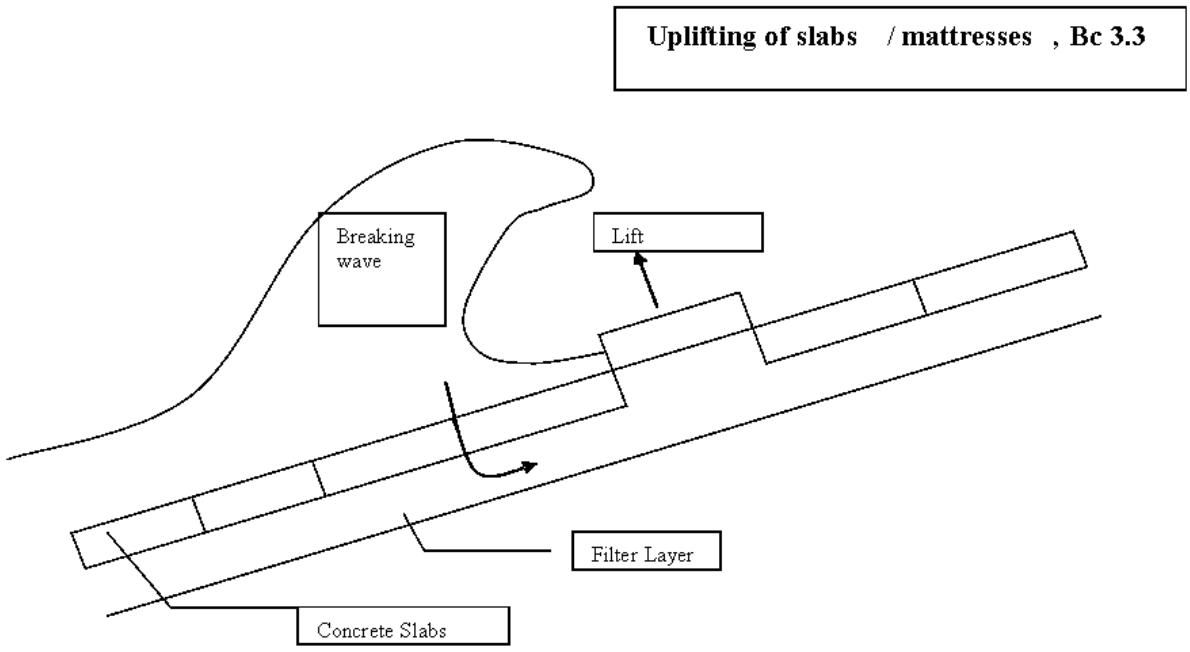
Bc 2.1d Uplift of revetment blocks (placed block revetments, block mats and concrete mattresses, gabions, geosystems) by wind or ship waves

Summary: Failure of revetment armour by up-lift forces dealt with by dimensioning armour thickness (and other factors) in relation to the wave height and wave steepness.



Limit State Equation:
The limit state equation is expressed by:

$$z = \Delta DF - H\xi_{op}^{0.67}$$

where:

H_{scr} = significant wave height at which blocks will be lifted out [m]
H = actual wave height in front of structure [m] due to wind or ship induced waves

Loading equations:
The loading can be due to either wind waves or ship waves.

Wind waves:
For wind waves the wave load is determined by the significant wave height H_s.
Ship waves:
For ship waves a distinction should be made between front and stern waves (both primary waves) and secondary waves.

Front wave: $H = \hat{h}_f$ and $\xi_{op} = 1$
Stern wave: $H = z_{max}$ and $\xi_{op} = 1$
Secondary wave: $H = H_i \cdot \cos\beta$ and $T = T_i$

with:

Resistance (strength) equations:

$$H_{scr} = \Delta \cdot D \cdot F \cdot \xi_{op}^{-0.67}$$

with:

$$\Delta = \frac{(\rho_s - \rho_w)}{\rho_w}$$

$$\xi_{op} = \frac{\tan\alpha}{\sqrt{H_s / \left(g \cdot T^2 / (2\pi)\right)}}$$

The value of F depends on the type of structure:
a) low stability : (k/k').(D/b) < 0.05 – 0.1
b) normal stability : 0.5 – 0.1 > (k/k').(D/b) > 0.05 – 0.1
c) high stability : (k/k').(D/b) > 0.5 – 0.1

$$\xi_{op} = \frac{\tan\alpha}{\sqrt{H_s / \left(g \cdot T^2 / (2\pi)\right)}}$$ <u>Primary waves:</u> *Front wave:* $\hat{h}_f = 0.11 \bullet + \hat{h}$ where the average water level depression can be determined on the basis of the following equation: $$\frac{v_s^{\,2}}{gh} = \frac{2\hat{h}/h}{(1 - A_s/A_c - \hat{h}/h)^{-2} - 1}$$ The water level depression $\hat{h}$ at the bank can now be determined as follows: $$\hat{h} = \bar{h} \cdot \left(1 + \frac{2yd}{A_c}\right) \quad for\ b_w / L_s < 1.5$$ $$\hat{h} = \bar{h} \cdot \left(1 + \frac{4yd}{A_c}\right) \quad for\ b_w / L_s \geq 1.5$$ *Stern wave*: $z_{max} = 1.5 \bullet \hat{h}$ <u>Secondary waves</u>: $$\frac{H_i}{h} = \zeta \cdot \left(\frac{s}{h}\right)^{-1/3} \cdot Fr^4$$ with $T_i \approx 0.82 \bullet v_s \bullet 2\pi/g$ ζ is the coefficient of proportionality, representing the ship's geometry. $\zeta = 1.2$ is a reasonable upper limit.	The range of stability factor F is: 5 < F < 15. The higher values refer to the presence of high friction and/or interlocking of a system.

Parameter definitions:

Δ	=	the relative density of cover layer [-]
D	=	characteristic thickness;
k	=	permeability of sublayer [m/s]; k'= permeability of top layer [m/s]
H_{scr}	=	significant wave height at which blocks will be lifted out [m]
H_s	=	significant wave height [m]
α	=	slope angle [-]
g	=	gravity acceleration = 9.81 [m/s^2]
T_p	=	wave period at peak of spectrum [s]
b	=	thickness of sublayer [m]
D	=	thickness of top layer [m]
v_s	=	ship's speed [m/s]
A_s	=	area of ship's cross section [m^2]
A_c	=	area of canal's cross section [m^2]
h	=	water depth [m]
y	=	the eccentricity ship in canal [m]
b_w	=	width of the waterway [m]
L_s	=	the length of the ship [m]
ζ	=	the coefficient of proportionality, representing the ship's geometry [-]
s	=	distance from the ship's sailing line [m]

Sources of failure mechanism equations / methods:

CUR/TAW (1992); Klein Breteler M. & Bezuijen A. (1991); Klein Breteler, M. & Pilarczyk, K.W. (1998); McConnell K.J. (1998); McConnell K.J. & Allsop N.W.H. (1999); Schiereck, G.J. (2001) TAW (2004)

Sources of uncertainties in failure equations / input parameters:
The wave parameters (H_s and T_p) and the properties that describe the structure, *e.g.* the permeability, the layer thickness, the composition of the filter and cover layer, are the most important input parameters in the design process of placed block revetments. Moreover, the stability factor F forms also a major uncertainty.

Remarks:
The displacement of a block occurs if the uplift pressure exceeds the weight of the block added with the additional forces, such as friction and inertia. The downward pressure is the "strength", the upward pressure is the "load". The stability equation is usually written in the following form.

$$\frac{H_{scr}}{\Delta D} = f\left(\frac{D}{b}\frac{k'}{k}\right)^{0.33} \xi_{op}^{\ -0.67} = F\xi_{op}^{\ -0.67}$$

where:
f = stability coefficient, mainly dependent on structure type, tanα and friction [-]
F = total (black box) stability factor [-]
H_{scr} = significant wave height at which blocks will be lifted out [m]

Status of Draft

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			WL / Delft		
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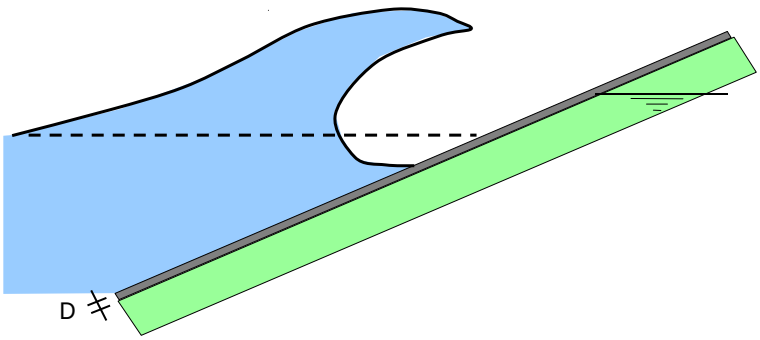
Bc 2.1f Failure of cover layer by wave impact (asphalt revetments)

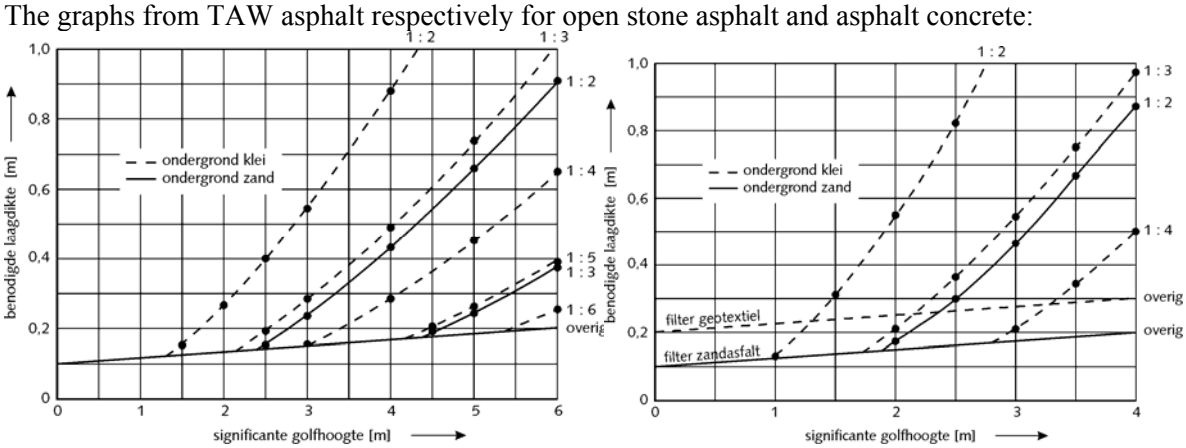
Summary: Plate type asphalt constructions (asphalt concrete, open stone asphalt, mastic and fully grouted stone) can be damaged by inward-acting wave forces. The method is based on ensuring a "safe" armour thickness in relation to the anticipated exposure.



Reliability equation:
PC-Ring page 36/37 for open stone asphalt and asphalt concrete suggests the following limit state equation:

$$z = D - D_{required}$$

Load / Resistance (strength) equations:
The graphs from TAW asphalt respectively for open stone asphalt and asphalt concrete:



From these graphs the required D can be derived. These graphs are only valid when the dwell time of the still water level is 17 hours over a length of 0.5 m and the mean wave period is $T_g = 3.5\ H_s^{1/2}$

Parameter definitions:

Sources of failure mechanism equations / methods:
PC-Ring page 36/37 for open stone asphalt and asphalt concrete
See TAW asphalt report page 125

Sources of uncertainties in failure equations / input parameters:

Remarks:

Status of Draft

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		N. Doorn	WL / Delft		
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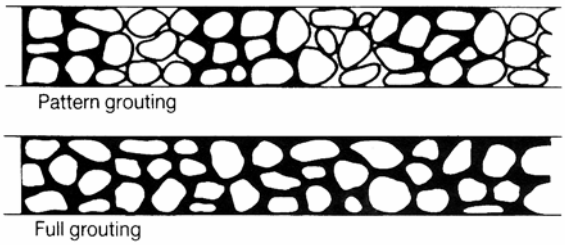
Bc 2.1g Erosion of revetment / cover layer (bound or grouted stone)

Summary: A set of empirical rules used to avoid direct damage to asphaltic revetments by wave loadings.



Reliability equation:
The same design method as for loose armourstone is used and the layer thickness is determined by the dimensions of the rock. However, a reduction factor can be applied depending on the degree of penetration, based on the following equation:

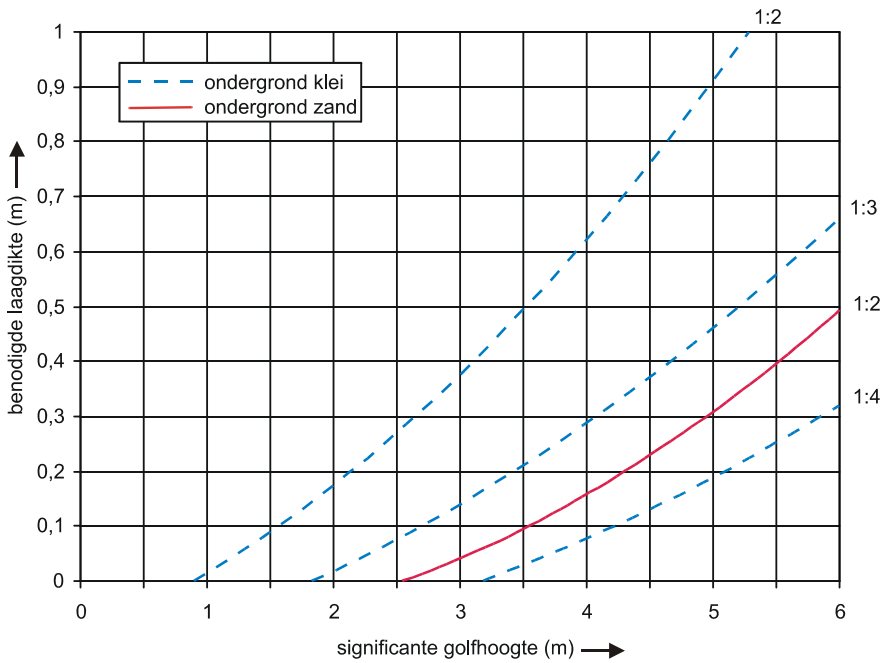
$$\frac{H_s}{\Delta D_{n50}} = \phi_u \phi_{sw} \frac{\cos\alpha}{\xi_p^{\,b}}$$

This formula is only valid for pattern grouting and not for full grouting!!! A LSE of this formula is defined as follows according to PC-Ring page 37:

$$Z = D_{n50} - \frac{H_s \xi_p^{\,b}}{\Delta \phi_u \phi_{sw} \cos\alpha}$$

If the voids are filled up to approximately 60% a value for the upgrading factor ϕ_u = 1.5 can be used. With a narrow (homogeneous) grading, and if monitored carefully during construction, this value can be increased up to ϕ_u = 2.0. For the stability parameter the value ϕ_{sw} = 2.25 can be used, however depending on the number of waves and the safety required this value may need to be modified. The parameter *b* depends on the interaction between the waves and the revetment. For revetments with pattern penetration the value *b* = 0.5 is recommended.

The required layer thickness for fully penetrated rock revetments can be found with the design graph given below, which has been compiled for hydraulic conditions as found in the Netherlands. The minimum layer thickness needed in the wave impact zone also depends on the stone diameter D_{n50}. To obtain a well penetrated revetment, the thickness needs to be more than 1.5 D_{n50}. For a fully penetrated rock revetment, the stone grading 5-40 kg is usually suitable although if required a stone grading of 10-60 kg can be used. Based on a rock density of ρ_r = 2650 kg/m^3 this leads to layer thickness of 0.30m for the grading 5-40 kg and 0.35m for the grading 10-60 kg.
For pattern penetrated rock revetments (for example following a pattern of dots or strips), the



This graph is made for pattern grouting. Note that this graph is only valid when the dwell time of the still water level is 17 hours over a length of 0.5 m and the mean wave period is $T_g = 3.5\ H_s^{1/2}$ (see TAW asphalt report, page 125), otherwise the program 'golfklap' has to be used.

Loading equations:
The loading is determined by the significant wave height H_s.

Resistance (strength) equations:
For fully penetrated rock revetments the resistance is influenced by the material below the coverlayer.
Resistance for pattern placed rock revetments, is expressed by the upgrading factor ϕ_u.

Parameter definitions:

H_s	= significant wave height [m]
Δ	= the relative density of cover layer, $\Delta = (\rho_r - \rho_w)/\rho_w$ [-]
D_{n50}	= nominal mean diameter [m]
α	= slope angle [-]
ϕ_u	= upgrading factor [-]
ϕ_w	= stability factor [-]
ξ_p	= breaker parameter based on the peak wave period T_p [-]

Sources of failure mechanism equations / methods:
TAW (2002) Technical report on Asphalt for Dikes

Sources of uncertainties in failure equations / input parameters:

Remarks:

Status of Draft

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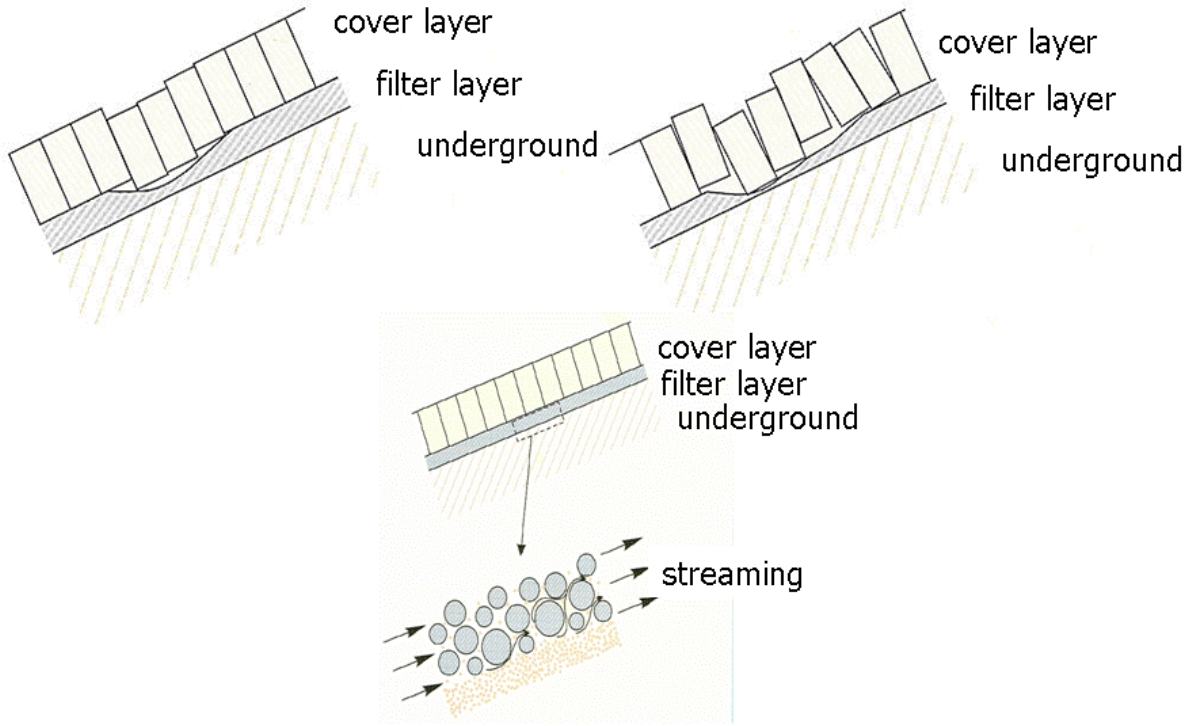
Bc 2.1h Erosion through sublayers (placed block revetments, block mats and concrete mattresses, gabions, geosystems)

Summary: A set of empirical rules used to avoid erosion of underlying materials through blockwork (or related) revetments under wave loadings. Waves can cause the migration of subsoil particles in the granular layer. Testing on the erosion of the sublayers only makes sense if the revetments themselves have a score 'unsafe' for the mechanisms 'stability of the cover layer' or 'migration of subsoil particles'.



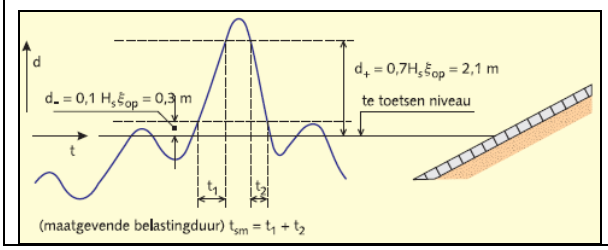
Reliability equation:
The reliability equation is expressed as follows:

$$Z = \left(t_{rg} + t_{rk}\right) - t_{sm}$$

where:

t_{rg} = residual strength of top layer and granular layer [h],
t_{rk} = residual strength of clay layer [h],
t_{sm} = normative duration of the load [h]

Loading equations:
The duration of the loading is expressed in terms of the duration parameter t_{sm} and it is defined as the time that a particular point on a dyke is exposed to wave attack during the storm (see Figure).



Resistance (strength) equations:
The residual strength of the top layer and the granular layer can be calculated as follows:

$$t_{rg} = 163.000 \cdot T_p \cdot \exp\left[-0.74 \cdot \sqrt{\left(H_s \cdot L_{0p}\right)}\right] / 3600$$

where

$$L_0 = \frac{g \cdot T_p^{\,2}}{2\pi}$$

Note that T_p is in second and t_{rg} is in hours.

Source: TAW, 2004a The values for d_+ and d_- are determined by the wave height H_s and the breaker parameter ξ_{op}. $$\xi_{op} = \frac{\tan\alpha}{\sqrt{H_s / \left(g \cdot T_p^{\,2} / (2\pi)\right)}}$$	According to PC-Ring, the remaining strength of the clay is: $$t_{rk} = \frac{0.4 L_k c_{rk}}{r^2 H_s^2}$$ where: $$L_K = \frac{d_K}{\sin\alpha}$$ The residual strength of the clay layer can only be taken into account under specific conditions.

Parameter definitions:

H_s	=	significant wave height [m]
L_{0p}	=	the wave length based of irregular wave on deep water [m].
T_p	=	peak wave period [s],
c_{rk}	=	coefficient for erosion resistance of sand [-]
d_k	=	thickness of remaining clay layer [m]
r	=	reduction factor for oblique wave attack [-]
α	=	slope of dike [-]

Sources of failure mechanism equations / methods:
TAW (2004a)

Sources of uncertainties in failure equations / input parameters:
The empirical coefficients and the quality of clay / sand (content) form the most important source of uncertainty.

Remarks:

Status of Draft

Date	Revision	Prepared by	Organisation	Approved by	Notes
			TAW		
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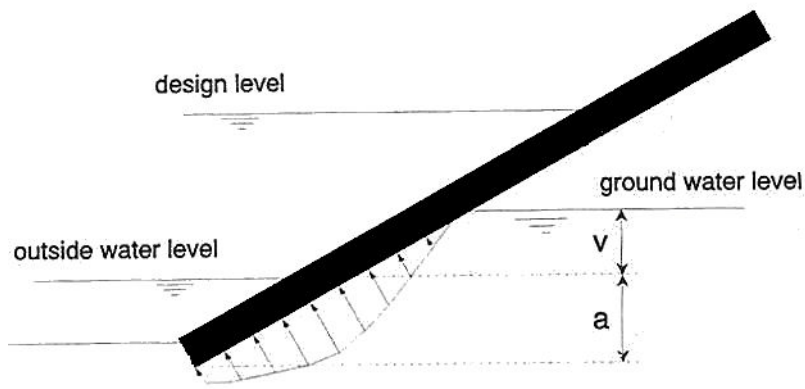
Bc 2.1j Uplift of cover layer (asphalt revetments)

Summary: Failure of asphaltic revetment layer by up-lift forces dealt with by dimensioning armour thickness (and other factors) in relation to the maximum water level difference.



Reliability equation:
Uplifting occurs if the uplifting pressure ("load") is higher than the downward forces ("strength"). According to PC-Ring page 35/36:

$$z = \Delta D - 0.21 \cdot Q_n (a + v) R_w$$

where:

v	=	water level difference (outside and internal) [m]
a	=	distance between outside water level and toe of asphalt revetment [m]
ΔD	=	relative density of the asphalt layer [-]

Loading equations:
1. outside water level at which the maximum uplift pressure occurs is lower than the average outside water level (a and v are defined according to the figure above):

$$H_{max,act} = \frac{v}{\pi} \cdot \arccos\left[2 \cdot \left(\frac{v + h \cdot \cos\alpha}{a + v}\right)^{\frac{\pi}{\theta}} - 1\right]$$

with:

$$a = 0.47\left(h_{MGWS} - h_{fo}\right)$$

$$v = 0.53\left(h_{MGWS} - h_{fo}\right)$$

$$\theta = \mathrm{arctg}(n) + \frac{\pi}{2}$$

$$Q_n = 0.9405 + 0.1275 \cdot \tan\alpha_u + 0.4229 \cdot \tan^2\alpha_u$$

2. outside water level by which the maximum uplift pressures occur is higher than the mean outside water level:

$$H_{max,act} = 0.21 \cdot Q_n \cdot (a + v) \cdot R_w$$

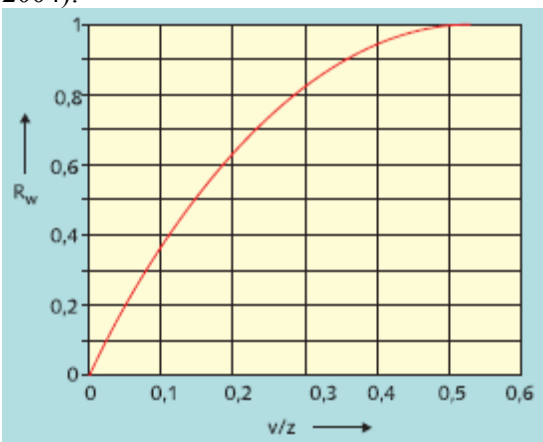
where:

$$Q_n = \frac{0.96}{\left(\cos\alpha\right)^{0.25}}$$

Resistance (strength) equations:
The relative density of the asphalt layer is defined as follows:

$$\Delta D = \frac{\rho_a - \rho_w}{\rho_w}$$

The reduction factor R_w can be graphically derived by means of the following figure (source TAW, 2004):



Parameter definitions:

Q_n	=	factor, depending on the slope angle α [-]
R_w	=	reduction factor factor, depending on the slope angle α [-]
n	=	the slope gradient (1:n) [-]
ρ_a	=	density of the revetment (asphalt mixture) [kg/m^3]
ρ_w	=	density of the water [kg/m^3]
H_{max}	=	maximum head difference [m]
d	=	thickness of asphalt layer [m]
α	=	slope angle [-]
h_{f0}	=	???
h_{MGWS}	=	???

Sources of failure mechanism equations / methods:
TAW (2004a); Van Herpen, J. A. (1998)

Sources of uncertainties in failure equations / input parameters:
The empirical coefficients and the quality of the asphalt form the most important source of uncertainty.

Remarks:
see also graphs in TAW asphalt for R_w and Q_n

Status of Draft

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			PC RING		
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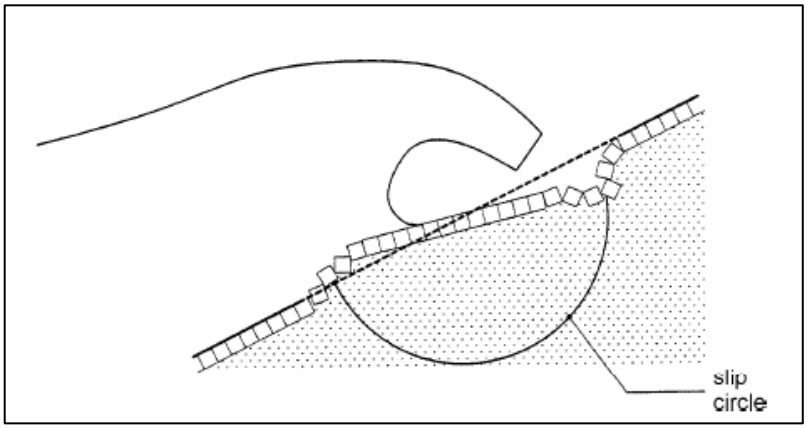
Bc 2.1k Uplift of revetment cover layer revetments due wave action

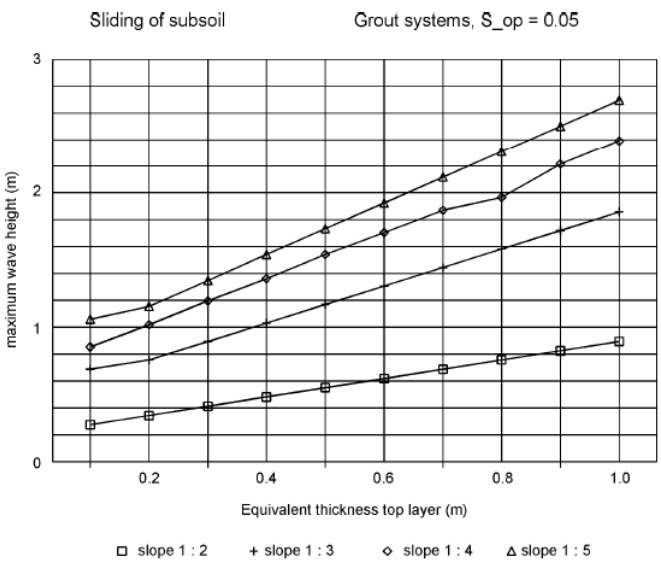
Summary: Potential failure of revetment layer by direct wave forces dealt with by dimensioning armour thickness (and other factors) in relation to the wave height and steepness.



