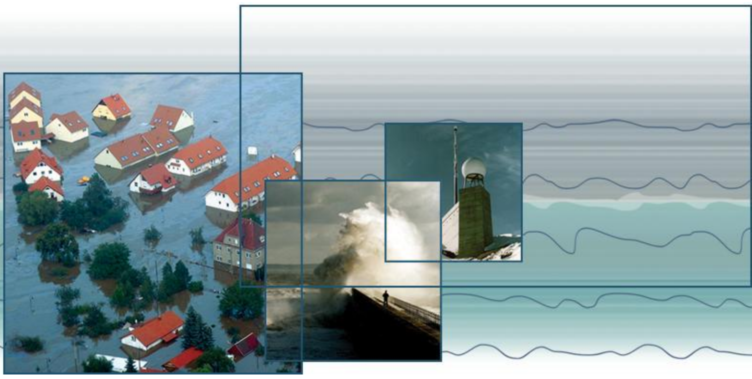
Integrated Flood Risk Analysis
and Management Methodologies

FLOODsite



Task 4 – Understanding and predicting failure modes

ACTIVITY 1.2: FAILURE MODES FOR REVETMENTS AND DUNES

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SUMMARY

The present report describes the research done within the framework of Activity 1.2 of Task 4 of the European Union FLOODsite project. It contains an inventory of the failure modes, predictive equations and related uncertainties for revetments and dunes, based work already undertaken in The Netherlands, including limit state equations and uncertainties. The methodology that is followed is based on the FLOODsite risk-source-pathway-receptor approach. Besides the inventory of limit state equations, this document also highlights 'indicator' processes for each defence type to facilitate the rapid identification of critical areas. This document provides a source of information upon which risk management tools and analyses may be based. The document may be updated and extended in the future as knowledge of structure performance increases.

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1. Introduction to Activity 1.2 of Task 4: Failure modes for revetments and dunes

1.1 Introduction

This report contains information on the research that is undertaken within Activity 1.2 of Task 4 of FLOODsite. Task 4 is part of Theme 1, which aims at a better understanding and new knowledge of the underlying physical processes and risk analysis tools as well as the required management methodologies. It consists of three Sub-Themes which reflect the key areas of the FLOODsite methodology, namely risk sources, risk pathways and risk receptors which will be discussed in more detail in this report. Task 4 is part of sub-Theme 1.2, which provides and extends knowledge on risk pathways. It describes the potential failure modes of flood defences. Other subjects that are studies within sub-theme 1.2 include influences by morphological changes, breaching of defences, the reliability of such kind of structures and the flood wave itself developing from breached structures.

1.2 Background

The methodology that is followed is based on the FLOODsite risk-source-pathway-receptor approach. The risk-source-pathway-receptor approach is a model that is used in risk assessment to identify the source of the kind of 'damage' that is studied (e.g. contamination or flooding), what the source may affect (receptor) and how the source may reach the receptor (pathway). From this information, a conceptual model is created displaying all sources, pathways and receptors (Figure 1).

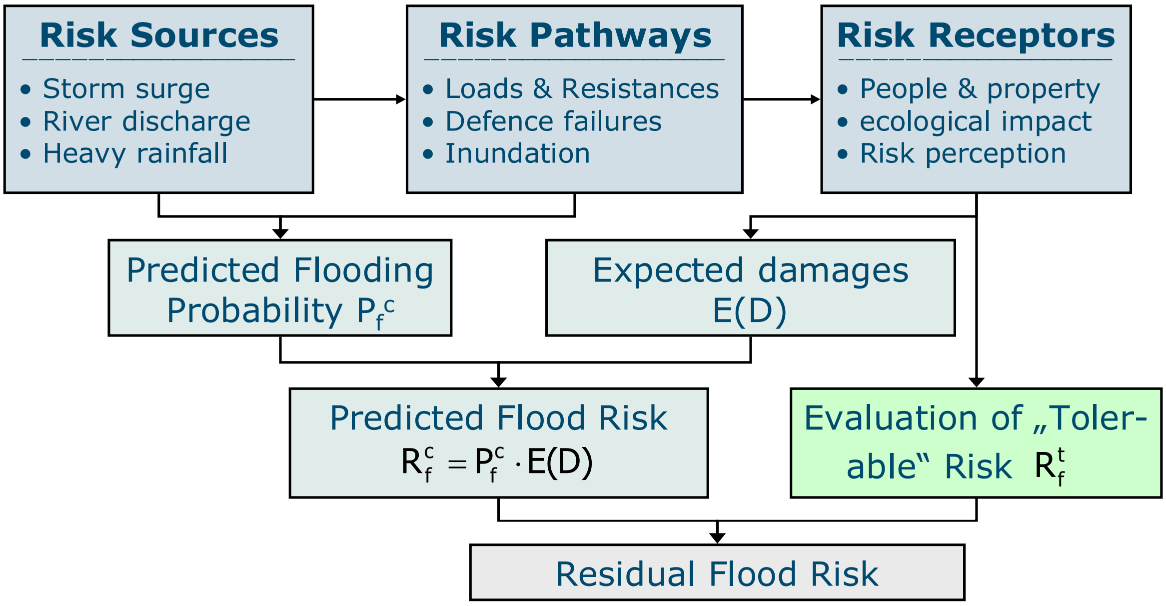

Figure 1.1 Methodology of Theme 1

The background philosophy of Theme 1 therefore is to provide methods and new knowledge to assess (i) an overall probability of failure for flood defences Pf; (ii) the expected economic and non-economic damages in the flood prone areas E(D); and (iii) the tolerable risk Rft by the receptors of risk.

Flood risks are inherently associated to the occurrence of extreme events. These events are poorly known in terms of:

- their probability of occurrence
- the behaviour of the system under such extreme events

- the enhancement of produced or perceived damage due to man-made
  - infrastructures
  - decisions

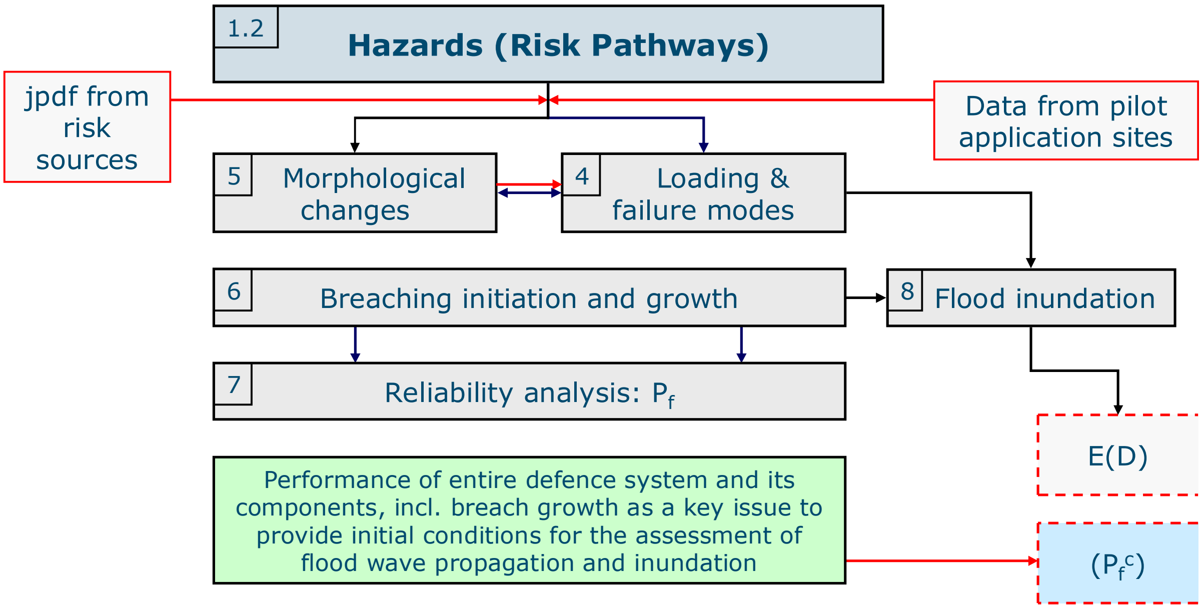

Figure 1.2 Structure of Sub-Theme

It can be seen from Figure 2 that Sub-Theme 1.2 is split into five Tasks (Task 4-8) dealing with loading and failure modes, morphological changes, breaching initiation and breach growth, reliability analysis and flood inundation. All Tasks in this Sub-Theme will help to deliver and understand the performance of the entire flood defence system and its components. This deliverable therefore essentially contributes to obtain the overall failure probability of flood defences.

This report describes the research undertaken within Action 2 of Activity 1 of Task 4. This activity collates existing information detailing defence failure mechanisms. The current report gives an inventory of literature written on the failure mechanisms of dunes and revetments. Within Activities 2 – 4 of Task 4 these failure mechanisms are analysed in more detail.

1.3 Objective

To develop an effective risk management approach, it is essential that the behaviour of the flood defence structures is understood for all load conditions and defence types. This demands consideration of a large number of independent and dependent variables including, for example, material properties, construction quality, structure response, operation and maintenance history, environmental loading etc. This activity will classify the range of revetment types (Chapter 2) and dunes (Chapter 3) that exist and their associated dominating failure modes. Current methodologies available for predicting the onset of component failure will be collated and their advantages and limitations reviewed.

The overall objective of this study is to obtain a fault tree for revetments and dunes that can be used for probabilistic analysis. The elements of the fault tree, such as individual failure modes that eventually lead to complete failure of the structure are described. Limit state equations for the failure modes are given if already available from the literature.

1.4 Approach

The methodology that is followed to derive the aforementioned objectives is threefold. It is essentially based on detailed literature review:

- Inventarisation and analysis of failure modes of dunes and revetments. This step also includes the derivation of limit state equations for each of the failure modes.
- Identification of input parameters and uncertainties for both these parameters and the limit state equations.
- Set-up of a fault tree for dunes and revetments and identification of the (temporal) interrelations of individual failure modes (and their input parameters).

1.5 Reader's guide

This report is the outcome of a literature survey on failure modes. After a first introductory chapter, the findings are presented in Chapters 2 and 3. In Chapter 2 the failure modes for different kinds of revetment are given. Chapter 3 describes the failure modes for dunes.

2. Failure modes for revetments

2.1 Introduction

The use of revetments, such as riprap, blocks and block mats, various mattresses, and asphalt in the construction of coastal and hydraulic structures is very common. In the present report the whole package that covers the core of a structure is considered to be revetment. Dependent on the type, a revetment can consist of a top layer, different types of sublayer (e.g., granular filters or geotextiles) and a sublayer. The revetment can fuction in different ways:

- as a necessary protection of the soil structure
- to reduce wave runup
- to improve the water resistance of the defence structure
- to reduce the amount of maintenance works
- to give the defence structure a more natural or esthetical appearance.

Mostly the function is a combination of these five points. For the present project only the first point will be considered, although the second and third point can be used as input parameters in the design process of a revetment.

Revetments may be classified into different categories, each having their own critical failure modes and corresponding determinant loads and required strength. Table 2.1 shows an overview of different type of revetments and their critical failure modes (Pilarczyk, 1998a; TAW, 1999)

Table 2.1 Review of revetments with critical modes of failure

type of cover layer	critical failure mode	determinant wave loading	strength
placed (pitched) block revetments	• lifting • bending • deformation • sliding	• overpressure • wave impact	• thickness, friction, interlocking • permeability including sublayer/geotextile
natural material: clay/grass	• erosion • deformation	• max. velocity • wave impact	• cohesion • grass-roots • quality of clay
loose units: sand/gravel and riprap	• initiation of motion • transport of material • profile deformation	• velocity field in waves • seepage	• weight, friction • dynamic ‘stability’ • permeability of sublayer/core
asphalt	• erosion • deformation • lifting	• max. velocity • wave impact • overpressure	• mechanical strength • weight
alternative revetments: block mats, concrete mattresses, gabions and geosystems	• initiation of motion • deformation • rocking • abrasian/corosion of wires • UV light	• max. velocity • wave impact • climate • vandalism	• weight • blocking • wires • large unit • permeability including sublayer

For natural material, such as sand, clay and grass, or interlocking units, such as concrete blocks and mats, the resistance of the protection is mainly determined by the friction, while for loose units, such as gravel and riprap, the determinative strength parameter is the cohesion. For concrete and asphalt slabs the mechanical strength is normative. As a result of the difference of strength properties, critical loadings are also different. Maximum velocities will be determined for clay/grass dikes and gravel/riprap, as they cause displacment of material, while uplift pressures and impacts are of more importance for paved revetments and slabs, as they tend to lift the protection. As these phenomena vary both in time and space, critical loading conditions vary both with respect to the position along the slope and the time during the passage of a wave. Instability for grass/clay and gravel/riprap will occur around the water level, where velocities are highest during the up and downrush. Moreover, wave impacts are more intense in the area just below the still water level. Instability of paved revetments without too much interlock occurs near the point of maximum downrush, where uplift forces are higher, just before the arrival of the next wave front. If the protection is pervious, uplift forces are strongly reduced. Instability will have occurred due to the combined effect of uplift and impact forces, just after wave breaking. Concrete slabs and asphalt will mainly respond to uplift forces at maximum loads and are distributed more evenly over a layer area, thus causing a higher resistance against uplift, compared with loose block pavement. The stability (or threshold conditions) for loose materials, such as sand and rock, can be improved by using composite systems, *i.e.*, grouting, mattresses, geosystems, *etc*.

In the following sections the relevant failure modes of the six different types of revetment will be discussed. Before discussing the different types of revetment, the theoretical background of wave loading and the structural response will be given. The theoretical part is mainly based on Klein Breteler *et al*. (1998) and Klein Breteler & Pilarczyk (1998).

2.2 Theoretical background of wave loads

2.2.1 Wave load

Wave attack on revetments will lead to a complex flow over and through the revetment structure (filter and cover layer). During the wave run-up the resulting forces by the waves will be directed opposite to the gravity forces. Therefore the run-up is less hazardous then the wave run-down. Wave run-down wil lead to two important mechanisms:

- The downward flowing water will exert a drag force on the cover layer and the decreasing freatic level will coincide with a downward flow gradient in the filter or in a gabion. The first mechanism can be schematised by a free flow in the filter or gabion with a typical gradient equalling the slope angle. It may result in sliding.
- During maximum wave run-down there will be an incoming wave that a moment later will cause a wave impact. Just before impact there is a 'wall' of water giving a high pressure under the point of maximum run-down. Above the run-down point the surface of the revetment is almost dry and therefore there is a low pressure on the structure. The high pressure front will lead to an upward flow in the filter or a gabion. This flow will meet the downward flow in the run-down region. The result is an outward flow and uplift pressure near the point of maximum wave run-down (Figure 2.1).

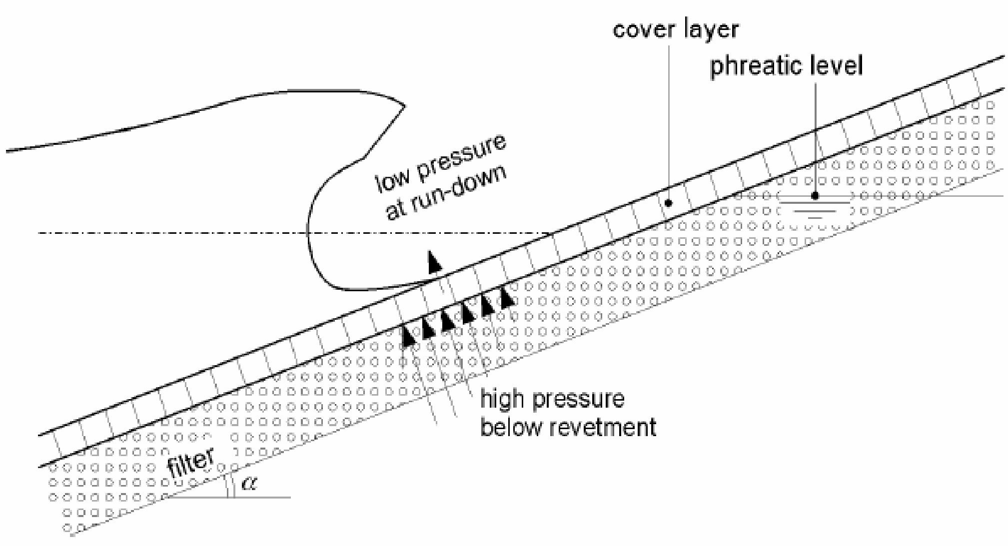

Figure 2.1 Pressure development in a revetment structure (Source: Klein Breteler et al., 1998)

The schematised situation can be quantified on the basis of the Laplace equation for linear flow:

$$\frac{\partial^2 \phi}{\partial y^2} + \frac{\partial^2 \phi}{\partial z^2} = 0 \qquad (2.1)$$

with:
$\varphi = \varphi_b$ = potential head induced in the filter or a gabion (m)
y = coordinate along the slope (m)
z = coordinate perpendicular to the slope (m)

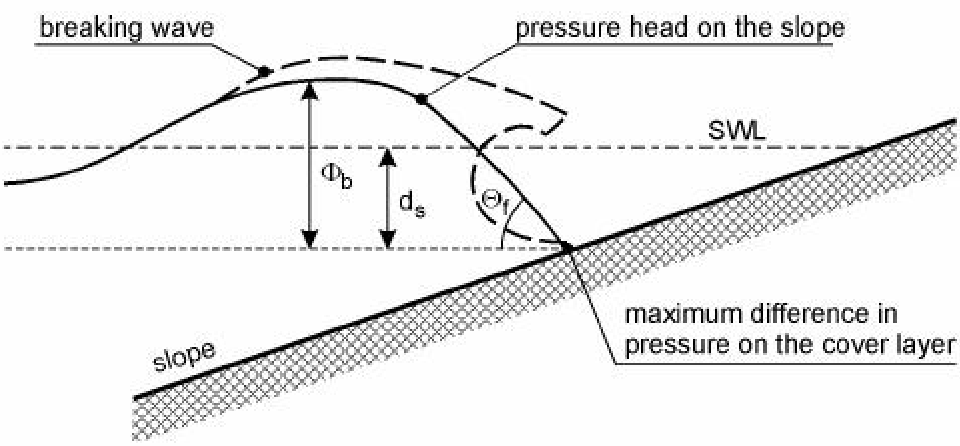

Figure 2.2 Schematization of pressure head on a slope (Source: Klein Breteler et al., 1998)

After complicated calculations the uplift pressure in the filter or a gabions can be derived. The uplift pressure is dependent on the steepness and height of the pressure front on the cover layer (which is dependent on the wave height, period and slope angle, see Figure 2), the thickness of the cover layer and the level of the phreatic line in the filter or a gabion. In case of riprap or gabions, it is not dependent on the permeability of the cover layer, if the permeability is much larger then the subsoil. The equilibrium of uplift forces and gravity forces leads to the following (approximate) design formula (Pilarczyk, 1998a):

$$\frac{H_{scr}}{\Delta D} = f\left(\frac{D}{\Lambda \xi_{op}}\right)^{0.67} \quad \text{with } \Lambda = \sqrt{\frac{bDk}{k'}} \tag{2.2a}$$

or

$$\frac{H_{scr}}{\Delta D} = f\left(\frac{D}{b}\frac{k'}{k}\right)^{0.33} \xi_{op}^{-0.67} = F\xi_{op}^{-0.67} \tag{2.2b}$$

where

H_{scr}	= significant wave height at which blocks will be lifted out	[m]
ξ_{op}	= $\tan\alpha/\sqrt{(H_s*g/(2\pi T_p^2))}$ = breaker parameter	[-]
α	= slope angle	[-]
g	= gravity acceleration = 9.81	[m/s^2]
T_p	= wave period at peak of spectrum	[s]
Λ	= leakage length	[m]
Δ	= $(\rho_s-\rho_w)/\rho_w$ = the relative density of cover layer	[-]
ρ_w	= density of water	[kg/m^3]
ρ_s	= density of protection material	[kg/m^3]
b	= thickness of sublayer	[m]
D	= thickness of top layer	[m]
k	= permeability of sublayer	[m/s]
k'	= permeability of top layer	[m/s]
f	= stability coefficient, mainly dependent on structure type, tanα and friction	[-]
F	= total (black box) stability factor	[-]

2.2.2 Structural response to wave load

There are two practical design methods available: the black-box model and the analytical model. In both cases, the final form of the design method can be presented as a critical relation of the load compared to strength, depending on the type of wave attack:

$$\left(\frac{H_s}{\Delta D}\right) = \text{function of } \xi_{op} \tag{2.3a}$$

For revetments, the basic form of this relation is:

$$\left(\frac{H_s}{\Delta D}\right) = \frac{F}{\xi_{op}} \text{ with maximum} \left(\frac{H_s}{\Delta D}\right) = 8.0 \tag{2.3b}$$

The advantage of this black-box formula is it's simplicity. The disadvantage, however, is that the value of F is known only very roughly for many types of structures.