Reliability equation:
Sliding occurs if the grain tension ("strength") decreases so strongly that insufficient sheer stress ("load") can be absorbed in the subsoil to prevent sliding. The design method with regard to the different failure mechanisms connected with elastic storage are presented in the form of design diagrams. An example is given in the figure (more diagrams and details: see Klein Breteler and Pilarczyk, 1998).

In the diagrams the permissible wave height is plotted against the thickness of the top layer and the slope gradient for a certain wave steepness S_{op}.

$$z = ???$$



Loading equations:	**Resistance (strength) equations:**
In the diagrams the load is expressed as the maximum wave height.	In the diagrams the strength is expressed in terms of thickness of the top layer. If the revetment construction consists of a top layer on a filter layer, the thickness of the filter

	layer may be replaced by the equivalent thickness D_{eq}, which is defined as: $D_{eq} = D + \frac{b}{\Delta_t}$ where $\Delta_t = (\rho_t - \rho_w)/\rho_w$ For sand-filled sytems ρ_t is equal to: $\rho_t = (1-n).\rho_s + n.\ \rho_w$ [kg/m^3]

Parameter definitions:

D_{eq} = equivalent thickness of the top layer [m]
D = real thickness of the top layer [m]
b = thickness of the filter layer [m]
Δ_t = $(\rho_t-\rho_w)/\rho_w$ = relative mass (weight) under water of the top layer [-]
ρ_t = density of the top layer [kg/m^3]
For sand-filled sytems ρ_t is equal to:
ρ = $(1-n).\rho_s + n.\ \rho_w$ [kg/m^3]
ρ = density of the sand [kg/m^3]

Sources of failure mechanism equations / methods:

Stoutjesdijk, T. (1996); Klein Breteler, M., K. Pilarczyk and T. Stoutjesdijk (1998);

Sources of uncertainties in failure equations / input parameters:

Remarks:

Status of Draft

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		N. Doorne	WL / Delft		
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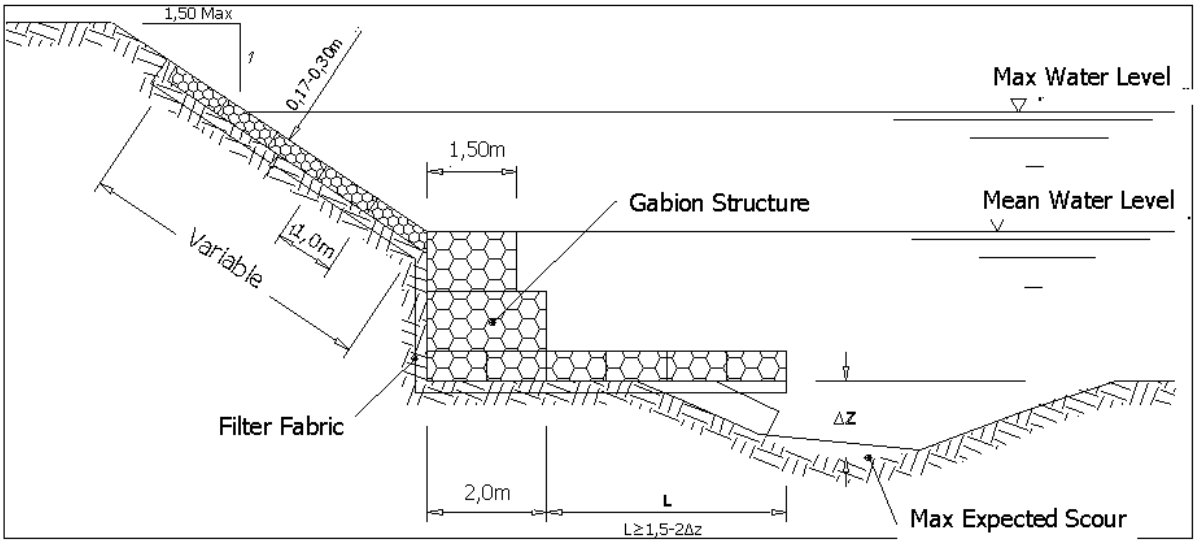
Bc 2.1m Erosion of cover layer (gabions)

Summary: Wave action on gabions can cause progressive or sudden damage. This method adapts the general method of Klein Breteler & Bezuijen.



Limit State Equation:
The limit state equation for static stability can be expressed by:

$$z = \phi_u \phi_{sw} \frac{\cos\alpha}{\xi_p^{2/3}} \Delta' D' - H_s$$

where:

H_s	=	significant wave height [m]
Δ	=	the relative density of cover layer, $\Delta = (\rho_r - \rho_w)/\rho_w$ [-]
D'	=	gabion thickness [m]
α	=	slope angle [-]
ϕ_u	=	upgrading factor [-]
ϕ_{sw}	=	stability factor [-]
ξ_p	=	breaker parameter based on the peak wave period T_p [-]

The limit state equation for dynamic stability can be expressed by:

$$z = \frac{F}{\xi_p^{1/2}} \Delta' D_n - H_s$$

where:

H_s	=	maximum bearable wave height [m]
D_n	=	nominal mean diameter [m]
F	=	stability factor [-]
ξ_p	=	breaker parameter based on the peak wave period T_p [-]
Δ	=	the relative density of cover layer, $\Delta = (\rho_r - \rho_w)/\rho_w$ [-]

Loading:
The loading is determined by the significant wave height H_s.

Resistance (strength) equations:
The static stability factors $\phi_u \phi_{sw}$ and the dynamic stability factor F represent the strength of the gabion against the loading.

	<u>Static stability</u> $H_s = \phi_u \phi_{sw} \frac{\cos\alpha}{\xi_p^{2/3}} \Delta' D'$ where: $\Delta = (\rho_r - \rho_w)/\rho_w$ $\xi_p = \frac{g \cdot T_p^2}{2 \cdot \pi}$ For $H_s/\Delta D' \approx 1 - 4$ the following values of the stability factors $\phi_u \phi_{sw}$ can be used: $\phi_u \phi_{sw} = 8$ with Δ' equal to the relative buoyant density of a unit ($\Delta' \approx 1$) and $D' \geq 1.8 D_n$ <u>Dynamic stability</u> $H_s = F \cdot \frac{\cos\alpha}{\xi_p^{1/2}} \Delta' \cdot D_n$ For $H_s/\Delta D_n \approx 6$ the following value of the stability factor can be used: F = 2 – 3, with Δ equal to the relative buoyant density of the rock material (usually $\Delta \approx 1.65$).

Parameter definitions:

H_s	=	significant wave height [m]
Δ	=	the relative density of cover layer, $\Delta = (\rho_r - \rho_w)/\rho_w$ [-]
D'	=	gabion thickness [m]
D_{n50}	=	nominal mean diameter [m]
α	=	slope angle [-]
ϕ_u	=	upgrading factor [-]
ϕ_{sw}	=	stability factor [-]
F	=	stability factor [-]
ξ_p	=	breaker parameter based on the peak wave period T_p [-]

Sources of failure mechanism equations / methods:
Klein Breteler M. & Bezuijen A. (1991); McConnell K.J. (1998)

Sources of uncertainties in failure equations / input parameters:

Remarks:
Although gabions are usually applied in the fluvial environment, they may be exposed to wave attack, which may cause erosion.

The primary requirement for a gabion or mattress of a given thickness is that it will be stable as a unit. The second requirement is that the dynamic movement of individual stones within the basket should not be too strong, because of the possible deformation of the basket and the abrasion of the mesh wires.

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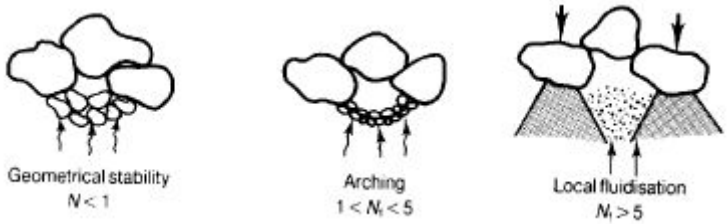
Bc 2.1n Filter instability and insufficient filter permeability

Summary: Local flow of pore water conveying fine particles of a granular material (from subsoil or structure) through the pores of the coarse particles of the same material (internal instability) or of an adjacent material (interfacial instability). Filters may also fail when they do not allow for sufficient transport of water to prevent unacceptable pressure build up.

Two approaches to filter instability

The most common methods to prevent such conveyance is the creation of 'geometrically tight' filters. Then, only the grain sizes are relevant. They need to vary between relatively narrow ranges. The ranges can be widened by 'geometrically open' filters, when the flow is limited and the slope of filter interfaces is dominantly horizontal, with the fines below the coarse grains (Figure, De Graauw et al 1983). Only the (conservative) 'geometrically tight' reliability equations are presented here.



Reliability equations

$$z = f\left(\frac{D_{s,f}}{D_{s,b}}; c_1; c_2\right)$$

CIRIA, CUR, SETMEF (2006) presents the following criteria:

to prevent internal instability (geometrically tight): $D_{60}/D_{10} < c_1$ with $c_1 = 10$, (if, however, $D_{60}/D_{10} > c_1$, no internal instability if each of the following criteria is met: $D_{10}/D_5 < c_1$ with $c_1 = 3$, $D_{20}/D_{10} < c_1$ with $c_1 = 3$, $D_{30}/D_{15} < c_1$ with $c_1 = 3$ and $D_{40}/D_{20} < c_1$ with $c_1 = 3$)

to prevent interface instability (geometrically tight): $D_{15f}/D_{85b} < c_1$ with $c_1 = 5$

to ensure adequate permeability: $D_{15f}/D_{15b} > c_1$ with $c_1 = 1\text{-}5$

BS 6349-7-1991 presents the following criteria:

$D_{15f} / D_{85b} < c_1$ with $c_1 = 4\text{-}5$

$c_1 < D_{15f} / D_{15b} < c_2$ with c_1 with $c_1 = 4$, $c_2 = 20\text{-}25$

Pilarczyk ed (1998) presents the following criteria:

$D_{60} / D_{10} < c_1$ with $c_1 = 10$

$D_{15f} / D_{85b} < c_1$ with $c_1 = 5$

$D_{15f} / D_{15b} > c_1$ with $c_1 = 1$ *)

*) the figure '5' is printed; but should be replaced with '1'

Loading equations:

The loading follows from gravity and pore flow, which are not included in the equations for geometrically tight filters.

Resistance (strength) equations:

The resistance depends on the grading characteristics of considered stone size and the orientation of the interface (last not included in the equations of geometrically tight filters).

Parameter definitions:

N= $n_f D_{15f}/D_{50b}$, where n_f = porosity of filter material

D_z = particle size corresponding to sieve size "z"

Index "f" refers to filter
Index "b" refers to base layer

Sources of failure mechanism equations / methods:
CIRIA, CUR, CETMEF (2006); BS 6349-7 (1991); De Graauw, A., van der Meulen, T. and Van der Does de Bye, M. (1983); Pilarczyk, K.W. ed (1998)

Sources of uncertainties in failure equations / input parameters:
- Variability of grain sizes within a granular material
- Uncertainty about permeability requirement

Remarks:
The permeability requirement largely depends on the location and the function of the filter. The presented equations are too conservative in many cases.
Only equations for granular filters are presented here. Similar ones are available for geotextile filters (CIRIA, CUR, CETMEF 2006).
Reliability equations for 'geometrically open' stable filters are available (CIRIA, CUR, CETMEF 2006). They are relatively complicated, as both pore flow parallel to the interface and perpendicular to the interface are involved. In case of strong perpendicular flow, distinction should be made between steady flow and reversing flow.

Status of Draft

Date	Revision	Prepared by	Organisation	Approved by	Notes
			CIRIA/CUR, British Standards		
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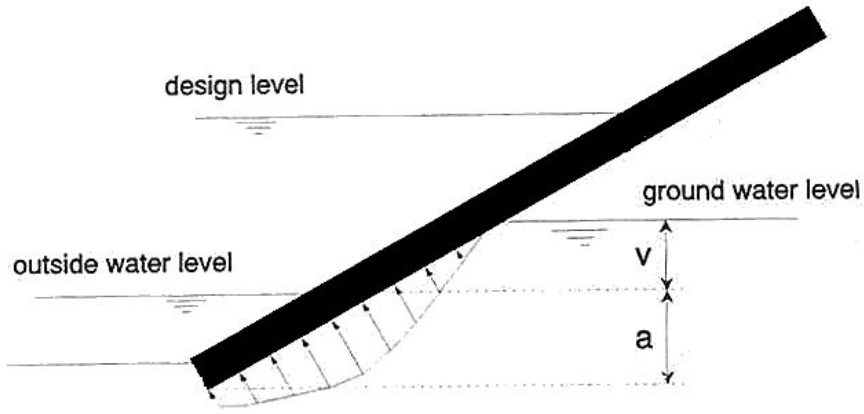
Bc 2.3a Sliding of cover layer (asphalt revetments)

Summary: Sliding of revetment facing / elements where the lower edge of the revetment is insufficiently supported, for instance when toe scour occurs. Driven by internal pressures exceeding imposed pressures during draw-down.



Reliability equation:
The driving load is expressed in terms of a head difference H_{max}. The strength is expressed in terms of a frictional criterion.
The reliability equation reads as follows:

$$z = H_{max} - H_{act}$$

where:

H_{max} = maximum bearable water level [m]
H_{act} = actual water level in front of ctructure [m]

Loading equations:

1. outside water level at which the maximum uplift pressure occurs is lower than the average outside water level (a and v are defined according to the figure above):

$$H_{max} = \frac{v}{\pi} \cdot \arccos\left[2 \cdot \left(\frac{v + h \cdot \cos\alpha}{a + v}\right)^{\frac{\pi}{\theta}} - 1\right]$$

with:

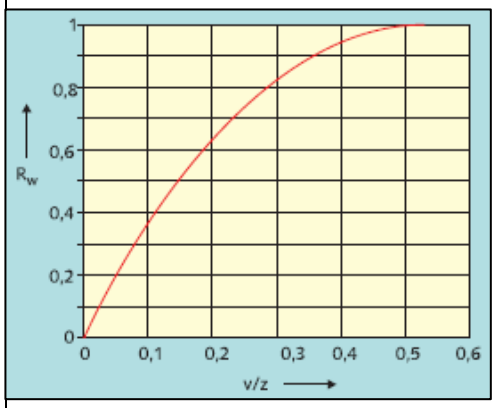
$$\theta = \mathrm{arctg}(n) + \frac{\pi}{2}$$

2. outside water level by which the maximum uplift pressures occur is higher than the mean outside water level: $H_{max} = 0.21 \cdot Q_n \cdot (a + v) \cdot R_w$

For the factor Q_n the following expression holds:

$$Q_n = \frac{0.96}{(\cos\alpha)^{0.25}}$$

The reduction factor R_w can be graphically derived by means of the following figure (source TAW, 2004):



Resistance (strength) equations:

The strength is expressed in terms of a frictional criterion. The required thickness h of the revetment can be established by:

$$h \geq \frac{H_{max}}{\cos\alpha} \cdot \frac{1}{\frac{\rho_w}{\rho_a} \cdot \left(1 - \frac{\tan\alpha}{f}\right) - 1}$$

Parameter definitions:

Q_n	=	a factor, depending on the slope angle α [-]
R_w	=	a reduction factor factor, depending on the slope angle α [-]
h	=	thickness of revetment [m]
n	=	the slope gradient (1:n) [-]
ρ_a	=	density of the revetment (asphalt mixture) [kg/m^3]
ρ_w	=	density of the water [kg/m^3]
H_{max}	=	maximum head difference [m]
φ	=	angle of internal friction of the subsoil [°]
θ	=	angle of internal friction between revetment and subsoil [°]
α	=	slope angle [°]
f	=	coefficient for friction [-], for θ < φ: f = tan θ, for θ ≥ φ: f = tan φ

Sources of failure mechanism equations / methods:

TAW (2004a)

Sources of uncertainties in failure equations / input parameters:

The empirical coefficients and the quality of the asphalt form the most important source of uncertainty. The fact that an asphalt revetment is used for several functions at the same time (traffic, water defence, recreation) might also lead to unusual loadings (and hence uncertainties) which have not been taken into account during the design.

Remarks:

Status of Draft

Date	Revision	Prepared by	Organisation	Approved by	Notes
		N. Doorne	Wl / Delft		
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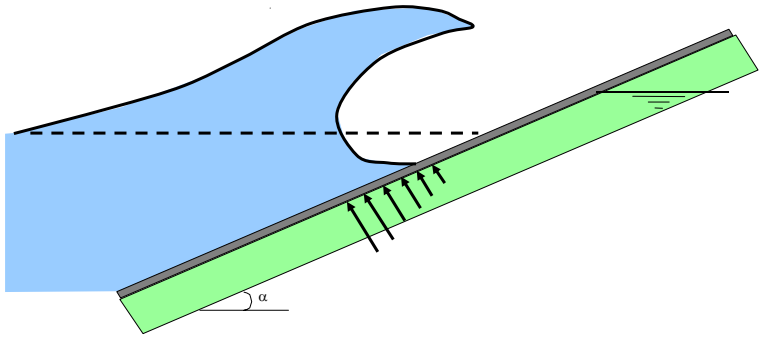
Bc 2.3b Uplift of revetment surface

Summary: Uplift of revetment surface / elements under wave draw-down, dealt with by dimensioning armour thickness (and other factors) in relation to the wave height and steepness.



Reliability equation:
The reliability function is expressed by:

$$z = G \cdot \cos\alpha - F_u$$

where:

G = mass force of revetment element [kN/m]
F_u =uplift force underneath the revetment [kN/m]

Loading equations:
Acting uplift force calculated from:

$$F_u = \frac{1}{2} \cdot p_a \cdot b \cdot \left(2 - \frac{b \cdot \sin\alpha}{z_{-98} + D \cdot \cos\alpha}\right)$$

with:

$$p_a = \rho_w \cdot g \cdot H_s \cdot [0{,}7 + 0{,}7 \cdot \tanh(\xi_d - 2{,}1)] + \rho_w \cdot g \cdot D \cdot \cos\alpha$$

$$\frac{z_{-98}}{H_s} = 0{,}7 + 0{,}7 \cdot \tanh(\xi_d - 2{,}1)$$

Resistance (strength) equations:
Mass force of revetment elements.

$$G = \gamma_s \cdot D \cdot b \cdot \cos\alpha$$

Parameter definitions:

F_u	=	wave induced uplift force [kN/m]
G	=	mass force of revetment elements [kN/m]
D	=	thickness of revetment elements [m]
b	=	width of revetment elements [m]
z_{98}	=	wave run-down being exceeded by 2% of all run-downs [m]
a	=	slope [°]

Sources of failure mechanism equations / methods:

Klein Breteler, M.; Pilarczyk, K.W.; Stoutjesdijk, T. (1998): Design of alternative revetments. *Proceedings International Conference Coastal Engineering (ICCE)*, ASCE, Kopenhagen, Dänemark, no. 26, Teil 2, S. 1587-1600.

Kortenhaus, A. (2003): Probabilistische Methoden für Nordseedeiche. Ph.D. thesis, Dissertation, Fachbereich Bauingenieurwesen, Leichtweiß-Institut für Wasserbau, Technische Universität Braunschweig, Braunschweig, Germany, 154 S.

Schüttrumpf, H. (2001): Wellenüberlaufströmung bei Seedeichen - experimentelle und theoretische Untersuchungen. Fachbereich Bauingenieurwesen, Technische Universität Braunschweig. Ph.D.

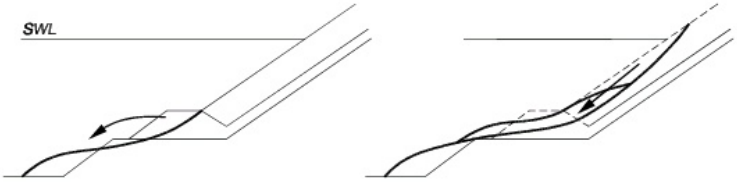
thesis, Mitteilungen Leichtweiß-Institut für Wasserbau der Technischen Universität, Braunschweig, Germany, S. 1-124.
Sources of uncertainties in failure equations / input parameters:
Remarks: Further formulae are available to calculate the uplift forces under wave attack. The aforementioned formulae include the description of Schüttrumpf (2001) for wave run-down (see Kortenhaus, 2003). See also Bc 2.1k

Status of Draft

Date	Revision	Prepared by	Organisation	Approved by	Notes
			LWI		
		ND??		WA / MM	
25/01/ 07	v3_4_p03		LWI		edited

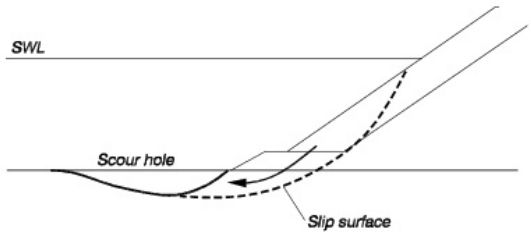
Bc 3.1a Bed scour by flow velocities

Summary: Onset of erosion to natural bed / natural slope by fluvial flows. This is usually addressed by dimensioning a protection layer. in the form of a toe berm, or in the form of a berm and additional apron (see Figure). Most rules-of-thumb are based on physical model tests performed for breakwaters or seawalls (wave attack).



Failure of main armor due to erosion of toe berm

- *Erosion starts at the "shoulder" of the toe and progresses towards the foot of the main armor. The toe is functioning as support for the main armor as long as the toe erosion do not cause undermining of the armor.*
- *Undermining of the armor layer causes sliding of armor units which eventually are armoring the remaining part of the toe.*



Sliding of main armour due to seabed scour

- *Formation of scour hole close to the foot of the structure due to wave and current action. The toe is functioning as support for the main armour as long as the toe erosion does not cause undermining of the armour.*
- *Reduced stabilizing forces causes slip failure to occur which results in sliding of armour.*

Reliability equation:
The wave and flow conditions determine the required width of the protection. The reliability equation can therefore be expressed in terms of the difference between the actual width of the toe protection and the required width. If the actual width is less than the required width, scour will occur which will lead to damage of the structure.

$$z = W - W_{act}$$

where:

W = width of scour hole at breakwater [m]
W_{act} = actual width of toe [m]

Loading equations:
For the trunk section of rubble-mound breakwaters, the width W_{act} of the scour hole can be determined on the basis of the following table (L is the wave length).

Slope of the breakwater	width W_{act} of the scour hole at the breakwater
vertical-wall	1.0 x (L/4)

Resistance (strength) equations:
The width of protection apron, l, may be selected as l = W, where the W is the width of the scour hole

The following minimum rule-of-thumb resulted from a survey carried out by Hales (1980) :

- minimum toe apron thickness: 0.6 – 1.0 m (1.0 – 1.5 m in Northwest US);
- minimum toe apron width 1.5 m (3 – 7.5 m in

breakwater	
1:1.2	0.6 x (L/4)
1:1.75	0.3 x (L/4)

For the head section of rubble-mound breakwaters, the width of the scour hole is :

$$W_{act} = \frac{AH_s}{\sinh(kh)}$$

in which H_s is the significant wave height and the following values of A can be used:
A = 3.3 for a complete scour protection (S/B=0);
A = 2.4 for S/B ≤ 0.01.

Northwest US) and
- Material: Quarry stone to 0.3 n diameter, gabions, mats, etc.

Parameter definitions:

W = width of scour hole [m]
l = width of apron [-]
k = wave number [m^{-1}]
h = water depth [m]
L = wave length [m]

Sources of failure mechanism equations / methods:
Coastal Engineering Manual (2001); Hales, L.Z. (1980); Sumer, B.M and J. Fredsøe (2002)

Sources of uncertainties in failure equations / input parameters:

Remarks:

Status of Draft

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			CEM		
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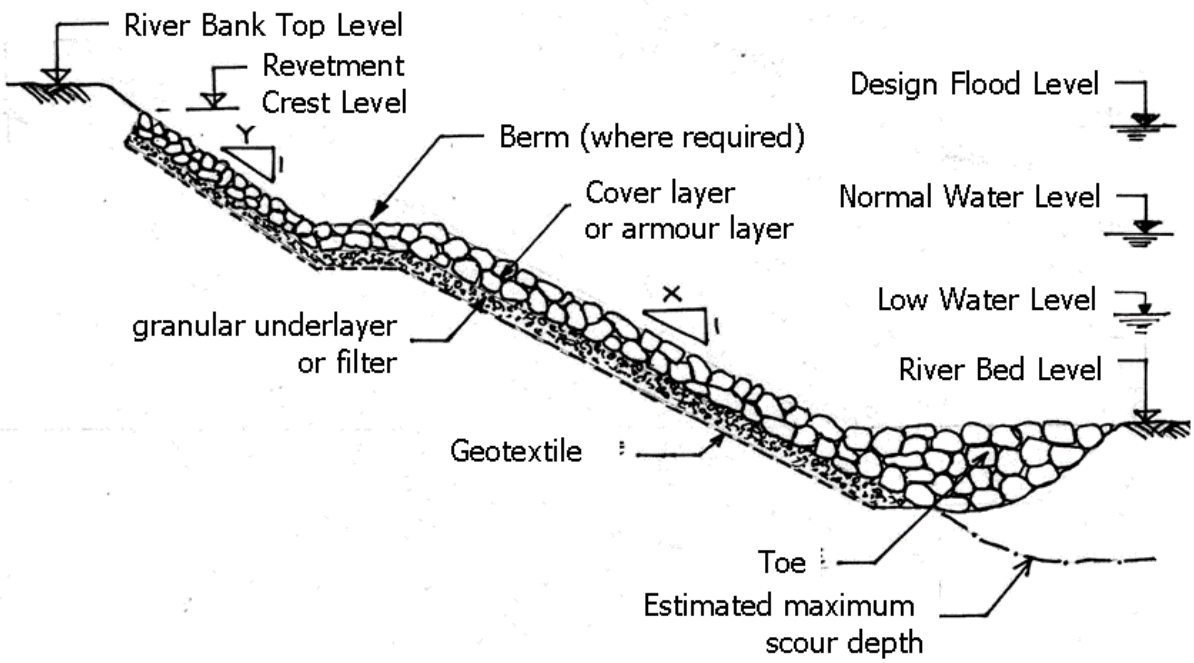
Bc 3.1b Erosion by flow velocities past defence (rock armour, riprap or gabions)

Summary: High flow velocities (and turbulence) past defence structures may cause erosion of the structure toe or of the cover layer. Toe or slope armour can be sized against current attack with various methods based on work by Isbash and Shields.