The analytical model is based on the theory for placed stone revetments on a granular filter (pitched blocks). In this calculation method, a large number of pshysical aspects are taken into account. In short, in the analytical model nearly all physical parameters that are relevant to the stability have been incorporated in the leakage length Λ. The final result of the analytical model may, for that matter, again be presented as a relation such as Eq. 2.3b where F = f(Λ).

With a system without a filter layer (directly on sand or clay, without gullies be formed under the top layer) not the permeability of the filter layer, but the permeability of the subsoil (eventually with gullies/surface channels) is filled in.

The wave attack on a slope can be roughly transformed into the maximum velocity component on a slope during run-up and run-down, U_{max}, by using the formula:

$$U_{\max} = p\sqrt{gH_s\xi_{op}} \tag{2.4}$$

where 1 < p < 1.5 for irregular smooth slopes.

2.2.3 Flow load stability

There are two possible approaches for determining the stability of revetment material under flow attack. The most suitable approach depends on the type of load:

- flow velocity: 'horizontal' flow, flow parallel to dike;
- discharge: downward flow at slopes steeper than 1:10, overflow without waves; stable inner slope.

When the flow velocity is known, or can be calculated reasonably accurately, Pilarczyk's relation (Pilarczyk; 1990, 1999) is applicable:

$$\Delta D = 0.035\frac{\Phi}{\Psi}\frac{K_T K_h}{K_s}\frac{u_{cr}{}^2}{2g} \tag{2.5}$$

in which:
Δ = the relative density of cover layer [-]
D = characteristic thickness;
for rock : $D = D_n = (M_{50}/\rho_s)^{1/3}$ = nominal diameter defined) [m] and $\Delta = (\rho_s-\rho_w)/\rho_w$ [-],
for blocks : D = thickness of the block and $\Delta = (\rho_s-\rho_w)/\rho_w$ [-],
for mattresses : D = d = average thickness of mattress and $\Delta = (1-n)\,.\,(\rho_s-\rho_w)/\rho_w$ [-]
with : n = bulk porosity of fill material. For common squarry stone $(1-n).(\rho_s-\rho_w)/\rho_w \approx 1$
u_{cr} = critical vertically-averaged flow velocity [m/s],
Φ = stability parameter [-],
Ψ = critical Shields parameter [-],
K_T = turbulence factor [-],
K_h = depth parameter [-],
K_s = slope parameter [-].

The five parameters are explained below.

Stability parameter Φ:
The stability parameter Φ depends on the application. Some guide values are shown in Table 2.2:

Table 2.2 Guide values for stability parameter Φ (Source: Klein Breteler et al.,1998)

Revetment type	Continuous top layer	Edges and transitions
Riprap and placed blocks	1.0	1.5
Block mats, gabions, washed-in blocks, geobags and geomattresses	0.5 to 0.75	0.75 to 1.0

Shields parameter Ψ :
With the critical Shields parameter Ψ the type of material can be taken into account:
• riprap, small bags : $\Psi \approx 0.035$
• placed blocks, geobags : $\Psi \approx 0.05$

- blockmats : $\Psi \approx 0.07$
- gabions : $\Psi \approx 0.07$ (to 0.10)
- geomattresses : $\Psi \approx 0.07$

Turbulence factor K_T:
The degree of turbulence can be taken into account with the turbulence factor K_T. Some guide values for K_T are:

- Normal turbulence:
  - abutment walls of rivers : $K_T \approx 1.0$
- Increased turbulence:
  - river bends : $K_T \approx 1.5$
  - downstream of stilling basins : $K_T \approx 1.5$
- Heavy turbulence
  - hydraulic jumps : $K_T \approx 2.0$
  - strong local disturbances : $K_T \approx 2.0$
  - sharp bends : $K_T \approx 2.0$ (to 2.5)
- Load due to water (screw) jet : $K_T \approx 3.0$ (to 4.0)

Depth parameter K_h:
With the depth parameter K_h, the water depth is taken into account, which is necessary to translate the depth averaged flow velocity into the flow velocity just above the revetment. The depth parameter also depends on the development of the flow profile and the roughness of the revetment.
The following formulas are recommended:

fully developed velocity profile:
$$K_h = \frac{2}{\left(\log\left(\frac{12h}{k_s}\right)\right)^2} \tag{2.6a}$$

non-developed profile:
$$K_h = \left(\frac{h}{k_s}\right)^{-0.2} \tag{2.6b}$$

very rough flow (h/ks < 5):
$$K_h = 1.0 \tag{2.6c}$$

In which:
h = water depth [m],
k_s = equivalent roughness according to Nikuradse [m].

In the case of dimensioning the revetment on a slope, the water level at the toe of the slope must be used for h. The equivalent roughness according to Nikuradse depends on the type of revetment/geosystem. For riprap, k_s is equal usually to one or twice the nominal diameter of the stones, for bags it is approximately equal to the thickness (d), for mattresses it depends of the type of mattress: k_s of about 0.05 m for smooth types and about the height of the rib for articulating mats.

Slope parameter K_s:
The stability of revetment elements also depends on the slope gradient under which the revetment is applied, in relation to the angle of internal friction of the revetment. This effect on the stability is taken into account with the slope parameter K_s, which is defined as follows:

$$K_s = \sqrt{1 - \left(\frac{\sin\alpha}{\sin\theta}\right)^2} = \cos\alpha\sqrt{1 - \left(\frac{\tan\alpha}{\tan\theta}\right)^2} \tag{2.7a}$$

or

$$K_s = \cos\alpha_b \tag{2.7b}$$

with:
θ = angle of internal friction of the revetment material [°],
α = transversal slope of the bank [°],
α_b = slope angle of river bottom (parallel along flow direction) [°].

The following values of θ can be assumed as a first approximation: 40° for riprap, 30° to 40° for sand-filled systems, and 90° for stiff and anchored mortar-filled mattresses and (cabled) blockmats (K_s = cosα). However, for flexible non-anchored mattresses and block mats (units without contact with the neighbouring units) this value is much lower, usually about 3/4 of the friction angle of the sublayer. In case of geotextile mattress and block mats connected to geotextile lying on a geotextile filter, θ is about 15° to 20°.

The advantage of this general design formula of Pilarczyk is that it can be applied in numerous situations. The disadvantage is that the scatter in results, as a result of the large margin in parameters, can be rather wide. With a downward flow along a steep slope it is difficult to determine or predict the flow velocity, because the flow is very irregular. In such case formulas based on the discharge are developed (Pilarczyk, 1998a).

2.3 Placed block revetments (including block-mats)

2.3.1 Introduction

Revetments and placed blocks or block-mats are often used as a protection of slopes of various coastal structures against wave attack. The blocks are placed adjacent to each other on a filter layer to form a relatively closed and smooth surface, which is easy to walk on. The wave forces due to wave run-up and run-down will be only small, because of the smooth surface. On the other hand, the uplift forces due to pressure fluctuations in the breaking waves are a considerable thread to the stability.

In general, a revetment system will consist of a number of layers, the principal of which are the cover layer, filter layer(s) and, as far as necessary, complementary sublayer(s). A revetment system must be designed as an integrated system of cover layer, sublayers and subsoil (see Figure 2.3).

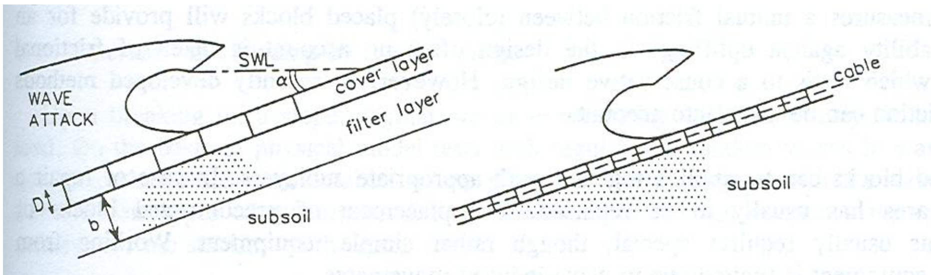

Figure 2.3 Examples of rock revetment structures (cross sections) (Source: Klein Breteler and Bezuijen, 1998)

2.3.2 Inventarisation of failure modes and fault tree

For a placed block revetmens four failure mechanisms can be discerned (Klein Breteler and Bezuijen, 1998; TAW, 2004):
1. Uplifting of blocks (PC)
2. Migration of subsoil particles through the granular filter and/or cover layer (PM)
3. Erosion through underlayers (PEU).
4. Shear/geotechnical instability (PS)

The cover or armour layer is the major protection of the structure and should resist external and internal loadings. The strength against external loadings can primarily be provided for by a sufficient weight of the armour elements. The internal loadings depend to a large extent on the permeability ration of cover and filter layer. Further on, the permeability of the core may affect the stability of the cover layer as far as the phreatic level inside the structure is concerned. Additional stability of the cover layer can be obtained by friction, interlocking or tensile forces. These forces may act between the elements of the armour layer and between the armour elements and the underlayers. Most of the artificial systems have been designed deliberately to mobilise these additional forces. The strength and the capacity of load reduction are often used interchangeably.

In The Netherlands for the assessment of placed block revetments a distinction is made between two different zones (TAW, 2004):
A. the outer slope below the Dutch normative high water level called 'toetspeil'.
B. the outer slope above 'toetspeil', crest or inner slope

The revetments in zone A should be checked for the following failure mechanisms: uplifting of blocks (PC), migration of subsoil through filter or cover layer (PM) and shear/geotechnical instability (PS). If the revetment in zone A shows deficiencies on the cover layer stability (PC) or if it shows migration of soil particles (PM) the revetment should be tested for erosion through the underlayers (PEU). If the score on this latter mechanism is sufficient, the revetment in zone A can be considered to be 'safe'. If the the structure shows signs of shear (PS) the revetment fails.

Revetments in zone B are checked for cover layer stability (PC). If the revetment in zone B shows deficiencies on the cover layer stability (PC) the revetment should be tested for erosion through the underlayers (PEU). If the score on this latter mechanism is sufficient, the revetment in zone B can be considered to be 'safe'.

Summarizing, if shear (PS) occurs the revetment fails. If the revetment shows deficiencies on the mechanisms cover layer stability (PC) or migration of soil particles (PM), and erosion through the underlayers (PEU) occurs simultaneously the structure fails as well.

Hence, failure can be expressed by:

{failure} = { {{ cover layer instability OR migration of subsoil particles } AND erosion } OR shear }

2.3.3 Analysis of failure modes and derivation of limit state equations

This section is mainly based on Klein Breteler and Bezuijen (1998) and TAW (2004).

Cover layer stability

Upon breaking on a slope, regular waves exert during each wave a cyclic hydraulic load. On the basis of physical model tests with regular and random waves in wave tanks good knowledge has been obtained of the relevant load phenomena within a wave cycle. For different types of revetments different moments or periods in each wave cycle are decisive for the stability of the cover layer. The external loads can be quantified by way of physical model tests and with numerical methods (Petit *et al.*, 1994; Van Gent *et al.*, 1994; Kuiper and Doorn, 2004). Most numerical methods will give a full description in space and time.

A much simpler approach towards a computation of the relevant wave loads is to abandon a full description of time and place dependent wave pressures on the slope, and to concentrate only on the instant of critical wave loads. For placed block revetments the most critical load situation occurs at the moment of maximum wave run-down (see also section 2.2.1). This is proved to have general validity for the structures with relatively low cover layer permeability.

During the wave run-down there is a large piezometric head gradient on top of the revetment (see Figure 2.4), caused by the simultaneous occurrence of run-down of the preceding wave and the arrival of the present. The piezometric head underneath the cover layer is a damped representation of the potential on top of the revetment, causing an uplift at the location of maximum wave run-down. The extend of the damping is influenced by the permeability ratio of the cover layer and the filter layer and also by the compressibility of the air/water-mixture in the filter. The latter is important for very fine granular filter (D_n < 3 mm) and will not be considered here.

The piezometric head over the cover layer during wave run-down can be quantified by considering the mass balance of the water in the filter and the Darcy flow equation (Figure 2.4)

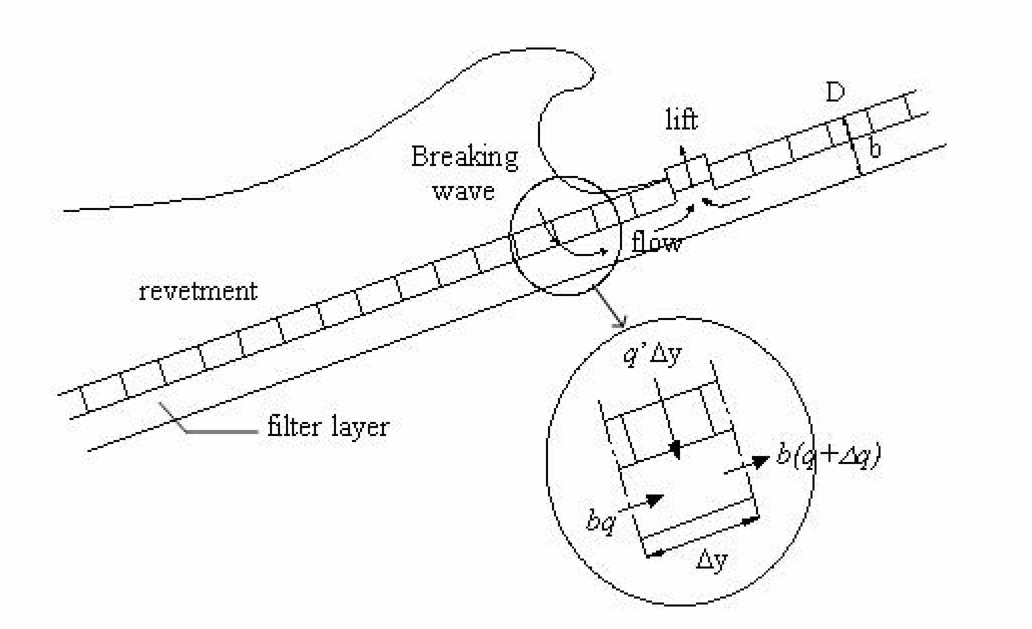

Figure 2.4 Mass balance in filter

The flow in the filter is quasi-static. In the filter layer a mean potential φ (piezometric head; pressure) can be derived in a plane perpendicular slope assuming the flow in the filter layer parallel to the slope. The flow in the cover layer is assumed to be perpendicular to the slope. The differential equation can be written as:

$$\frac{d^2\phi}{dy^2} = \frac{\phi - \phi_T}{\Lambda^2} \tag{2.8}$$

with Λ the leakage length as defined previously and φ_T the piezometric head over the cover layer. The leakage length can be seen as a piece of protection, in which the flow resistance through cover layer and filter layer are the same. This parameter is a meaure of the pressure head difference on the cover layer for given wave forces. A solution of Eq. 2.8 was presented by Wolsink (see Burger *et al.*, 1990):

$$\phi_w = \left(\frac{1}{2}\Lambda \cdot \cos\alpha \cdot \tan\theta \cdot \left(1 - \exp\left[\frac{-\phi_b}{\Lambda \cdot \cos\alpha \cdot \tan\theta} \right] \right) + \frac{1}{2}\Lambda \sin\alpha \right) \left(1 - \exp\left[\frac{-2z_1}{\Lambda \sin\alpha} \right] \right) \tag{2.9}$$

with:
φ_w = maximum piezometric head over the cover layer [m],
φ_b = maximum piezometric head [m],
α = slope angle [°],
θ = steepness of the wave front [°],

z_1 = phreatic level in filter layer relative to the point where the wave front meets the revetment [m],.

The resulting formula for the maximum gradient in the filter layer is:
1. maximum downward gradient:

$$i = \sin\alpha \qquad (2.10)$$

2. maximum upward gradient:

$$i = \cos\alpha \tan\theta \left(1 - \exp\left[\frac{-\phi_b}{2\Lambda \cdot \cos^2\alpha \cdot \tan\theta}\right]\right) - \frac{\sin\alpha}{2}\exp\left[\frac{-\phi_b}{2\Lambda \cdot \cos^2\alpha \cdot \tan\theta}\right] \qquad (2.11)$$

Equations (2.9) and (2.11) are presented in Figure 2.5 and Figure 2.6. It is clear that the uplift pressure over the cover layer increases as the leakage length Λ increases and the steepness of the wave front θ increases. But the larger the Λ, the smaller is the upward gradient i in the filter.

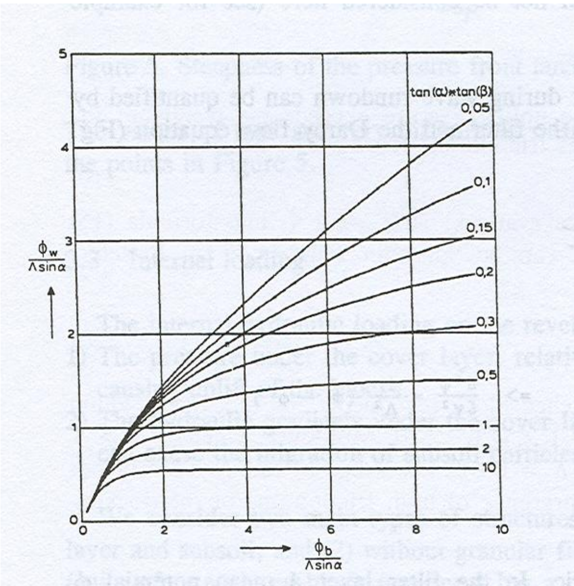

Figure 2.5 Uplift (tanβ=cotθ). (Source: Klein Breteler & Bezuijen, 1998).

The above formulae for the loads are derived for regular wave attack. Experiments show that especially the large waves cause instability and that the number of waves during a storm plays a minor role. On comparing the piezometric head on the slope under regular wave attack the following simple rule of thump is derived for the wave height at threshold if damage:

$$\left(\frac{H}{H_s}\right)_{damage} = 1.4 \qquad (2.13)$$

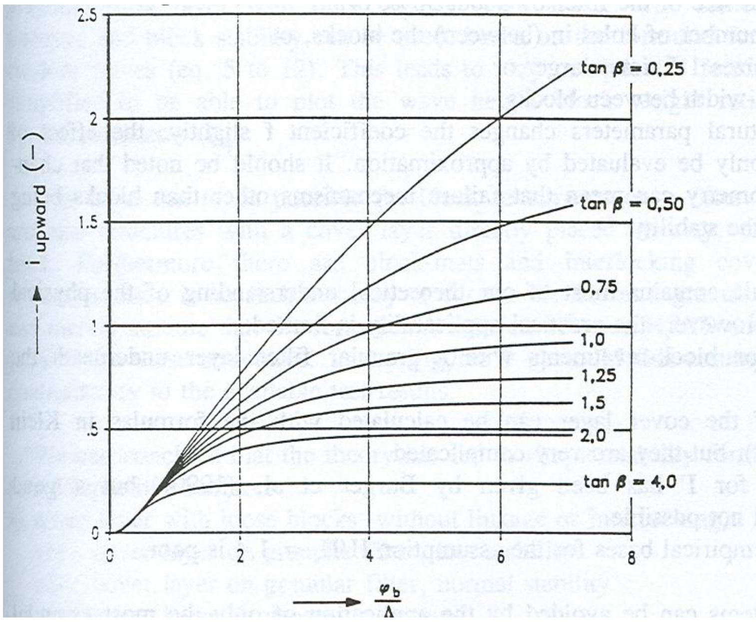

Figure 2.6 Max. upward gradient (tanβ=cotθ). (Source: Klein Breteler & Bezuijen, 1998).

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 limit state is:

$$\phi_w = \Gamma \Delta D \cos\alpha \tag{2.14}$$

with:
Γ = a coefficient representing friction, inertia *etc.* [-].
A lower boundary for Γ has been given by Burger *et al.* (1990), but a good quantification is still not possible.

Combining Equations (2.13) and (2.14) leads to a complicated stability formula that can be approximated by (Klein Breteler, 1991):

$$\frac{H_{scr}}{\Delta D} = f\left(\frac{D}{b}\frac{k'}{k}\right)^{0.33} \zeta_{op}^{-0.67} \tag{2.15}$$

with:
H_{scr} = the critical wave height at which blocks are lifted up [m].

The formulae work properly for placed/pitched block revetments and blockmats within the following range: 0.01 < k'/k<1 and 0.1<D/b<10. Moreover, when D/Λ> 1 use D/Λ=1 and when D/Λ<0.01 use D/Λ=0.01. The range of stability coefficient F is: 5 < F < 15. The higher values refer to the presence of high friction and/or interlocking of a system.

In practice the stability formulae is applied in it's most essential form and completed with emperical data from large-scale model studies. Partly based on the general trends in the results of model tests, the stability formula then reads:

$$\frac{H_{scr}}{\Delta D} = F \cdot \zeta_{op}^{-0.67} \tag{2.16}$$

The value of F depends on the type of structure, characterised:

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

The theory presented here cannot straightforwardly be extended to other types of revetments and random wave attack. For these structures there is no such theory as for the blocks on a granular filter. Therefore, it is merely assumed that the form of the relation between $H_s/\Delta D$ and ξ_{op} (like Equation 2.16) is also valid for these structures.

The results of the research presented above is incorporated in Dutch guidelines for the design of coastal strucuteres. The guideline for the assessment of safety (TAW, 2004) presents the following procedure for the testing of placed block revetments on the stability of the cover layer.

Assessment procedure for the outer slope (zone A and the part of zone B that is on the outer slope, where the layer thickness in zone B can be increased by a factor of 25%).

1. Make a first assessment on the basis of the 'behaviour' of the revetment. If it appeared that the revetment experienced damage, in the sense that blocks are lifted out, during seasonal condition it can be assumed that the same and worse will happen during design conditions. If this is the case the revetment is considered to be 'unsafe'. If this is not the case the assessment can be continued with step 2. Note that less severe damage, such as bending and deformation, can also be an indication of cover layer instability but also of other failure mechanisms.

2. Use a so-called black-box formula for the assessment of the safety of the cover layer (see also Section 2.2.1). Three different types of placed block revetment structures are discerned (hence, three series of black-box formulae):
   1. a placed block revetment on geotextile on sand or clay
   2. a placed block revetment on good erosion resistant clay (C1) or on moderate/bad erosion resistent clay (C2/C3). For situation C2 or C3 no black-box formulae exist, since the revetment will score 'insufficient' on the mechanism 'transport of material through underlayer (PEU).
   3. a placed block revetment on a granulair layer with a favourable construction (A), a normal construction (B) or an unfavourable construction (C).

   The exact criteria for the subdivision can be found in TAW (2004).

   The black-box formulae read:

   <u>Placed block revetment on geotextile on sand or clay (type 1):</u>
   Condition for score 'good':
   $$\begin{aligned}&\text{if } 0.6 < \zeta_{op} \le 2.2 \text{ then } H_s/\Delta D < 4.31\zeta_{op}^{-0.926}\text{; or}\\ &\text{if } 2.2 < \zeta_{op} \le 5.0 \text{ then } H_s/\Delta D < 11.0\zeta_{op}^{-4} + 0.09\zeta_{op} + 1.38\end{aligned} \quad (2.17a)$$
   Condition for score 'bad':
   $$\begin{aligned}&\text{if } 0.6 < \zeta_{op} \le 2.2 \text{ then } H_s/\Delta D > 6.78\zeta_{op}^{-0.588}\text{; or}\\ &\text{if } 2.2 < \zeta_{op} \le 5.0 \text{ then } H_s/\Delta D > 17.0\zeta_{op}^{-2} + 1.84\zeta_{op} - 3.25\end{aligned} \quad (2.17b)$$
   For intermediate values the score is 'doubtful'.

   <u>Placed block revetment on good clay (type 2):</u>
   Condition for score 'good':
   $$\begin{aligned}&\text{if } 0.6 < \zeta_{op} \le 2.4 \text{ then } H_s/\Delta D < 3.75\zeta_{op}^{-1.001}\text{; or}\\ &\text{if } 2.4 < \zeta_{op} \le 5.0 \text{ then } H_s/\Delta D < 8.0\zeta_{op}^{-4} + 0.02\zeta_{op} + 1.25\end{aligned} \quad (2.18a)$$
   Condition for score 'bad':

$$\text{if } 0.6 < \zeta_{op} \leq 2.1 \text{ then } H_s / \Delta D > 6.1\zeta_{op}^{-0.75}\text{; or}$$
$$\text{if } 2.1 < \zeta_{op} \leq 5.0 \text{ then } H_s / \Delta D > 11.0\zeta_{op}^{-2} + 0.98\zeta_{op} - 1. \quad (2.18b)$$

For intermediate values the score is 'doubtful'.

<u>Placed block revetment on a granular layer on a favourable construction with C_{silt} = 1.0 (type 3a):</u>
Condition for score 'good':

$$\text{if } 0.6 < \zeta_{op} \leq 2.2 \text{ then } H_s / \Delta D < 4.58\zeta_{op}^{-0.903}\text{; or}$$
$$\text{if } 2.2 < \zeta_{op} \leq 5.0 \text{ then } H_s / \Delta D < 14.5\zeta_{op}^{-4} + 0.17\zeta_{op} + 1.27 \quad (2.19a)$$

Condition for score 'bad':

$$\text{if } 0.6 < \zeta_{op} \leq 2.2 \text{ then } H_s / \Delta D > 7.12 C_{silt}\zeta_{op}^{-0.539}\text{; or}$$
$$\text{if } 2.2 < \zeta_{op} \leq 5.0 \text{ then } H_s / \Delta D > C_{silt}(17.8\zeta_{op}^{-1.5} + 2.54\zeta_{op} - 0.632) \quad (2.19b)$$

For intermediate values the score is 'doubtful'.

<u>Placed block revetment on a granular layer on a normal construction with C_{silt} = 1.0 (type 3b):</u>
Condition for score 'good':

$$\text{if } 0.6 < \zeta_{op} \leq 2.0 \text{ then } H_s / \Delta D < 4.08\zeta_{op}^{-1.014}\text{; or}$$
$$\text{if } 2.0 < \zeta_{op} \leq 5.0 \text{ then } H_s / \Delta D < 11.0\zeta_{op}^{-4} + 0.03\zeta_{op} + 1.25 \quad (2.20a)$$

Condition for score 'bad':

$$\text{if } 0.6 < \zeta_{op} \leq 2.1 \text{ then } H_s / \Delta D > 6.68 C_{silt}\zeta_{op}^{-0.723}\text{; or}$$
$$\text{if } 2.1 < \zeta_{op} \leq 5.0 \text{ then } H_s / \Delta D > C_{silt}(12.0\zeta_{op}^{-1.5} + 1.5\zeta_{op} - 3.12) \quad (2.20b)$$

For intermediate values the score is 'doubtful'.

<u>Placed block revetment on a granular layer on unfavourable construction with C_{silt} = 1.0 (type 3c):</u>
Condition for score 'good':

$$\text{if } 0.6 < \zeta_{op} \leq 2.0 \text{ then } H_s / \Delta D < 3.07\zeta_{op}^{-1.014}\text{; or}$$
$$\text{if } 2.0 < \zeta_{op} \leq 5.0 \text{ then } H_s / \Delta D < 6.5\zeta_{op}^{-4} + 0.02\zeta_{op} + 1.09 \quad (2.21a)$$

Condition for score 'bad':

$$\text{if } 0.6 < \zeta_{op} \leq 2.3 \text{ then } H_s / \Delta D > 5.08 C_{silt}\zeta_{op}^{-0.785}\text{; or}$$
$$\text{if } 2.3 < \zeta_{op} \leq 5.0 \text{ then } H_s / \Delta D > C_{silt}(13.8\zeta_{op}^{-4} + 0.26\zeta_{op} + 1.53) \quad (2.21b)$$

For intermediate values the score is 'doubtful'.

3. If the revetment turns out 'not safe' on the basis of these black-box formulae the analytical model should be used. The analytical model incorporates the effect of the permeability of the top layer and the granular layer. The software package ANAMOS uses the analytical method. ANAMOS calculates the stability of the cover layer under loading conditions for irregular waves. Three failure mechanisms are tested in the program (see also Figure 2.7): uplifting of individual blocks out of the slope, sliding of a number of blocks down the slope and penetration of the base layer into the filter (causing the cover layer to settle). The wave loading is first schematised to a single pressure distribution on the slope. With this pressure distribution, the pressures under the coverlayer are calculated assuming a steady state situation. The resulting pressure difference induces flow in the filter layer, which can cause infiltration from the base layer.

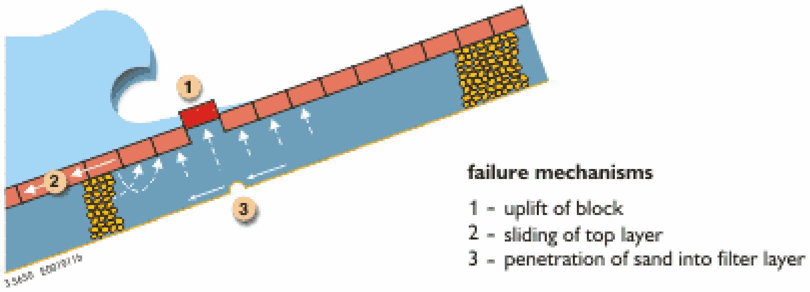

Figure 2.7 Failure mechanisms modelled in ANAMOS

Within the analytical method three criteria are used:

1. no movement for the top layer element under consideration during wave impact of individual waves with a height as high as H_s during normative conditions.
2. a maximum movement of 10% of the the top layer thickness for the top layer element under consideration during wave impact of individual waves with a height as high as $H_{2\%}$ during normative conditions.
3. the general stability criterion $H_s/\Delta D \leq \xi^{-2/3}$ must be fulfilled.

4. If the revetment turns out 'not safe' on the basis of the analytical method an advanced assessment should be carried out. Three methods are available to come to an advanced judgement of the revetment:
   1. refinement of the standard guidelines on the basis of local conditions
   2. application of more accurate calculation methods
   3. consideration of the strength that is already demonstrated

   In practice, an advance judgement consists of a combination of these three methods. As an example of the first method one can think of testing of the local permeability properties. With regard to the second method one can think of using more advanced models such as Steenzet/ZSteen or TRITON. The former can accurately model the piezometric head and the pressure as a function of time (GeoDelft, 2000). The model TRITON is a time-domain Boussinesq-type wave model that can accurately compute the wave conditions at the toe of the structure (Borsboom; 2000, 2001).

Asssessment procedure for the crest and the inner slope (Zone B without the outer slope)
This zone can be of importance for the assessment of the crest height. For a matter of fact, placed block revetments are not used very often for the protection of the crest or the inner slope of a dyke or dam. The assessment consists of four steps.

1. Preselection: if the crest height is higher than the normative high water level ('toetspeil'), increased with the 2% wave run-up level (this is the run-up level that is exceeded by 2% of the waves), the wave load on the crest and the inner slope is considered to be very low. Hence, the revetment is considered to be 'safe'. For dykes with a crest height lower than the normative high water level, increased with the significant wave height H_s, an advance assessment is necessary (step 4). For all intermediate values the assessment can continue with step 2.
2. Make an assessment on the behaviour of the revetment. See the description of the procedure for outer slopes for an explanation of the assessment of the behaviour of the revetment.
3. For the load due to overtopping a simple rule exists which lead to a score 'safe', 'doubtful' or 'unsafe'. In this rule the weight per surface unit is compared with the so-called fictitious wave height. These rules are:

   For favourable placed block revetments (well-clamped, washed-in and/or silted natural stones of which the crest edge is rounded or poured with asphalt) the rules are:

   - 'safe' : $\Delta D \geq 1/6 \cdot (\text{'toetspeil'} + z_{2\%} - h_{cr})$ (2.22a)

- 'doubtful' : $1/6 \cdot ('\text{toetspeil}' + z_{2\%} - h_{cr}) > \Delta D \geq 1/12 \cdot ('\text{toetspeil}' + z_{2\%} - h_{cr})$ (2.22b)
- 'unsafe' : $\Delta D < 1/12 \cdot ('\text{toetspeil}' + z_{2\%} - h_{cr})$ (2.22c)

For all other placed block revetments the rules are:

- 'safe' : $\Delta D \geq 1/4 \cdot ('\text{toetspeil}' + z_{2\%} - h_{cr})$ (2.23a)
- 'doubtful' : $1/4 \cdot ('\text{toetspeil}' + z_{2\%} - h_{cr}) > \Delta D \geq 1/12 \cdot ('\text{toetspeil}' + z_{2\%} - h_{cr})$ (2.23b)
- 'unsafe' : $\Delta D < 1/12 \cdot ('\text{toetspeil}' + z_{2\%} - h_{cr})$ (2.23c)

where:
$z_{2\%}$ = wave run-up level exceeded by 2% of the waves [m]
h_{cr} = crest height [m]

4. If the score in step 3 is 'doubtful' the revetment can be subjected to an advanced assessment method. See the description of the procedure for outer slopes for an explanation of the advanced assessment.

The guideline for the assessment of safety (TAW, 2004) also gives a a rule for the assessment of the cover layer stability for flow along the revetment. This rule reads:

- 'safe' : $\Delta D \geq 0.44 \dfrac{u^2}{g}$ (2.24a)
- 'doubtful' : $0.44 \dfrac{u^2}{g} \geq \Delta D \geq 0.16 \dfrac{u^2}{g}$ (2.24b)
- 'unsafe' : $\Delta D < 0.16 \dfrac{u^2}{g}$ (2.24c)

where:
u = the depth-averaged flow velocity [m/s]

Migration of subsoil through the filter or cover layer

The 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. The damage mechanism shows as some stones that are sunk compared to the adjacent stones, or as a gradually increasing S-profile develops. Some minor settlements is hardly effecting the stability against wave action, but it must warn us that it will get worse during every serious wave attack (storm). Loss of coherence of the cover layer is the final stage and the failure is at hand. No problems will arise if the granular filter or geotextile on the subsoil is geometrically sandtight:

- granular filter on sand: $D_{50} / d_{50} < 5$ (2.25a)
- geotextiles on sand: $O_{90} / d_{90} < 1$ (2.25b)
- geotextiles on clay or silt: $O_{90} / d_{90} < 1$ and $O_{90} < 100 \mu m$ (2.25c)

where:
D_x = grain size of the filter [m]
d_x = grain size of the subsoil [m]
O_{90} = average diameter of the standardized sand fraction, of which 90% remains on the geotextile after a sieve test under defined conditions [m]

Unfortunately these criteria are often difficult to meet. A more advanced requirement is based on hydrodynamic sandtightness, *viz.* the internal flow must not be capable of washing out the subsoil

material (even though the openings of the geotextile are much larger than the subsoil grains). This arises from:
- the hydrodynamic forces on the subsoil are greatly reduced by the geotextile.
- the cohesion forces on the particles do not allow small particles to be washed away.

The hydrodynamical sandtightness criteria can be applied in the majority of structures because hydraulic loads usually are low in the vicinity of the subsoil (see Figure 2.8). Only in some case, in which the geotextile or subsoil filter interface is very close to the surface of the structure and, provided the hydraulic loads are heavy (for example breaking waves), the geotextiles of filter should be geometrically sandtight (see Figure 2.9).

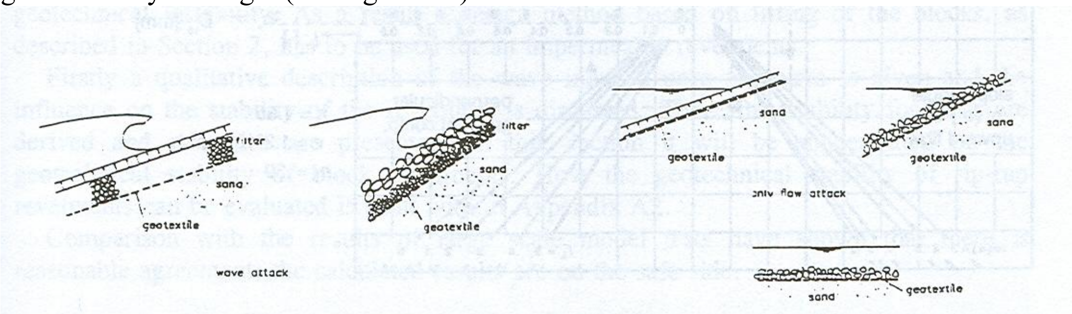

Figure 2.8 Examples of structures in which hydrodynamically sandtight geotextiles can be applied (Source: Klein Breteler & Bezuijen, 1998)

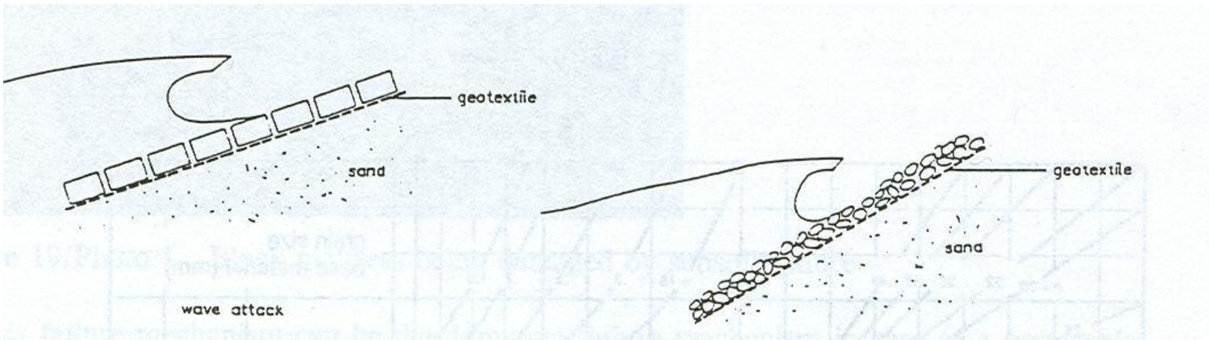

Figure 2.9 Examples of structures in which geometrically sandtight structures are necessary (Source: Klein Breteler & Bezuijen, 1998)

The critical hydraulic gradient can be read from Figure 2.10. In the upper right panel the ratio of O_{90} and D_{b90} is given as a function of the permeability of the geotextile. This can be translated to a thickness of the geotextile (upper left panel). The lower left panel shows the corresponding grain size in the filter. In the lower right panel this is translated to the porosity of the filter which yields a critical gradient.