Reliability equation:
The reliability equation reads as follows:

$$z = D_a - D_r$$

where:

D_a = actual (existing) characteristic size of the protection element [m]
D_r = required characteristic size of the protection element corresponding to the actual boundary condition [m]

Loading equations:
Use one of the following formulae is suggested:

Pilarczyk criteria: $D_r = \frac{\phi_{sc}}{\Delta}\frac{0.035}{\psi_{cr}} k_h k_{sl}^{-1} k_t^2 \frac{U^2}{2g}$ (i)

where:

$$\Delta = \left(\frac{\rho_r - \rho_w}{\rho_w}\right)$$

Escarameia and May: $D_r = c_T \frac{u_b^2}{2g\Delta}$ (ii)

- Pilarczyk's equation uses a mobility parameter ψ_{cr}, which can be considered as the relative strength of the protection element. Values of the mobility parameter

Resistance (strength) equations:
- D_a can easily be determined from as built conditions or intended design size of protection. It could be the nominal mean diameter of stones (for rock revetment) or gabion thickness (for gabion system).

are listed below:

rip-rap	ψ_{cr} = 0.035
box gabions and gabion mattresses	ψ_{cr} = 0.07
rockfill in gabions	ψ_{cr} < 0.10

For Pilarczyk's equation the load is represented by the depth-averaged flow velocity U and the turbulence coefficient k_t. Typical values for the turbulence coefficient k_t are presented below:

normal turbulence level:	$k_t^2 = 1.0$
non-uniform flow, increased turbulence in outer bends:	$k_t^2 = 1.5$
non-uniform flow, sharp outer bends:	$k_t^2 = 2.0$

Pilarczyk introduces a velocity profile factor for which the following expressions are available:

fully developed logarithmic velocity profile: $k_h = 2/(\log^2 (1 + 12h/k_s))$ where k_s = roughness height (m), k_s = 1 to $3D_n$ for rip-rap
not fully developed velocity profile: $k_h = (1 + (h/D_n))^{-0.2}$
shallow rough flow ($h / D_n < 5$): $k_h \approx 1$

Escarameia and May use the near bed velocity u_b, defined at 10% of the water depth above the bed. If data are not available an estimation can be made in relation to the depth average velocity U as: u_b = 0.74 to 0.90 U
Escarameia and May present the following equations for the turbulence coefficient c_T used in their equation.

rip-rap (valid for $r \geq 0.05$):	$c_T = 12.3\ r - 0.20$
gabion mattresses (valid for $r \geq 0.15$):	$c_T = 12.3\ r - 1.65$

Typical turbulence levels are given below:

straight river reaches	r = 0.12
edges of revetments in straight reaches	r = 0.20
bridge piers, caissons and spur dikes, transitions	r = 0.35 – 0.50
downstream of hydraulic structures	r = 0.60

Parameter definitions:

U	=	depth averaged flow velocity [m/s]
u_b	=	near bed velocity [m/s]
Δ	=	the relative density of cover layer, $\Delta = (\rho_r - \rho_w)/\rho_w$ [-]
D_a	=	actual (existing) characteristic size of the protection element nominal mean diameter / gabion thickness, [m]
D_r	=	the required characteristic size of the protection element/ or required thickness protection layers corresponding to the actual boundary condition.
D_{n50}	=	nominal mean diameter [m]

ϕ_{sc}	=	stability correction factor [-]
ψ_{cr}	=	mobility parameter of protection element [-]
k_h	=	velocity profile factor [-]
k_{sl}	=	side-slope factor [-]
k_t	=	turbulence factor [-]
c_T	=	turbulence coefficient [-]

Sources of failure mechanism equations / methods:
CUR (1995); Escarameia, M. (1998)

Sources of uncertainties in failure equations / input parameters:

Remarks: D_a is determined from (ii) and is the nominal mean diameter (D_{n50}) of stone applied for given protection system, not a thickness of the system.

Also see Ab 3.1

Status of Draft

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			CUR / TAW		
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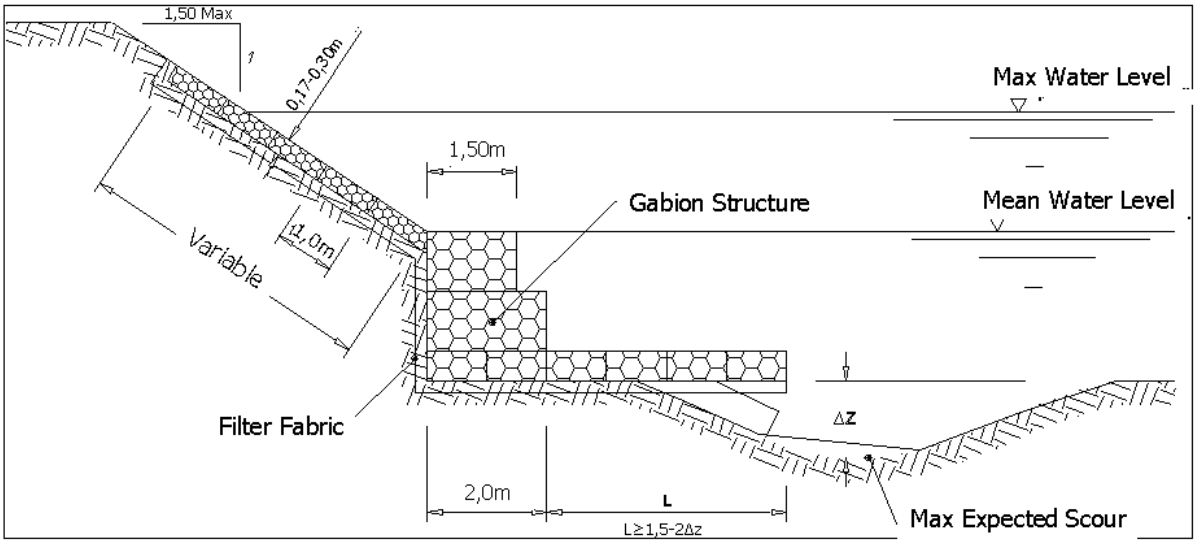
Bc 3.1c Erosion of cover layer (gabions) by flow velocities

Summary: High flow velocities (and turbulence) past defence structures may cause erosion of the structure toe or of the cover layer. Toe or slope armour can be sized against current attack with various methods based on work by Isbash and Shields.



Reliability equation:
The reliability equation is expressed by:

$$z = \psi_c - \psi$$

where:

ψ_c = critical shear stress [-]
ψ = existing shear stress [-]

Loading equations:
The existing Shields parameter can be estimated as follows:

$$\psi = \frac{u_*^2}{\Delta g D}$$

Resistance (strength) equations:
Shields:

$$\psi_c = \frac{\tau_c}{(\rho_r - \rho_w)\cdot gD} = \frac{u_{*c}^2}{\Delta g D} = f(Re_*)$$

where:

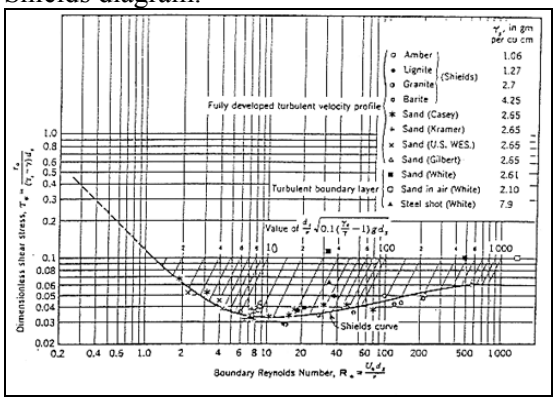
$$\Delta = \left(\frac{\rho_r - \rho_w}{\rho_w}\right)$$

$$D \geq 0.7\frac{U^2}{2g\Delta}\left(1 - \frac{\sin^2\alpha}{\sin^2\phi}\right)^{-0.5} \text{ (Izbash)}$$

The above equation is a modified version of Izbash equation originally meant for horizontal bed. The coefficient 0.7 is the suggested Isbash's constant for embedded stone.

For a flat bed, to ensure that the armor stone will not move, the shields parameter must be less than the critical shields parameter (ψ_c).

The plot of ψ_c vs Reynolds number is the well known Shields diagram. For turbulent flow the value of ψ_c is nearly constant in the range of 0.06.
Shields diagram:



Parameter definitions:

U	=	current velocity [m/s]
α	=	slope angle [-]
ϕ	=	angle of internal friction [-]
τ_c	=	critital shier stress [N/m^2]
ρ_r	=	density of cover layer elements [kg/m^3]
ρ_w	=	density of water[kg/m^3]
u*	=	???

Sources of failure mechanism equations / methods:
Izbash, S.V. (1935); Izbash, S.V. and K.Y. Zkhaldre (1970); Schiereck, G.J. (2001); Shields, A. (1936)

Sources of uncertainties in failure equations / input parameters:

Remarks:
Although gabions are usually applied in the fluvial environment, they may be exposed to wave attack, which may cause erosion.
The primary requirement for a gabion or mattress of a given thickness is that it will be stable as a unit. The second requirement is that the dynamic movement of individual stones within the basket should not be too strong, because of the possible deformation of the basket and the abrasion of the mesh wires.
The basic approach is to compute the near bottom flow induced shear stress. The stone stability is then determined by comparing the frictional resistance of embedded stone with the flow induced shear stress. Either the Izbash equation or the Shields diagram can be used to compute start of motion.

Status of Draft

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		N. Doorne	WL / Delft		
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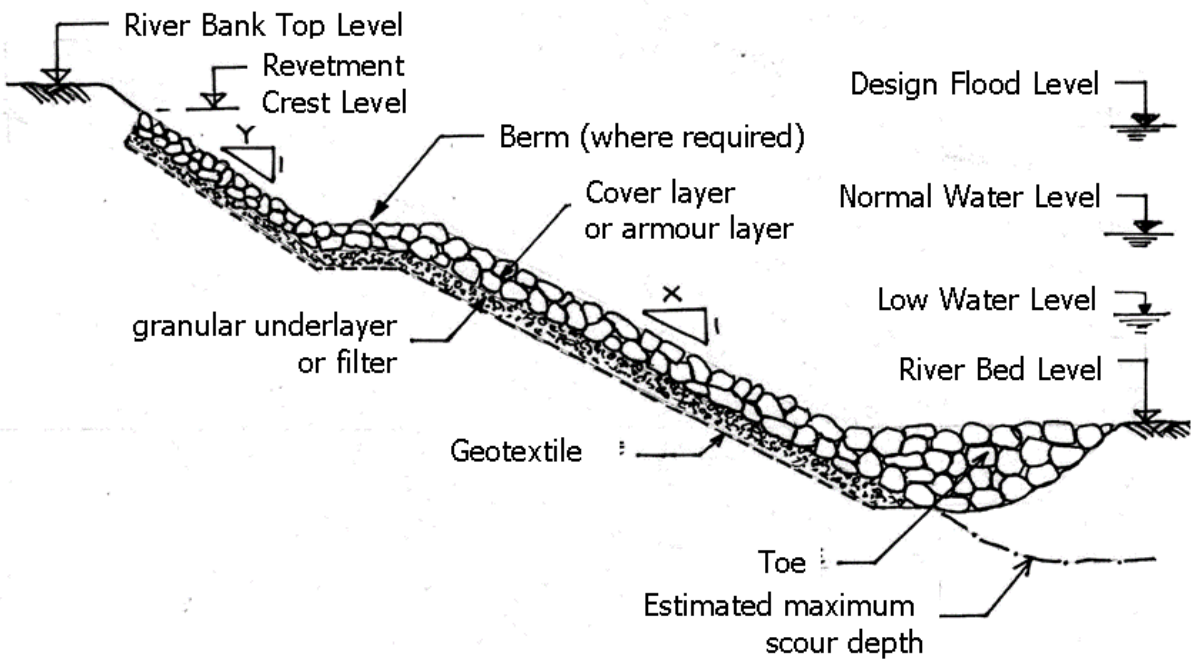
Bc 3.1d Erosion by flow velocities past defence (Linked concrete blocks)

Summary: High flow velocities (and turbulence) past defence structures may cause erosion of the structure toe or of the cover layer. The stability of concrete blocks forming the cover layer is primarily determined by the density of the concrete and the thickness of the cover layer.



Reliability equation:
The reliability equation is given by:

$$z = D_{req} - D_{act}$$

where:

D_{act} = actual revetment thickness [m]
D_{req} = required revetment thickness [m]

Loading equations:
Actual revetment thickness D_{act}

Resistance:
Required revetment thickness D_{req} according to Pilarczyk (1990):

$$D = \frac{\phi_{sc}}{\Delta} \frac{0.035}{\psi_{cr}} k_h k_{sl}^{-1} k_t^2 \frac{U^2}{2g}$$

where:

$$\Delta = \left(\frac{\rho_r - \rho_w}{\rho_w} \right)$$

For Pilarczyk's equation the load is represented by the depth-averaged flow velocity U and the turbulence coefficient k_t. Typical values for the turbulence coefficient k_t

are presented below:

normal turbulence level:	$k_t^2 = 1.0$
non-uniform flow, increased turbulence in outer bends:	$k_t^2 = 1.5$
non-uniform flow, sharp outer bends:	$k_t^2 = 2.0$

Pilarczyk introduces a velocity profile factor for which the following expressions are available:

fully developed logarithmic velocity profile:

$$k_h = 2/(\log^2 (1 + 12h/k_s))$$

where k_s = roughness height (m), k_s = 1 to $3D_n$ for rip-rap

not fully developed velocity profile:

$$k_h = (1 + (h/D_n))^{-0.2}$$

shallow rough flow ($h / D_n < 5$):

$$k_h \approx 1$$

Pilarczyk's equation uses a mobility parameter ψ_{cr}, which can be considered as the relative strength of the protection element. Values of the mobility parameter are listed below:

rip-rap	$\psi_{cr} = 0.035$
Cable blocks and asphalt mats	$\psi_{cr} = 0.07$
rockfill in gabions	$\psi_{cr} < 0.10$

Parameter definitions:

- U = depth averaged flow velocity [m/s]
- u_b = near bed velocity [m/s]
- D = block thickness [m]
- ϕ_{sc} = stability correction factor [-]
- ψ_{cr} = mobility parameter of protection element [-]
- k_h = velocity profile factor [-]
- k_{sl} = side-slope factor [-]
- k_t = turbulence factor [-]
- ρ_r = density of revetment elements [kg/m^3]
- ρ_w = density of water[kg/m^3]

Sources of failure mechanism equations / methods:

Escarameia, M. (1998); Pilarczyk K W (1990)

Sources of uncertainties in failure equations / input parameters:

It has been assumed that there is suitable restraint at toes of banks and edges. Cellular blocks rather than solid units allow release of pressure that may build up behind a revetment

Remarks:

Status of Draft

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		R. Bettess	HRW		
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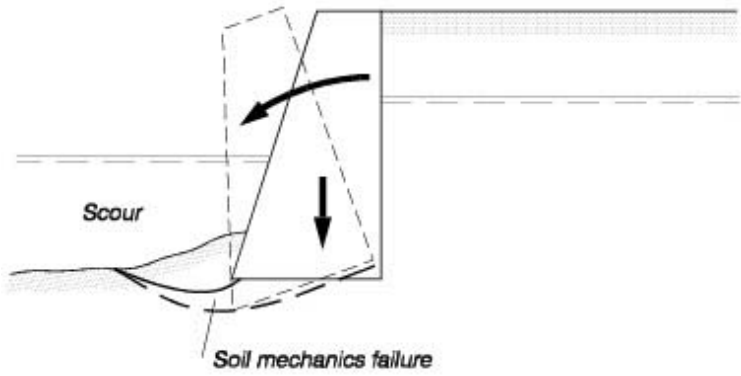
Ca 2.1a Erosion (scour) of bed without artificial protection

Summary: Bed scour is the primary initiating failure mechanism for seawalls, leading to undermining, loss of fill, local or global slips, armour slide, etc. Unprotected natural beds may scour substantially in current flows or waves. Scour in front of vertical walls is more severe than for slopes / mound, driven by enhanced bed pressures / velocities / turbulence. This mechanism uses simple rules to estimate scour depth of granular materials under wave attack.



Seaward overturning and settlement of gravity wall

- *Scour in front of the wall reduces both the passive resistance and the bearing capacity of the foundation soil.*
- *The resulting load from the active backfill pressure, the high groundwater table and the weight of the wall cause a bearing capacity failure in the soil resulting in a foreward overturning and some settlement of the wall.*

Reliability equation:
The wave and flow conditions determine the depth of scour and/or required width of protection. The reliability equation can therefore be expressed in terms of the difference between the actual width of the toe protection and the required width. If the actual width is less than the required width, scour will occur which will lead to damage of the structure.

$$z = S_{req} - S_m$$

Loading equations:
Normally incident, nonbreaking, regular waves incident upon an impermeable vertical wall (Xie, 1981, 1985):

$$S_m = H \cdot \frac{0.4}{\left[\sinh(kh)\right]^{1.35}}$$

where
S_m = maximum scour depth at node (L /4 from wall)

Normally incident, nonbreaking irregular waves (Hughes & Fowler, 1991):

$$S_m = \langle u_{rms} \rangle_m \cdot \frac{0.05}{\left[\sinh(k_p h)\right]^{0.35}}$$

The value of $<u_{rms}>_m$ was given by Hughes (1992) as:

$$\frac{\langle u_{rms} \rangle_m}{g k_p T_p H_{m0}} = \frac{\sqrt{2}}{4\pi \cosh(k_p h)} \cdot \left[0.54 \cosh\left(\frac{1.5 - k_p h}{2.8} \right) \right]$$

Resistance (strength) equations:
Required scour width S_{req}

Parameter definitions:

S_m = maximum scour depth at node (L /4 from wall)
H = incident regular wave height
H_{m0} = zero-th moment wave height
g = gravity (=9.81m/s^2)
h = water depth [m]
k = incident regular wave number [-]
L = incident regular wavelength [m]
T_p = wave period of the spectral peak [s]
k_p = wave number associated with the spectral peak by linear wave theory [-]
$<u_{rms}>_m$ = root-mean-square of horizontal bottom velocity
d_e = the depth of sheet-pile penetration below the seabed [m]
φ = angle of internal friction of the soil (varies from about 26° to 36° [°]
d_s = the depth at the structure toe [m]

Sources of failure mechanism equations / methods:

Coastal Engineering Manual (2001), Eckert, J.W. (1983); Hughes, S. A. (1992); Hughes, S. A., & Fowler, J. E. (1991); Sumer, B.M and J. Fredsøe (2002); Xie, S.-L. (1981); Xie, S.-L. (1985)

Sources of uncertainties in failure equations / input parameters:

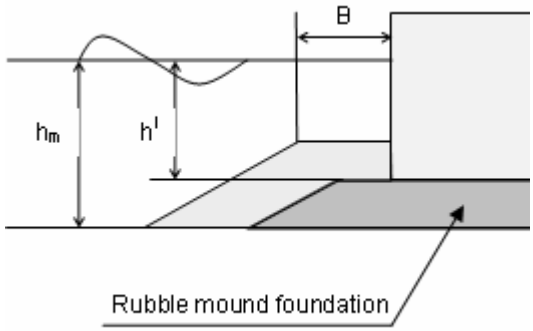
Remarks:

Status of Draft

Date	Revision	Prepared by	Organisation	Approved by	Notes
		FB??		WA / MM	
25/01/ 07	v3_4_p03		LWI		edited

Ca 2.1b Erosion of toe protection to vertical structures by waves

Summary: Bed scour is the primary initiating failure mechanism for seawalls, leading to undermining, loss of fill, local or global slips, armour slide, etc. Unprotected natural beds may scour substantially in current flows and/or waves. Scour in front of vertical walls is more severe than for slopes / mound, driven by enhanced bed pressures / velocities. This mechanism uses simple rules to denominate representative armour size to resist damage of granular protection berms against wave damage.



Reliability equation:
The reliability equation reads as follows:

$$z = D_{n50} - D_{n50,act}$$

where:

$D_{n50,act}$ = actual stone diameter [m]
$D_{n,50}$ = required stone diameter for given damage level [m]

Loading equations:
Madrigal & Valdes (1999):

$$D_{n50} = \frac{H_s}{\left(5.8 \cdot \left(h'/h_m\right) - 0.6\right)} \cdot N_{od}^{-0.19}$$

where:

$$\Delta = \left(\frac{\rho_r - \rho_w}{\rho_w}\right)$$

with range of validity:

$0.5 < h'/h_m < 0.8$
$7.5 < h'/D_{n50} < 17.5$
$0.3 < B/h_m < 0.55$

The formula uses the damage level N_{od} (actual number of displaced stones related to a width, along the longitudinal axis of the structure, of one nominal diameter) as an indicator of the strength.

- $N_{od} = 0.5$, start of damage
- $N_{od} = 2.0$, acceptable damage
- $N_{od} = 5.0$, severe damage

Resistance (strength):
Actual stone diameter $D_{n50,act}$

Parameter definitions:

H_s = ignificant wave height [m]
D_{n50} = nominal mean diameter [m]

N_{od} = damage level [-]
h_m = water depth [m]
h' = water depth at the toe (including the coverlayer) [m]
ρ_r = density of revetment elements [kg/m^3]
ρ_w = density of water[kg/m^3]

Sources of failure mechanism equations / methods:
USACE (2003); Madrigal, B.G. & Valdés, J.M. (1995); Brebner A. & Donnelly D. (1962)

Sources of uncertainties in failure equations / input parameters:

Remarks:
This approach should replace the previous method by Brebner & Donnelly (1962), see also SPM / CEM.
For vertical gravity structures, the toe provides protection to the foundation of the structure. For sheet piled structures, an armoured toe may provide scour protection to prevent the passive soil volume from eroding.

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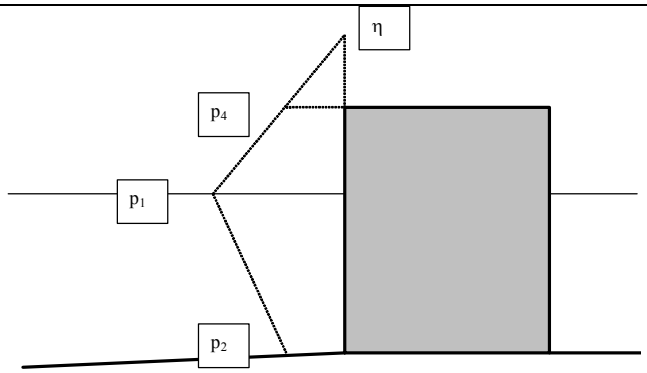
Ca 2.2a Bulk sliding (landward) of wall / element by direct wave force

Summary: Failure of mass wall element by sliding or overturning movement driven by long-duration wave momentum force. Other methods may be available for short-duration impulsive loadings, but the resistance equations will require dynamic analysis of wall and foundation..



Wave force F_h given by notional pressures p_1, p_2 and p_4. Pressure p_2 derived from p_1 and η*.

Reliability equation:
Failure occurs when the horizontal force F_h exceeds net friction force F_r between wall and foundation:

$$z = F_r - F_h$$

where:

F_r = net friction force [kN/m]
F_h = horizontal force due to wave load [kN/m]

Loading equations:
Horizontal wave load, F_h, given by equations by Goda (1985, 2000) for pressures p_1, p_2 and p_4 and elevation η.

Resistance (strength) equations:
Friction force F_r:

$$F_r = \mu \cdot \left(M_g - B_u - F_u\right)$$

Up-lift force F_u given by Goda, but adjusted for foundation material / conditions.

Buoyancy force, B_u, given by structure geometry, water level, and density of water. Structure weight force given by dry mass, M_g.

Friction coefficient μ for roughness of caisson / wall base and foundation material

Parameter definitions:

p_1 = maximum pressure at the water level [kN/m]
p_3 = pressure at the bottom [kN/m]
p_4 = pressure because of wave overtopping [kN/m]
β = wave obliquity, plan angle of wave direction to normal at the wall [°]
H_d = design wave height (sometimes given simply as = 2.2 H_s) [m]
D = water depth on top of the top layer of the sill [m]

$\lambda_1, \lambda_2, \lambda_3,$ = modification factors, depending on the geometry and the nature of the wall
$\alpha_1, \alpha^*, \alpha_3, \alpha_4,$= wave pressure coefficients, depending on the wave conditions and the geometry
h = water depth in front of the sill [m]
h' = water depth on top of the foundation of the wall [m]
h_c = height difference between the still water level and the top of the wall [m]
K_s = shoaling coefficient
K_r = refraction coefficient
K_d = diffraction coefficient
$H_{s,0}$ = significant wave height in deep water [m]

Sources of failure mechanism equations / methods:
Allsop N.W.H. (2000); Goda Y. (1974); Goda Y. (1985); Goda Y. (2000)

Sources of uncertainties in failure equations / input parameters:
Discussion from PROVERBS / MCS.
Guidance on model uncertainties;
Identify data on parameter uncertainties, s.d., distribution types;

Remarks:

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			HRW,NWHA		
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Ca 2.2b Bulk sliding (seaward) of wall / element –ve wave force

Summary: Failure by sliding or overturning movement of a wall element seaward driven by wave trough and perched water table behind element. May need to include soil wedge failure, see Cb 1.2

Reliability equation:

Failure when horizontal force exceeds net friction force between wall and foundation, or net overturning moment. Failure when horizontal force (wave and geotechnical) exceeds net friction force between wall and foundation:

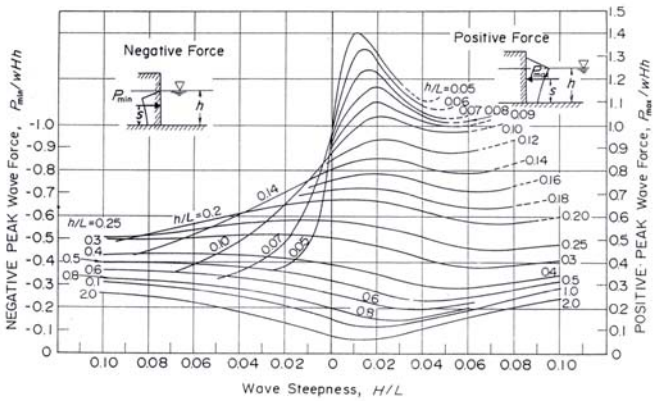
$$z = F_r - \left(F_{h\,min} + F_{soil}\right)$$

where:

F_r = net friction force [kN/m]
F_{hmin} = horizontal negative force due to wave load [kN/m]
F_{soil} = friction force between wall and foundation [kN/m]

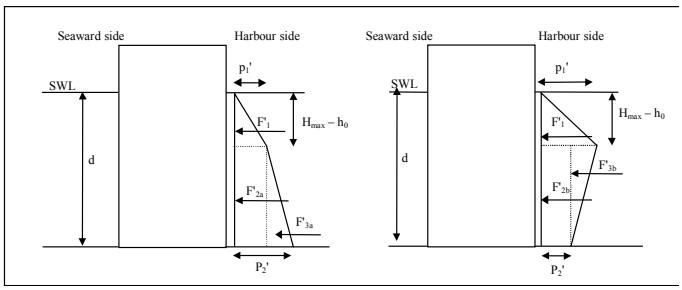
Loading equations:

Horizontal wave load estimates , F_h, might be given by Goda's graphical method, see below:



A method with more recent validation is given by Sainflou's equations modified by McConnell et al (1999) to give $F_{h,\,min}$. The probabilistic approach for $F_{hmin1/250}$ uses mean and standard deviation for ratio of $F_{hmin,test}$ to $F_{hmin,Sainflou}$. The mean value of the ratio of measured to Sainflou predictions was 1.126 and the σ was 0.1508, giving CoV = 13%.

$$F_{hmin} = 1.126 \cdot F_{Sainflou} \pm 13\%$$



Resistance (strength) equations:

Friction force F_r:

$$F_r = \mu \cdot \left(M_g - B_u - F_u\right)$$

Resistance given by friction force resisting sliding and/or weight resisting overturning:

Buoyancy force, B_u, given by structure geometry, water level, and density of water. Structure weight force given by dry mass, M_g.

Friction coefficient μ for roughness of caisson / wall base and foundation material. Note: this method is not appropriate where the main resistance is driven by cohesion.

Parameter definitions:

p_1' = $\rho g\,(H-h_0)$
p_2' = $\rho g H / \cosh(2\pi d/L)$
h_0 = $(\pi H^2/L)\coth(2\pi d/L)$

$F'_1 = (H_{max}-h_0)\, p_1'/2$
$F'_{2a} = p_1' (d-H_{max}+h_0)$ (if $p_2' > p_1'$)
$F'_{2b} = p_2' (d-H_{max}+h_0)$ (if $p_2' < p_1'$)
$F'_{3a} = (p_2'-p_1') (d-H_{max}+h_0)/2$ (if $p_2' > p_1'$)
$F'_{3b} = (p_1'-p_2') (d-H_{max}+h_0)/2$ (if $p_2' < p_1'$)

The total negative force is given by:

$F_{h\text{-}ve,\ Sainflou} = (H_{max}-h_0)p_1'/2+(p_1'+p_2') (d-H_{max}+h_0)/2$

The design equations are, probabalistic:

$F_{hmin} = 1.126 \cdot F_{Sainflou} \pm 13\%$

and deterministic:

$F_{hmin} = 1.27 \cdot F_{Sainflou}$

Sources of failure mechanism equations / methods:

Allsop N.W.H. (2000) *Wave forces on vertical and composite walls* Chapter 4 in Handbook of Coastal Engineering, pages 4.1-4.47, Editor J. Herbich, ISBN 0-07-134402-0, publn. McGraw-Hill.

Goda Y. (2000) "Random seas and maritime structures, 2nd edition" ISBN 981-02-3256-X, World Scientific Publishing, Singapore.

McConnell K.J., Allsop N.W.H. & Flohr H. (1999) *Seaward wave loading on vertical coastal structures* Proc. Coastal Structures '99, Santander, pp 447-454, ISBN 90 5809 092 2, publn. Balkema, Rotterdam.

Sources of uncertainties in failure equations / input parameters:

Guidance on model uncertainties;
Identify data on parameter uncertainties, s.d., distribution types;

Remarks:

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				WA / MM	
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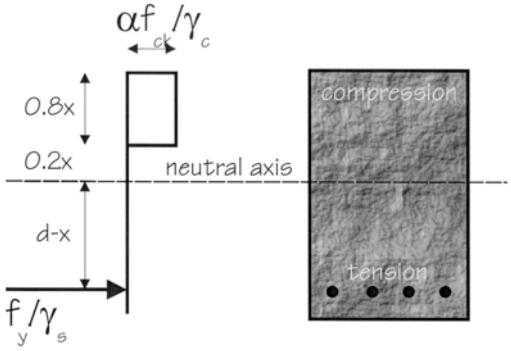
Ca 2.3 Local surface failure of wall

Summary: Flexural failure of a reinforced concrete member. Equivalent unit width beam (with no compressive reinforcement), spanning one-way continuously over at least 6 equal-span cells



Reliability equation:

The reliability equation for flexural failure in an under-reinforced section is given by (see, for example, O'Brien & Dixon, 1995):

$$z = (\rho_r\, d^2\, f_y / \gamma_s)(1-(0.4\rho_r f_{ck}\gamma_c / \alpha\, f_{ck}\gamma_s))-(0.08pL^2)/\, \gamma_d$$

where:

ρ_r	=	area ratio of steel reinforcement with respect to the concrete cross-sectional area D [-]
d	=	depth of the section from the compression face to the centre of the tensile steel reinforcement [m]
f_y	=	characteristic yield strength of steel reinforcement [mPa]
α	=	coefficient which takes account of the long-term affects on the compressive strength and of the unfavourable effects resulting from the way in which the load is applied [-]
f_{ck}	=	characteristic compressive strength of concrete [MPa]
p	=	net uniformly distributed pressure acting on the member [MPa]
L	=	the effective span distance between the supports [m]
γ_c, γ_s	=	partial safety factors [-]
γ_d	=	model factor [-]

Loading equations:

Net uniformly distributed pressure p:

In the case of the front wall, p is the arithmetic sum of the applied wave loading and the internal cell pressure. The factor 0.08 is chosen as a representative value for the maximum (mid-span) bending moment occurring in the middle of the outer-most span of a caisson with 6, or more, cells.

Resistance (strength) equations:

Characteristic yield strength of steel reinforcement f_y: LN(460MPa, 10MPa)

Characteristic compressive strength f_{ck} of concrete: LN (40-60MPa, 4-8MPa). EC2 denotes a concrete with characteristic cylinder strength of 30MPa and characteristic cube strength of 37MPa, as grade C30/37 concrete. Other grades include C35/45, C40/50, C45/55 and C50/60. Concrete of grade at least C40/50 should generally by used in a marine environment to limit the chloride diffusion.

Coefficient α:

Adopt α=0.85 as a default value

Parameter definitions:

ρ_r	=	area ratio of steel reinforcement with respect to the concrete cross-sectional area D [-]
d	=	depth of the section from the compression face to the centre of the tensile steel

		reinforcement [m]
f_y	=	characteristic yield strength of steel reinforcement LN(460MPa, 10MPa) [mPa]
α	=	coefficient which takes account of the long-term affects on the compressive strength and of the unfavourable effects resulting from the way in which the load is applied (adopt α=0.85 as a default value) [-]
f_{ck}	=	characteristic compressive strength of concrete LN(40-60MPa, 4-8MPa) [MPa]
p	=	net uniformly distributed pressure acting on the member (in the case of the front wall, p is the arithmetic sum of the applied wave loading and the internal cell pressure) [MPa]
L	=	the effective span distance between the supports [m]
γ_c, γ_s	=	partial safety factors [-]
γ_d	=	model factor [-]

Sources of failure mechanism equations / methods:
Oumeraci et al. (2001).

Sources of uncertainties in failure equations / input parameters:

Remarks:
For limit state equations for shear failure of a reinforced concrete member and cracking in a flexural reinforced concrete member see Oumeraci et al. (2001)

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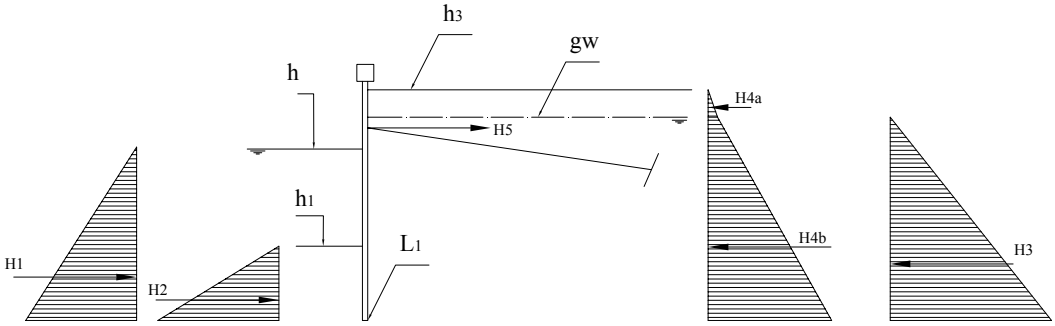
Cb 1.2a Overturning failure of wall element, insufficient strength of tie rod

Summary: Simple overturning failure of wall element driven by static water level differences and/or geotechnical loading. May occur after other effects have reduced soil strength (see e.g. Ba 1.5d), or under negative wave loads (see also Ca 2.2b). This sub-mechanism occurs when the tie rod fails.



Reliability equation:
The tie rod supports the sheet pile wall in taking on the forces. Failure of the tie rod occurs if the stress occurring in the tie rod exceeds the tensile strength of the steel.

$$z = m_1 F_u - m_2 . F_{tot}$$

where:

F_u	=	tensile foce capacity of the tie rod [kN]
F_{tot}	=	total occuring forcein the tie rod [kN]
m_1, m_2	=	factors taking the model uncertainty into account [-]

Loading equations:
Total occuring force on the sheet pile wall F_{tot}:

$$F_{tot} = H_5 \cdot \frac{w_a}{\cos\alpha}$$

where:

$$H_5 = \left(H_4 + H_3 - \left(H_1 + H_2\right)\right)$$

with:

$$H_1 = 0.5 \cdot \gamma_w \left(h - L_1\right)^2$$

$$H_2 = 0.5 \cdot K_p \left(\gamma_s - \gamma_w\right) \cdot \left(h_1 - L_1\right)^2$$

$$H_3 = 0.5 \cdot \gamma_w \left(g_w - L_1\right)^2$$

$$H_{4a} = 0.5 \cdot K_a \cdot \gamma_a \left(h_3 - g_w\right)^2$$

$$H_{4b} = K_a \cdot \gamma_d \left(h_3 - g_w\right) + \left(g_w - L_1\right) + 0.5 \cdot K_a \cdot \left(\gamma_s - \gamma_w\right)\left(g_w - L_1\right)^2$$

Resistance (strength) equations:
Maximum bearable force F_u:

$$F_u = A_s \cdot f_s$$

Parameter definitions:
Parameter definitions:

h	=	the river water level [mLD]
g_w	=	the ground water level [mLD]
γ_s	=	the volumetric weight of the saturated soil [kN / m3]

γ_d	=	the dry volumetric weight of the soil [kN / m3]
γ_w	=	the volumetric weight of water [kN / m3]
L_1	=	the toe level of the sheet pile wall [mLD]
h_1	=	the ground level in front of the sheet pile wall on the river side [mLD]
h_3	=	the ground level next to the sheet pile wall on the land side [mLD]
K_a	=	the coefficient for active horizontal grain force [-]
K_p	=	the coefficient for passive horizontal grain force [-]
w_a	=	the distance between two tie rods [m]
A_s	=	the total area of the tie rod [m2]
f_s	=	the yield stress of the steel, net of any factoring [kN/m2]
α	=	the angle of inclination of the tie rod [˚]

Sources of failure mechanism equations / methods:

Sources of uncertainties in failure equations / input parameters:
Baecher & Christian (2003); CUR (1997); Vrouwenvelder et al. (2001)

Remarks:
See also Cc 1.2b

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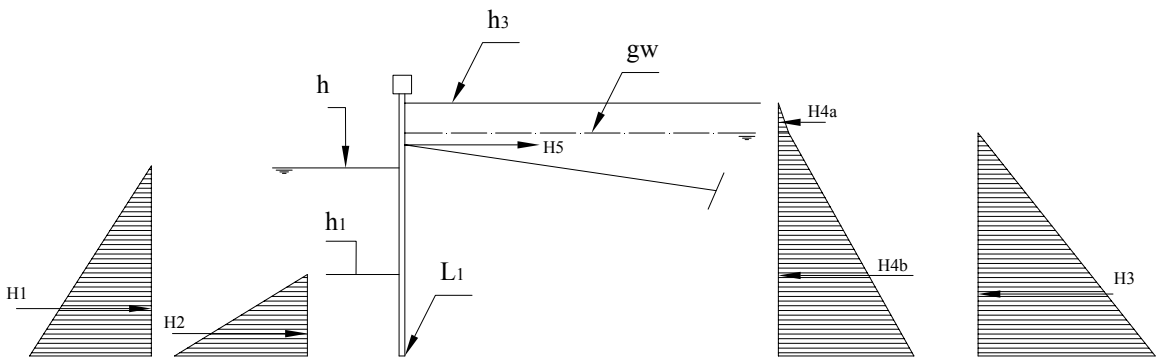
Cb 1.2b Overturning failure of wall element, insufficient strength of soil at anchor

Summary: Simple overturning failure of wall element driven by static water level differences and/or geotechnical loading. May occur after other effects have reduced soil strength (see e.g. Ba 1.5d), or under negative wave loads (see also Ca 2.2b). This sub-mechanism occurs when the anchor fails the soil.



Reliability equation:
The anchor head transfers the force from the tie rod to the soil. Failure occurs if the stress exerted by the anchor head exceeds the shear strength of the soil.
The reliability equation reads as follows:

$$z = m_1 \cdot F_r - m_2 \cdot F_{tot}$$

where:

H_r	=	force capacity of the soil around the anchor head [kN]
H_5	=	total occuring force in the anchot [kN]
m_1, m_2	=	factors taking the model uncertainty into account [-]

Loading equations:
The horizontal force on the sheet pile wall F_{tot}:

$$F_{tot} = H_5$$

where:

$$H_5 = \left(H_4 + H_3 - \left(H_1 + H_2\right)\right) \cdot w_a$$

with:

$$H_1 = 0.5 \cdot \gamma_w \left(h - L_1\right)^2$$

$$H_2 = 0.5 \cdot K_p \left(\gamma_s - \gamma_w\right) \cdot \left(h_1 - L_1\right)^2$$

$$H_3 = 0.5 \cdot \gamma_w \left(g_w - L_1\right)^2$$

$$H_{4a} = 0.5 \cdot K_a \cdot \gamma_a \left(h_3 - g_w\right)^2$$

$$H_{4b} = K_a \cdot \gamma_d \left(h_3 - g_w\right) + \left(g_w - L_1\right) + 0.5 \cdot K_a \cdot \left(\gamma_s - \gamma_w\right)\left(g_w - L_1\right)^2$$

Resistance (strength) equations:
The maximum force the tie rod can withstand based on the strength of the soil is defined as follows:

$$F_r = 0.5\left(\alpha + \beta - 1\right) h_a d_a^2 \gamma_d \left(\frac{1+\sin(\varphi)}{1-\sin(\varphi)} - \frac{1-\sin(\varphi)}{1+\sin(\varphi)}\right) - q h_a d_a \left(\alpha + \beta - 1\right) \frac{1-\sin(\varphi)}{1+\sin(\varphi)}$$

where:

$$\alpha = b_a / h_a$$

Parameter definitions:

h	=	the river water level [mLD]
g_w	=	the river water level [mLD]
γ_s	=	the volumetric weight of the saturated soil [kN / m^3]
γ_w	=	the volumetric weight of water [kN / m^3]
L_1	=	the toe level of the sheet pile wall [mLD]
h_1	=	the ground level in front of the sheet pile wall on the river side [mLD]
h_3	=	the ground level next to the sheet pile wall on the land side [mLD]
K_a	=	the coefficient for active horizontal grain force [-]
K_p	=	the coefficient for passive horizontal grain force [-]
w_a	=	the distance between two tie rods [m]
h_a	=	the height of the anchor head [m]
b_a	=	the width of the anchor head [m]
d_a	=	the depth of the bottom of the anchor head [m]
γ_d	=	the dry volumetric weight of the soil [kN / m^3]
α	=	ba/ha [-]
β	=	factor according to Buchholz [-]
q	=	surcharge load behind the anchored sheet pile wall [kN / m^2]
φ	=	the angle of internal friction of the soil [°]

Sources of failure mechanism equations / methods:

Sources of uncertainties in failure equations / input parameters:
Baecher & Christian (2003); CUR (1997); Vrouwenvelder et al. (2001);

Remarks:
Further validation of (alpha + beta -1) component of resistance equation required. (MWM/NB)

See also Cb 1.2a and Cc 1.2b

Status of Draft

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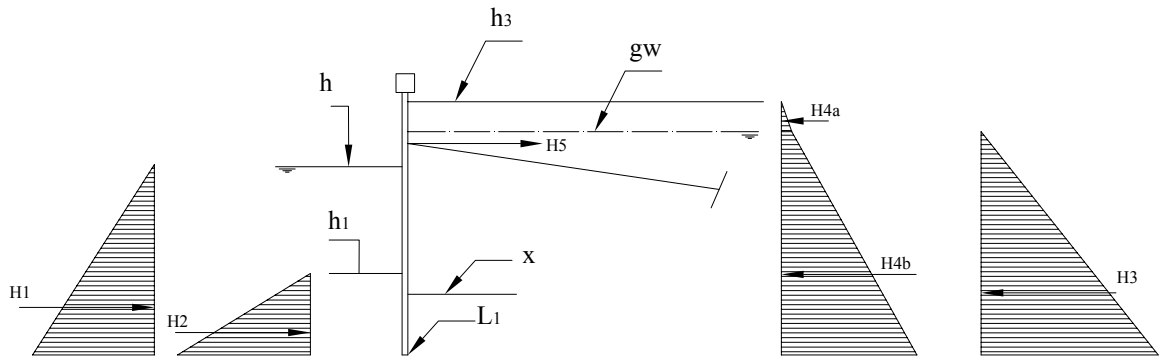
Cb 1.2c Failure of sheet pile wall element in bending

Summary: Simple bending failure of wall element driven by static water level differences and/or geotechnical loading. May occur after other effects have reduced soil strength (see e.g. Ba 1.5d), or under negative wave loads (see also Ca 2.2b). This sub-mechanism occurs when the sheet piles fail.



Reliability equation:
Failure occurs if the capacity of the sheet pile cross section is exceeded by the actual bending moments. From the maximum bending moment in the sheet pile, a maximum tensile stress in the sheet pile wall can be derived, using the moment of inertia and the height of the section. That tensile stress is compared against the yield stress of the sheet pile steel

The reliability equation reads as follows:

$$z = m_1 \cdot f_s - m_2 \cdot \sigma_b$$

where:

f_s	=	yield stress of the steel cross section [kN/m^2]
σ_b	=	maximum tensile stress in the sheet pile [kN/m^2]
m_1, m_2	=	factors taking the model uncertainty into account [-]

Loading equations:
The maximum tensile stress in the sheet pile cross section is:

$$\sigma_b = M_{max} \cdot \frac{z}{I_z}$$

$$M_{max} = \max\left(M_1; M_2; M_3; M_4\right)$$

The maximum and minimum moments are found where the shear force in the cross section is 0. M_{max} is the highest of those maxima and minima. The shear force in the cross section at a level x can be found with the following equations:

$$H_1 = 0.5 \cdot \gamma_w \cdot \left(h - x\right)^2$$
$$H_2 = 0.5 \cdot K_p \cdot \left(\gamma_s - \gamma_w\right)\left(h_1 - x\right)^2$$

$$H_3 = 0.5 \cdot \gamma_w \left(g_w - x\right)^2$$

Resistance (strength) equations:
The yield stress f_s of the steel sheet pile cross section (net of any factoring) determines the limit of the tensile stress.

f_s = ???

$$H_{4a} = 0.5 \cdot K_a \cdot \gamma_d \left(h_3 - g_w\right)^2$$

$$H_{4b} = K_a \cdot \gamma_d \left(h_3 - g_w\right) + \left(g_w - L_1\right) + 0.5 \cdot K_a \cdot \left(\gamma_s - g_w\right) + \left(g_w - L_1\right)$$

$$H_{4b} = K_a \cdot \left(\gamma_s - \gamma_w\right)\left(h_3 - x\right)^2$$

$$H_5 = \left(H_4 + H_3 - \left(H_1 + H_2\right)\right)$$

The maximum moment can be found combining shear forces with the arm of the force:

$$M_1 = 0.5 \cdot \gamma_w \left(h - x\right)^2 \cdot 1/3\left(h - x\right)$$

$$M_2 = 0.5 \cdot K_p \left(\gamma_s - \gamma_w\right)\left(h_1 - x\right)^2 \cdot 1/3\left(h_1 - x\right)$$

$$M_3 = 0.5 \cdot \gamma_w \left(g_w - x\right)^2 \cdot 1/3\left(g_w - x\right)$$

$$M_{4a} = 0.5 \cdot K_a \cdot \gamma_d \left(h_3 - g_w\right)\left(1/3\left(h_3 - g_w\right) + g_w - x\right)$$

$$M_{4b} = 0.5 \cdot K_a \cdot \gamma_d \left(h_3 - g_w\right)\left(g_w - x\right) \cdot 1/3\left(g_w - x\right) + 0.5 \cdot K_a \left(\gamma_s - \gamma_w\right)\left(g_w - x\right)^2 \cdot 1/3\left(h_3 - x\right)$$

Parameter definitions:

h	=	the river water level [mLD]
g_w	=	groundwater level [mLD]
γ_d	=	the volumetric weight of the dry soil [kN / m3]
γ_s	=	the volumetric weight of the saturated soil [kN / m3]
γ_w	=	the volumetric weight of water [kN / m3]
L_1	=	the toe level of the sheet pile wall [mLD]
h_1	=	the ground level in front of the sheet pile wall on the river side [mLD]
h_3	=	the ground level next to the sheet pile wall on the land side [mLD]
K_a	=	the coefficient for active horizontal grain force [-]
K_p	=	the coefficient for passive horizontal grain force [-]
z	=	the distance between the centre of gravity and the outer edge of the sheet pile profile [m]
I_z	=	the moment of inertia of the sheet pile cross section [m4/m]
M_{max}	=	the maximum bending moment in the sheet pile wall

Sources of failure mechanism equations / methods:

Sources of uncertainties in failure equations / input parameters:
Baecher & Christian (2003); CUR (1997); Vrouwenvelder et al. (2001);

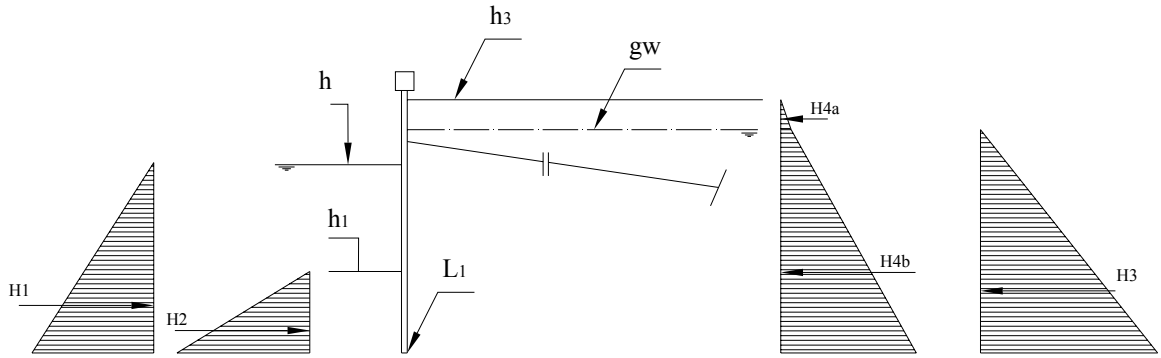
Remarks:
See also Cb 1.2a,b and Cc 1.2b

Status of Draft

Date	Revision	Prepared by	Organisation	Approved by	Notes
		FB	HRW	WA	
25/01/ 07	v3_4_p03		LWI		edited

Cb 1.2d Rotation failure of sheet pile wall after loss of tie rod

Summary: Rotational failure of wall element driven by static water level differences, geotechnical loading and/or negative wave loads (see also Ca 2.2b). May occur after other effects have reduced soil strength (see e.g. Ba 1.5d). This sub-mechanism occurs when the sheet piles fail in rotation.



Reliability equation:
Collapse of the sheet pile wall after failure of the tie rod depends on moment equilibrium around the toe of the sheet pile.
The reliabilty equation reads as follows:

$$z = m_{spa,m,R} \cdot M_r - m_{spa,m,S} \cdot M_l$$

where:

M_r	=	resulting resisting moment [kNm]
M_l	=	resulting loading moment [kNm]
$m_{spa;m;R}$	=	model factor for the strength [-]
$m_{spa;m;S}$	=	model factor for loading [-]

Loading equations:
The resulting loading moment is taken around the toe of the sheet pile wall and is built up as follows:

$$M_l = M_3 + M_{4a} + M_{4b}$$

where:

$$M_3 = 0.5 \cdot \gamma_w \left(g_w - L_1\right)^2 \cdot 1/3\left(g_w - L_1\right)$$

$$M_{4a} = 0.5 \cdot K_a \cdot \gamma_d \left(h_3 - g_w\right)\left(1/3\left(h_3 - g_w\right) + g_w - L\right)_1$$

$$M_{4b} = 0.5 \cdot K_a \cdot \gamma_d \left(h_3 - g_w\right)\left(g_w - L_1\right) \cdot 1/3\left(g_w - L_1\right) + 0.5 \cdot K_a \left(\gamma_s - \gamma_w\right)\left(g_w - L_1\right)^2 \cdot 1/3\left(h_3 - L_1\right)$$

Resistance (strength) equations:
The resulting resisting moment is taken around the toe of the sheet pile wall and is built up as follows:

$$M_r = M_1 + M_2$$

where:

$$M_1 = 0.5 \cdot \gamma_w \left(h - L_1\right)^2 \cdot 1/3\left(h - L_1\right)$$

$$M_2 = 0.5 \cdot K_p \cdot \left(\gamma_s - \gamma_w\right)\left(h_1 - L_1\right)^2 \cdot 1/3\left(h_1 - L_1\right)$$

Parameter definitions:

h	=	the river water level [mLD]
g_w	=	groundwater level [mLD]
γ_s	=	the volumetric weight of the saturated soil [kN / m^3]
γ_w	=	the volumetric weight of water [kN / m^3]
L_1	=	the toe level of the sheet pile wall [mLD]
h_1	=	the ground level in front of the sheet pile wall on the river side [mLD]
h_3	=	the ground level next to the sheet pile wall on the land side [mLD]
K_a	=	the coefficient for active horizontal grain force [-]
K_p	=	the coefficient for passive horizontal grain force [-]

Sources of failure mechanism equations / methods:
Sources of uncertainties in failure equations / input parameters: Baecher & Christian (2003); CUR (1997); Vrouwenvelder et al. (2001);
Remarks: See also Cb 1.2a and Cc 1.2b

Status of Draft

Date	Revision	Prepared by	Organisation	Approved by	Notes
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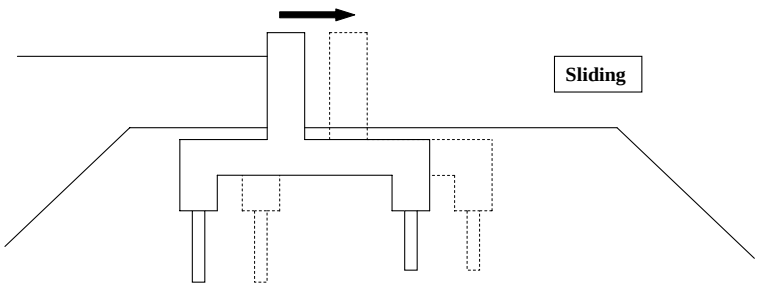
Cc 1.2ai Sliding failure of wall element, no waves

Summary: Simple sliding failure of crown wall or similar element driven by static water level differences. Most likely after other effects have reduced soil strength (see e.g. Ba 1.5d).



Reliability equation:
Failure occurs when horizontal force from hydrostatic water level difference exceeds net shear strength of foundation or wall / foundation junction

$$z = T_{max} - T$$

where:

T	=	actual horizontal force [kN/m]
T_{max}	=	maximum bearable force [kN/m]

Loading equations:
Hydrostatic water level difference giving horizontal load, and pressure gradients.
Actual horizontal force due to water level is calculated for different cases:

For sealed slopes:

$$T = W_H$$

where:
horizontal force due to water

$$W_H = \gamma_W \cdot \frac{h_W^2}{2}$$

Resistance (strength) equations:
Soil shear strength or wall / foundation friction
Maximum bearable force:

$$T_{max} = \mu \cdot (G - W_v)$$

where:

vertical uplift force underneath structure

$$W_v = \gamma_W \cdot \frac{h_W}{2} \cdot l$$

Parameter definitions:

l	=	length of slide line [m]
γ_i	=	volume weight of cover layer [kN/m^3]
γ_w	=	volume weight of water [kN/m^3]
h_w	=	water level in front of structure [m]
μ	=	sliding factor [-]
G	=	weight force of structure [kN/m]

Sources of failure mechanism equations / methods:
Abramson, L.W., Lee, T.S., Sharma, S., Boyce, G.M., (2002)

Sources of uncertainties in failure equations / input parameters:
Sliding is assumed to happen at the interface between concrete structure and subsoil. For simplicity reason the sliding surface is assumed to be flat, there are no slabs or curtains underneath the structure. If the structure is built with slabs a working assumption could be that no sliding will occur.

Remarks:
Bulk sliding of complex structures is scarce where flat sliding surfaces may lead to such failures. This mechanism may also be unrealistic as this simple representation of friction can only be applied when the material under the wall behaves as a granular material. That would in turn probably allow seepage / piping flows to act before this failure mechanism.

See also Ca 1.2

Status of Draft

Date	Revision	Prepared by	Organisation	Approved by	Notes
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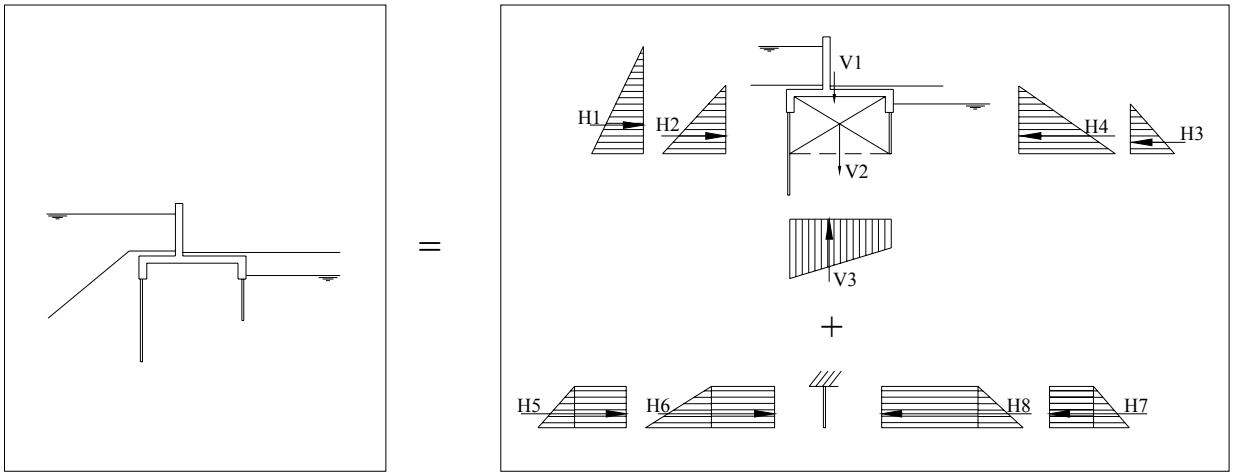
Cc 1.2aii Sliding failure of wall element, no waves

Summary: Simple sliding failure of crown wall or similar element driven by static water level differences. Most likely after other effects have reduced soil strength (see e.g. Ba 1.5d).