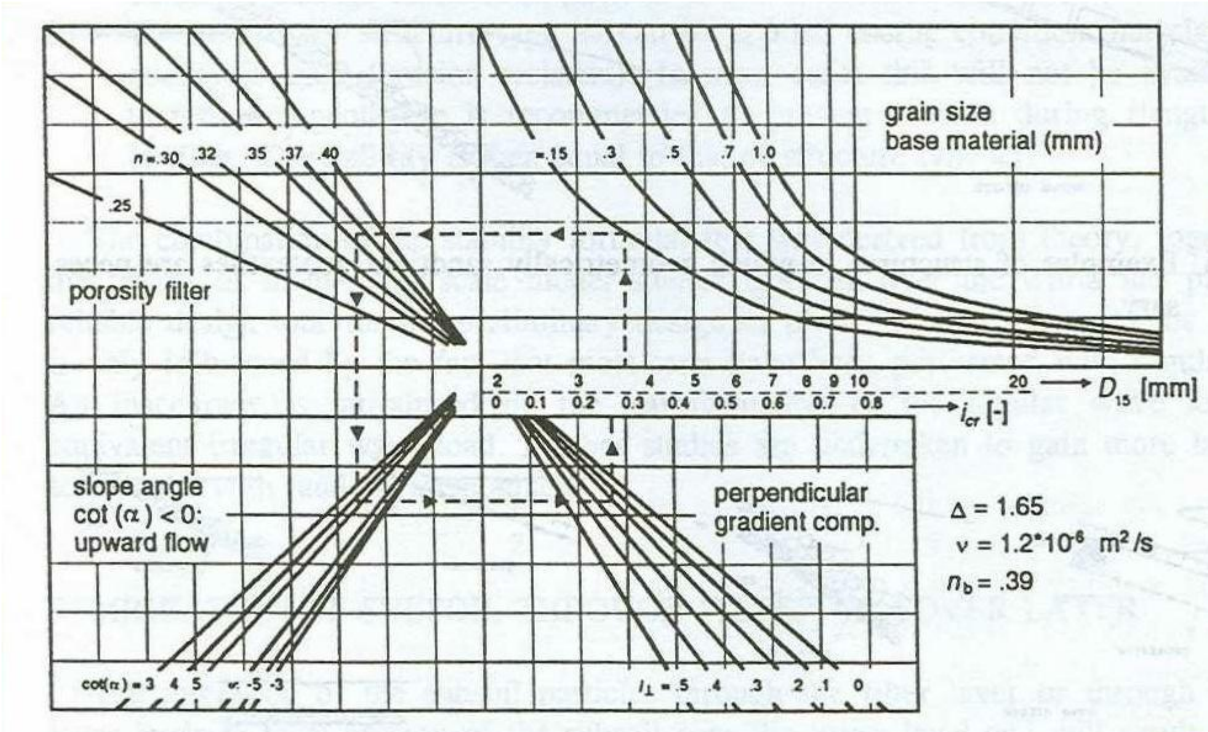

Figure 2.10 Calculation rules for critical gradient i_{cr} for granular filter on sand (Source: Klein Breteler & Bezuijen, 1998)

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

- good clay (colloid content = 39%; d_{50} = 9 μm; d_{90} = 80 μm):

$$i = \frac{0.03}{n^2 D_{15}} \tag{2.26a}$$

- medium and poor clay (colloid content = 20%; 42 μm < d_{50} < 130 μm; 100 μm < d_{90} < 400 μm):

$$i = \frac{0.01}{n^2 D_{15}} \tag{2.26b}$$

- fine sand (d_{50} = 90 μm; d_{90} = 130 μm):

$$i = \frac{0.001}{n^2 D_{15}} \tag{2.26c}$$

where:
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].
The value i can be calculated with Equations (2.10) and (2.11). If gradients larger than i_{cr} can be expected (structures like in Figure 2.9), then a geometrically sandtight geotextile or filter is recommended.

Erosion through the subayers/subsoil (PEU)

Testing on the erosion of the sublayers makes only sense if the revetments themselves have a score ‘unsafe’ for the mechanisms ‘stability of the cover layer’ or ‘migration of subsoil particles’. The residual strength can be seen as a resistance against erosion. The revetment has a positive score on the mechanism if the erosion resistance of the cover layer, the ganular layer and the clay layer are together more than the normative duration of the load:

$$t_{rg} + t_{rk} > t_{sm} \quad (2.27)$$

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

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. The duration t_{sm} starts as soon as the water depth d reaches certain lower limit d_- and it stops as soon as the water depth d exceeds the upper limit d_+. Hereafter the duration t_{sm} starts again as soon as the water depth d has decreased again to the upper limit d_+ and it stops as soon as the water depth d is less than the lower limit d_-. See Figure 2.11 for details.

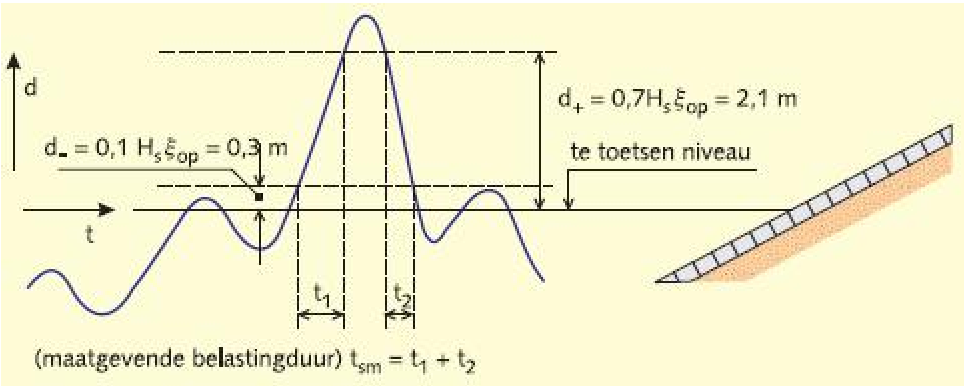

Figure 2.11 Determination of normative duration of loading t_{sm} (Source: TAW, 2004)

The values for d_+ and d_- are determined by the wave height Hs and the breaker parameter ξ_{op}. Moreover, a difference is made between perpendicular and oblique wave attack. The design rules are as follows:
for perpendicular wave attack (angle of incidence < 20º):

$$d_- = 0.1 \cdot H_s \cdot \xi_{op}$$
$$d_+ = 0.7 \cdot H_s \cdot \xi_{op} \quad (2.28a)$$

and for oblique wave attack (angle of incidence > 20º):

$$d_- = 0.3 \cdot H_s \cdot \xi_{op}$$
$$d_+ = 0.5 \cdot H_s \cdot \xi_{op} \quad (2.28b)$$

The resulting loading duration holds for one particular position on the slope. It is not possible to determine beforehand which level is crucial. Higher on the slope the duration is less but the wave load is more severe.

The residual strength of the top layer and the granular layer can be calculated as follows:

$$t_{rg} = 163.000 \cdot T_p \cdot \exp\left[-0.74 \cdot \sqrt{\left(H_s \cdot L_{0p}\right)}\right] / 3600 \quad (2.29)$$

where
L_{0p} = the wave length based of irregular wave on deep water [m].
Note that T_p is in second and t_{rg} is in hours.

The residual strength of the clay layer can only be taken into account if the following three criteria are fulfilled:
1. the score of the top layer on shear is at least ‘sufficient’,
2. the normative wave height H_s is less than 2,
3. the thickness of the clay layer is more than 0.4 m.

If the dyke has clay core up to the normative high water level the residual strength of the clay layer is equal to 24 hours. Hence Equation (2.27) than changes into:

$$t_{sm} - t_{rg} < 24\,[\text{hour}] \tag{2.30}$$

If this criterion is not fulfilled a more detailed rule must be applied, according to the Dutch guideline for the assessment of safety (TAW, 2004). Four parameters for the determination of the residual strength of the clay layer are taken into account:
1. the measure of deformation of the clay. A good indication for this whether the clay is regularly below water level. Mean high water level (MHW) or the averaged water level in the river (AWR) are used a the reference level,
2. the erosion category of the clay (C1, C2 or C3),
3. the significant wave height H_s,
4. the thickness of the clay layer d_c.

Table 2.3 Detailed determination of residual strength of clay layer (Source: TAW, 2004)

	level	below MHW or AWR + 1m				above MHW or AWR + 1m			
	H_s [m]	0.2	0.5	1.0	>1.6 <2.0	0.2	0.5	1.0	>1.6 <2.0
erosion resistance low (C3)	thickness clay layer [m]								
	<0.4 m	0	0	0	0	0	0	0	0
	0.7 m.	2	1.5	1.5	1	2	1.5	1.5	1
	1.0 m.	3.5	3	3	2	3.5	3	3	2
	1.2 m.	5	4.5	4.5	3	5	4.5	4.5	3
good (C1) and moderate (C2)	<0.4 m	0	0	0	0	0	0	0	0
	0.7 m.	4	3	2	1.5	3.5	2.5	1.5	1
	1.0 m.	7.5	6	4	3	6.5	5	3	2
	1.2 m.	11	9	6	4.5	9.5	7.5	4.5	3

If the required residual strength of the clay layer is more than the residual strength according to Table 2.3 an advanced assessment is necessary. See the section above on the advanced assessment methods.

Shear/geotechnical stability (PS)

Shear is the failure mechanism at which a part of the revetment (only the cover layer or the cover layer in combination with sublayers), as a consequence of hydraulic loading, moves along a shear plane parallel to the slope. Whether or not shear occurs depends on the following parameters: the steepness of the slope, the composition of the structure, the significant wave height H_s, or the presence of sand between the clay of the cover layer and the clay of the core.

To analyze the strength of the subsoil it is assumed that the subsoil consists of granular material that can be described as a friction material. Stability is guaranteed as lang as the ratio between shear stress and normal stress is smaller than the tangent of the friction angle Φ:

$$\frac{\tau}{\sigma} < \tan\Phi \tag{2.31}$$

where:

τ = shear stress on a plane in the subsoil [kN/m^2],
σ = normal stress on the same plane [kN/m^2].

Without any water movement the calculation of the normal and shear stress in a plane parallel to a slope is straightforward, leading to the well-known relation that the slope angle cannot exceed the friction angle. In case of water movement in the subsoil and thus a non-hydrostatic pressure distribution, the influence of the pore pressure on the normal and shear stress has to be included in the calculation. This can lead to a failure surface that is different from the plane parallel to the slope. Therefore generally the stability has to be evaluated by a slip circle analysis or finite element calculation. Generally the pore pressure distribution in the subsoil underneath a revetment under wave attack has to be calculated by numerical methods. Bezuijen (1991) has developed a simplified procedure for permeable revetments that leads to a minimum revetment weight per square meter to prevent subsoil instability, including the influence of the pore pressure distribution. For the exact details of this procedere, see the corresponding publication. Only the resulting expressions for the shear and normal stress will be presented here. These expressions read:

$$\sigma = \left[\sigma_b + (1-n_s)(\rho_s - \rho) g z_0\right]\cos\alpha - \rho g \cdot \Delta\phi \tag{2.32}$$

$$\tau = \left[(1-n_s)(\rho_s - \rho) g z_0\right]\sin\alpha \tag{2.33}$$

$$\sigma_b = \left\{(\rho_c - \rho) D + [1-n_s]\left(\rho_f - \rho\right) b\right\} \tag{2.34}$$

where:
z_0 = critical depth [m],
n_s = the porosity of the subsoil [-],
ρ_f = density of the filter grains [kg/m^3],
ρ_c = density of top layer elements [kg/m^3],
ρ_s = density of protection material [kg/m^3],
b = thickness of the filter layer [m].
It is now possible, on the basis of Equations (2.30) – (2.33), to derive a minimum value of the weight of the revetment and filter layer (σ_b) necessary to achieve a stable revetment. With a toe protection or anchoring this is:

$$\sigma_b \geq \frac{\rho g \cdot \Delta\phi}{\cos\alpha} - (1-n_s)\cdot(\rho_s - \rho)\cdot g z_0 \cdot \left(1 - \frac{\tan\alpha}{\tan\Phi}\right) \tag{2.35}$$

To use this relation it is necessary to define $\Delta\varphi$ and the critical depth z_0 at which the slip surface occurs. In this report $\Delta\varphi$ is assumed to be equal to the run-down value. The following relation can be used for the run-down ($R_{d2\%}$) for irregular waves:

For $\xi_{op} < 4.5$: $\dfrac{R_{d,2\%}}{H_s} = -0.33 \cdot \xi_{op}$ (2.36a)

For $\xi_{op} \geq 4.5$: $\dfrac{R_{d,2\%}}{H_s} = -1.50$ (2.36b)

The critical depth z_0 can be determined with consolidation theory (Bezuijen, 1991):

$$z_0 = \frac{1}{2} \cdot L_{es} \cdot \sqrt{\pi} \tag{2.37}$$

with:

$$L_{es} = \sqrt{T \cdot c_v} \tag{2.38}$$

and

$$c_v = \frac{k}{\rho_w \cdot g \cdot n \cdot w'} \tag{2.39}$$

where:
w' = the compressibility of the pore water with air [m^2/N],

In the equation of c_v it is assumed that the soil skeleton is very stiff compared to the stiffness of the water-air mixture. Normally this assumption is valid, since a few percent air in the pore water decreases the compressibility considerably to values lower than the compressibility of densified sand. The permeability of the subsoil is most accurately determined from permeability tests. A first approximation can be obtained, based on the grain size and porosity of the soil (Den Adel, 1989):

$$k = \frac{g}{160 \cdot \nu} \cdot \frac{n_s^{\,3} \cdot d_{15}^{\,2}}{(1-n_s)^2} \tag{2.40}$$

where:
ν = kinematic viscosity (= 1.2 10^{-6}) [m^2/s],

As a relation for the compressibility of the air-water mixture w', the following relation can be used (Verruijt, 1969):

$$w' = w + \frac{s}{p_a} \tag{2.41}$$

where:
p_a = atmospheric pressure (= 1.10^5) [N/m^2],
s = air content (normally between 1 and 10) [%].

The Dutch guideline on the assessment of safety uses the following criterion:

$$\Delta D + b_f + b_k > \min\left\{0.11 \cdot H_s \cdot \left(\frac{\tan\alpha}{gH_s/(2\pi T_p^{\,2})}\right)^{0.8} ; 1.5 \cdot H_s\right\} - 1334 \cdot (1 - 1.19 \cdot \tan\alpha) \cdot D_{15} \cdot \sqrt{T_p} \tag{2.42}$$

where:
b_f = thickness of the cover layer [m],
b_c = thickness of the cohesive layer [m],
α = local slope angle [°],
D_{15} = grain size diameter of sand underneath the revetment which is exceeded by 15% of the material based on the weight [m].

In fact Equation (2.38) is the result of substitution of the following values in Equations (2.35) – (2.40):

n_z	= 0.4 [m],
$\Delta_z = (\rho_s - \rho)/\rho$	= 1.65 [-],
$(1\text{-}n)\Delta_f = (1\text{-}n)(\rho_f - \rho)/\rho$	= 1. [-],
W_L	= 0.1 [-],
Φ	= 40 [°],
$\cos \alpha$	= 1 [-].

Figure 2.12 shows the value of z_0 as a function of the grain size of the subsoil for T = 5s and n = 0.5 and various percentages of air content. Since the underwater weight of the subsoil is roughly 10 kPa and the loading on this type of revetment goes to several meters, the mechanism discussed here is of relevance if z_0 is also smaller than a few meters. Therefore, Figure 2.12 shows that geotechnical instability has to be considered when the subsoil consists of material with an average grain diameter of less than 1 mm.

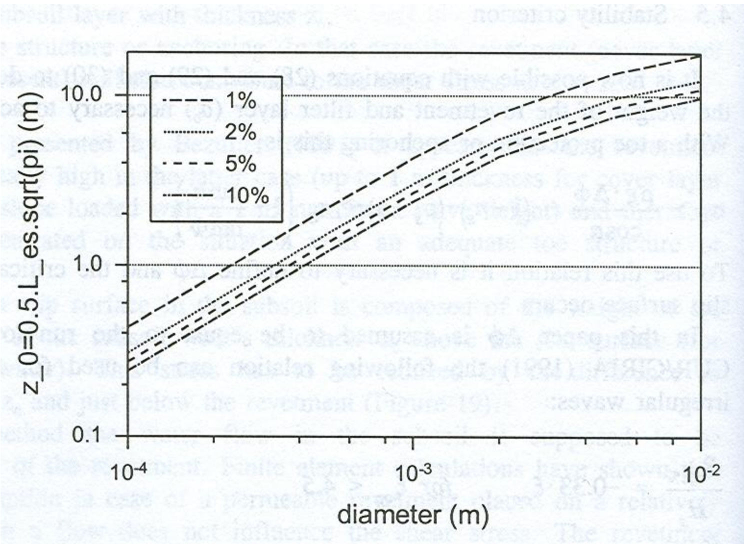

Figure 2.12 Influence of several parameters on z_0 (Source: Klein Breteler & Bezuijen, 1998)

2.3.4 Identification of input parameters and uncertainties

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. The permeability of the different layers is probably the parameter that is most difficult to estimate. However, good tuning of the permeabilities of the cover layer and the sublayers (including geotextile) is an essential condition for a balanced design of a placed block revetment. The presence of transition structures can seriously change the permeability properties and hence the critical loading. This should be incorporated in a proper design of a placed block revetment.

The empirical coefficients in all design formulae form a second source of uncertainty. However, due to a the research programme ‘knowledge gaps regarding block revetments’ (‘Kennisleemtes Steenbekledingen’), initiated by the Department of Road and Hydraulic Engineering (DWW) and the Directorate Zeeland, project office Seadefence (Directie Zeeland, PBZ) of the Dutch Directorate General of Public Works and Water Management (Rijkswaterstaat), a major research project has been carried out. The purpose of this research project is to increase the knowledge about placed block revetments, which will lead to improved methods for safety assessments and design.

2.4 Natural material (grass and clay)

2.4.1 Introduction

A grassed clay dike revetment is one of the types of revetments used with the aim of preventing erosion of a dyke by breaking waves. On as suitable clay layer the active construction of a grass cover is not really necessary. Good accompanying of the natural evolution is fundamentally sufficient. If the spontaneously growing vegetation is mowed and removed once or twice a year, the growing of woody plant species will be prevented and the result will be a grassland consisting of grasses and forbs.

For the evaluation of the strength of a grass revetment the following subdivision of the grass is used (see also Figure 2.13):

- the top soil: this is the upper part of the clay layer. It consists of turf and the root system;
- the subsoil: the part of the clay layer with mainly substrate and none of only few rootage.

In general the erosion protection against hydraulic loadings is supplied by a cover layer of clay on top of the core of the dyke, with a grass cover on top. In the case of heavy hydraulic loadings, the function of the clay layer is not only to deliver nourishment and moisture to the vegetation, but also to contribute to the erosion resistance in cases in which the grass cover is not yet developed or when the grass cover is temporarily or locally absent. In this case the function of the clay is quite comparable with the residual strength of the clay layer underneath a placed concrete block revetment.

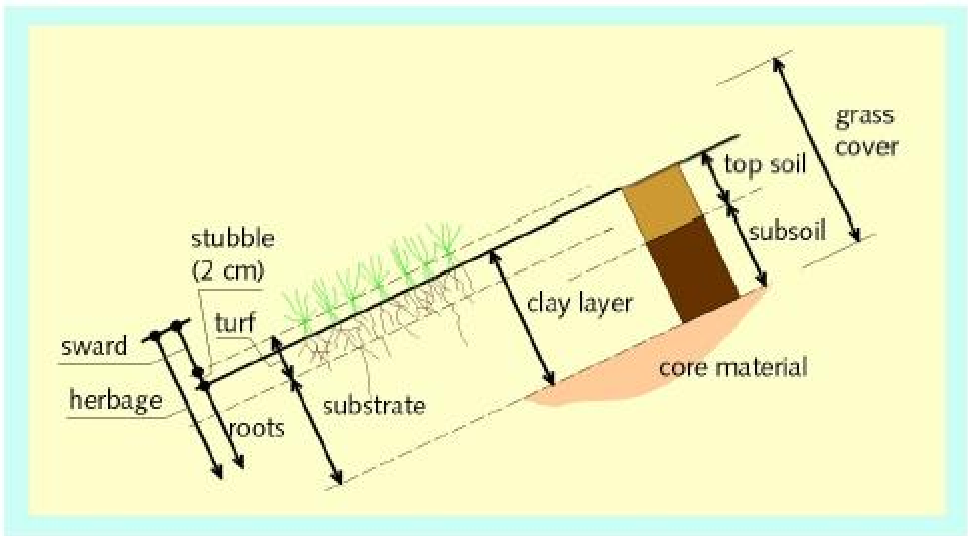

Figure 2.13 Structure and division of a grass cover (Source: Rijkswaterstaat)

2.4.2 Inventarisation of failure modes and fault tree

For a grass revetment six failure mechanisms can be discerned (TAW, 2004):

1. Washing out of loose soil particles and small lumps between the roots. If this leads to equally distributed erosion over a large surface this is usually not considered as severe damage. However, this mechanism may also lead to such transport of material that the cover surface will become uneven or that the vegetation will be disturbed.
2. Sudden washing out of larger lumps as a consequence of water pressure differences between pores and cracks in the substrate and the water outside. Unevennesses in the cover surface further these pressure differences.
3. Breaking through of the sod by strong erosion developed locally.
4. Complete or partial tearing, or breaking open of sod due to wave impact or flow along the grass cover.
5. Erosion of all sublayers of the clay after the stubble has been removed by previous erosion mechanisms (residual strength).