Reliability equation:

Failure occurs when horizontal (and up-lift) forces exceed sliding resistance. The simplest representation of this is to balance the hydrostatic loads by a friction resistance driven in turn by the (net) weight force of the crown wall. In practice, this mechanism may be unrealistic as this simple representation of friction can only be applied when the material under the wall behaves as a granular material. That would in turn probably allow sepage / piping flows to act before this failure mechanism.

The reliability equation is expressed by:

$$z = m_{c,s,R} \cdot \tan\delta \cdot \sum V - m_{c,s,S} \cdot \sum H$$

where:

$\sum V$	=	total resulting vertical force [kN/m]
$\sum H$	=	total resulting horizontal loading force [kN/m]
$m_{c;s;R}$	=	model factor for the horizontal resistance [-]
$m_{c;s;S}$	=	model factor for the horizontal loading force [-]

Loading equations:

$$H_{total} = (H_1 + H_2 + H_5 + H_6) - (H_3 + H_4 + H_7 + H_8)$$

where:

Main structure (concrete wall & mobilised soil):

$$H_1 = 0.5 \cdot \gamma_w (h - L_3)^2$$

$$H_2 = 0.5 \cdot K_a (\gamma_s - \gamma_w) \cdot (h_1 - L_3)^2$$

$$H_3 = 0.5 \cdot \gamma_w (g_w - L_3)^2$$

$$H_4 = 0.5 \cdot K_p (\gamma_s - \gamma_w)(h_3 - L_3)^2$$

Sheet pile cut-off:

$$H_5 = \gamma_w (h - L_3)(L_3 - L_1) + 0.5 \cdot \gamma_w (L_3 - L_1)^2$$

Resistance (strength) equations:

$$V_{total} = \tan\delta \cdot (V_1 + V_2 - V_3)$$

where:

V_1 is the weight of the concrete structure - depends on the geometry of the wall.

V_2 is the vertical weight of the mobilised soil:

$$V_2 = \gamma_s \cdot h_s \cdot B$$

V_3 is the upward hydraulic force

$$V_3 = \gamma \cdot (g_w - L_3) \cdot B + 0.5 \cdot \gamma_w \cdot (L_3 - L_1) \cdot B$$

$$H_6 = K_p(\gamma_s - \gamma_w)(h_1 - L_3)(L_3 - L_1) + 0.5 \cdot K_p(\gamma_s - \gamma_w)(L_3 - L_1)^2$$

$$H_7 = \gamma_w(g_w - L_3)(L_3 - L_1) + 0.5 \cdot \gamma_w(L_3 - L_1)^2$$

$$H_8 = K_a(\gamma_s - \gamma_w)(h_3 - L_3)(L_3 - L_1) + 0.5 \cdot K_a(\gamma_s - \gamma_w)(L_3 - L_1)^2$$

Parameter definitions:

h	=	river water level [mOD]
g_w	=	ground water level [mOD]
d	=	friction coefficient [-]
γ_s	=	the volumetric weight of the saturated soil [kN / m^3]
γ_w	=	the volumetric weight of water [kN / m^3]
L_1	=	the level of the longest sheet pile cut-off [mOD]
L_3	=	the level of the shortest sheet pile cut-off [mOD]
h_1	=	the level of the crest in front of the concrete wall on the river side [mOD]
h_3	=	the level of the crest in front of the concrete wall on the land side [mOD]
K_a	=	the coefficient for active horizontal grain force [-]
K_p	=	the coefficient for passive horizontal grain force [-]
B	=	the width of the concrete structure between extensions [m]
h_s	=	the height of the mobilised soil [m]

Sources of failure mechanism equations / methods:

Sources of uncertainties in failure equations / input parameters:

Remarks:
See also Cc 1.2

Status of Draft

Date	Revision	Prepared by	Organisation	Approved by	Notes
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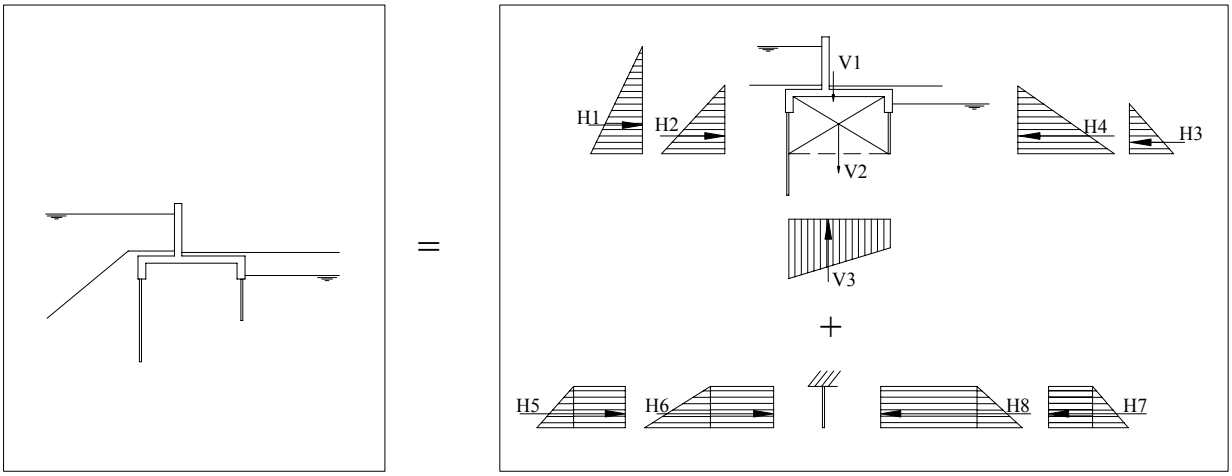
Cc 1.2b Overturning failure of wall element, no waves

Summary: Simple overturning failure of crown wall or similar element driven wall by static water level differences. Most likely to occur after other effects have reduced soil strength (see e.g. Ba 1.5d).



Reliability equation:

When the water level reaches the concrete wall, a horizontal hydraulic force is exerted against the wall. This force can overturn the concrete structure. Resisting forces are the weight of the structure and the pressures of the ground keeping the structure into place. Overturning is assumed to occur when tensile stress occurs in the foundational plane. This assumption leads to the following limit state function:

$$z = m_{c,o,R} \cdot 1/6 \cdot B_c - m_{c,o,S} \cdot \frac{\sum M}{\sum V}$$

where:

B_c	=	width of the base of the concrete structure
$\sum M$	=	the resulting moment [kNm / m]
$\sum V$	=	resulting vertical force acting on the concrete wall structure [kN / m]
$m_{c,o,;R}$	=	model factor for the strength model [-]
$m_{c,o;S}$	=	model factor for the loading model [-]

Loading equations:

The resulting moment is taken around the centre of the base of the mobilised soil and is built up as follows:

Main *structure (concrete wall & mobilised soil):*

$M1 = 0.5 .\gamma_w(h - L_3)^2. 1/3(h - L_3)$

$M2 = 0.5.Ka .(\gamma s -\gamma w)(h1 - L3)2. 1/3(h1 - L3)$

$M3 = 0.5 .\gamma_w(gw - L_3)^2. 1/3(gw - L_3)$

$M4 = 0.5.K_p .(\gamma_s -\gamma_w)(h_3 - L_3)^2. 1/3(h_3 - L_3)$

Sheet pile cut-off:

$M5 = \gamma_w(h - L_3)(L_3 - L_1) .1/2(L_3 – L_1)+$
$0.5 .\gamma_w(L_3 - L_1)^2 .2/3(L_3 – L_1)$

$M6 = K_p .(\gamma_s -\gamma_w)(h_1 - L_3)(L_3 - L_1) . 1/2(L_3 – L_1)$
$+0.5.K_p .(\gamma_s -\gamma_w)(L_3 - L_1)^2 .2/3(L_3 – L_1)$

$M7 =\gamma_w(gw - L_3) (L_3 - L_1) .1/2(L_3 – L_1)+$
$0.5 .\gamma_w(L_3 - L_1)^2.2/3(L_3 – L_1)$

Resistance (strength) equations:

V_1 is the weight of the concrete structure in kN per metre - depends on the geometry of the wall.

V_2 is the vertical weight of the mobilised soil in kN/m:

$$V_2 = \gamma_s h_s .B$$

V_3 is the upward hydraulic force:

$$V_3 = \gamma_w. (gw - L_3) .B + 0.5. \gamma_w. (L_3 - L_1) .B$$

$M8 = K_a .(\gamma_s - \gamma_w)(h_3 - L_3)(L_3 - L_1) .1/2(L_3 - L_1) + 0.5.K_a .(\gamma_s - \gamma_w)(L_3 - L_1)^2.2/3(L_3 - L_1)$

Moments due to vertical forces:

$Mv1 = b_{gr} . V1$

$Mv2 = 0$

$Mv3 = 0.5. \gamma_w. (L_3 - L_1) .B.(1/2B-1/3B)$

Resulting moment

$\Sigma M = M1+M2-M3-M4-M5-M6+M8+M9 - Mv1+Mv3$

Parameter definitions:

h	=	river water level [mOD]
gw	=	ground water level [mOD]
γ_s	=	volumetric weight of the saturated soil [kN / m^3]
γ_w	=	volumetric weight of water [kN / m^3]
L_1	=	level of the longest sheet pile cut-off [mLD]
L_3	=	level of the shortest sheet pile cut-off [mLD]
h	=	level of the crest in front of the concrete wall on the river side [mLD]
h_3	=	level of the crest in front of the concrete wall on the land side [mLD]
K_a	=	coefficient for active horizontal grain force [-]
K_p	=	coefficient for passive horizontal grain force [-]
B	=	width of the concrete structure between extensions [m]
b_g	=	distance between the centre of gravity of the concrete structure and the centre of the mobilised soil [m]
h_s	=	height of mobilised soil [m]
LD	=	the local datum to which all levels are reduced, probably the prevailing land survey datum (ODN in UK)

Sources of failure mechanism equations / methods:

Sources of uncertainties in failure equations / input parameters:

Remarks:

See Ca 1.2

Status of Draft

Date	Revision	Prepared by	Organisation	Approved by	Notes
		FB	HRW	WA / MM	
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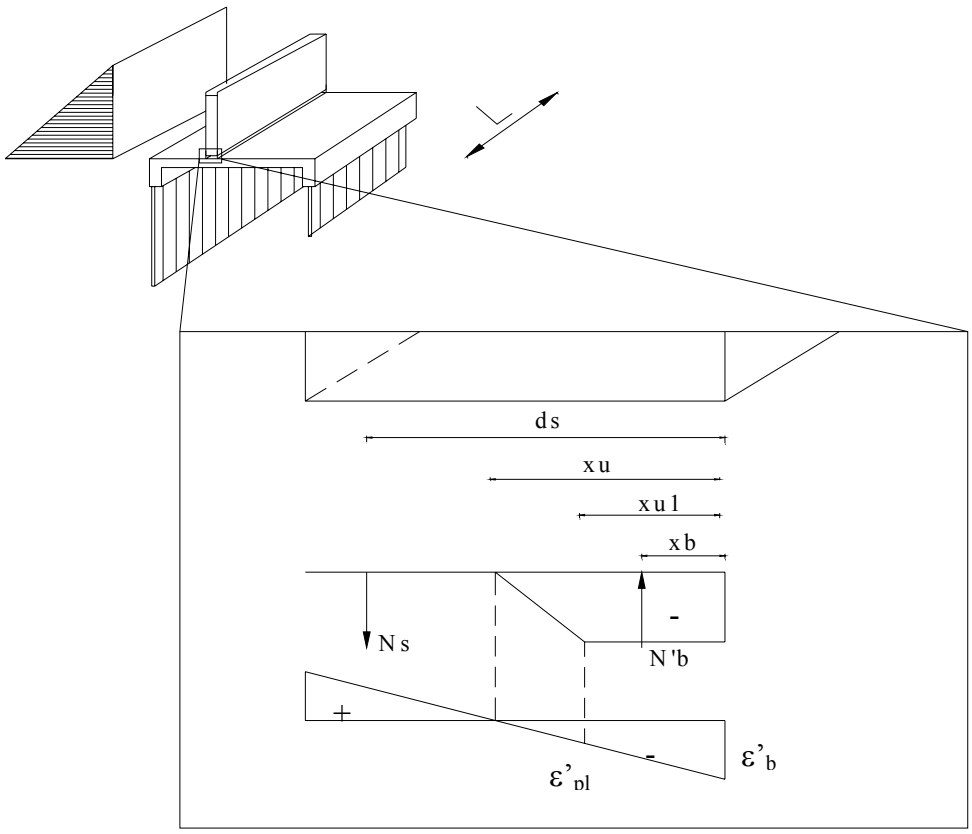
Cc 1.2c Bending failure of wall element, no waves

Summary: Simple structural failure of cantilever wall element or similar driven by static water level differences.



Reliability equation:

The horizontal hydraulic force exerted by the river water level and the ground resting against the riverside of the concrete wall cause bending moments in the vertical slab of the wall. Failure of the vertical slab occurs when there is insufficient reinforcement to take on the tensile stress due to the bending moment.

The relialbility equation is expressed by:

$$z = m_{c,b,R} \cdot M_u - m_{c,b,S} \cdot M_d$$

where:

M_u	=	the maximum moment the cross section can take on [kN/m]
M_d	=	actually occurring moment exerted by the hydraulic and geotechnical loading [kN/m]
$m_{c;b;R}$	=	model factor for the strength model [-]
$m_{c;b;S}$	=	model factor for the loading model [-]

Loading equations:

Resulting moment:Moments are taken about the base of the vertical cantilever

$$M_d = M_1 + M_2 - M_3$$

where:

Resistance (strength) equations:

$$M_u = N_s \cdot (d_s - x_b)$$

where:

$$N_s = A_s f_s$$

$$M_1 = 0.5 \cdot \gamma_w \left(h - \left(h_c - d_4\right)\right)^2 \cdot 1/3\left(h - \left(h_c - d_4\right)\right)$$

$$M_2 = 0.5 \cdot K_a \left(\gamma_s - \gamma_w\right)\left(h_1 - \left(h_c - d_4\right)\right)^2 \cdot 1/3\left(h_1 - \left(h_c - d_4\right)\right)$$

$$M_3 = 0.5 \cdot K_p \left(\gamma_s - \gamma_w\right)\left(h_3 - \left(h_c - d_4\right)\right)^2 \cdot 1/3\left(h_3 - \left(h_c - d_4\right)\right)$$

$$x_b = \frac{\frac{1}{2} x_{u1}{}^2 f'_b + \frac{1}{2}\left(x_u - x_{u1}\right)\left(\frac{1}{3}\left(x_u - x_{u1}\right) + x_{u1}\right) f'_b}{x_{u1} f'_b + \frac{1}{2}\left(x_u - x_{u1}\right) f'_b}$$

with:

$$x_u = \frac{N_s}{\frac{1}{2}\left(1 + \frac{\varepsilon'_b - \varepsilon'_{pl}}{\varepsilon'_b}\right) L \cdot f'_b}$$

$$x_{u1} = \frac{\varepsilon'_b - \varepsilon'_{pl}}{\varepsilon'_b} x_u$$

Parameter definitions:

h	=	river water level [mLD]
h_c	=	crest level of the concrete wall [mLD]
h_1	=	ground level on the riverside of the concrete wall [mLD]
h_3	=	ground level on the landside of the concrete wall [mLD]
d_4	=	height of the vertical slab of the concrete wall [m]
γ_w	=	volumetric weight of water [kN / m^3]
γ_s	=	volumetric weight of saturated soil [kN / m^3]
N_s	=	total tensile force in the steel reinforcement [kN]
A_s	=	total area of steel reinforcement in the concrete cross section [m^2]
f_s	=	yield strength of reinforcement steel [kN/m^2]
x_u	=	pressure zone in the concrete [m]
x_{u1}	=	plastic pressure zone in the concrete [m]
x_b	=	distance of the resulting pressure in the concrete from the edge [m]
ε'_{bu}	=	ultimate strain of the concrete [-]
ε'_{pl}	=	plasticity strain of the concrete [-]
f'_b	=	cubic pressure strength of the concrete [kN/m^2]
L	=	length of the concrete slab [m]
K_a	=	coefficient for active horizontal grain force [-]
K_p	=	coefficient for passive horizontal grain force [-]

Sources of failure mechanism equations / methods:

Sources of uncertainties in failure equations / input parameters:

Remarks:
See also Ca 1.2

Status of Draft

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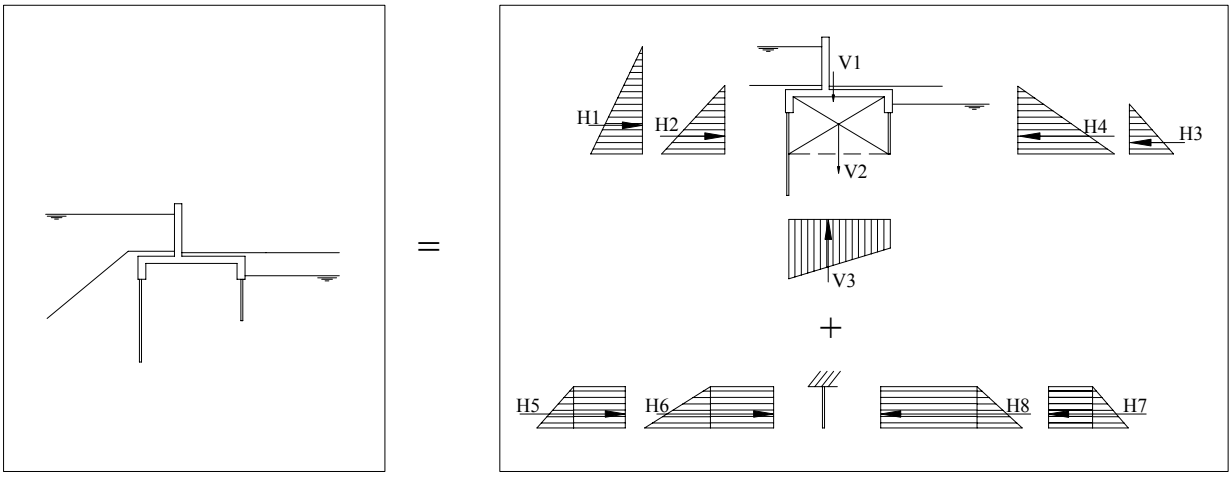
Cc 1.2d Shear failure of wall element, no waves

Summary: Simple shear failure in crown wall element driven by static water level differences.



Reliability equation:

The horizontal hydraulic force exerted by the river water level and the ground resting against the riverside of the concrete wall cause shear stress at the base section of the vertical slab. Failure of the vertical slab occurs if the concrete cross section has insufficient width or shear strength to take on the horizontal force. The approach below applies to concrete slabs without reinforcement for shear stress.

The reliability equation is expressed by:

$$z = m_{c,sh,R} \cdot \tau_u - m_{c,sh,S} \cdot \tau_d$$

where:

τ_u	=	the maximum shear stress the cross section can withstand [N/mm^2]
τ_d	=	actually occurring shear stress exerted by the hydraulic and geotechnical loading [N/mm^2]
$m_{c;sh;R}$	=	model factor for the strength model [-]
$m_{c;sh;S}$	=	model factor for the loading model [-]

Loading equations:

$$\tau_d = H_1 + H_2 - H_3$$

$$H_1 = 0.5 \cdot \gamma_w \left(h - \left(h_c - d_4\right)\right)^2$$

$$H_2 = 0.5 \cdot K_a \left(\gamma_s - \gamma_w\right)\left(h_1 - \left(h_c - d4\right)\right)^2$$

$$H_3 = 0.5 \cdot K_p \left(\gamma_s - \gamma_w\right)\left(h_3 - \left(h_c - d_4\right)\right)^2$$

The shear forces are determined in the base of the vertical concrete slab:
Resulting shear force
$\Sigma H = H1+H2-H3$

Resistance (strength) equations:

$$\tau_u = \tau_1 \quad \leq \quad \tau_2$$

where:

$$\tau_1 = 0.4 f_b k_\lambda k_h \sqrt[3]{\omega_0}$$

with:

$$k_\lambda = 1.0$$

$$k_h = 1.6 - d_2 \qquad \geq 1.0$$

$$\omega_0 = \frac{100 A_s}{L d_2} \qquad \leq 2.0$$

$$\tau_2 = 0.2 f'_b \, k_n k_\theta$$

with:

$$k_n = 1.0$$

$$k_\theta = 1.0$$

Parameter definitions:

h	=	river water level [mOD]
h_c	=	crest level of the concrete wall [mOD]
h_1	=	the ground level on the riverside of the concrete wall [mOD]
h_3	=	the ground level on the landside of the concrete wall [mOD]
d_4	=	the height of the vertical slab of the concrete wall [m]
d_2	=	the width of the vertical slab [m]
A_s	=	total area of steel reinforcement in the concrete cross section [m^2]
γ_w	=	the volumetric weight of water [kN/m^3]
γ_s	=	the volumetric weight of saturated soil [kN/m^3]
f'_b	=	the cubic pressure strength of the concrete [kN/m^2]
f_b	=	the cubic tensile strength of the concrete [kN/m^2]
ω_0	=	reinforcement percentage [-]
τ_1	=	the maximum shear stress the cross section can take on, if no shear stress reinforcement is present [kN/m^2]
k_λ, k_h	=	coefficients [-]
k_n, k_θ	=	coefficients [-]
L	=	length of the concrete slab [m]
K_a	=	the coefficient for active horizontal grain force [-]
K_p	=	coefficient for passive horizontal grain force [-]

Sources of failure mechanism equations / methods:

Sources of uncertainties in failure equations / input parameters:

Remarks:
See also Ca 1.2

Status of Draft

Date	Revision	Prepared by	Organisation	Approved by	Notes
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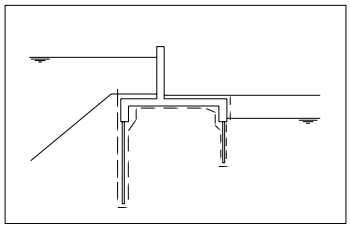
Cc1.5 Piping under parapet wall

Summary: Piping directly underneath sheet pile cut-off

Sketch of failure mechanism:



Reliability equation:
Failure due to piping directly underneath the sheet pile cut-off is taken into account if the water level exceeds the ground water level in the earth bank behind the wall. This ensures a positive water head over the concrete structure, which drives the piping process. One of the requirements is that the water level persists long enough for the piping process to initiate
The reliabilty equation is expressed by:

$$z = m_{c,p,R} \cdot \Delta h_c - \Delta h_a$$

where:

Δh_c	=	critical head difference [m]
Δh_a	=	actual head difference [m]
$m_{c;p;R}$	=	model factor for the strength model [-]

Loading equations:
The head over the concrete structure:

$$\Delta h_a = h - g_w$$

Resistance (strength) equations:
The critical head associated with the piping process:

$$\Delta h_c = \left(L_v + 1/3 \cdot L_h\right) / c_t$$

Parameter definitions:

h	=	the river water level [mOD]
g_w	=	the groundwater level behind the concrete structure [mOD]
L_v	=	the vertical seepage length [m]
L_h	=	the horizontal seepage length [m]
c_t	=	the creep ratio [-]

Sources of failure mechanism equations / methods:
Terzaghi (1967)

Sources of uncertainties in failure equations / input parameters:

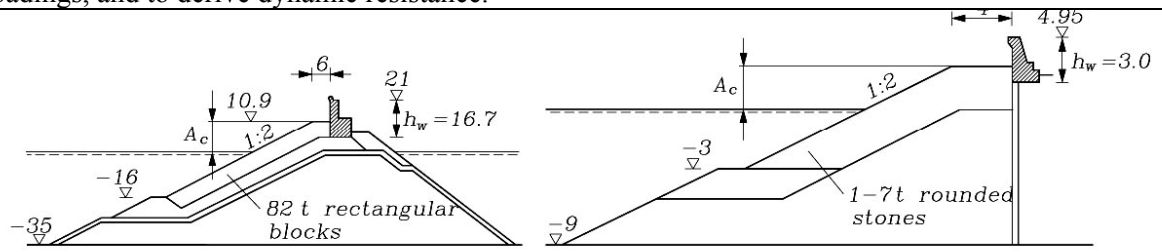
Remarks:

Status of Draft

Date	Revision	Prepared by	Organisation	Approved by	Notes
		FB	HRW		
25/01/ 07	v3_4_p03		LWI		edited

Cc 2.2a Bulk sliding of wall or wall element, direct wave force

Summary: Simple sliding failure of elevated crown wall element driven by direct wave forces. Simple methods will give quasi-static loads, but more complex methods are needed to give impulsive loadings, and to derive dynamic resistance.



Reliability equation:
Failure when horizontal force exceeds net friction force between wall and foundation.
The reliability equation is expressed by:

$$z = F_r - F_{h,0.1\%}$$

where:

F_r	=	friction force [kN/m]
$F_{h,1.0\%}$	=	horizontal wave load [kN/m]

Loading equations:
Horizontal wave load, F_h, given by equations like:

$$F_{h,0.1\%} = \rho \cdot g \cdot h_w \cdot L_{op} \cdot \left(\alpha + \beta \cdot \frac{H_s}{A_c} \right)$$

Force equations and parameters α and β will need to be adjusted for front slope armour type (roughness, thickness, porosity etc). Values of coefficients are given by CIRIA / CUR (1991) or Bradbury et al (1988)

Resistance (strength) equations:
Friction force given by :

$$F_r = \mu \cdot \left(M_g - B_u - F_u \right)$$

Up-lift force F_u given from reduction applied to intensity of F_h, adjusted for foundation material / conditions.

Buoyancy force, B_u, given by structure geometry, water level, and density of water. Structure weight force given by dry mass, M.

Friction coefficient μ for roughness of caisson / wall base and foundation material

Parameter definitions:

$F_{h,0.1\%}$	=	peak horizontal wave force in 1000 waves [kN/m]
F_u	=	wave driven up-lift force acting on underside of crown wall [kN/m]
H_s	=	significant wave height [m]
h_w	=	height of crown wall element [m]
A_c	=	crest freeboard (without crown wall) [m]
L_{op}	=	length of peak period wave in deep water [m]
μ	=	coefficient of friction between crown wall base and underlying material [-]
M_g	=	dry mass of crown wall element [kg] or [t]
B_u	=	buoyant up-lift force at static water level [kN/m]
ρ	=	density of water [kg/m^3]
α	=	coefficient for determination of horizontal wave load [-]
β	=	coefficient for determination of horizontal wave load [-]

Sources of failure mechanism equations / methods: CIRIA, CUR, CETMEF (2006); Bradbury A.P. & Allsop N.W.H. (1988); Bradbury A.P., Allsop N.W.H. & Stephens R.V. (1988); Martin F.L. (1999); Martin F.L., Losada M.A. & Medina R. (1999); Pedersen J. (1996); Pedersen J. & Burcharth H.F. (1992);
Sources of uncertainties in failure equations / input parameters: Discussion from PROVERBS / MCS and in Rock Manual. Guidance on model uncertainties; Identify data on parameter uncertainties, s.d., distribution types;
Remarks:

Status of Draft

Date	Revision	Prepared by	Organisation	Approved by	Notes
				WA / MM	
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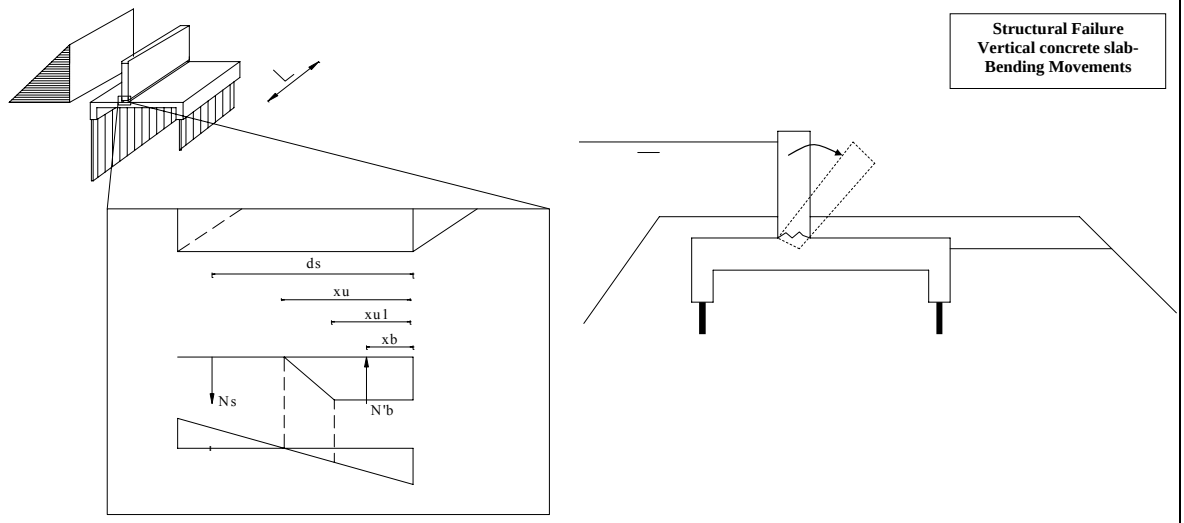
Cc 2.2b Bending failure of wall element by direct wave force

Summary: Simple structural failure of cantilever wall element or similar driven by wave loadings. See Cc 1.2c for static water version.



Reliability equation:

The horizontal hydraulic force exerted by wave loads resisted by ground against the landside of the concrete wall cause bending moments in the vertical slab of the wall. Failure of the vertical slab occurs when there is insufficient reinforcement to take on the tensile stress due to the bending moment.

The reliability equation is expressed by:

$$z = m_{c,b,R} \cdot M_u - m_{c,b,S} \cdot M_d$$

where:

M_u	=	maximum moment the cross section can take on [kNm]
M_d	=	actually occurring moment exerted by the hydraulic and geotechnical loading [kNm]
$m_{c;b;R}$	=	model factor for the strength model [-]
$m_{c;b;S}$	=	model factor for the loading model [-]

Loading equations:

$$M_d = F_{h,0.1\%} \cdot ???$$

Moments should be taken around the base of the vertical concrete slab for wave forces:

$$F_{h,0.1\%} = \rho \cdot g \cdot h_w \cdot L_{op} \cdot \left(\alpha + \beta \cdot \frac{H_s}{A_c} \right)$$

Force equations and parameters α and β will need to be adjusted for front slope armour type (roughness, thickness, porosity etc). Values of coefficients are given by CIRIA / CUR (1991) or Bradbury et al (1988)

Resistance (strength) equations:

$$M_u = N_s \cdot (d_s - x_b)$$

where:

$$N_s = A_s f_s$$

$$x_b = \frac{\frac{1}{2} x_{u1}^2 f'_b + \frac{1}{2}(x_u - x_{u1})\left(\frac{1}{3}(x_u - x_{u1}) + x_{u1}\right) f'_b}{x_{u1} f'_b + \frac{1}{2}(x_u - x_{u1}) f'_b}$$

with:

$$x_u = \frac{N_s}{\frac{1}{2}\left(1 + \frac{\varepsilon'_b - \varepsilon'_{pl}}{\varepsilon'_b}\right) L \cdot f'_b}$$

$$x_{u1} = \frac{\varepsilon'_b - \varepsilon'_{pl}}{\varepsilon'_b} x_u$$

Parameter definitions:

d_4	=	height of the vertical slab of the concrete wall [m]
N_s	=	total tensile force in the steel reinforcement [kN]

A_s	=	total area of steel reinforcement in the concrete cross section [m^2]
f_s	=	yield strength of reinforcement steel [kN/m^2]
x_u	=	pressure zone in the concrete [m]
x_{u1}	=	plastic pressure zone in the concrete [m]
x_b	=	distance of the resulting pressure in the concrete from the edge [m]
ε'_{bu}	=	ultimate strain of the concrete [-]
ε'_{pl}	=	plasticity strain of the concrete [-]
f'_b	=	cubic pressure strength of the concrete [kN/m^2]
L	=	length of the concrete slab [m]
$F_{h,0.1\%}$	=	peak horizontal wave force in 1000 waves [kN/m]
h_w	=	height of crown wall element [m]
L_{op}	=	length of peak period wave in deep water [m]
α	=	coefficient for determination of horizontal wave load [-]
β	=	coefficient for determination of horizontal wave load [-]

Sources of failure mechanism equations / methods:
Bradbury A.P. & Allsop N.W.H. (1988); Bradbury A.P., Allsop N.W.H. & Stephens R.V. (1988); CIRIA / CUR (1991); Jensen O.J. (1983); Jensen O.J. (1984); Martin F.L. (1999); Martin F.L., Losada M.A. & Medina R. (1999); Pedersen J. (1996); Pedersen J. & Burcharth H.F. (1992);

Sources of uncertainties in failure equations / input parameters:

Remarks:

Status of Draft

Date	Revision	Prepared by	Organisation	Approved by	Notes
		FB??	HRW	WA / MM	
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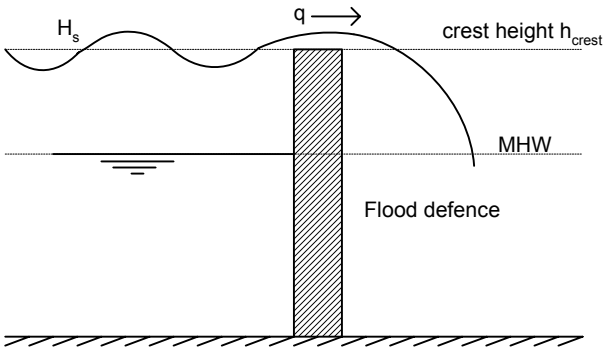
Da 2.5 Crest level too low – wave overtopping

Summary: Damage to stability of structure when wave overtopping exceed limit for the structure or flooding of the area behind the structure in case of large volumes of overtopping water. The "load" is the actual wave overtopping rate, q. The "strength" is a critical discharge, qc, at which the resistance of the structure is not exceeded or at which the maximum storage capacity behind the flood defence is not exceeded.



Limit state equation:
Overtopping is allowed as long as the stability of the structure is not endangered and as long as there is enough storage capacity behind the structure to store the overtopping volume water. Failure occurs if the wave-overtopping rate exceeds the admissible discharge rate over the defence or if the overtopping affects the stability of the defence. The "load" is the actual wave-overtopping rate; the "strength" is the highest discharge rate at which the maximum storage capacity behind the flood defence is not exceeded (q_{adm}). The limit state function is expressed by:

$$z = q_{adm} - q$$

where:

q_{adm} = admissible wave overtopping rate [l/(s·m)]
q = actual wave overtopping rate [l/(s·m)]

Loading equations:
The basic formula for the overtopping rate over vertical structures is as follows:

$$q = 0.13\sqrt{gH_s^3}\exp\left(-3.0\frac{h_{crest}}{H_s}\frac{1}{\gamma_\beta\gamma_n}\right)$$

In this formula the force due to wave breaking in front of the structure is not taken in account.

The maximum wave overtopping occurs with perpendicular waves. If the significant waves approach the structure under an angle, an influence factor can be used:

$\gamma_\beta = 1$ for $\beta \le 20^\circ$

$\gamma_\beta = \cos(\beta - 20^\circ)$ for $\beta > 20^\circ$ with a minimum of $\gamma_b = 0.7$

The wind can also have an influence on the wave overtopping. If a high overtopping rate is allowed, the influence of the wind can be neglected. (This is the case with most structures) However, for limited overtopping

Resistance (strength) equations:
Admissible wave overtopping rate for example $q_{adm} = 10\ l/s/m$
or maximum storage capacity behind the flood defence:
$q_{adm} = V_B / t$
where:

$V_B = K / B$
$K = A \cdot h_{wlr}$

rates the wind can play a role on the overtopping rate. An example with coupures. If q > 10 l/s per m, the wind has no influence on the overtopping rate If q < 1 l/s per m, the overtopping rate has to be enlarged with a factor 3 For situations in between an interpolation can be done. A flood defence with a protruding shape can reduce the wave overtopping considerably. The protrusion needs minimal dimensions to receive a reduction: $\gamma_n = 1.0$ if $h_{crest}/H_s \leq 0.5$ and $\gamma_n = 0.7$ if $h_{crest}/H_s > 1$ If a protrusion is absent $\gamma_n = 1$.	

Parameter definitions:

q	=	mean discharge due to overtopping [m^3/s/m]
g	=	gravitational acceleration [m/s^2]
H_s	=	significant wave height at the structure [m]
h_{crest}	=	crest height above SWL [m]
γ_β	=	influence factor for oblique wave attack
γ_n	=	influence factor for the shape of the structure
V_B	=	the maximum volume of water during a high water period per unit of width of the structure [m^3/m]
t	=	period of constant loading [s]
K	=	storage capacity [m^3]
B	=	total width of the structure [m]
A	=	surface behind the gate [m^2]
h_{wlr}	=	allowable water level rise [m]

Sources of failure mechanism equations / methods:
TAW (2003) "Leidraad Kunstwerken"

Sources of uncertainties in failure equations / input parameters:

Remarks:

Status of Draft

Date	Revision	Prepared by	Organisation	Approved by	Notes
			TAW		
		AK, PV			
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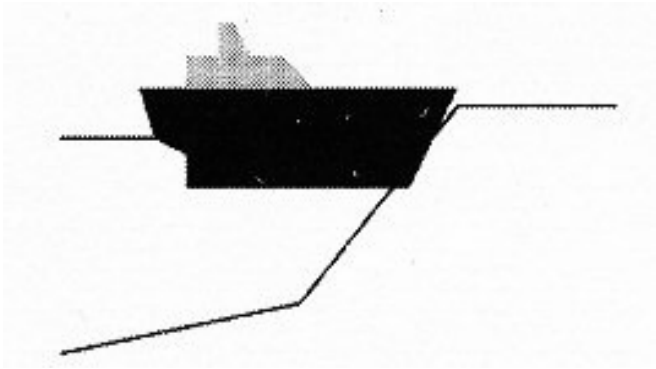
Da 4.1 Ship impact

Summary: Local failure of the flood defence occurs under conditions of ship impacts onto the outer face of the structure. The collision loads can be derived from an empirical formula. This failure mechanism is especially important for structures like sluices.

Sketch of failure mechanism:



Limit state equation:

The probability of failure of the flood defence due to a ship collision is a combination of three components:

- Distribution of the exceedence probability of collision loads.
- The effect of a certain collision load on the water defence.
- The probability of flooding when a collision has affected the flood defence.

A limit state function for the collision can be described with the following expression:

$$z = F_r - \sum F$$

where:

F_r = overall strength of structure [kN]
ΣF = total of the loads[kN]

Loading equations:

The total of the loads is the sum of the normal loads on the structure and extra ship collision force in kN.

For calculating vessel impact on bridge piers AASHTO (1998) gives a formula, where the maximum impact force on the pier F_v, is based on the dead-weight tonnage of the vessel, DWT and the vessel velocity:

$$F_v(\text{kips}) = 8.15u\sqrt{DWT}$$

This approach is equivalent to the one-degree-of-freedom model which is set up for storm debris.

Resistance (strength) equations:

The resistance of the structure is depending on the strength of the different parts of the structure, especially the gates. The strength is also given in kN.

Parameter definitions:

F_v = the maximum impact force on the pier [kN]
u = the vessel velocity [m/s]
DWT = dead-weight tonnage of the vessel [long tons]

Sources of failure mechanism equations / methods: TAW (2003)'Leidraad Kunstwerken'; Haehnel, R.B.; Daly, S.F. (2004)
Sources of uncertainties in failure equations / input parameters: This approach is only valid if both inertia and the stiffness of the structure are large enough that the structure itself does not move in any appreciable amount in response to the impact. Curbach and Proske (2003, 2004); Curbach M., Proske D.(2004); Proske D., Curbach M. (2003)
Remarks:

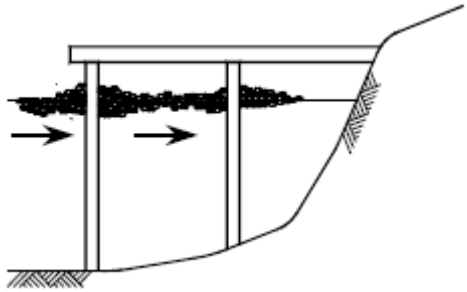
Status of Draft

Date	Revision	Prepared by	Organisation	Approved by	Notes
		PvG	TUD		
07/07/06		PV			drafted
25/01/ 07	v3_4_p03		LWI		edited

Da 4.2a Ice accumulation

Summary: Extra forces on a structure are created due to ice accumulation against a structure.

Sketch of failure mechanism:



Limit state equation:

Ice accumulation

As a result of a slight current, ice can accumulate against a structure. A current beneath the ice causes forces in the ice. Due to the accumulation of ice against the structure in slowly flowing water, a horizontal load is created.

The force is limited to the magnitude of the force that is necessary to fail the ice sheet in crushing, bending, buckling, splitting, or a combination of these modes. The highest effective pressure on a structure can be expected from ice crushing. Therefore only the ice crushing failure modes are described. This mode is depending on the indentation speed and the aspect ratio (D/h).

The limit state function can be described with the following expression:

$$z = F_r - \sum F$$

where:

F_r = overall strength of structure [kN]

ΣF = total of the loads[kN]

Loading equations:

The total of the loads is the sum of the normal loads on the structure and extra ice force in kN.

The ice force F on bridge piers resulting from moving ice, can be estimated with CSA (2000) and Aashto (1994) resulting in the following equation:

$$F = \min \begin{cases} F_c \\ F_b \end{cases} \quad \text{for } D/h < 6$$

and

$$F = F_c \quad \text{for } D/h > 6$$

where:

$$F_c = C_a pDh$$

$$F_b = C_n ph^2$$

with:

$$C_a = \left(5 \cdot h/D + 0.1\right)^{0.5}$$

Resistance (strength) equations:

The resistance of the structure is depending on the strength of the different parts of the structure, especially the gates. The strength is also given in kN.

$C_n = 0.5 \cdot \tan(\alpha + 15°)$	

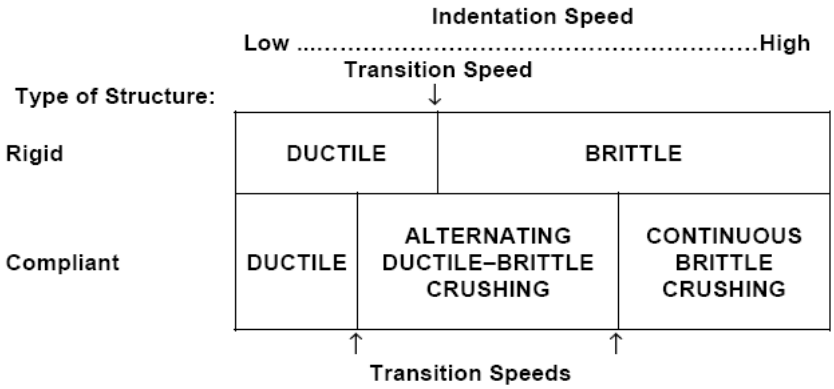
Parameter definitions:

ΣF = the sum of the forces [kN]
F = the sum of the ice loads [kN]
F_c = horizontal force when ice floes fail by crushing over full width of the pier [kN]
F_b = horizontal force when ice floes fail in bending against a sloping pier [kN]
D = the pier width [m]
h = ice thickness [m]
F_c = horizontal force when ice floes fail by crushing over full width of the pier [kN]
α = slope of the pier from the downstream horizontal (< 75°)
p = effective ice crushing pressure for which following values have been recommended.

0.7 MPa (101.5 psi)	*Ice breaks up at melting temperature and is somewhat disintegrated.*
1.1 MPa (159.5 psi)	*Ice breaks up or moves at melting temperature, but the ice moves in large floes and is internally sound.*
1.5 MPa (217.5 psi)	*Ice breaks up or moves at temperatures considerably below its melting point. Even higher pressures are recommended for ice temperatures 2 or 3 °C below melting temperature.*

Sources of failure mechanism equations / methods:
USACE (2002)

Sources of uncertainties in failure equations / input parameters:
The effective pressure depends also on the mode of ice crushing, which in turn depends on the rate of indentation, or the relative speed of an ice feature with respect to a structure. In the figure below an impression is given of the different crushing modes.



The highest loading occurs at the transition speed, where ductile deformation as well as brittle crushing occurs. This type of loading acts like a dynamic load.

Remarks:

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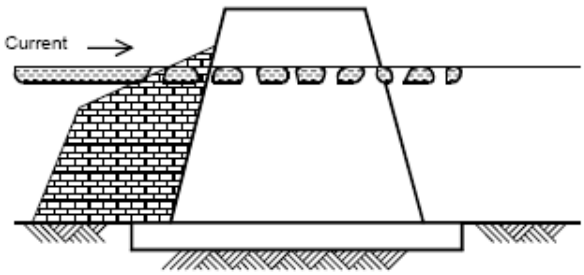
		PvG	TUD		
07/07/06		PV			drafted
25/01/ 07	v3_4_p03		LWI		edited

Da 4.2b Ice collision

Summary: Failure due to ice collision against the structure.

Sketch of failure mechanism:



Collision with ice

Limit state equation:

The dynamic horizontal load on a structure is caused by colliding blocks of ice, which are carried along by the wind and the current. This is particularly of importance for structures in rivers and along coasts with a considerable current. The impact of large ice sheets that on structures can be compared with the impact of ship collisions.

The force is limited to the magnitude of the force that is necessary to fail the ice sheet in crushing, bending, buckling, splitting, or a combination of these modes. The highest effective pressure on a structure can be expected from ice crushing. Therefore only the ice crushing failure modes are described. This mode is depending on the indentation speed and the aspect ratio (D/h).

The limit state equation is expessed by:

$$z = F_r - \sum F$$

where:

F_r = overall strength of structure [kN]

ΣF = total of the loads[kN]

Loading equations:

The total of the loads is the sum of the normal loads on the structure and extra loads due to the ice collisions in kN.

The extra load due to the ice collisions can be calculated with the following formula:

$$F = p_e \cdot A$$

For a head-on collision in which a moving ice comes to stop against a structure, the initial kinetic energy is dissipated to crush a certain volume V of ice at an effective pressure p_e is given:

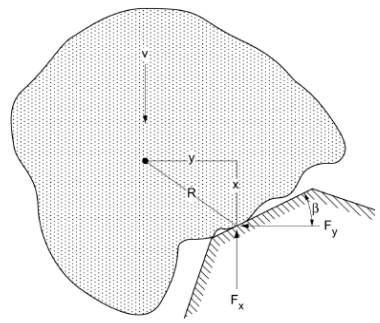
$$\frac{Mv^2}{2} = p_e \cdot V$$

An eccentric impact will rotate the ice floe, and the ice feature will retain a portion of initial energy after the impact.

$$\frac{Mv^2}{2} = \frac{Mv^2}{2}\left[\frac{(Y/R)^2}{1+(R_g/R)}\right] + (1+\mu\tan\beta)p_e V$$

Resistance (strength) equations:

The resistance of the structure is depending on the strength of the different parts of the structure, especially the gates. The strength is also given in kN.



Parameter definitions:

ΣF	=	the sum of the forces [kN]
F	=	the ice collision load [kN]
p_e	=	the effective pressure [kN/m]
A	=	area of ice crushing [m^2]
M	=	the mass of the ice feature [kg]
v	=	the velocity of the ice feature [m/s]
V	=	the estimated volume of the crushed ice [m^3]
Y	=	eccentricity of the center of gravity from the point of impact [m]
R	=	distance to the center of gravity from the point of impact [m]
R_g	=	radius of gyration of the ice feaure about the vertical axis through its center of gravity [m]
μ	=	ratio of tangential force to normal force in the contact area [-]
β	=	ratio of local geometry ($\beta = 0$ for head-on impact) [-]

Sources of failure mechanism equations / methods:

USACE (2002)

Sources of uncertainties in failure equations / input parameters:

Remarks:

Status of Draft

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		PvG	TUD		
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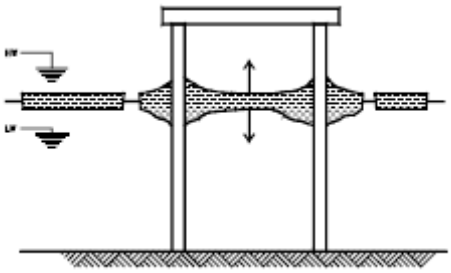
Da 4.2c Ice attachment

Summary: Marine structures that become frozen into an ice sheet are subjected to vertical ice forces as the ice sheet responds to changes in water level. Typically, the uplifting load resulting from changes in water level governs the design of light-duty, pile-founded docks common in marinas.

Sketch of failure mechanism:



Ice attachment

Limit state equation:

The static vertical load on a structure as a result of the attachment of ice onto a structure is of particular importance when water levels vary. Ice attachment can occur anywhere between the high and the low water levels. Under water this mass causes an upward force and above water it results in a downward load on the structure.

The limit state equation is expessed by:

$$z = F_r - \sum F$$

where:

F_r = overall strength of structure [kN]

ΣF = total of the loads[kN]

Loading equations:

The total of the loads is the sum of the normal loads on the structure and extra loads due to the ice in kN. Theoretical estimates of vertical ice loads depend on the assumed mode of ice failure, which is often difficult to ascertain for a particular situation.

Resistance (strength) equations:

The resistance of the structure is depending on the strength of the different parts of the structure, especially the gates. The strength is also given in kN.

Parameter definitions:

Sources of failure mechanism equations / methods:

USACE (2002)

Sources of uncertainties in failure equations / input parameters:

Remarks:

Zabilansky (1998) plotted the shear stress with respect to the ratio of pile diameters thickness and defined a best fit formulation:

$$\sigma = 300/(d/h)^{0.6}$$

σ = shear stress [kPa]

d = diameter of the pile [m]

h = ice thickness [m]

The equation for the pull out force is gives as:
$P = \sigma \pi d h$
P = pull out force [kN]

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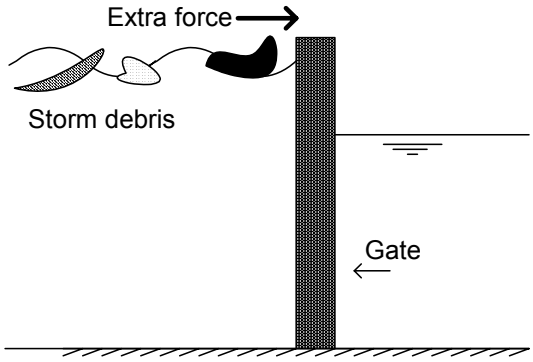
Da 4.3 Storm debris

Summary: Local failure of the flood defence occurs under conditions of storm debris impacts on the structure.

Sketch of failure mechanism:



Limit state equation:
Storm debris could cause a higher load at the gates. The structure has to be designed for these extra loads.
The limit state equation is expessed by:

$$z = F_r - \sum F$$

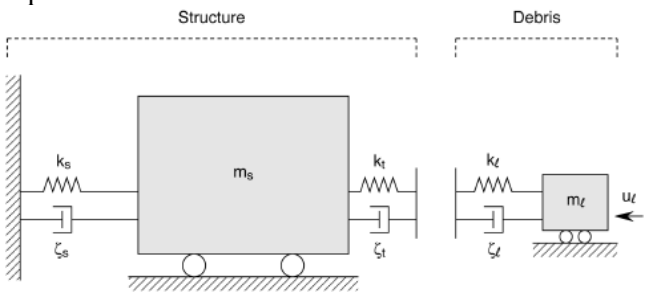
where:

F_r = overall strength of structure [kN]
ΣF = total of the loads[kN]

Loading equations:
The total of the loads is the sum of the normal loads on the structure and extra loads due to the storm debris in kN.
The extra loads due to the storm debris can be derived from a one-degree of freedom model if the structure can be considered to be rigid:

$$F_{i,max} = u_l \sqrt{\hat{k}(m_l + Cm_f)}$$

In this formulation the effect of damping was neglected, because it was assumed that the collision occurs over a very short duration.
A picture of the model is shown below:



Resistance equations:
The resistance of the structure is depending on the strength of the different parts of the structure, especially the gates. The strength is also given in kN.

Parameter definitions:

ΣF = the sum of the forces [kN]

$F_{i,max}$	=	the maximum impact force [kN]
u_l	=	velocity of the storm debris [m/s]
$\hat{k}$	=	the effective contact stiffness of the collision [kg/s^2]
m_l	=	the mass of the storm debris [kg]
C	=	the added mass coefficient [-] The value of C depends on may variables, including the object's geometry, its degree of submergence, its orientation with respect to the direction of acceleration and its natural vibration frequency.
m_f	=	the mass of the fluid displaced by the object [kg]

Sources of failure mechanism equations / methods:
TAW (2003) 'Leidraad Kunstwerken'; Haehnel, R.B.; Daly, S.F. (2004)

Sources of uncertainties in failure equations / input parameters:
For the determination of the maximum debris force the contact-stiffness approach is chosen. Other approaches are possible, like the impulse-momentum approach and the work-energy approach. These approaches are equivalent to the contact-stiffness approach. For a description of the approaches and a derivation of the formula, see Haehnel, Maximum impact force of woody debris on floodplain structures.
Laboratory tests (Haehnel 2004) demonstrated that the construction material of the impacted face of the rigid structure (wood, steel, or concrete) has no effect, in itself, on the maximum impact force.
In some cases the structure is designed to restrain ship collisions. In that case the ship collisions loads are probably more normative than the storm debris loads. If the structure is not endangered by ship collisions or the structure is not designed to restrain ship collisions, the structure has to be checked for storm debris.
In case of a sluice, debris could hinder the functioning of the gates. This results in a higher failure probability.

Remarks:

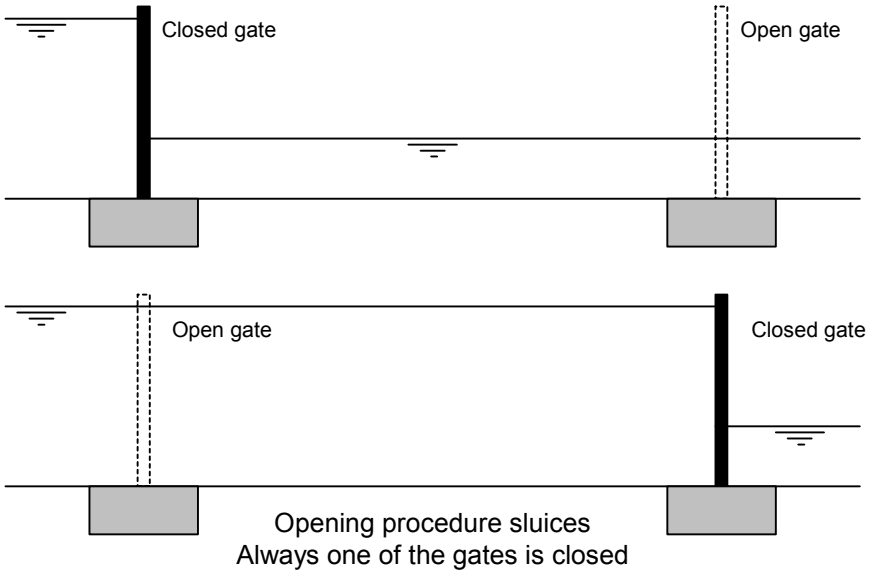
Status of Draft

Date	Revision	Prepared by	Organisation	Approved by	Notes
		PvG	TUD		
07/07/06		PV			drafted
25/01/ 07	v3_4_p03		LWI		edited

Da 5.1a Closing failure mechanisms double gated sluices

Summary: The failure probability of the closing process in double gate sluices is very small, because the probability that the gate is open when the water levels are too high, is very small.

Sketch of failure mechanism:



Limit state equation:
Analysis of the failure probability of the function as flood defence in case of non-closure starts with an inventory of the intended use. For example navigation locks and inlet sluices always have one gate closed. The sluice is in this way nearly always closed. In the case that all the gates of these types of sluices are opened, non-closure of the gates has to be monitored. However the probability that all the sluice gates are open and the outer water level is higher than the posterior quays is small.
For a guard lock or a storm surge barrier this probability is around 1.0, because the gates of these structures are almost always open.

Loading equations:	**Resistance equations:**

Sources of failure mechanism equations / methods:
TAW (2003)

Sources of uncertainties in failure equations / input parameters:
The failure of the closing operation is mainly depending on human failure. More information about this subject can be found in Heart (1988) and Kirwan (1994).