6. Sliding of the grass cover along a slide circle through the under layer due to satiation or ground water flow (shear).

The first four of these mechanisms concern erosion mechanisms of the stubble due to wave wave impact, wave runup, wave overtopping or flow. The fifth mechanism concerns erosion of the under layer and the last concerns the shear along a slip plane. This mechanisms can be clustered as follows:

- Erosion by wave impact (GEI)
- Erosion by wave run-up (GER)
- Erosion by wave overtopping (GEO)
- Erosion through ‘underlayers’ (GEU)
- Shear (GS)

The assessment of the grass revetment is dependent on the age and the location on the dyke profile. In the first four years after construction the grass revetment the grass has not developed enough to give protection against erosion. Therefore, additional measures (*e.g.*, coverage or additional monitoring) should be taken during high water if the age of the grass revetment is less than four years.

For grass revetments which have passed this ‘development phase’ the assessment is dependent on the location on the dyke profile. In The Netherlands for the assessment of grass a distinction is made between four different zones:
A. the outer slope with a probability more than 1/10 year of being exposed to wave impact (slamming)
B. the outer slope between the height coupled to the 1/10 probability and ‘toetspeil’
C. the outer slope above the ‘toetspeil’
D. between crest and the inner slope

For revetments in zone A there is no direct danger of erosion if the behaviour of the dyke is good. The person who is responsible for the management of the dyke should, on the basis of his or her own experience, check whether the quality level is sufficient. If not the grass revetment should be tested as if it positioned in zone B. All grass revetments in zone A are checked for shear (GS).

Revetments in zone B are checked for erosion by wave impact (GEI) and shear (GS). If the score on the erosion mechanism is insufficient, the revetment will also be checked for erosion through the underlayers (GEU).

For zone C the revetment is checked on the mechanism erosion by wave runup (GER). If the score on the erosion mechanism is insufficient, the revetment will also be checked for erosion through the underlayers (GEU).

Revetments in zone D are checked for erosion by wave overtopping (GEO) and shear (GS). Only the inner slope, not the crest, is checked on shear. If the score on the erosion mechanism is insufficient, the revetment will also be checked for erosion through the underlayers (GEU).

Summarizing, of the five mechanims GEI, GER, GEO, GEU and GS, the first three can be clustered to one mechanism, since a particular location on the dyke is exposed to either wave impact, wave runup or wave overtopping.

The top-event in the fault tree can then be expressed by:

{failure} = {erosion OR shear}

The probability that erosion occurs is determined by the chance that erosion by wave load (GEI, GER or GEO) and erosion through the underlayers occur simultaneously:

{erosion} = {erosion by waves AND erosion through underlayers}

Erosion by waves can be split into three partial mechanisms:

{erosion by waves} = {erosion by wave impact OR erosion by wave runup
OR erosion by wave overtoping}

2.4.3 Analysis of failure modes and derivation of limit state equations

For the failure mechanisms the following hydraulic loadings are responsible: water level, wind and ship waves, flow caused by rainfall and by overtopping. In The Netherlands the wave load is more important than the flow load. In the breaker zone wave impact is normative, above that level the water flow due to runup and rundown gives the maximum load and below the breaker zone the orbital movement of the waves is the normative mechanism. For slopes with a steepness of less than 1:5 the breaking waves are reduced by the water layer caused by the wave rundown.

The remainder of this section is mainly based on TAW (2004).

Loading

Erosion by wave impact (GEI)

The parameters that determine the wave load on the dyke in the wave impact zone are the significant wave height at the toe of the dyke (Hs), the wave period (Tp), the slope angle α_i (where i denotes 'impact'), the development in time of the water level during the storm en the durence of the storm.

The slope angle α_i is the average slope over an area of 1.5Hs below the normative water level of the location that is be assessed (with the 'toetspeil' as a limit for the upper boundary).

The time t_i that the asssessed location is within the wave impact zone is equal to the time that it takes for the water to rise and fall over a height of 0.5Hs above the assessed location (see Figure 2.14).

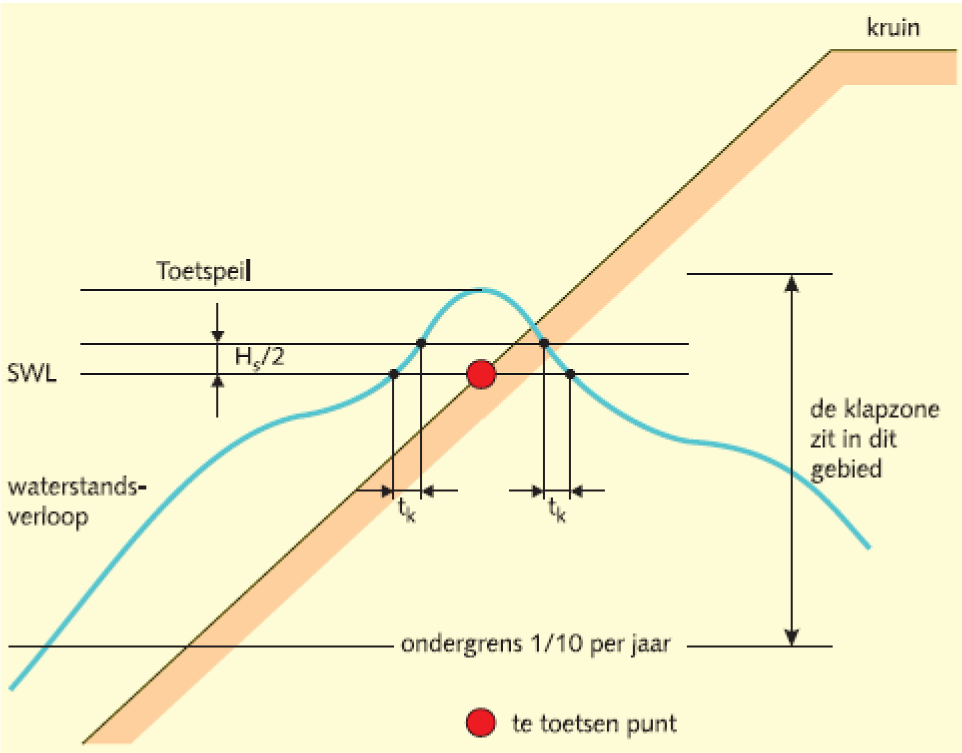

Figure 2.14 Determination of duration of loading in wave impact zone t_i (Source: TAW, 2004)

Note that the duration t_k in the Figure is the same as the duration t_i.

Erosion by wave runup (GER)

The parameters that determine the wave load on the dyke in the wave runup zone are the design velocity v_r of the flow velocity during a wave period, the slope angle α_r (where r denotes 'runup'), the development in time of the water level during the storm en the durence of the storm.

The slope angle α_r is the average slope over an area of 'toetspeil' + 1.5Hs and 'toetspeil' - 1.5Hs. If the level 'toetspeil' – 1.5Hs is below the level of the toe of the dyke, the toe is the lower boundary. It the level 'toetspeil' + 1.5Hs is above the crest level, the crest level is considered to be the upper boundary. If there is a berm present the this berm determines the boundary. For the grass revetment below the berm the berm is the upper boundary, whereas for the revetment above the berm the berm is the lower boundary.

The time t_r that the asssessed location is within the wave runup zone is equal to the time that it takes for the water to rise and fall over a certain height which is equal to the height difference between 'toetspeil' and a level at h_A below the point that is tested. The parameter h_A is equal to the wave runup level with respect to still water level, coupled to a fictitious overtopping discharge q equal to 0.1l/m/s on a infinite long slope with a slope angle equal to that of the outer slope (see

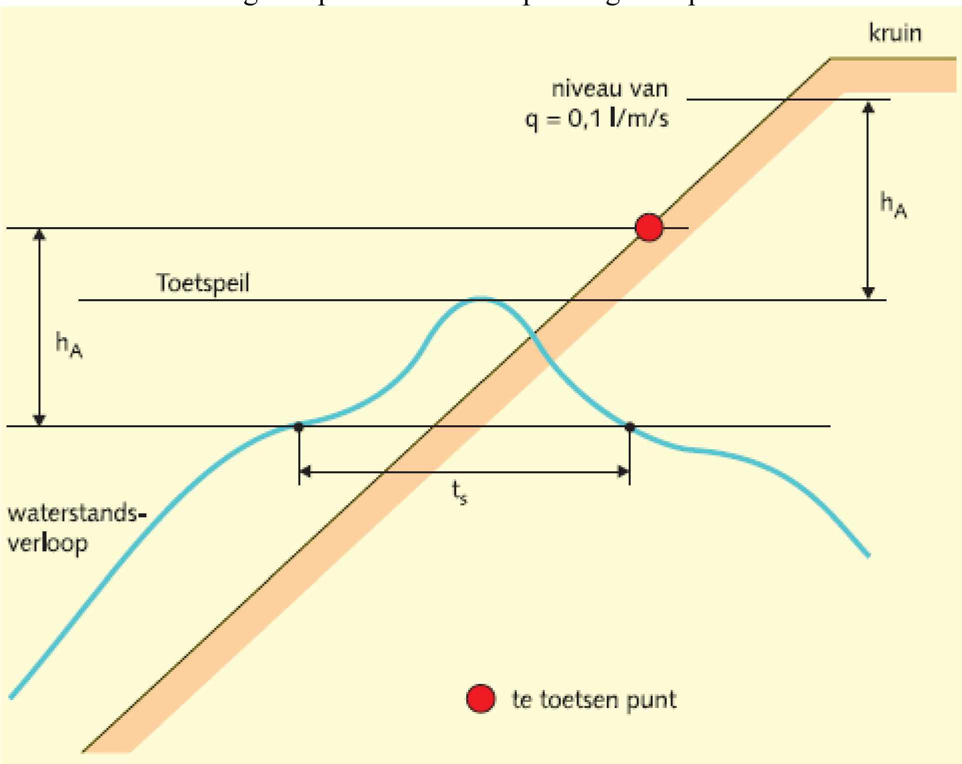

Figure 2.15).

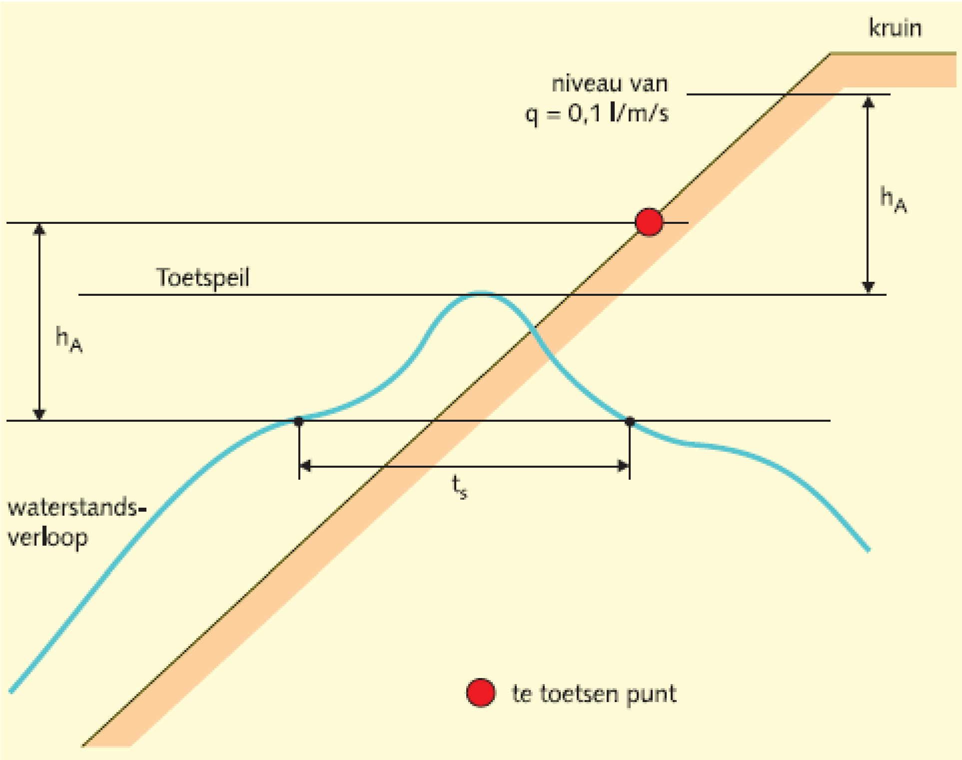

Figure 2.15 Determination of duration of loading in wave run-up zone t_r (Source: TAW, 2004)

Note that the duration t_s in the Figure is the same as the duration t_r.

Erosion by wave overtopping (GEO)

The load due to wave overtopping is expressed in terms of a overtopping discharge. In case of water flowing downwards the aboveground parts of the vegetation have a shielding effect. This does not hold for larger waves with varying flow directions. In these cases the damaging effect of the flow can no longer be neglected and the sublayers will be exposed to the waves and flow.

The time t_r for the points on the crest and the inner slope is determined similar to the time t_r for the outer slope.

Erosion through the underlayers (GEU)

This mechanism is described in the paragraph on placed block revetments. See the section concerned for an explanation and the rules for the determination of the normative duration.

Shear (GS)

For shear the wave height Hs and wave steepness are of importance. The wave steepness can be derived from the wave period Tp.

Strength

Erosion by wave impact (GEI), wave runup (GER) and wave overtopping (GEO)

The erosion stability of the whole stubble is the strength for the failure mechanisms erosion by wave impact' (GEI), wave runup (GER) and wave overtopping (GEO). The strength can be expressed in terms of the quality the grass stubble and the erosion stability of the clay in the stubble.

Dependent on the 'root density' at a particular height the grass stubble is considered to be of 'bad', 'weak', 'moderate' or 'good' quality. The erosion stability of the clay in the stubble is dependent on the thickness of the clay cover and the composition of the clay.

Acceptable wave load in wave impact zone
For wave impact of a short duration the following relation holds:

$$a \cdot H_s \cdot \tan\alpha < \; ca.\,1 \qquad (2.43)$$

where the following values for a can be used:
- grass of good quality : a = 4 [-]
- grass of moderate quality : a = 4√2 [-]

For wave impact of a long duration the maximum duration of the wave impact can be determined as follows:

$$t_{k\max} = \frac{d}{3600 \cdot \gamma \cdot C_E \cdot (4 \cdot H_r \cdot \tan\alpha)^2} \qquad (2.44)$$

where

t_{kmax}	= maximum duration of the wave load	[hr]
d	= reference erosion depth (usually chosen equal to 0.5 m)	[m]
C_E	= grass erosion coefficient	[$m^{-1}s^{-1}$]
γ	= safety coefficient = 2	[-]
H_r	= δH_s	[m]
δ	= $0.5\ H_s^{-0.25}\ T_p^{0.5}$	[-]
T_p	= peak period = $4\ H_s^{0.5}$	[s]

Note that a simpler variations on Equation 2.44 are also presented in literature, *e.g.*, :

$$t_{k\max} = \frac{d}{3600 \cdot \gamma \cdot C_E \cdot H_s^{\;2}} \qquad (2.45)$$

The following values can be used for C_E (Seijffert and Verheij, 1998):
grass of good quality : $C_E = 0.5 – 1.5\ 10^{-6}$ [$m^{-1}s^{-1}$]
grass of average quality : $C_E = 1.5 – 2.5\ 10^{-6}$ [$m^{-1}s^{-1}$]
grass of poor quality : $C_E = 2.5 – 3.5\ 10^{-6}$ [$m^{-1}s^{-1}$]

For the considered location t_k must be smaller than t_{kmax} ($t_k < t_{k\max}$).

Acceptable wave load in other zones
For all parts in the outer slope, the crest and the inner slopes for which the discharge is less than 0.1 l/s no additional demands are made besides the general requirements originating from the point of view of good management and maintenance.

In other cases the maximum wave load can again be described in terms of a maximum duration of the wave load.

The design value of the velocity can be determined according to the following formula:

$$v_r = m_1 \cdot \frac{H_s}{T_p} \cdot (0.085 - \frac{H_s}{L_0}) \cdot (1 - \frac{z}{z_q})^{0.5} \cdot \tan\alpha \qquad (2.46)$$

where

v_r = the design velocity which is approximately 50% of the maximum 2% flow velocity in the wave runup zone [m/s]

m_1 = 700 [-]

z = level on outer slope with respect to SWL [m]

z_q = wave runup level, corresponding to a discharge of q = 0.1 l/m/son a infinite slope with the same slope angle as the outer talud [-]

The design velocity for crest and inner slope are taken equal to the velocity in the outer slope. The value of t_s for points above the wave impact zone can be corrected (reduced) for drying. Figure 2.19 shows how the maximum duration of wave load t_{smax} can be graphically derived. The value t_{sr} is related to t_s according to the following relationship:

$$t_{sr} = m_2 \cdot t_s \tag{2.47}$$

where

$m_1 \quad = 1 - \dfrac{z}{z_q}$ [-]

The design criterion is: $t_{sr} < t_{s\max}$.

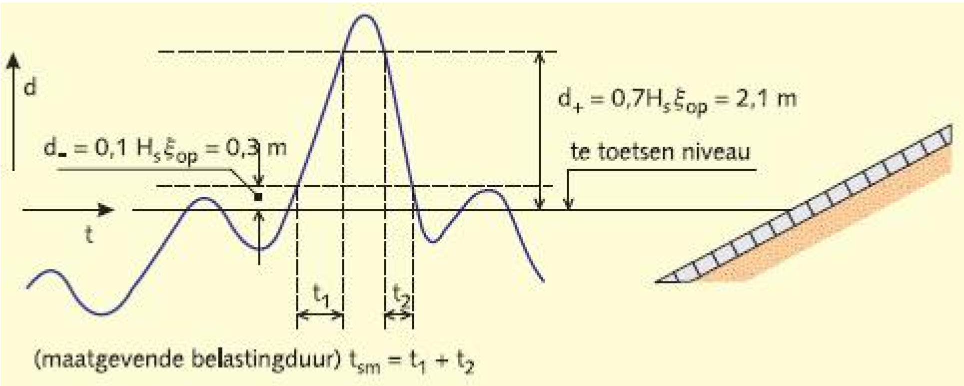

Figure 2.16 Determination of normative duration of loading t_{sm} (Source: TAW, 2004)

Erosion through the underlayers

The contribution of the clay layer to the safety of the defence structure is considered to be the residual strength of the grass cover. This failure mechanism and the assessment of the safety is the same as the mechanism ‘erosion through underlayer’ for placed block revetments. See Section 2.3.2. for details.

Shear

A distinction is made between shear in the outer slope and shear in the inner slope. For shear in the outer slope the failure mechanism and the assessment is the same as in the case of a placed block revetment. See Section 2.3.2. for details.

If the overtopping discharge is more than 0.1 l/m/s the stability of the inner slope should be assessed for the overtopping water that infiltrates in the soil structure. The way the inner slope can become instable by overtopping water is, amongst others, dependent on the composition of the structure. Two characteristic situations can be discerned:
- a soil structure with a clay core, covered with a layer of clay with a high root density.
- a soil structure with a sandy core, covered with a layer of clay.