Remarks:

Status of Draft

Date	Revision	Prepared by	Organisation	Approved by	Notes
		PvG	TUD		
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Da 5.1b Closing failure mechanisms single gated structures

Summary: The closing failure of a gate in a single gated structure can be of major importance in case the closing procedures of the structure are not established properly. The probability of failure due to non-closing can be determined by taking into account the frequency of exceedance of the structure's critical water level and four procedures that enhance a reliable closure of the structure

Limit state equation:
A limit state equation cannot be established in great detail due to all the human factors involved. In general we could state that the structure fails in case the volume of water flowing in during a non-closure is higher than the allowable volume (there is usually a storage capacity in the system; the amount of water flowing through a structure is in most cases limited):

$$Z = V_{open} - V_{allow} \quad (1)$$

The probability of failure can be defined as:

$$P_{fa} = P\left\{V_{open} > V_{allow}\right\} \quad (2)$$

In the Netherlands, this probability of failure has to be smaller than 10% of the area's safety standard:

$$P_{fa} = P\left\{V_{open} > V_{allow}\right\} < 0.1 \cdot \text{norm} \quad (3)$$

The determination of all volumes flowing through, integrated over the frequency of occurrence of the water levels is rather complicated. Instead, the following approach is used to determine P_{fa}. This approach is based on the allowable volumes.

$$P_{fa} = P\left\{V_{open} > V_{allow}\right\} = P_{nc} \cdot N\left\{V_{open} > V_{allow} \mid nc\right\} = P_{nc} \cdot n_j \quad (4)$$

The determination of P_{nc} and n_j is elaborated below.

Loading equations:
n_j can be regarded as the main loading variable; n_j is a function of:

- Open retaining height (ORH)
- Frequency of exceedance water level curve

The Open Retaining Height (ORH) is an important parameter in the reliability assessment. The ORH is the critical water level for an opened structure. The OHR depends on the composition of the system (i.e. storage allows for more water flowing in and thus a higher ORH) and on the functions of the systems (i.e. dewatering or high

Resistance equations:
P_{nc} is the probability of non-closure of the structure; hence it can be regard as the resistance equation. There are 4 aspects that influence P_{nc}:

1. High water warning system
2. Procedure for mobilisation people
3. Closing procedure of the structure
4. Reliability of the gates

P_{nc} can be determined with a simple and with an advanced method

Simple determination P_{nc}:

level retention). Figure 1 shows the relation between the different water levels. The other important parameter is the frequency exceedance curve of the local water levels, which also highly depends on local circumstances.

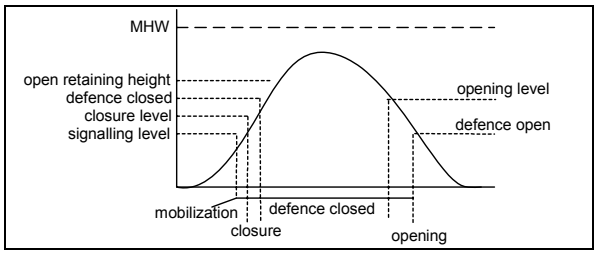

Figure 1: Different relevant water levels during a floodwave

n_j can be determined by a simple method and by an advanced method

Simple determination n_j:
The simple determination of n_j involves a simple determination of the ORH. The simple determination results in a conservative approximation. In case the structure is not considered reliable enough, the advanced determination can be used. The determination is based on the assumption that P_{nc} is maximally 10^{-3} (see *Resistance equations* section) and n_j is maximally 1/10 per year. A simple determination of the ORH can be:

- The closure level
- The retaining height minus 0.3 meter.

If the exceedance frequency of one of these 'simple' ORHs is less than 1/10 per year, and if the simple P_{nc} are fulfilled, the structure is considered reliable.

Advanced determination n_j:
In case the structure does not pass the simple determination, a more advanced method can be applied. The advanced determination of n_j involves the following steps:

1. Determine the functions of the structure and the different possible states during the functions
2. Determine the ORH per state
3. Determine the frequency of exceedance of the ORHs
4. Determine the probability of non closure per state (see *Resistance equations* section)
5. Check whether the norm is fulfilled with

In case a list of conditions for all four aspects is fulfilled, as well as the simple conditions for n_j, no additional research is needed and the structure is considered reliable. These conditions, for instance, involve the presence of multiple gates and the presence of two independent high water warning systems. For more information is referred to TAW (2003).

Advanced determination P_{nc}:
In case of the more advanced method, the probability of failure can be approximated with:

$$P_{nc} = 10^{-E} \qquad (5)$$

Where P_{nc} = Probability of non-closure [1/year]
E = lowest value of E_i (i = 1 to 4)
E_i = score according to a questionnaire, based on the structure for each of the 4 aspects mentioned above.

For more detailed information about the questionnaire is referred to TAW (2003).
.

equation (4) The determination of the functions and of the ORH strongly depend on the type of the structure and local circumstances. An overview of the possibilities is presented in TAW (2003).	

Parameter definitions:

V_{open} = Expected volume flowing through
V_{allow} = Allowable volume flowing in
P_{fa} = Probability of failure due to non-closure [1/year]
norm = Area's safety standard (in the Netherlands an probability of exceedance of 1/1.250, 1/2.000, 1/4.000 of 1/10.000 per year)
P_{nc} = Probability of non-closure [1/demand]
n_j = $N\{V_{open}>V_{allow}|nc\}$ = frequency of exceedance of the allowable volume flowing through an opened structure [demand/year]

Sources of failure mechanism equations / methods:
TAW (2003), Technical Advisory Committe on Flood Defences, '*Leidraad Kunstwerken*' (*Guide Hydraulic Structures*), published at www.tawinfo.nl

Sources of uncertainties in failure equations / input parameters:
In the loading part, the main source of uncertainty is the water level. The exceedance frequence of the water level that is critical for a structure, however, depends on the structure's functions and local circumstances.

In the resistance part, the main sources of uncertainty are divided in four groups that are all related to the closing procedure: the high water warning system, the mobilisation procedure of the persons involved, the closing procedure and the reliability of the gates. These uncertainties are best qualified on the basis of a qualitative method.

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5. Gaps in Knowledge and Methods

The information contained in the failure mode templates of this report has been drawn from a wide range of sources and expertise. In undergoing this process it has become clear that:

- The wide range of processes and scientific disciplines that they cover make it very difficult to find experts who are able to advise on all failure modes. Content has to be drawn and merged across disciplines
- There tends to be different 'standard' methods applied both in different scientific communities (e.g. costal, fluvial, hydraulic, geotechnical) as well as in different nations

Logically, the summary matrix (Section 3) highlights areas where knowledge and methods are not clear, as shown by the red shading. In practice, some of these areas may well be addressed by logical derivation from other similar failure / loading conditions or by finding the appropriate existing expertise to advise. The aim of this report is to provide a definitive record of asset failure modes which may form the basis of a living document which can be updated as more knowledge is aquired.

However, development of this document did highlight some areas which show as clear gaps in knowledge. The three main areas were:

- Transitions
- Time dependent processes
- Point structures

5.1 Transitions

Transitions between asset types can be a weak point within a flood defence system. Experience from the floods in New Orleans has confirmed this. A very high percentage of asset failures occurred at transition points.

Research and guidance is therefore required to a) Understand the processes that occur during failure at a transition and b) to provide practical design guidance to avoid the continued construction of defences with such weak points.

5.2 Point Structures

Point structures come in many varying designs and sizes. They invariably entail transitions between asset types and can contain a large number of ways in which performance of the flood defence system may be compromised. Information on point structure performance appears difficult to collate, is therefore an area requiring further attention and maybe an area where further RTD is required to establish real performance issues.

5.3 Time Dependent Processes

Present capabilities for system modelling typically do not allow for time varying processes. For example, how asset condition may deteriorate during continued hydraulic loading or over longer periods of time (i.e. intra storm loading; longer term deterioration). Equally, system models often assume failure once part of the asset is deemed to have failed. For example, once grass cover has eroded during embankment overtopping, rather than considering the real evolution of a breach through the embankment. A longer discussion of these points is included under Section 2.4.

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The following references have been compiled as the various failure mode templates have been developed. Given the nature of this report, the templates have been and will continue to be updated as new approaches are discovered or developed. As such, references are in a continual process of being updated. Any [references] presented within brackets are still within the review process, with the expectation of being confirmed / updated in later versions or editions of this document.

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8. Notation

The notation list below (Ver 5.0) is drawn from technical content developed within the failure mode templates. Given that the templates draw on science and processes from a range of technical disciplines (e.g. Hydraulics, structures, soil mechanics etc.) there is invariably duplication in the use symbols. No attempt has been made to refine the use of symbols – rather to list all, with summaries of template specific symbols also given within each template.

a	Slope of the pier from the downstream horizontal < 75°	°
A	Acceleration	m/s^2
A	Adhesion	N/m^2
A	Area	m^2
A	Area of ice crushing	m^2
A	Coefficient used in various empirical formulae	
A	Empirical factor according to Larson et al. 2004, A = 1.34.10-2	-
Ac	Area of canal's cross section	m^2
A_e	Erosion area on rock profile	m^2
A_o	Amplitude of horizontal wave motion at bed	m
a_s	Flow direction	°
A_s	Area of ship's cross section	m^2
A_t	Area of structure cross section	m^2
b	Coefficient used in various empirical formulae	-
b	Empirical factor according to Larson et al. 2004, b = 3.19.10-4	-
b	Location parameter for the river discharge Q	-
b	Ratio of local geometry b = 0 for head-on impact	-
b	Width of segment, element or slice	m
B	Channel width	m
B	Crest width	m
B	Structure width at crest, in horizontal direction normal to face	m
B_a	Width of armour berm at crest	m
B_B	Berm width	m
BL_c	Blockiness, volume of block divided by volume of enclosing XYZ orthogonal	-
B_n	Bulk number cross-section of stones	-
Bu	Buoyant up-lift force at static water level	kN/m
B_w	Structure width, often at toe level	m
B_{wl}	Structure width at still water level	m
c	Factor representing the erosion sensitivity of the sand layer	-
C	Chezy coefficient	$m^{1/2}/s$
C	Propagation celerity of waves	m/s
C, c'	Drained cohesion of soil	N/m^2
C_c	Compression index	-
cE	Grass quality after Verheij et al., 1998	m.s-1
cg	Coefficient that represents the erosion endurance of the grass.	m·s
C_k	Creep coefficient	-
C_{pl}	Plunging wave stability coefficient, Van der Meer equations	-
cr	Crest retreat caused by the storm	m
C_r	Coefficient of wave reflection	-
c_{RB}	Factor representing the erosion sensitivity of the dike body	-
c_{RK}	Factor representing the erosion sensitivity of the clay cover	-
C_s	Empirical coefficient according to Larson et al. 2004	-
C_s	Recompression index	-
C_{su}	Surging wave stability coefficient, Van der Meer equations	-
c_T	turbulence coefficient	-

C_t	Coefficient of wave transmission	-
C_u	Undrained cohesion of soil	N/m^2
c_V	Critical flow velocity	-
c_V	Dissipation coefficient	-
C_v	Consolidation coefficient	m²/s
c_w	Cohesion due to root penetration	kPa
d	Depth	m
d	Inclination of surcharge	°
d	Structure crest height relative to bed level breakwaters, dams,..	m
d	Thickness of certain layer, element	m
d	Thickness or minimum axial breadth given by the minimum distance between two parallel straight lines between which an armour block can just pass	m
D	Particle size, or typical dimension	m
D	water depth on top of the top layer of the sill	m
D_*	Non-dimensional sediment grain diameter	-
D'	Basket or mattress thickness	m
D_{15}	15% value of sieve curve	m
D_{50}	Sieve diameter, diameter of stone which exceeds the 50% value of sieve curve	m
d_{70}	70%-pass grain diameter	m
D_{85}	85% value of sieve curve	m
D_{85}/D_{15}	Armour grading parameter	-
d_a	Depth of the bottom of the anchor head	m
D_{c15}	Diameter of particle fraction of coarse material	mm
d_{ca}	Difference of level between crown wall and armour crest, $d_{ca} = R_c - R_{ca}$	m
D_{eq}	Equivalent thickness of the top layer	m
D_f	Degree of fissuration	-
D_{f15}	15% non-exceedance diameter of filter layer from grading curve, indicating permeability of the filter	m
D_{f85}	Diameter of particle fraction of fine material	mm
D_h	Difference between local water depth in front of dike and water level in the floodplain	m
D_h	Existing pressure head gradient	-
D_I	Indicative grain size diameter	m
d_k	Thickness of remaining clay layer	m
d_{loc}	localised scour along the toe structure at outer bend	m
D_n	Nominal block diameter, $D_n= M/\rho_r^{1/3}$	m
D_{n50}	Nominal mean diameter, $D_{n50}= M_{50}/\rho_r^{1/3}$	m
d_o	The water level in front of or at upstream of the dyke	m
d_r	Depth of gap	m
D_r	Required characteristic size of the protection element corresponding to the actual boundary condition	m
D_s	Size of the equivalent volume sphere	m
d_{scour}	Depth of scour at toe of protection	m
dw	Depth of the grass roots	m
dw	Width of the dune	m
D_z	Block size corresponding to sieve size z	m
Dz	Particle size corresponding to sieve size "z"	mm
E	Young's Modulus	N/m²
e, e_o	Void ratio, e=n/1-n and initial void ratio	-
e'_{bu}	Ultimate strain of the concrete	-
e'pl	Plasticity strain of the concrete	-

E_d	Design value of the effect of actions	Unit of the parameter
E_d	Energy absorbed or dissipated	N/m
E_g	Velocity of erosion of the grass revetment	m/s
E_i	Incident wave energy	N/m
E_k	Characteristic value of the effect of actions	Unit of the parameter
E_r	Reflected wave energy	N/m
es	Fraction of air pore	-
E_t	Transmitted wave energy	N/m
$E_{\eta\eta}$	Energy density of a wave spectrum	m²s
f	Frequency of waves = 1/T	s^{-1}
f	Stability coefficient, general, mainly dependent on structure type, tanα and friction	-
F	Factor of safety geotechnical, defined as ultimate resistance/required resistance	-
F	Fetch length	m
F	Ice collision load	kN
F^*	Non-dimensional freeboard parameter $F^*=R_c/H_s^2s/2\pi^{1/2}$	-
f'_b	Cubic pressure strength of the concrete	kN/m2
f_A	Factor for mean force due to wave impact	-
fb	Cubic tensile strength of the concrete	kN/m2
f_c	Friction factor for current	-
F_d	Design value of an action	Unit of the parameter
fg	Quality factor for grass revetment	-
f_G	Factor for force due to mass of soil	-
F_h	Horizontal force due to wave load	kN/m
F_H	Horizontal force on caisson or crown wall element	N/m
f_i	Stability increase factor for armourlayers with stepped or bermed slopes	-
F_k	Characteristic value of an action	Unit of the parameter
f_p	'Peak' frequency of wave spectrum	s^{-1}
f_{pmax}	Factor for pmax	-
Fr	Friction force	kN/m
Fr	Froude number, $Fr=U/gh^{1/2}$	-
Fr	Overall strength of structure	kN
f_s	Yield stress of the steel, net of any factoring	kN/m^2
F_U	Uplift force on caisson or crown wall element	N/m
f_w	Wave friction factor	-
g	Gravitational acceleration, 9.81 m/s^2	m/s²
g	Reduction factor; g = gf gb , taking into account the effects of oblique wave attack.	-
G	Mass force of a certain element, segment	kN/m
G	Shear modulus	N/m²
g'	Saturated volume weight of soil	kN/m^3
gb	Influence factor for oblique wave attack	-
gc, gs	Partial safety factors	-
gd	Model factor	-
gd	Volumetric weight of the dry soil	kN/m^3
g_f	Roughness of the seaward slope	-
g_{f-c}	Roughness at the crest	-
g_{fg}	Unit weight of the fine grained natural soil beneath the embankment saturated	kg/m^3
g_G	Velocity coefficient	-

G_i	Mass force of slice	-
g_s	Volumetric weight of saturated soil	kN/m^3
g_{sat}	Unit weight of the saturated part of the embankment	kg/m^3
g_{unsat}	Unit weight of the unsaturated part of the embankment	kg/m^3
g_w	Groundwater level	m
g_w	Volumetric weight of water	kN/m^3
h	Height of a element, segment	m
h	Ice thickness	m
h	River water level	m
h	Water depth	m
H	Actual wave height in front of structure due to wind or ship induced waves	m
H	Incident regular wave height	m
H	Wave height, from through to crest	m
h´	Water depth at the toe including the coverlayer	m
$H_{1/10}$	Mean height of highest 1/10 fraction of waves	m
$H_{1/3}$	Significant wave height based on time domain analysis, average of highest 1/3 of all wave heights	m
$H_{2\%}$	Wave height exceeded by 2% of waves	m
h_a	Height of the anchor head	m
H_{act}	Actual water level in front of ctructure	m
h_b	Water level in the floodplain / dike ring	m
h_B	Water depth above berm	m
$h_{B,0}$	Layer thickness at beginning of inner slope	m
h_c	Critical water level	m
h_c	Height difference between the still water level and the top of the wall	m
h_c	Water depth above structure crest	m
$h_{c,s}$	Crest height after the storm, with reference to the intersection between the water level and the beach slope, point 0,0	m
h_{cd}	Critical crest level of the dike	m
h_{crest}	Crest height above SWL	m
h_{crit}	Critical pressure head gradient for piping	-
h_{crit}	Critical water level difference between water level in front and behind dike	m
h_d	Crest level of the dike	m
h_E	Actual energy height of water overflowing the flood defence	m
$h_{E,\ adm}$	Admissible energy height of water calculated from critical overflow discharge	m
h_f	Depth of intersection point between original berm and reshaped berm	m
h_{gp}	Crest height of the dune which respects to SWL	m
H_i	Ship induced wave height	m
H_{m0}	Significant wave height calculated from the spectrum, $H_{mo}=4\sqrt{m_o}$	m
H_{max}	Maximum wave height in a record	m
H_o	Deep water wave height	m
H_o	Stability number, $H_o=N_s=H_s/\Delta D_{n50}$	-
$h_{o,k}$	Critical water level above crown height	m
H_oT_o	Dynamic stability number, $H_oT_o=N_{sd}=N_sT_mg/D_{n50}^{1/2}$	-
h_{over}	Depth of flow over local crest	m
h_{ow}	Maximum outer water level during a high water wave	m
h_p	Critical head difference according to Sellmeyer	m
$h_{pr\ t}$	Predicted water level with a lead-time t	m
H_r	Force capacity of the soil around the anchor head	kN
h_s	Water depth at a distance of l/2L or $5H_{max}$ seaward of structure toe	m
H_s	Significant wave height	m
$H_{s,crit}$	Critical significant wave height for a failure process	m
H_{sb}	Significant wave height during depth-limited breaking	m
H_{si}	Incident significant wave height	m

H_{so}	Deep water significant wave height	m
h_t	Water depth above transition in composite slope	m
h_t	Water depth at structure toe	m
i	hydraulic gradient	-
i	Hydraulic gradient of phreatic water level	-
I	Potential gradient: $I=\nabla p/\gamma$	-
i_b	Gradient of river bed	-
I_c	Continuity index	-
i_{cr}	Critical hydraulic gradient	-
I_D	Density index e_{max}-e/e_{max}-e	-
i_n	Transversal hydraulic gradient	-
i_p	Longitudinal hydraulic gradient	-
I_p	Plasticity index of soil	-
I_{s50}	Point load strength index	N/m²
i_w	Wind-induced gradient of still water surface	-
Iz	Moment of inertia of the sheet pile cross section	m^4/m
j	Internal friction angle	°
j	Shear strength of undrained soil	kN/m^2
j´	Effective angle of friction	°
k	Incident regular wave number	-
k	Permeability coefficient according to Darcy	m/s
k	Permeability of core material	m/s
k	Roughness factor by Strickler	m
k	Wave number, $k=2\pi/L$	m^{-1}
K	Modulus of compressibility	N/m²
K	Stability or velocity factor rock stability	-
K	Storage capacity	m^3
k*	Empirical factor , e.g. k* = 1.0, see Schüttrumpf 2001	-
K’	Velocity loading factor rock stability	-
K_a	Coefficient for active horizontal grain force	-
k_a, k_u	Permeability of armour or underlayer	
k_B	Influence factor for berm width	-
k_c	Modified layer coefficient for concrete armour units	
k_d	Slope reduction factor for critical bed shear stress on a slope normal to the flow direction	-
K_d	Diffraction coefficient	-
K_d	Diffraction coefficient	-
K_D	Stability coefficient, Hudson equation	-
k_d, k_f	Coefficients for consideration of the crest width Bk, and sharpcrestedness of the weir Rk	-
k_h	Horizontal seismic coefficient	-
k_h	Influence factor for berm level	-
k_h	velocity profile factor	-
k_h	Velocity profile factor	-
K_h	Depth parameter	-
k_l	Slope reduction factor for critical bed shear stress on a slope along the flow direction	-
k_p	Packing density for concrete armour units	
k_p	wave number associated with the spectral peak by linear wave theory	-
K_p	Coefficient for passive horizontal grain force	-
K_R	Refraction coefficient	-
k_r, k_r’	Factor, similar to k_d, but for wave run-up/down	-
k_s	Bed roughness	m
k_s	Rock fabric strength	-
k_s	Shape coefficient for concrete armour units	-

K_s	Slope parameter	-
K_S	Shoaling coefficient	-
k_{sl}	Slope reduction factor for critical bed shear stress, $k_{sl}=k_l.k_d$	-
k_t	Layer thickness coefficient	-
k_t	Turbulence amplification factor for current velocity	-
K_T	Turbulence factor	-
k_w	Wave amplification factor for bed shear stress	-
K_{wa}	Modulus of compressibility for water with air	N/m²
l	Length of horizontal sliding surface beneath embankment	m
l	Length of slide line	m
l	Width of apron	-
L	Characteristic response distance geotechnics	m
L	Effective span distance between the supports	m
L	Horizontal leakage length	m
L	Length of the concrete slab	m
L	Wave length, in the direction of propagation	m
L_b	Width of the dike body	m
L_B	Length of outer slope	m
lD	Length of seepage line	m
L_k	Seepage length	m
L_k	Width of the clay cover layer	m
LK_h	horizontal seepage length	m
Lk_{inside}	Width of the inside clay cover layer, that can be considered as the total width of the embankment.	-
LK_v	Vertical seepage length	m
L_{ms}, L_{ps}	Wave length at the structure of mean and peak period wave	m
L_o, L_0	Offshore or deep water wave length, $L_o=gT^2/2\pi$	m
L_{om}, L_m	Offshore or deep water wave length of mean period T_m	
L_{op}, L_p	Offshore or deep water wave length of peak period T_p	
lp	Length of seepage line	m
l_R	Effective width	m
L_s	Length of 'splash-area'	m
L_s	Length of the ship	m
L_s	Wave length in shallow water at structure toe	m
$L_{s,act}$	Actual width of the protection	m
lt	Partial length of the dike at the inner toe	-
m	Coefficient of friction between crown wall base and underlying material	-
m	Mean outer slope	-
m	Model factor	-
m	Ratio of tangential force to normal force in the contact area	-
m	Seabed slope gradient	-
M	Mass of an armour unit	kg
M	Mass of the ice feature	kg
M	Overturning moment	kNm/m
m_0	Zeroth moment of wave spectrum	m²s
M_{50}	Median mass of an armour unit	kg
M_A	Driving moment	kNm
MBT	Maximum available time for the closure process	s
M_d	Actually occurring moment exerted by the hydraulic and geotechnical loading	kNm
M_{em}	Effective mean mass	kg
mf	Mass of the fluid displaced by the object	kg
M_g	Dry mass of crown wall element [kg] or [t]	kg / t
m_h	Model uncertainty factor for damping	-
m_l	Mass of the storm debris	kg

M_{max}	Maximum bending moment in the sheet pile wall	kNm
m_n	n-th moment of spectrum	$m^{2+n}s$
m_v	Coefficient of volume change	m²/N
m_{ve}	Coefficient of volume change (elastic)	m²/N
M_y	Block mass where y percent of the total sample mass is lighter than M_y	
n	Kinematic viscosity of water	Pa.s
n	Mannings coefficient of bed roughness	$m^{1/3}/s$
n	Number of layers	-
n	Slope gradient 1:n	-
N	$nf \cdot D_{15f}/D_{50b}$, where nf = porosity of filter material	-
N	Normal force	kN
N	Number of waves over the duration T_r of a storm, record, or test, $N=T_r/T_m$	-
N_a	Total number of armour units in area considered	-
N_d	Number of armour units displaced in area considered	-
nj	N{Vopen > Vadm │nc} = frequency of exceeding of the admissible inflow volume with an opened flood defence	-
N_{od}	Number of displaced units per width D_n across armour face	-
N_{ow}	Number of overtopping waves	-
N_s	Stability number, $N_s=H_s/\Delta D_{n50}$	-
N_s^*	Spectral stability number, $N_s^*=N_sH_s/L_p^{-1/3}$	-
N_{sd}	Dynamic stability number, $N_{sd}=N_sT_mg/D_{n50}^{1/2}$	-
n_v	Volumetric fictitious porosity, volume of voids as proportion of total volume	-
n_v	Volumetric porosity	-
n_w	Number of waves until erosion of dike core is completed	-
n_X	Scale factor of parameter X, $n_X = X_p/X_m$	-
O, O_i	Opening size in geotextile, i%	-
p	Effective ice crushing pressure	kN/m^2
p	Net uniformly distributed pressure acting on the member in the case of the front wall, p is the arithmetic sum of the applied wave loading and the internal cell pressure	Mpa
p	Pore water pressure, or wave -induced pressure	N/m²
p	Porosity	
P	Mean force within gap	kN
P	Notional permeability factor, defined by van der Meer	-
P	Permeability parameter 0.1<P<0.6	-
p_a	Atmospheric pressure at sea level	N/m²
PA	Mean force due to impact	kN
pc	Retreat of the shingle beach crest	m
pe	Effective pressure	kN/m
P_f	Failure probability	-
p_i	Wave impact pressure	N/m^2
pm	Maximum pressure	kN/m^2
p_p	Wave pulsating pressure	N/m^2
P_t	Percentage of the time that wave overtopping takes place	-
p_u	Wave uplift pressure	N/m^2
P_{wh}	Horizontal component tof water pressure hydrostatic	kN/m^2
p_x	Probability density function of x	x^{-1}
P_x	Probability that x will not exceed a certain value or cumulative probability density of x	- or $year^{-1}$
q	Direction angle of segment	°
q	Mean overtopping rate	l/s.m
q	Specific discharge	$m^3/s.m$
q	Surcharge load behind the anchored sheet pile wall	kN/m^2
Q	Discharge	m^3/s

Q	External load surcharge	kN
Q	Mean overtopping discharge per metre run of crest	$m^3/s.m$
Q^*	Non-dimensional overtopping discharge, $q/gH_s^{31/2}.s/2\pi^{1/2}$	-
q_0	Actual wave overtopping discharge	$m^3/s.m$
q_0	Wave overtopping rate for crest freeboard Rc = 0 Schuttrumpf	l/s.m
q_B	Bedding layer	°
q_c	Critical overtopping discharge	l/s.m
q_{cv}	Critical overtopping discharge for saturation	m^2/s
qG	Grass quality between 0,0 and.1,0	-
q_i	Direction angle of the slice	°
Q_i	Volume of single erosion segment	m^3/m
q_M	Material quality 1,0 for Sand	-
r	Density of water	kg/m^3
r	Radius of slip circle	m
r	Reduction factor for oblique wave attack	-
r	Relative intensity of turbulence	-
R	Distance to the center of gravity from the point of impact	m
R	Force due to cohesion and soil reaction	kN
R	Hydraulic radius	m
R	Maximum bearable shear force of clay layer	kN/m^2
R	Shear strength of entire system	kN/m^2
R	Strength descriptor in probabilistic calculations	
R^*	Non-dimensional freeboard, $R^*=R_c/H_s\sqrt{s/2\pi}$	-
R’	Equivalent rock roughness	-
R'_d	Run-down level, due to ship-induced waves	m
R'_u	Run-up level, due to ship-induced waves	m
R_b	Dimensionless crest height	-
R_c	Crest freeboard relative to still water	m
R_{ca} / A_c	Crest freeboard relative to still water with regard to rock armour	m
R_d	Run-down level, relative to still water level	m
$R_{d2\%}$	Run-down level, below which only 2% pass	m
Re	Reynolds number, Re=UD/ν	-
Re^*	Reynolds number with regard to shear velocity U_*	-
Rec	Recession of berm	m
R_g	Radius of gyration of the ice feaure about the vertical axis through its center of gravity	m
$R_{i;d}$	Design value of a resistance	Unit of the parameter
$R_{i;k}$	Characteristic value of a resistance	Unit of the parameter
r_r	Area ratio of steel reinforcement with respect to the concrete cross-sectional area D	-
r_r	Density of cover layer elements	kg/m^3
r_s	Density of sand particle	t/m^3
R_u	Run-up level, relative to still water level	m
$R_{u2\%}$	Run-up level exceed by only 2% of run-up tongues	m
rz	Reduction factor without berm	-
rz*	Reduction factor with berm	-
s	Distance from the ship’s sailing line	m
s	Wave steepness, $s=H/L_o$	-
S	Actual shear force due to wave overtopping	kN/m^2
S	Force due to current velocity and weight	kN
S	Loading descriptor in probabilistic design	
S	Shear strength of soil	kN/m^2
S	Stiffness modulus of asphalt	kN/m^2

S’	Equivalent rock strength	N/m²
sb	maximum tensile stress in the sheet pile	kN/m^2
sB	Control variable inner slope	-
S_d	Non-dimensional damage, $S_d=A_e/D_{n50}^2$ calculated from mean profiles or separately for each profile line, then averaged	-
SF	Stability factor	-
s_m	Wave steepness for mean period wave, $s_m=2\pi H_s/gT_m^2$	-
Sm	Maximum scour depth at node L /4 from wall	m
s_p	Wave steepness for peak period wave, $s_p=2\pi H_s/gT_p^2$	-
S_r	Degree of saturation	-
Su	Undrained shear strength of the fine grained soil	kN/m^2
t	Period of constant loading	s
t	Time, variable	s
T	Actual horizontal force	kN/m
T	Shear resistance in gap	kN/m^2
T	Thickness	m
T	Tidal period	s
T	Typical geotechnical response period	s
T	Wave period	s
t_a, t_u, t_f	Thickness of armour and underlayer or filter layer in direction normal face	m
tan_o	Slope of the initial dune profile simplified	-
tan_s	Slope of the initial dune profile simplified	-
T_C	Required time for the closing phase	s
t_d	Actually occurring shear stress exerted by the hydraulic and geotechnical loading	N/mm^2
t_e	Overflow duration	h
t_e	Time for erosion of sand core over the erosion length lc	s
T_{el}	Time scale of elastic response	s
Ti	Shear resistance in gap	-
tl	Toe level of initial dune profile	m
T_m	Mean wave period	s
T_m	Mean wave period	s
T_M	Required time for the mobilization phase	s
$T_{m-1,0}$	Spectral wave period, also called the energetic wave period	s
$T_{m-1,0}$	Spectral wave period, $T_{m-1,0}=m_{-1}/m_0$	s
tmax	Maximum duration for erosion of grass layer	h
Tmax	Maximum bearable force	kN/m
To	Wave period parameter for dynamic stability number HoTo, $To=T_m g/D_{n50}^{1/2}$	
T_p	Spectral peak period, inverse of peak frequency	s
T_{Pf}	Return period	years
T_{ph}	Time scale of phreatic response	s
t_{PHL}	time for complete seepage through the dike	h
T_{pl}	Time scale of plastic response	s
Tr	Root tensile strength	kN/m^3
T_R	Duration of wave record, test or sea state	
t_{RB}	Time needed to damage dike body	s
t_{RK}	Time needed to damage clay cover	s
t_{RT}	Time needed to damage grass revetment	s
u	Maximum bearable overtopping velocity for existing rubble mound structure at crest	m/s
u	Pore water pressure at slice	kN/m^2
u	Vessel velocity	m/s
u	Water pore pressure at segment	kN/m^2
U	Horizontal depth-mean current velocity	m/s
	Horizontal cross-sectional mean current velocity in rivers	

u_*	Shear velocity, $u_* = \tau_b/\rho$	m/s
u, v, w	Local velocities, usually defined in x, y, z directions	m/s
u'	Fluctuating velocity component	m/s
u0	wave run-up velocity at still water level	m/s
u1%	Existing average wave overtopping velocity at crest	m/s
u1%	maximum velocity depth-averaged at the rear side of the crest during a wave overtopping event, exceeded by 1% of the incident waves	m/s
u_b	Near bed velocity	m/s
uB,2%	Overtopping velocity Schüttrumpf and Van Gent, 2003	m/s
U_{cr}	Depth-averaged critical current velocity	m/s
U_g	Velocity in gap of closure dam	m/s
ui	Water pore pressure at slice	kN/m^2
uk	Average velocity on the inner slope	m/s
ul	Velocity of the storm debris	m/s
u_o	Maximum sea-bed wave induced orbital velocity	m/s
U_p	Propeller thrust velocity	m/s
U_r	Return current	m/s
U_w	Wind speed	m/s
U_z	Wind speed at a height of zm above sea surface	m/s
Uα	Pore water force	kN
Uβ	Surface water force	kN
$\hat{u}$□	Peak bottom velocity	m/s
v	Seepage flow velocity	m/s
v	the velocity of the ice feature	m/s
V	Individual overtopping volume per metre run of crest	m^3/m
V	Volume	m^3
vA	Actual velocity	m/s
vA	Velocity of wave run-up	m/s
vA	Velocity on inner slope	m/s
VB	Maximum volume of water during a high water period per unit of width of the structure	m3/m
vB,0	Velocity at beginning of inner slope	m/s
vc	Critical flow velocity	m/s
V_e	Equilibrium fall velocity in water	
V_{max}	Maximum individual overtopping volume	m^3/m
vs	Ship's speed	m/s
vZB	Acceleration factor for erosion of sand	-
w	Fall velocity of the sand particles.	m/s
w	Gap width between slabs or blocks	m
w	Width of shingle beach, determined as narrow / wide and condition grade	m
W	Water surface width	m
W	Weight	N
W	Width of scour hole	m
W'	Submerged weight	N
w_a	Distance between two tie rods	m
w_{ab}	Water content due to absorption by weight	%
Wact	Actual width of toe	m
x, y, z	Distances along orthogonal axes	m
X, Y, Z	Block dimensions of enclosing cuboid system	m
XA	Crest retreat position	m
xb	Distance of the resulting pressure in the concrete from the edge	m
Xc	Critical horizontal position of point C ensuring the mimum required dune width	m
X_d	Design value of a material property	Unit of the parameter

xi	Horizontal length of single erosion segment	m
ximp	x-coordinate of initial impact point at outer slope with regard to dike toe	m
X_k	Characteristic value of a material property	Unit of the parameter
xs-1,0	Surf-similarity parameter, defined as xs-1,0 = tan a / 2p /g·Hs /Tm-1,020.5	-
x_u	Pressure zone in the concrete	m
x_u	x- coordinate of leaking point at the inner berm	m
x_{u1}	Plastic pressure zone in the concrete	m
x_w	x- coordinate of intersection point of still water level and outer slope	m
y	'importance-of-structure' factor >1: engineering judgement factor	-
y	Eccentricity ship in canal	m
y	Existing shear stress	-
Y	Eccentricity of the center of gravity from the point of impact	m
y_{bs}	Depth of water at bend	m
y_c	Critical shear stress	-
y_{imp}	y-coordinate of initial impact point at outer slope with regard to dike toe	m
y_u	Average flow depth A/W in the channel upstream of the bend	m
z	Distance between the centre of gravity and the outer edge of the sheet pile profile	m
z	Half width triangular load = 0.5 H	m
Z	Limit state equation Z = R-S	Unit of the parameter
z_{98}	Wave run-up height according to Schüttrumpf 2001 or others	m
z_{98}	Wave run-up height at slope 2% probability of exceedence	m
z_a	Static rise in water level due to storm surge	m
z_o	Bed roughness length	m
z_o	Reference level of vertical velocity profile	m
Zo	Initial, unscoured bed level adjacent to toe of protection	m
z_s	Depth of slope area effected by flow	m

9. Glossary

Definitions for the following terms have been drawn from a range of sources (as referenced at the end of this section).

Accuracy - Closeness to reality.

Armour – Protective layer or mound, usually formed by rock or concrete units.

Asphalt – Flexible revetment material formed by stone, filler and bitumen.

Beach – Emergent region of sand or shingle, often subject to wave action, may extend landward to dunes or ridge.

Bias - Disposition to distort the significance of the information.

Blockwork, block revetment – Revetment cover layer formed by concrete (or stone) blocks (usually rectangular) fitted closely together, resisting hydraulic loads by weight and interblock friction.

Block mat – Concrete blocks cabled together, used to form revetment armour.

Boil - Concentrated outflow of seepage water, for example through a crack, channel or hole in a covering layer

Breach – Hole or gap cut in a flood defence, generally through a bank.

Capillary zone - Unsaturated zone of the soil above the phreatic plane (close to full saturation)

Catchment area - Area from which water runs off to a river

Characterisation - Process of expressing the observed or predicted behaviour of a system and it's components.

Coastal defence – Defence(s) to protect against erosion and/or to mitigate flooding by the sea. Coastal defences with to protect against flooding may be termed sea defences; defences primarily to protect against erosion may be termed coastal protection.

Cohesion - Attraction between soil particles (usually fine), by which they are maintained in a fixed mass without the application of external forces.

Collapse - Occurrence of inadmissible large deformations of a structure, such that it loses coherence.

Conditional probability - Likelihood of some event given the prior occurrence of some other event.

Confidence interval - Measure of degree of (un)certainty of an estimate. Usually presented as a percentage.

Consequence - Impact such as economic, social or environmental damage or improvement resulting from a flood.

Consolidation - Expulsion of water from pores of permeable and compressible soil under the influence of load leading to a reduction of volume.

Correlation - Measure of the extent to which a change in one variable tends to correspond to a change in another. One measure of linear dependence is the correlation coefficient p.

Cover layer – Outer layer of a revetment system, acts as the armouring protecting inner layers from direct hydraulic loads

Cracking – Loss of coherence for layer or element intended to be coherent.

Critical element – System element, the failure of which will lead to the failure of the system.

Defence system - Two or more defences acting to achieve common goals (e.g. maintaining flood protection to a given area).

Design objective - Objective (put forward by a stakeholder), describing the desired performance of an intervention, once implemented.

Design standard - Performance indicator that is specific to the engineering of a particular defence to meet a particular objective under a given loading condition. Note: the design standard will vary with load, for example there may be different performance requirements under different loading conditions.

Dependence - Extent to which one variable depends on another variable. Dependence affects the likelihood of two or more thresholds being exceeded simultaneously. When it is not known whether dependence exists between two variables or parameters, guidance on the importance of any assumption can be provided by assessing the fully dependent and independent cases (see also correlation).

Deterministic process / method - Method or process that adopts precise, single-values for all variables and input values, giving a single value output.

Drainage ditch - Also known as 'Soke' ditches or 'Delph' ditches, typically found close to the inward toe of an embankment to drain seepage or overflow water, and control water levels through the embankment. Larger 'Delph' ditches arise if embankment material is taken directly from the ground behind the bank.

Dune - Embankment of sand, usually formed by wind combined with vegetation that catches and holds the sand. May be a long-term or short-term feature depending to exposure.

Element - Component part of a system

Element life - The period of time over which a certain element will provide sufficient strength to the structure with or without maintenance.

Embankment - Bank formed artificially from (usually local) materials to exclude water from a given area. The main embankment structure provides mass obstruction against water. The embankment "outward face" is exposed directly to water, and the "inward face" is on the landward side and hence not normally exposed directly to water. The "embankment crest" is the top of the embankment, typically flat and (ideally) several metres wide.

Erosion - Processes of wearing away of land or defence surface by mechanical action (e.g. water, wind, ice)

Error - Mistaken calculations or measurements with quantifiable and predictable differences.

Event tree - Graphical representation of chains of events that might result from some initiating event, a few of which, should they occur, would lead to failure.

Exfiltration – Outward flow of water from a permeable layer, perhaps from beneath a dike

Exposure - Quantification of the receptors that may be influenced by a hazard (flood), for example, number of people and their demographics, number and type of properties etc.

Expected annual frequency - Expected number of occurrences per year (reciprocal of the return period of a given event).

Extrapolation - Inference of unknown data from known data, for instance future data from past data, by analysing trends and making assumptions.

Failure - Inability to achieve a defined performance threshold (response given loading). "Catastrophic" failure describes the situation where the consequences are immediate and severe, whereas "prognostic" failure describes the situation where the consequences only grow to a significant level when additional loading has been applied and/or time has elapsed.

Failure cause - Reason why a component fails.

Failure mechanism - The way in which a structure collapses (shearing, piping), or fails to retain water (Flood defence failure mechanism).

Failure mode - Description of one of a number of ways in which a defence or system may fail to meet a particular performance indicator.

Fault tree – Diagram or description showing different chains of events / failure modes leading to a top failure event. Different events are expressed in boxes and are mutually connected through logical gates, e.g. OR, AND, etc.

Filter or filter layer – Granular (usually) layer intended to prevent transport by seepage flows of finer particles from core or foundation.

Flood - A temporary covering of land by water outside its normal confines.

Flood control (measure) — A structural intervention to limit flooding and so an example of a risk management measure.

Flood damage - Damage to receptors (buildings, infrastructure, goods), production and intangibles (life, cultural and ecological assets) caused by a flood.

Flood defences - Defences (structures / systems) protecting against flooding by river or sea.

Flood forecasting system— System designed to forecast flood occurrence before the flood event:

Flood hazard map - Map with the predicted or documented extent of flooding, with or without an indication of the flood probability.

Flood level - Water level during a flood.

Flood management measures - Actions that are taken to reduce either the probability of flooding or the consequences of flooding or some combination of the two.

Flood peak - Highest water level recorded in a river during a flood.

Floodplain - Part of alluvial plain that would be naturally flooded in the absence of engineered interventions.

Flood protection (measure) – Measure protecting a certain area from inundation.

Flood risk management - Action taken to mitigate risk; or the complete process of risk assessment, options appraisal and risk mitigation.

Flood risk zoning - Delineation of areas with different possibilities and limitations for investments, based on flood hazard maps.

Flood warning system (FWS) — A system designed to warn members of the public of the potential of imminent flooding. Typically linked to a flood forecasting system.

Flooding System (in context) - In the broadest terms, *a system* may be described as the social and physical domain within which risks arise and are managed. An understanding of the way a system behaves and, in particular, the mechanisms by which it may fail, is an essential aspect of understanding risk. This is true for an organisational system like flood warning, as well as for a more physical system, such as a series of flood defences protecting a flood plain.

Foreshore - Natural area along coastline between active wave zone (swash zone) and hinterland. May include mudflats or slatings in estuaries and beaches on open coasts.

Fragility - The propensity of a particular defence or system to fail under a given load condition. Typically expressed as a *fragility function curve* relating load to probability of failure. Combined with descriptors of decay or deterioration, fragility functions enable future performance to be described.

Functional design - The design of an intervention with a clear understanding of the performance required of the intervention.

Functional failure - Failure of a flood or coastal defence to perform its function without damage to the structure, e.g. occurrence of high overtopping discharges over a defence leading to unacceptable damage to the assets defended, but without damage / failure of the defence itself.

FMECA - Failure Mode, Effects and Criticality Analysis - an analysis of all the ways in which a structure can fail, organised in a cause consequence and effects diagram.

Gabion – Rectangular basket formed by wire or rod mesh (occassionally geogrid) filled by stones, used to form armour layers and/or retaining walls.

Geogrid – Man made reinforcement grid / strips / lattice intended to strengthen soil or placed material

Geotextile – Man made fabric intended to perform some geotechnical function(s), usually filtration and separation.

Ground - Soil, rock and fill existing in place before construction or other operation starts.

Harm - Disadvantageous consequences — economic, social or environmental.

Hazard - A physical event, phenomenon or human activity with the *potential* to result in harm. A hazard does not necessarily lead to harm.

Heave - Creation of quicksand by vertical expulsion of groundwater

Horizontal seepage - Forming of channels or hollow spaces on the side of a hydraulic structure as a consequence of the erosion of the ground

Hydraulic soil failure – Loss of grain contact in the ground due to water overpressure; for cohesive soil this leads to uplifting and cracking; to heave in the case of non-cohesive soil.

Human reliability - Probability that a person correctly performs a specified task.

Ignorance – Lack of knowledge.

Infiltration - Penetration of water into permeable subsoil.

Inner face or slope - Sloping face of a dike body or embankment on the landward side.

Internal erosion - Transport of finer soil particles washed through the pores of the coarser grains of the same layer, may also be termed 'suffosion' or 'suffusion'.

Intervention - A planned activity designed to effect an improvement in an existing natural or engineered system (including social, organisation/defence systems).

Inundation - Flooding of land with water. (NB: In certain European languages this can refer to deliberate flooding, to reduce the consequences of flooding on nearby areas, for example.)

Joint probability - The probability of specific values of one or more variables occurring simultaneously. For example, extreme water levels in estuaries may occur at times of high river flow, times of high sea level or times when both river flow and sea level are above average levels. When assessing the likelihood of occurrence of high estuarine water levels it is therefore necessary to consider the joint probability of high river flows and high sea levels.

Judgement - Decisions taken arising from the critical assessment of the relevant knowledge.

Knowledge - Spectrum of known relevant information.

Knowledge uncertainty - Uncertainty due to lack of knowledge of all the causes and effects in a physical or social system. For example, a numerical model of wave transformation may not include an accurate mathematical description of all the relevant physical processes. Wave breaking aspects may be parameterised to compensate for the lack of knowledge regarding the physics. The model is thus subject to a form of knowledge uncertainty. Various forms of knowledge uncertainty exist, including:
Process model uncertainty – All models are an abstraction of reality and can never be considered true. They are thus subject to process model uncertainty. Measured data versus modelled data comparisons give an insight into the extent of model uncertainty but do not produce a complete picture.
Statistical inference uncertainty - Formal quantification of the uncertainty of estimating the population from a sample. The uncertainty is related to the extent of data and variability of the data that make up the sample.
Statistical model uncertainty - Uncertainty associated with the fitting of a statistical model. The statistical model is usually assumed to be correct. However, if two different models fit a set of data equally well but have different extrapolations/interpolations then this assumption is not valid and there is statistical model uncertainty.

Leakage flow - Flow of water under or past a structure intended to retain water.

Likelihood - A general concept relating to the chance of an event occurring. Likelihood is generally expressed as a probability or a frequency.

Limit state - The boundary between safety and failure.

Load – Force or environmental factors to which the flood or coastal defence may be subjected, such as high river flows, water levels and wave heights; also force (or pressure) applied to structure or element.

Macro-stability - Resistance to formation of a slip plane in the slope and subsoil.

Micro-stability - Resistance to erosion of slopes caused by the expulsion of water.

Monte Carlo simulation - Level III probabilistic calculation method where a large number of simulations are performed. In each simulation, numbers are drawn randomly from the distribution functions associated with the parameters in the strength and loading models, subsequently the value of the limit state function is calculated for the joint draw. The number of times that Z is smaller than zero is divided by the total number of simulations to estimate the total probability of failure.

Natural variability - Uncertainties that stem from the assumed inherent randomness and basic unpredictability in the natural world and are characterised by the variability in known or observable populations.

Outer slope - Sloping face of dike body or embankment on the seaward or river side

Overall stability – Resistance to failure that involves the structure and underlying foundation

Overflowing - Water flow over the crest own of the defence because the water level in front is higher than the defence crown or crest.

Overtopping – Intermittent water flows over the top of the defence driven by wave action with the (still) water level below the crest of the defence.

Overtopping discharge – Mean volume of water passing over a defence averaged over a given time (typically 500 or 1000 waves). Typically expressed as a discharge per metre run of defence.

Parameters - The parameters in a model are the "constants", chosen to represent the chosen context and scenario.

Pathway (in context) – Route that a hazard takes to reach Receptors. A pathway must exist for a Hazard to be realised.

Performance - Degree to which a process or activity succeeds when evaluated against some stated aim or objective.

Performance indicator - Measurable objective of a project, scheme or policy. These may be detailed engineering performance indicators, such as acceptable wave overtopping rates, armour damage, or conveyance capacity, or more generic indicators such as public satisfaction.

Phreatic surface (or plane) – Surface or level of groundwater, perhaps elevated under rainfall or wave overtopping.

Piping - A type of erosion (and water flow) in which soil particles are transported by strong groundwater flow via a channel-like pattern or pipe within a soil. Piping may occur through or below a defence if the intensity of groundwater flow exceeds a certain level.

Post-flood mitigation - Measures and instruments after flood events to remedy flood damages and to avoid further damages.

Precautionary Principle - Where there are threats of serious or irreversible damage, lack of full scientific certainty shall not be used as a reason for postponing cost-effective measures to prevent environmental degradation.

Precision — degree of exactness regardless of accuracy.

Pre-flood mitigation - Measures and instruments in advance to a flood event to provide prevention (reducing flood hazards and flood risks by e.g. planning) and preparedness (enhancing organisational coping capacities).

Preparedness – The ability to ensure effective response to the impact of hazards, including the issuance of timely and effective early warnings and the temporary evacuation of people

Probability — Measure of strength of belief that an event will occur. For events that occur repeatedly, the probability of an event is estimated from the relative frequency of occurrence of that event, out of all possible events.

Probabilistic method - Method in which the variability of input values and the sensitivity of the results are taken into account to give results in the form of a range of probabilities for different outcomes.

Probability density function (distribution) - Function which describes the probability of different values across the whole range of a variable (for example flood damage, extreme loads, particular storm conditions etc).

Probabilistic reliability methods - These methods attempt to define the proximity of a structure to fail through assessment of a response function. They are categorised as Level III, II or I, based on the degree of complexity and the simplifying assumptions made (Level III being the most complex).

Process model uncertainty - See *Knowledge uncertainty.*

Progressive failure - Failure where, once a threshold is exceeded, significant (residual) resistance remains enabling the defence to maintain restricted performance. The immediate consequences of failure are not necessarily dramatic but further, progressive, failures may result eventually leading to a complete loss of function.

Random events – Events which have no discernible pattern..

Receptor - Receptor refers to the entity that may be harmed (a person, property, habitat etc.). For example, in the event of heavy rainfall *(the source)* flood water may propagate across the flood plain *(the pathway)* and inundate housing *(the receptor)* that may suffer material damage *(the harm or consequence).* The vulnerability of a receptor can be modified by increasing its resilience to flooding.

Recovery time – Time taken for an element or system to return to its prior state after a perturbation or applied stress.

Reliability index - Probabilistic measure of the structural reliability with regard to any limit state.

Residual life - Residual life of a defence is the time to when the defence is no longer able to achieve minimum acceptable values of defined performance indicators (see below) in terms of its serviceability function or structural strength.

Residual risk - Risk that remains after risk management and mitigation measures have been implemented.

Resilience - Ability of a system or defence to react to and recover from the damaging effect of realised hazards.

Resistance – The ability of a system to remain unchanged by external events.

Response (in context) - The reaction of a defence or system to environmental loading or changed policy.

Response function - Equation linking the reaction of a defence or system to the environmental loading conditions (e.g. overtopping formula) or changed policy.

Return period - The expected (mean) time (usually in years) between the exceedence of a particular extreme threshold. Return period is traditionally used to express the frequency of occurrence of an event, although it is often misunderstood as being a probability of occurrence.

Revetment - A protective layer covering part or all of any embankment face. The protective layer may be natural (e.g. grass), man-made (e.g. concrete) or a combination of different materials.

Rip-rap - Armour layer(s) formed by wider-graded quarry stones, usually placed in bulk to approx 3 stones thickness or greater; c.f. rock armour.

Risk - Risk is a function of probability, exposure and vulnerability. Often, in practice, exposure is incorporated in the assessment of consequences, therefore risk can be considered as having two components — the probability that an event will occur and the impact (or *consequence)* associated with that event.
Risk = Probability multiplied by consequence

Risk analysis - A methodology to objectively determine risk by analysing and combining probabilities and consequences.

Risk assessment - Comprises understanding, evaluating and interpreting the perceptions of risk and societal tolerances of risk to inform decisions and actions in the flood risk management process.

Risk management measure - An action that is taken to reduce either the probability or consequences or some combination of the two

Risk mapping - The process of establishing the spatial extent of risk (combining information on probability and consequences). Risk mapping requires combining maps of hazards and vulnerabilities. The results of these analyses are usually presented in the form of maps that show the magnitude and nature of the risk.

Risk reduction - The reduction of the likelihood of harm, by either reduction in the probability of a flood occurring or a reduction in the exposure or vulnerability of the receptors.

Risk profile - The change in performance, and significance of the resulting consequences, under a range of loading conditions. In particular the sensitivity to extreme loads and degree of uncertainty about future performance.

Risk significance (in context) — The separate consideration of the magnitude of consequences and the frequency of occurrence.

Robustness – Capability to cope with external stress. A decision is robust if the choice between the alternatives is unaffected by a wide range of possible future states of nature.

Rock armour – Armour layer(s) formed by narrow-sized quarry stones, usually placed individually to approx 2 stones thickness; c.f. rip-rap.

Sand boil – Concentrated outflow of water from a defence in which sand is eroded and ejected; often linked to piping.

Saturation - Water flowing into and through permeable layers and forced up through less permeable or cover layers.

Scale - Difference in spatial extent or over time or in magnitude; critical determinant of vulnerability, resilience etc; or a geometric ratio between prototype and model.

Scenario – A plausible description of a situation, based on a coherent and internally consistent set of assumptions. Scenarios are neither predictions nor forecasts. The results of scenarios (unlike forecasts) depend on the boundary conditions of the scenario.

Scour - Erosion of beach or soil at the toe of a defence by flowing water or wave action.

Seepage – Flow of water under or through a flood defence, driven by the hydraulic head over the defence.

Seepage distance – Distance that seepage water travels below ground before it emerges.

Sensitivity - Refers to either: the resilience of a particular receptor to a given hazard. For example, frequent sea water flooding may have considerably greater impact on a fresh water habitat, than a brackish lagoon; or: the change in a result or conclusion arising from a specific perturbation in input values or assumptions.

Sensitivity Analysis - The identification at the beginning of the appraisal of those parameters which critically affect the choice between the identified alternative courses of action.

Serviceability limit states - States when deformations, displacements or non-structural damage affect the intended function of a structure in terms of comfort and appearance.

Standard of service - The measured performance of a defined performance indicator.

Severity — The degree of harm caused by a given flood event.

Shingle - Coarse (and relatively single size) granular material, of sizes classed as gravel or cobbles, often of hard flint origin, and forming re-shaping beaches or ridges.

Soil structures - Manmade soil bodies (dikes, dams, embankments, etc.), able to retain water and withstand erosion by water flow or waves.

Source — The origin of a hazard (for example, heavy rainfall, strong winds, surge etc).

Squeezing - Horizontal movement in subsoil under a defence, at an angle to its axis.

Statistic - A measurement of a variable of interest which is subject to random variation.

Strength (of materials) – A limit state of stress*

Structure - Organised combination of connected parts designed to provide some measure of rigidity. This includes fill placed during construction.

Structural failure - Failure of a defence in the form of damage of the structure, e.g. breach or collapse.

Suffosion - A type of failure in which fine soil particles are transported around / between coarser material of the same layer.

Susceptibility – The propensity of a particular receptor to experience harm.

System - An assembly of elements, and the interconnections between them, constituting a whole and generally characterised by its behaviour. Applied also for social and human systems.

System failure - Cessation of proper functioning of structure or system as a whole.

System state - The condition of a system at a point in time.

Ultimate limit state - Limiting condition beyond which a structure or element no longer fulfils any measurable function in reducing flooding.

Uncertainty - A general concept that reflects our lack of sureness about someone or something, ranging from just short of complete sureness to an almost complete lack of conviction about an outcome.

Up-lift - Form of hydraulic soil failure where a covering layer is lifted up by water pressure acting from any underlying aquifer or other permeable layer.

Validation - is the process of comparing model output with observations of the 'real world'.

Variability - The change over time of the value or state of some parameter or system or element where this change may be systemic, cyclical or exhibit no apparent pattern.

Variable – A quantity which can be measured, predicted or forecast which is relevant to describing the state of the flooding system e.g. water level, discharge, velocity, wave height, distance, or time. A prediction or forecast of a variable will often rely on a simulation model which incorporates a set of parameters.

Vulnerability – Characteristic of a system that describes its potential to be harmed. This can be considered as a combination of susceptibility and value.

Wave overtopping - Phenomenon where water is driven over the top of a defence onto the land behind when the (still) water level is lower than the crest of the defence, but water is driven over by wind or vessel waves.

Wave overtopping discharge – Mean rate of water (per unit time) that overtops the defence per metre run of defence

Wave run-up – Height above (still) water level to which a wave runs up to the slope (often the 2% wave run-up - exceeded by 2% of the waves)

Wave run-down - Height below the (still) water level reached by the wave surface running down up the slope.

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