Instability of structure with a clay core

The permeability of the clay at the surface is, as a consequence of little cracks and rootage higher than the permeability of the clay in the core of the structure. The water that infiltrates during overtopping will cause an increase in the water pressure on the interface between the core and the cover layer. This increase in water pressure will reduce the particle pressure which can, in the case of a relatively steep slope, cause the top layer to slide down. Besides, due to the absorption of water the cover layer can become softened which might also lead to local shear of the slope.

Instability of structure with a sandy core
If the core of the structure has a much higher permeability than the cover layer (in the direction perpendicular to the slope) infiltration of the overtopping water might lead to an increase of the phreatic level in the soil structure. An increase of the phreatic level might lead to superficial shear of the top layer and/or washing out of the sand underneath the top layer.

If the overtopping discharge is sufficiently low (< 0.1 l/m/s) or the steepness of the inner slope is less than 1:4 the inner slope is considered to be 'safe'. For other cases a more detailed and advanced testing should be performed. In practice this will mean bringing in scale model testing or mathematical methods based on Bishop's slip circle method. This method takes the equilibrium of moments in the total sliding part and the that of the vertical forces in the separate constituting lamellae into account. The horizontal equilibrium of forces is not considered. It is assumed that, that the occurrence of a slip circle implies failure.

2.4.4 Identification of input parameters and uncertainties

In section 2.4.3 the input parameters are given for all different failure mechanisms. For most cases the wave parameters and the course of the water level were considered most important. These input parameters form a first source for uncertainties. Since the limit state equations are all based on empirical tests, the coefficients in these equations form a second source of uncertainty. As mentioned in section 2.4.3 the age of the cover layer (*i.e.*, the development of the grass) is also a source of uncertainty.

2.5 Loose units (sand, gravel, riprap)[1]

2.5.1 Introduction

The stability of loose materials, from sand to rock, is investigated rather extensively and the proper design criteria are available. However, the stability of randomly dumped quarried rock can often be substantially improved by taking special measures in the form of composite systems, such as grouting, pitched stone, mattresses, *etc*.

Gravel and natural stone material are used for dyke construction as top layers for the slope or toe protection and as foundation/filter layer underneath the revetment. The criteria for these materials concern mainly the grain sizes, the strength and the weathering of the stone material and the unit weight (of importance for the resistance against currents and wave action). Requirements for grain sizes in relation to filter purposes are based on general filter criteria, which are (Lindenberg & De Groot, 1998):

- suffosion : $D_{15} / d_{85} < 4 - 5$
- permeability : $D_{15} / d_{15} < 4 - 5$
- internal erosion : $U = D_{60} / d_{10} < 10$

In which U is the uniformity factor and D represents the grain diameter of the upper layer consisting of coarser material (filter layer), d the grain diameter of the base layer consisting of finer material.

[1] This section is mainly based on CUR (1995), Van der Meer (1998) and TAW (1999)

2.5.2 Inventarisation of failure modes and fault tree

With regard to revetments consisting of loose units the following critical failure modes should be considered (CUR, 1995):

- Movement of cover layer elements (initiation of motion): Waves and currents determine the lift and drag forces acting on the stones of the cover layer. The inertial forces are also determined by the stone characteristics. The stone weight, but also the forces due to friction and interlocking with other stones, are the stabilizing forces (Figure 2.17). The (loss of) balance of all these dynamic forces may result in a great variety of the above mentioned stone movements (Sections 5.2.1 – 5.2.5 Rock Manual). These responses may be allowed for in the design, but care is needed to avoid responses large enough to initiate other failure modes such as damage of the filter layer.

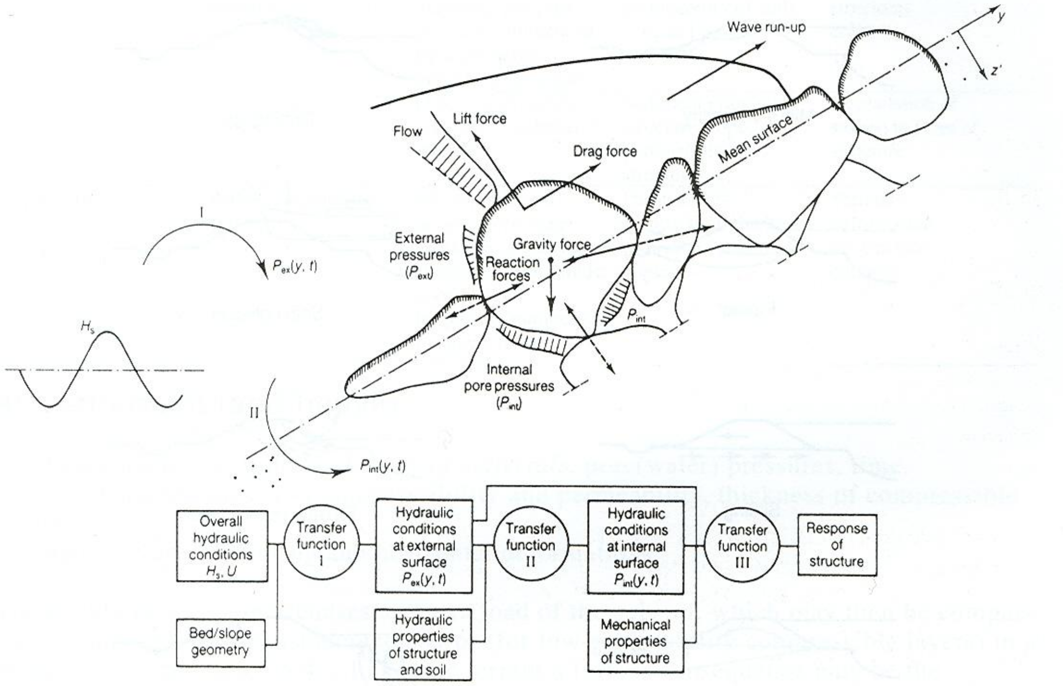

Figure 2.17 A rock system and it's responses (Source: CUR, 1995)

- Sliding of parts of the structure (transport of material): The stability of the rock slope is determined by the slope angle, specific weight, pore pressures and by the internal friction and cohesion (interlocking). Horizontal accelerations are also important during earthquakes (Section 5.2.6 Rock Manual) or wave shock loading. Sliding can occur anywhere along (local) failure planes in the structure and/or subsoil where the effective shear resistance is not sufficient, but preferentially along interfaces between different materials (*e.g.*, armour and sublayer/geotextile) because the local friction is reduced here. Sliding of an entire structure (river bank, breakwater), often including subsoil, is referred to as overall stability. The subsoil also takes part in supporting the structure and the generation of excess pore pressures and liquefaction in any fine layers beneath rock structures may thus endanger the stability of the rock structures (*e.g.*, toe structure). Excess pore pressures can be caused by dynamic loading or by rapid draw-down of the external water level (Section 5.6.3.2 Rock Manual). Crest structures may also move (slide) under wave loading when the friction between the structure and the underlying rock is not sufficient. Local sliding of a toe structure but also overall sliding can be initiated by excessive scour development.

- Erosion (profile deformation): Waves and currents cause resulting water movements near the sea bed, which may generate a sediment transport. Interactions with the structure (wave reflection, currents, generation of turbulence) may affect the natural sediment transport of bed or beach materials. Relative to the natural sediments most structures can be considered to be rigid and non-erodable, although some may be impermeable to sediment, and therefore only partly stop the transport processes. Local scour may lead to slopes, which will, provided they are sufficiently steep, initiate sliding. Erosion of sediments, despite of the presence of a bed protection can be the result of malfunctioning of the cover layer and/or filter layers. This is a mechanism which may follow from damage of the cover layer.

The three failure mechanisms mentioned above can occur all three simultaneously, although the first mechanism (movement of cover layer elements) will often be the predecessor of the second mechanism (sliding of parts of the structure). Failure can be expressed as by:

{failure} = {instability of revetment OR erosion}

Instability of the revetment can be expressed by:

{instability of revetment} = {movement of top layer elements OR sliding of parts of structure}

2.5.3 Analysis of failure modes and derivation of limit states

Movement of cover layer elements (initiation of motion)

During the last 50 years, many methods have been developed for the prediction of rock or grain size of top layer elements designed for wave attack. Three of them will be treated in more details: the Hudson formula (used in the Shore Protection Manual, 1984), the formula derived by Van der Meer (1998) and the formula derived by Van Gent (2003).

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

$$\frac{H_s}{\Delta D_{n50}} = \frac{\left(K_D \cot\alpha\right)^{1/3}}{1.27} \tag{2.48}$$

in which:
K_D = stability coefficient [-]
H_s = significant wave height of the incident waves at the toe of the structure [m]

For design purposes it might be acceptable that 0 to 5% of the armour stones are displaced from the region between the crest and a level of one wave height below still water; the wave height to be used for this purpose could be the design wave height.

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

With Equation (2.48) and the above described values for K_D an armour size can be calculated corresponding with 0 to 5% damage. Higher damage percentages have been determined as a function of the wave height for several armour types. Table 5.2.2.5.a shows $H_s/H_{s;D=0}$ as a function of the damage percentage D. H_s is the design wave height corresponding to damage D and $H_{s;D=0}$ is the design wave height corresponding for 0 to 5 % damage, generally referred to as “no damage” condition.

Table 2.4 Hs/Hs;D=0 as a function of cover-layer damage and type of armour type

Armour type	Relative wave height	Damage D (%)[1]						
		0 – 5 %	5 – 10%	10 – 15 %	15 – 20 %	20 – 30 %	30 – 40 %	40 – 50 %
Quarry stone (smooth)	$H_s/H_{s;D=0}$	1.00	1.08	1.14	1.20	1.29	1.41	1.54
Quarry stone (rough)	$H_s/H_{s;D=0}$	1.00	1.08	1.19	1.27	1.37	1.47	*1.56*[2]

[1] all values for breakwater trunk, randomly placed armour in two layers and non-breaking waves on the foreshore
[2] italic values are interpolated or extrapolated

The use of Equation (2.48) is for situations with a fixed damage level, namely 0 to 5% of the armour stones displaced out of the region of primary wave attack. The use can be extended for other damage percentages with

Table 2.4.

Van der Meer (1988)
For deep water conditions Van der Meer (1988) derived formulae to predict the stability of rock armour in straight rock slopes with crests exceeding the maximum run-up level. These formulae were based, amongst other work, on earlier work by Thompson and Shuttler (1975) and a large amount of model tests for which the majority of them were performed with relatively deep water at the toe. The formulae make use of a distinction between “plunging conditions” and “surging conditions”:

For “plunging conditions” ($\xi_m < \xi_c$):

$$\frac{H_s}{\Delta D_{n50}} = c_{plunging} \xi_m^{-0.5} P^{0.18} \left(\frac{S}{\sqrt{N}} \right)^{1/5} \tag{2.49}$$

and for “surging conditions” ($\xi_m \geq \xi_c$):

$$\frac{H_s}{\Delta D_{n50}} = c_{surging} \xi_m^{P} P^{-0.13} \tan\alpha^{-0.5} \left(\frac{S}{\sqrt{N}} \right)^{1/5} \tag{2.50}$$

in which:

N = number of incident waves at toe [-]
ξ_m = $\tan\alpha / \sqrt{(2\pi/g \cdot H_s/T_m^2)}$ = surf similarity parameter using the mean wave period T_m [-]
$c_{plunging}$ = surging parameter = 6.2 [-]
$c_{surging}$ = surging parameter = 1.0 [-]
P = permeability parameter (between 0.1 and 0.6) [-]

Note that the use of a geotextile reduces the permeability which may cause that larger material is needed than without a geotextile. See Figure 2.18 for the determination of the value of P.

The transition from plunging to surging waves is derived from the structure slope (and not from the slope of the foreshore), and can be calculated using a critical value of ξ_c:

$$\xi_c = \left[\frac{c_{plunging}}{c_{surging}} P^{0.31} \sqrt{\tan\alpha} \right]^{\frac{1}{P+0.5}} \tag{2.51}$$

For slope angles more gentle than 1:4 (cot$\alpha \geq 4$) both Equation (2.49) and (2.50) are to be used, irrespective of whether the surf-similarity parameter ξ_m is smaller or larger than the transition value ξ_c. It is recommended to use the most conservative approach of Eqs (2.49) and (2.50), *i.e.* the equation leading to the largest stone diameter. Because of this, for relatively small wave steepnesses a discontinuity can occur at cotα=4. Users of these formulae should be aware of this feature in the formulae.

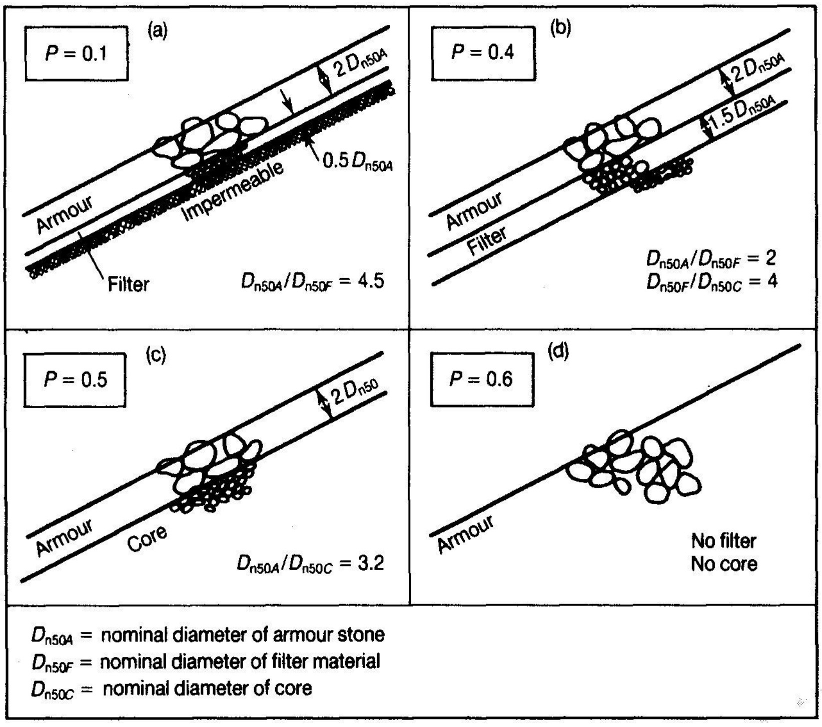

Figure 2.18 Permeability factor P for the formulae by Van der Meer (1988)

It should be noted that Eqs **Error! Reference source not found.** and **Error! Reference source not found.** are limited to a single storm event. The damage levels can be characterised as follows:

- Start of damage; corresponding to “no damage” in the formula by Hudson (1953, 1959);
- Intermediate damage;
- Failure; corresponding to reshaping of the armour layer such that the filter layer under a 2 D_{n50} thick armour layer is visible.

The limits of S depend mainly on the slope angle of the structure. For a 2 D_{n50} thick armour layer the values in Table 5.2.2.5.b can be used.

Table 2.5 Design values of S for a 2 Dn50 thick armour layer

Slope	**Damage level *S* (-)**		
cotα	start	intermediate	failure
1.5	2	3 - 5	8
2	2	4 - 6	8
3	2	6 - 9	12

4	2	8 - 12	17
6	2	8 - 12	17

Although a damage level of S=2-3 is often used for design purposes, in some cases it might be a feasible approach to apply higher damage levels of S=4-5. This depends for a great deal on the desired life cycle of the structure.

Van Gent *et al.* (2003)
The data-set described in Van Gent *et al* (2003) includes conditions with shallow water and conditions with deep water. This data-set was used to obtain the modifications to the formulae by Van der Meer (1988) as described above. This data-set was also used to obtain a more simple stability formula. This formula can be used if no, or not sufficiently accurate, information is available on input parameters such as the wave height $H_{2\%}$, the wave period $T_{m-1,0}$, or the permeability factor P. The simple stability formula by Van Gent *et al* (2003) reads:

$$\frac{H_s}{\Delta D_{n50}} = 1.75 \cot\alpha^{0.5} \left(1 + D_{n50-core} / D_{n50}\right) \left(\frac{S}{\sqrt{N}}\right)^{1/5} \qquad (2.52)$$

The influence of the permeability of the structure is incorporated by using the ratio $D_{n50\text{-}core} / D_{n50}$. The influence of filters is not accounted for in this ratio, which means that no filter or a rather standard filter of 2-3 layers thick is assumed here. Note that the use of a geotextile reduces the permeability which may cause that larger material is needed than without a geotextile. When the core consists of rock material with a very wide grading, it is recommended to use the $D_{n15\text{-}core}$ (which corresponds in most cases reasonably well to the lower limit of the grading) instead of the $D_{n50\text{-}core}$. Equation (2.52) is based on the same tests as the modification of the formulae by Van der Meer (1988) as proposed in Van Gent *et al.* (2003). This stability formula can be used for shallow water conditions, as well as for deep water conditions.

Sliding of parts of the structure (transport of material)[2]

The sliding of revements under wave attack is often more complicated in view of the low permeability of the cover layer comparing to the subsoil or at least to a filter layer just underneath the cover layer. With many revetments stability against sliding requires a structure at the toe or anchoring at the upper end of the slope protection. Formulae have been developed to calculate the force at the toe structure and the anchoring force.

Erosion (profile deformation)

2.5.4 Identification of input parameters and uncertainties

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.

The reliability of the empirical formulae depends on the differences due to random behaviour of rock slopes, accuracy of measuring damage and curve fitting of the test results. Van der Meer and Van Gent *et al.* give standard deviations of their empirical coefficients. They differ from 6.5 – 8% for the formula of Van der Meer and Van Gent *et al.* to 18% for Hudson and approximately 10% for Van Gent *et al.*

[2] This section is mainly based on CUR (1995).

2.6 Asphalt revetments[3]

2.6.1 Introduction

Asphalt is a visco-elastic material which means that it's properties vary with temperature and duration of the loading. This makes asphalt very suitable for hydraulic applications since it can withstand short duration loadings such as wave impacts firmly and at the same time can adjust to loadings with a longer duration such as settlements of the subsoil. Furthermore, asphalt mixes act as a plate and therefore the construction thickness can be limited compared to traditional constructions such as concrete blocks, loose stones, *etc*. Several mix types can be used in hydraulic engineering, *e.g.* asphaltic concrete, mastic, grouting mortars, dense stone asphalt, lean sand asphalt, open stone asphalt. Of these the following mixtures can be considered sand and water tight ('closed asphalt'): mastic, grouting mortars and dense stone asphalt. Open stone asphalt and lean asphalt are 'open asphalt types' and hence water permeable. For the following, the different asphalt mixtures are subdivided in the classes 'open' or 'closed'. No further subdivision are made.

The various applications of asphalt mixes in hydraulic engineering are (Agema, 1984):

1. protection against currents and waves:
   - slope and crest revetments of dykes, breakwaters, groynes, *etc.*
   - revetments of harbour basins, banks of canals, *etc.*
   - bottom and toe protection of banks, sluices, storm barriers, *etc.*
2. sealing or reducing water transport:
   - slope and bottom cover canals, reservoirs to reduce or prevent loss of water. In this respect also bituminous cores are applied
   - control of ground water levels in adjacent areas
   - prevention of pollution of (ground)water and soils in nearby areas
   - reduction of ground water flow, levels and pressures in and under dyke bodies, dams, sluices, *etc.*
3. bunds: application of lean sand asphalt as bund or core material under circumstances where sand is unable to resist the hydraulic forces.
4. filter constructions: application as filter material as a transition between the sand core of a dyke onto an open revetment or between a sand replenishment behind a rubble mound dam.
5. combination: combination of the functions mentioned above.

In the past the only function of the revetment to be dealt with in the design was the protection of the dyke or the bank against erosion. Nowadays more functions, such as traffic, landscape/ecology and recreation, are of importance which should be taken into account when designing the revetment. It should be noted that often asphalt revetments are one-layer systems. One layer has to fulfil all functions. In some cases more-layer systems occur. Then the relevant functions can be fulfilled by more layers. Within the present research only the water defence function will be taken into account. The water defence function can be subdivided into protection against erosion and water permeability.

The functions of the revetment can differ depending on the location on the dyke or bank. Therefore the revetment can be subdivided into different zones, each with there own hydraulic loading.

<u>River dykes</u>:
Asphalt is mainly applied as a revetment on the outer slope of a river dyke at locations where a grass cover provides insufficient protection. The following hydraulic loading zones can be distinguished:
A. River dyke with a low foreshore (see Figure 2.19):
- zone I: zone constantly below the water level. This zone is loaded by currents and waves.
- zone II: zone between the mean high water level and mean low water level. This zone is only present when the river dyke is in a tidal area. This zone is frequently loaded by waves and currents. After a period with high river discharges uplift pressures may develop here.

[3] This section is mainly based on Van Herpen (1998) and TAW (2002b).

- zone III: zone between the mean (high) water level and the design level. This zone is subject to waves and currents. Loeading is less frequent but heavier.
- zone IV: zone above the design level. This zone is subject to wave run-up.

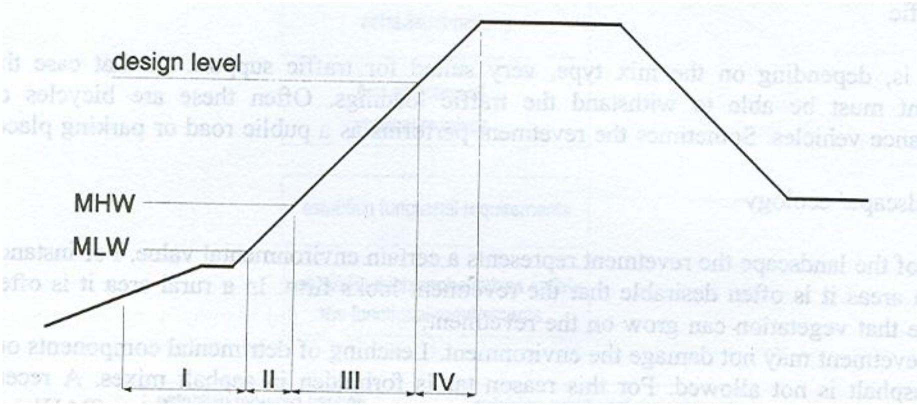

Figure 2.19 Hydraulic loading zones on riverdyke with low foreshore (Source: Van Herpen, 1998)

B. River dyke with a high foreshore (see Figure 2.20):
- zone III: zone between the toe of the dyke and the design level. This zone is subject to waves and currents. In the lower parts uplift pressures may occur.
- zone IV: zone above the design level. This zone is subject to wave run-up.

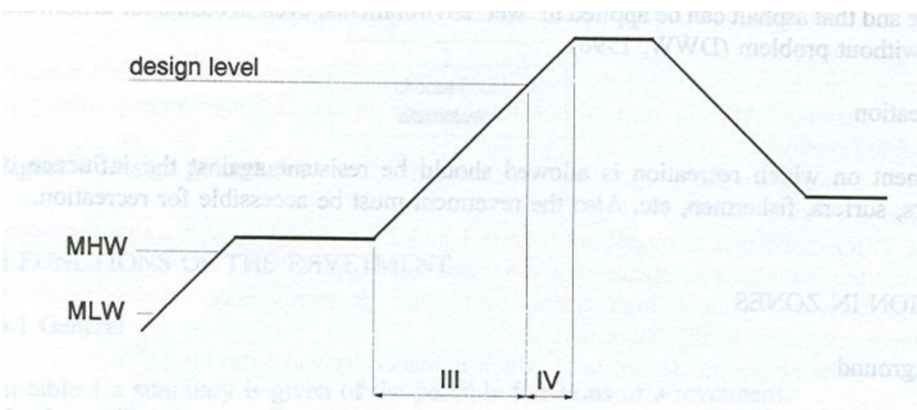

Figure 2.20 Hydraulic loading zones on riverdyke with high foreshore (Source: Van Herpen, 1998)

Lake dykes:
At lake dykes normally a rather constant water level is present. The the water level will be higher only occassionally. The following hydraulic loading zones can be distinguished (see Figure 2.21):

- zone I: zone below the mean water level. This zone is loaded by waves and currents.
- zone II: zone between the mean water level and the design level. This zone is occasionally loaded by larger waves and currents.
- zone III: zone above the design level. This zone is loaded by wave run-up.

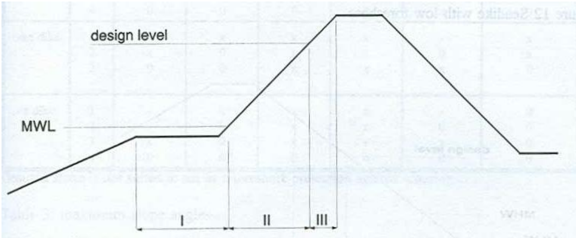

Figure 2.21 Hydraulic loading zones on lake dyke (Source: Van Herpen, 1998)

Sea dykes:
A distinction can be made between dykes with a higher and dykes with a lower foreshore.
A. Sea dyke with a low foreshore (see Figure 2.22):

- zone I: zone constantly below the water level. This zone is loaded primarily by currents. A foreshore protection, if present, can be threatened by scouring. Possible uplift of the protection mattress by waves or currents should be taken into account.
- zone II: zone between the mean high water level and mean low water level. This zone is frequently loaded loaded by waves and currents. After a high water level uplift pressures may develop under an impermeable revetment.
- zone III: zone between the mean (high) water level and the design level. This zone is subject to waves and currents. Loeading is less frequent but heavier.
- zone IV: zone above the design level. This zone is subject to wave run-up.

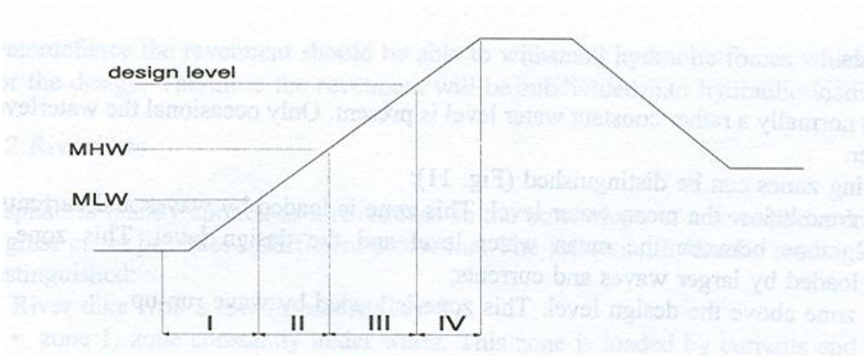

Figure 2.22 Hydraulic loading zones on sea dyke with low foreshore (Source: Van Herpen, 1998)

B. Sea dyke with a high foreshore (see Figure 2.23):

- zone I: this zone is a combination of zone 1 and 2 for a sea dyke with a low foreshore.
- zone III: the zone between mean high water level and the design level. This zone is loaded by waves and currents. After a high water level uplift pressures may develop under an impermeable revetments in the lower parts.
- zone IV: zone above the design water level which is loaded by wave run-up.

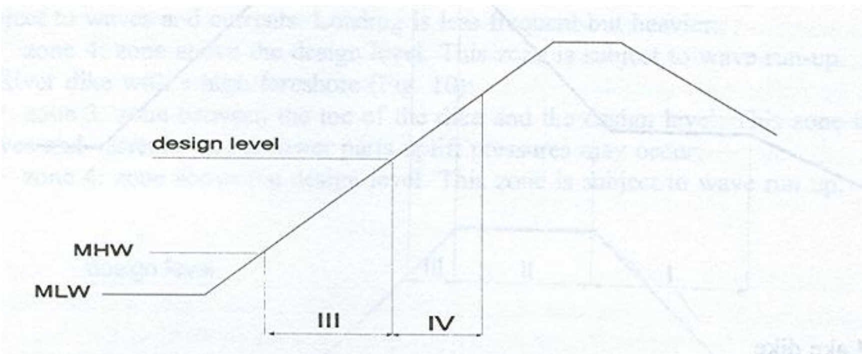

Figure 2.23 Hydraulic loading zones on sea dyke with higher foreshore (Source: Van Herpen, 1998)

Table 2.6 shows the different hydraulic loadings for the revetment in each zone.

Table 2.6 Hydraulic loadings per zone and mix type

dyke type	zone	mix type	static uplift	wave impacts	currents	scouring	dynamic uplift
river dyke	I	closed			x	x	x
		open			x		
	II	closed	x	x	x		
		open		x	x		
	III	closed	x	x	x		
		open		x	x		
	IV	closed			x		
		open			x		
lake dyke	I	closed			x		
		open			x		
	II	closed	x	x	x		
		open		x	x		
	III	closed			x		
		open			x		
sea dyke	I	closed			x	x	x
		open			x	x	
	II	closed	x	x	x		
		open		x	x		
	III	closed	x	x	x		
		open		x	x		
	IV	closed			x		
		open			x		

2.6.2 Inventarisation of failure modes and fault tree

The Dutch guideline for the assessment of safety (TAW, 2004) makes a distinction into four failure mechanisms:
1. transport of material (washing out of the soil body) (AT)
2. failure of the top layer due to wave impact (as a consequence of fatigue) (AW)
3. uplifting of the top layer due to wave pressure (possibly in combination with the under layer) (AU)
4. failure of the sub layer after failure of the top layer (AS)

The first mechanism (AT) is caused by currents and waves. This mechanism is relevant for all zones. Failure on this mechanism leads to failure of the revetment. Failure on the second (AW) or third mechanism (AU) does not lead to immediate failure of the structure. Only if these mechanisms occur in combination with the fourth mechanism (AS) the revetment will fail.

The top-event in the fault tree can then be expressed by:

{failure} = {transport of material OR 'other mechanisms'}

'Other mechanisms' are a composition of the following event:

{'other mechanisms'} = { {wave impact OR uplift pressures} AND failure of sub layer}

Note that the event 'other mechanism' is not relevant for asphalt revetments in zone IV. For revetments in these zone the only failure mechanism that can occur is transport of material (AT). The failure mechanism due to wave impacts (AW) is only relevant for revetments in the wave impact zone (zone II and III). For open revetments the mechanism 'uplift pressures' (AU) is not relevant.

2.6.3 Analysis of failure modes and derivation of limit states

Transport of material (AT)

The resistance to transport of material is determined by the ratio of the size of the pores in the top layer en the size of the loose material in the sub layer. There is no general quantitive assessment for this failure mechanism. TAW (2004) advises visual inspection in situ, where possible local damage on the revetment can be classified according to length of possible cracks, seams or even gaps in the revetment.

Wave impact (AW)

For this failure mechanism the loading is formed by the wave attact during a certain normative storm, expressed in the time signal of the significant wave height H_s and the spectral peak period T_p. Dependent on the form of the wave spectrum the averaged wave period is 10 to 30% lower than the peak period. The course of time of the surface elevation is of importance for the fatigue of the revetment.

Plate type asphalt constructions (asphalt concrete, open stone asphalt, mastic and fully grouted stone) are loaded by impact forces. The situation can be schematised as follows (see Figure 2.24):

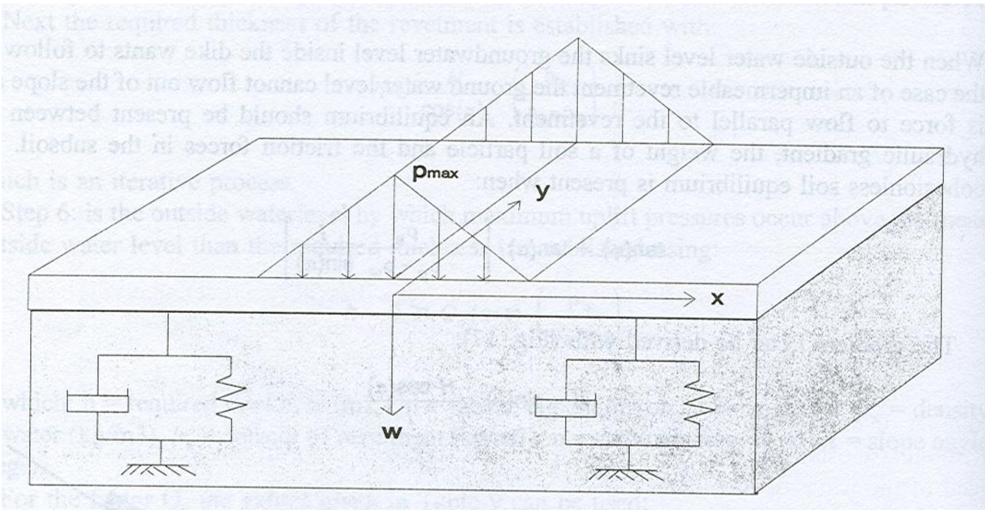

Figure 2.24 Schematization of wave impact on an asphalt revetment (Source: Van Herpen, 1998)

The maximum bending stress σ in the revetment is (at x = 0):

$$\sigma = \frac{p_m}{4\beta^2 \cdot \beta z}\left[1-\exp(-\beta z)\cdot\left(\cos(\beta z)+\sin(\beta z)\right)\right]\cdot\frac{6}{h^2} \qquad (2.53)$$

in which

$$\beta = \sqrt[4]{\frac{3c(1-\nu^2)}{S\cdot h^3}} \qquad (2.54)$$

where:

p_m	= maximum pressure	[N/m^2]
h	= revetment thickness	[m]
z	= half width triangular load = 0.5 H	[m]
c	= modulus of subgrade reaction	[N/m^3]
S	= stiffness modulus of asphalt	[N/m^2]

ν = Poissons' ratio of asphalt [-]

Inserting the maximum allowable bending stress σ_b into Equation (2.53) will result in the required thickness. However, under normal conditions (average wave heights, normal asphalt quality) this will result into very thin asphalt thicknesses. In that case a minimum thickness should be adopted to ensure that the asphalt layer is (at least) a solid plate. For asphalt concrete and asphalt mastic this minimum thickness will be 10 – 15 cm, for open asphalt 12 cm when using stones with a diameter of 20 – 40 mm and 10 cm when using stones with a diameter of 16 – 22 mm. For fully grouted stones this is at least 1.5 times the nominal stone size.

The wave impact is in fact a pressure acting over a certain width. It is schematized as a triangular load in which the maximum pressure p_{max} is:

$$p_m = \rho_w \cdot g \cdot K_{pu} \cdot H_s \tag{2.55}$$

where
K_{pu} = impact factor depending on the slope angle [-]

The width of the impact load is estimated to be equal to the height of the wave H. Table 2.5 shows values of K_{pu} for different slope angles.

Table 2.7 Impact factor for wave loadings on asphalt revetments (Source: Van Herpen, 1998).

slope angle	impact factor K_{pu}
1:2	3.2
1:3	4.25
1:4	5.0
1:6	3.3

The variable H represents the height of a single wave. In reality a revetment will be loaded by irregular waves with different heights and frequency. In the design the significant wave height H_s can be used as H.

Asphalt is sensitive to fatigue, which means that a load of a certain magnitude may occur a number of times before the asphalt collapses. When using H_s as the representative wave height it should be established how many times it must occur in order to represent the total fatigue load to the wave field. For Dutch circumstances this number can be established with:

$$n_s = 3.75 \cdot \frac{T}{3.5 \cdot H_s^{0.5}} \cdot 0.1 \tag{2.56}$$

where
n_s = number of loadings of H_s in order to represent the wave field [-]
T = load duration [s]

The subsoil is characterized by the modulus of subgrade reaction (see Table 2.8):

Table 2.8 Modulus of subgrade reaction (Source: Van Herpen, 1998).

subsoil		modulus of subgrade reaction c (N/m^3)
sand	- very well compacted (98)	3.10^8
	- well compacted (95)	2.10^8
	- moderately compacted (90)	1.10^8
clay		5.10^7
lean sand asphalt		$> 5.10^8$

The allowable bending stress of the asphalt can be established with the help of Figure 2.25. Values of the stiffness modulus can be found in Table 2.9. These values refer to asphalt of reasonable quality. The Poissons' ratio is 0.35. When calculating the thickness of a fully grouted stone layer the material properties of mastic can be taken. The revetment thickness calculated has to be multiplied with a factor 1.75 to achieve the required thickness.

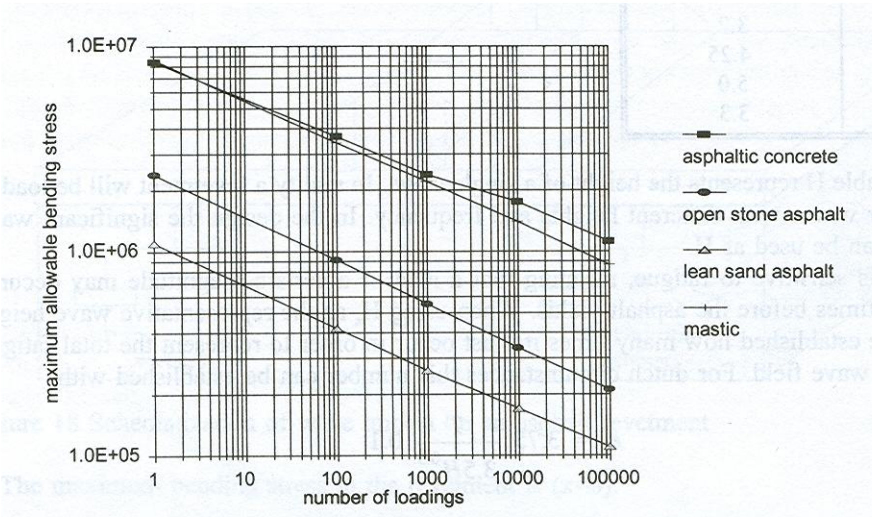

Figure 2.25 Allowable bending stress (Source: Van Herpen, 1998)

Table 2.9 Stiffness modulus of asphalt mixtures (Source: Van Herpen, 1998).

mix type	stiffness modulus (N/m^2)
asphalt concrete	7.10^9
open stone asphalt	7.10^8
mastic	1.10^9

For partially grouted stone the same design rule can be applied as for loose stones. An upgrading factor is to be used to include forces on the grouting mortar. The following formula can be used:

$$\frac{H_s}{\Delta D_n} \leq \psi_u \cdot \Phi \cdot \frac{\cos\alpha}{\xi_z^{\,0.5}} \tag{2.57}$$

with

$$\xi_z = \tan\alpha \cdot \frac{1.25 T_z}{H_s^{\,0.5}} \tag{2.58}$$

where:
Φ = 2.25 [-]

T_z	= mean wave period	[s]
D_n	= mean particle size of stone	[m]
ψ_u	= upgrading factor	[-]

When 30% of the surface is filled with mortar (surface grouting) the upgrading factor ψ_u varies between 1.0 and 1.05. When 60% of the surface is filled (partial grouting) the upgrading factor is between 1.5 and 1.9. For a small (homogeneous) gradiation of riprap and good maintenance a factor ψ_u = 2.0 can be applied.

Uplift pressures (AU)

Below a water impermeable revetment hydraulic uplift pressures under the revetment may occur when the outside water level is lower than the inside ground water level in the dyke. The design is based on three criteria, the uplift criterion, the sliding criterion and the equilibrium criterion. For mastic slabs a fourth criterion exists (dynamic uplift pressures).

Uplift criterion:

The design procedure consists of five steps as indicated below. Note that the method presented here provides safe values for the required layer thickness. More accurate values of the maximum uplift pressures can be established with non-stationary ground water calculation programs.

1. the design ground water level is the average of the design outside water level and the mean or normal water level. For sea dykes the mean water level is the average of mean high water level and mean low water level. The distance between the design ground water level and the lower edge of the revetment is a + v (see Figure 2.26)

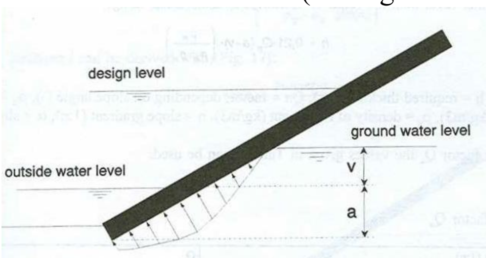

Figure 2.26 Variables a and v (Source: Van Herpen, 1998)

2. if the design ground water level is located below the revetment then a design for uplift pressures is not required.
3. the influence of an impermeable toe construction or foreshore protection must be taken into account by adding an additional length to (a + v): (a + v + q) or (a + v + r) (see Figure 2.27).

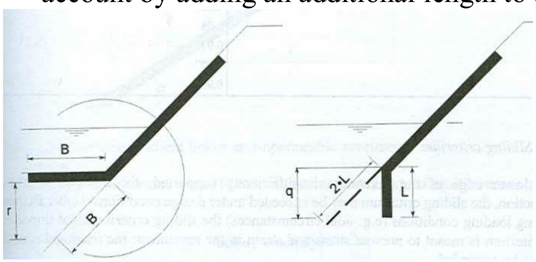

Figure 2.27 The influence of a toe construction or foreshore protection (Source: Van Herpen, 1998)

4. the outside water level at which the maximum pressure occurs is at the level 0.53*(a+v) below the design ground water level.
5. if the outside water level at which the maximum uplift pressure occurs is located below the average outside water level then the mean water level is taken equal to the outisde water level at which the maximum uplift pressure occurs. The maximum head difference (H) can now be calculated as:

$$H = \frac{v}{\pi} \cdot \arccos\left[2 \cdot \left(\frac{v + h \cdot \cos\alpha}{a + v}\right)^{\frac{\pi}{\theta}} - 1\right] \tag{2.59}$$

with:

$$\theta = \operatorname{arctg}(n) + \frac{\pi}{2} \quad (2.x) \tag{2.60}$$

The required thickness h of the revetment can now iteratively be estatblished with the following relation:

$$h \geq \frac{H}{\cos\alpha} \cdot \left(\frac{\rho_w}{\rho_a - \rho_w}\right) \tag{2.61}$$

where
ρ_a = density of the revetment (asphalt mixture) [kg/m^3]

6. If the outside water level by which the maximum uplift pressures occur are above the mean outside water level, then the required thickness can be calculated with:

$$h = 0.21 \cdot Q_n \cdot (a + v) \cdot \left(\frac{\rho_w}{\rho_a - \rho_w}\right) \tag{2.62}$$

where

Q_n	= a factor, depending on the slope angle α	[-]
R_w	= a reduction factor factor, depending on the slope angle α	[-]
n	= the slope gradient (1:n)	[-]

For the factor Q_n the following expression holds:

$$Q_n = \frac{0.96}{(\cos\alpha)^{0.25}} \tag{2.63}$$

If the revetment is closed at the bottom a reduction factor R_w should be applied.

$$h = 0.21 \cdot Q_n \cdot (a + v) \cdot R_w \cdot \left(\frac{\rho_w}{\rho_a - \rho_w}\right) \tag{2.64}$$

The value of this reduction factor can be graphically derived from Figure 2.25 (where z = a + v).

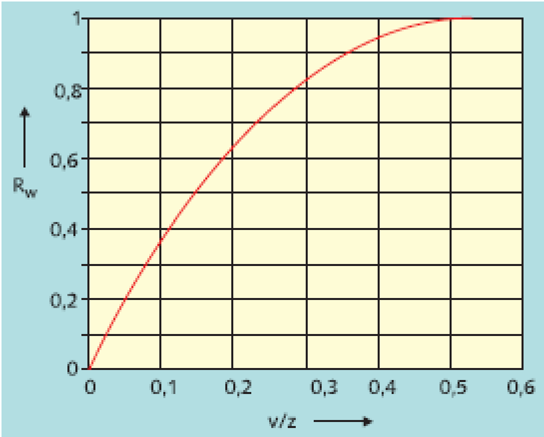

Figure 2.28 Reduction factor R_w (Source: TAW, 2004)

Sliding criterion:
If the lower edge of the revetment is sufficiently supported, for instance with a toe construction, the sliding criterion may be exceeded under design conditions. Under frequently occuring loading conditions (*e.g.* tidal circumstances) the sliding criterion is of importance. This criterion is meant to prevent stress and strain in the revetment. The frictional resistance criterion must be met. The required thickness of the revetment can be established by:

$$h \geq \frac{H}{\cos\alpha} \cdot \frac{1}{\frac{\rho_w}{\rho_a} \cdot \left(1 - \frac{\tan\alpha}{f}\right) - 1} \tag{2.65}$$

where
H = head difference [m]
φ = angle of internal friction of the subsoil [°]
θ = angle of internal friction between revetment and subsoil [°]
f = coefficient for friction [-]
for θ < φ: f = tan θ
for θ ≥ φ: f = tan φ

Equilibrium criterion:
When the outside water level decreases, there is a pressure on the ground water to follow. However, in case of an impermeable revetment the ground water cannot flow out of the slope and it is forces to flow parallel to the revetment. An equilibrium should be present between the hydraulic gradient, the weight of the soil particles and the friction forces in the subsoil. For cohesionless soil the equilibrium reads:

$$\tan\varphi = \tan\alpha \cdot \left[1 + \frac{\rho_w}{\rho_g - \rho_a} \cdot \frac{i}{\sin\alpha}\right] \tag{2.66}$$

where
ρ_q = density of the subsoil [kg/m^3]
The gradient i can be derived with:

$$i = \sin\alpha - \frac{H \cdot \cos\alpha}{l} \tag{2.67}$$

See Figure 2.29 for an explanation of the variables H and l.

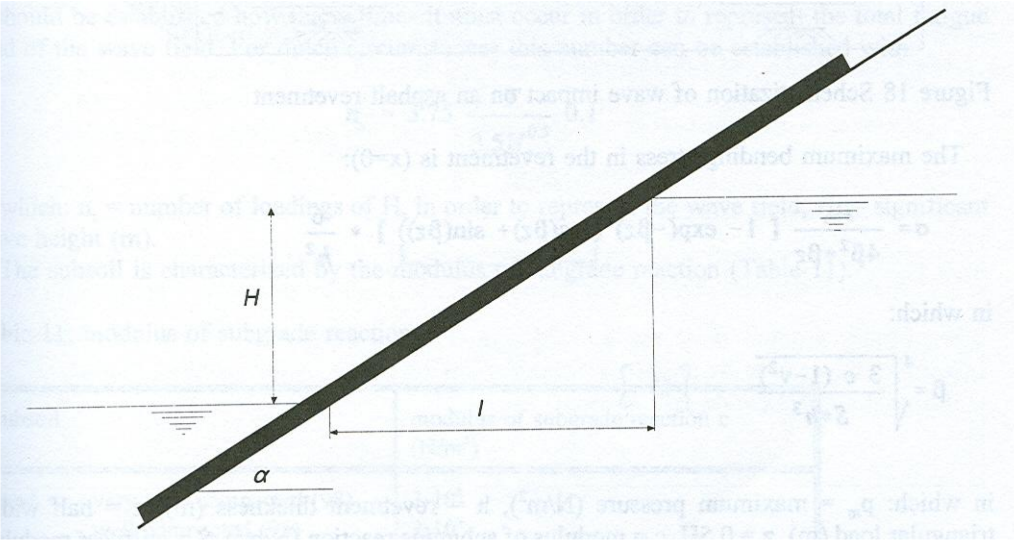

Figure 2.29 Hydraulic gradient underneath an impermeable revetment (Source: Van Herpen, 1998)

The worst case is present when $H/l \rightarrow 0$. In that case $i = \sin\alpha$, so equilibrium is guaranteed when:

$$\tan\alpha \leq \tan\varphi \cdot \left(1 - \frac{\rho_w}{\rho_g}\right) \tag{2.68}$$

This means that, *e.g.* in the case of sand, the slope angle should be no steeper than 1:4.

Dynamic uplift pressures:
For mastic slabs the dynamic uplift pressure can also be relevant. Differences in hydraulic pressures above and below the slab can occur due to alternating currents and waves. These pressure differences may also cause the slab to lift up. When this occurs rather frequently this may eventually lead to failure.

In the case of wave action two situations can be distinguished:

1. the wave length is larger than the length of the slab. In this case the following design criterion can be used: the maximum pressure difference must be smaller than the weight of the slab, or:

$$h \geq \frac{\rho_w}{\rho_a} \cdot \frac{H}{2} \tag{2.69}$$

2. the wave length is smaller than the length of the slab.
   Under normal circumstances uplift pressures are not relevant in this case. More important wil be possible scouring at the edges of the mat.

Failure of the sub layer after failure of the top layer (AS)

The assessment of the strength on the mechanism 'failure of the sub layer' is the same as in the case of placed block revetments. On the basis of the normative storm duration it can be computed how long it takes for the construction to fail. If this duration is longer than the duration of the loading, the residual strength of the revetment can be considered sufficient. See section 2.3.3 for details.

The residual strength of the top layer is dependent on the score on the other failure mechanisms of the top layer. If the score on 'uplifting' is sufficient and only the score on 'wave impact' is insufficient, the residual strength of the top layer can be computed with a mathematical model such as GOLFKLAP, which is a model that computes whether a layer is resistent to fatigue as a consequence of frequent wave impact loading (Klein Breteler & Coeveld, 2004). If the score on 'uplift pressures' is insufficient no residual strength of the top layer can be taken into account.

For asphalt revetment, generally no sublayers are present. This does especially hold for the location where the revetments is loaded during design conditions. The cases where some form of residual strength of a sublayer can be taken into account are limited. The residual strength of the sublayers is dependent on the material composition of the sublayer.

Clay:
For the assessment of the residual strength of a sublayer of clay the assessment procedure is the same as in the case of placed block revetments. See section 2.3.3 for details.

Sand asphalt:
Underneath an open asphalt revetment, sometimes a sublayer of sand asphalt exists. The residual strenght of this layer is dependent on the score on the failure mechanisms of the top layer. If the score on 'uplifting' is sufficient and only the score on 'wave impact' is insufficient, the residual strength of the top layer can be computed with a mathematical model such as GOLFKLAP. Note that the sand asphalt layer is not taken into account twice (as part of the top layer and as a separate sublayer). If the score on 'uplift pressures' is insufficient the sand asphalt layer can assumed to have failed as well.

Geotextiles:
Geotextiles have the function of being sandtight. The geotextile can give some minor additional safety to the construction but this cannot be quantified. In the assessment the residual strength of a sublayer of geotextiles can therefore not be taken into account.

Materials of loose grains:
For this type of sublayer the same holds a for geotextiles. Hence, the residual strength of this layer cannot be taken into account.

2.6.4 Identification of input parameters and uncertainties

The most important input parameter of an asphalt revetment is the thickness of the layer. The most important loading parameter is the pressure head difference, which is a function of the water levels (inside and outside the dyke) and the slope angle of the dyke.

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.

2.7 Alternative (open) revetments[4]

2.7.1 Introduction

In the present section the following alternative open revetments are treated:

1. Block mats:
A block mat is a slope that is made of blocks that are joined together to form a "mat" (see Figure 2.30). The interconnection may consist of cables from block to block, or of hooks connecting the blocks, or of a geotextile on which the blocks are attached with pins, glue or other means. The spaces between the blocks are usually filled with rubble, gravel or slag. Block mats are more stable than a setting of loose blocks, because a single stone cannot be moved in the direction perpendicular to the

[4] This section is mainly based on Klein Breteler *et al.* (1998), Klein Breteler and Pilarczyk (1998) and Pilarczyk (1998b)

slope without moving other nearby stones, This is the most important difference from a setting of clambed stones, where the presence of loose stones must always be taken into account.

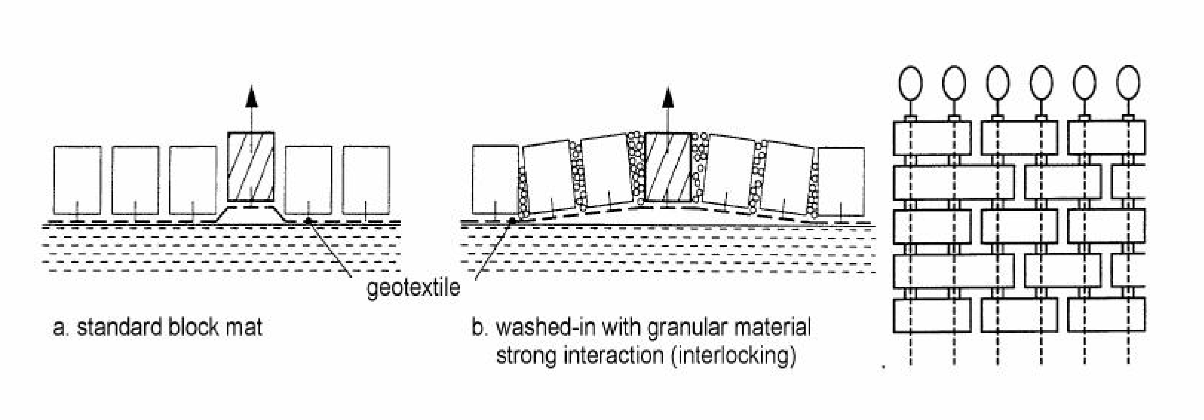

Figure 2.30 Example of block mats (Source: Klein Breteler et al., 1998)

2. Concrete-filled mattresses:
Characteristic of concrete mattresses are the two geotextiles with concrete or cement between them. The geotextiles can be connected to each other in many patterns, which results in a variety of mattress systems, each having its own appearance and properties. Some examples are given in Figure 2.31.

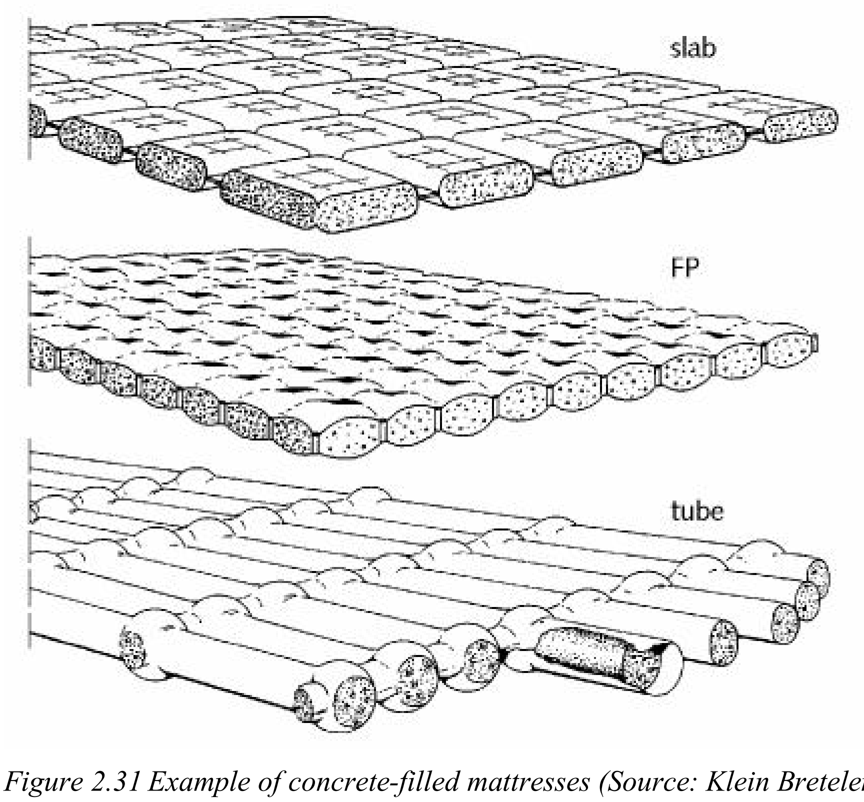

Figure 2.31 Example of concrete-filled mattresses (Source: Klein Breteler et al., 1998)

3. Gabions:
Gabions are made of rectangular baskets of wire mesh, which are filled with stones. The idea of the protection system is to hold the rather small stones together with the wire mesh. Waves and currents would have easily washed away the small stones, but the wire mesh prevents this. A typical length of gabions is 3 to 4 m, a width of 1 to 3 m and a thickness of 0.3 to 1 m. The gabions with small thickness (less then 0.5 m) and large length and width are usually called Reno-mattresses. An important problem of

this protection system is the durability. Frequent wave or current attack can lead to a failure of the wire mesh because of the continuously moving grains along the wires, finally cutting through. Another problem is the corrosion of the mesh. Therefore meshes with plastic coating or corrosion resistant steel are used. On the other hand the system is less suitable where waves and currents frequently lead to grain motion.

4. Geosystems:
Geotextile systems as bags, mattresses, tubes and containers filled with sand or mortar can be a good and mostly cheaper alternative for more traditional materials/systems as rock, concrete units or asphalt. Geosynthetics can be used for erosion prevention in revetments, but also as reinforcement of new or old dykes and for drainage, filters, impermeable membranes and the seperation of layers of construction material. Until very recently, geosystems were mostly applied as temporary structures. The reason for that was their relatively low resistance to the hydraulic loadings (wave and current attack), the lack of proper design criteria and low durability with respect to UV-radiation and vandalism. However, the increased demand in recent years for reliable methods for protective structures and the shortage of natural materials in certain geographical regions and/or limited dimensions and quality of available rock have led to the application of other systems (including geotextile systems) and to research concerning the design of these new systems. Where possible, the results of these research activities will be presented in the following section. However, compared to the other revetments the knowledge on geosystems is relatively limited.

2.7.2 Inventarisation of failure modes and fault trees

The revetments treated in the present section are for a great part subject to the same loading as the placed block revetments. Equations 2.2 – 2.5 can well be applied to compute the wave load and flow load stability. The major difference compared to placed block revetments is the soil-mechanical stability. There are three aspects of soil-mechanical stability:

- elastic storage
- liquefaction (softening)
- drop in the water level

Elastic storage

Elastic storage in the subsoil is connnected with the permeability and stiffness of the grain skeleton and the compressibility of the pore water (the mixture of water and air in the pores of the grain skeleton). Because of these characteristics, wave pressures on the top layer are passed on delayed and damped to the subsoil of the revetment construction and to deeper layers (as seen perpendicular to the slope) of the subsoil. This phenomenon takes place over a larger distance or depth as the grain skeleton and the pore water are stiffer. If the subsoil is soft or the pore water more compressible (because of the presence of small air bubbles) the compressibility of the system increases and large damping of the water pressures over a short distance may occur. Because of this, alternately water undertension and overtension may develop in the subsoil and corresponding to this an increasing and decreasing grain pressure. Elastic storage can lead to the following damage mechanisms (Stoutjesdijk, 1996):

- lifting of the top layer
- partial or full sliding of the top layer
- sliding of the subsoil (Figure 2.32).

For the stability of the top layer, elastic storage is particularly of importance if the top layer is placed directly on the subsoil without granular filter.

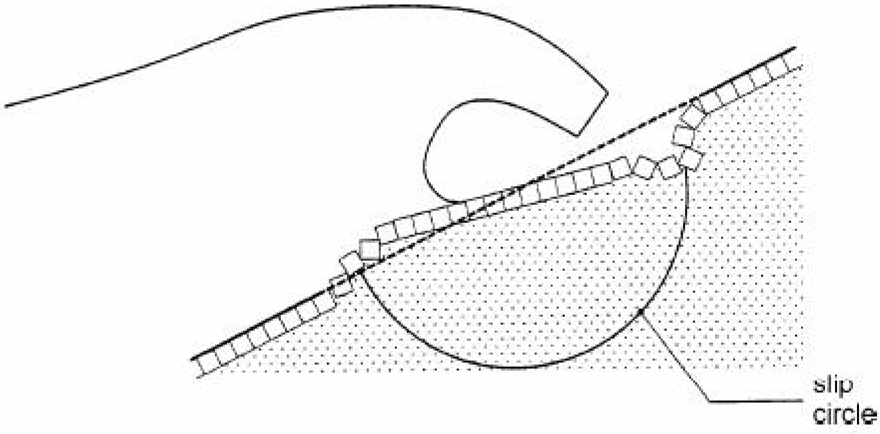

Figure 2.32 Schematised development of S-profile and possible local sliding in the base (Source: Klein Breteler et al., 1998)

The durability of the subsoil may be jeopardized if, because of elastic storage, the grain tension decreases so strongly that insufficient sheer stress 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 Figure 2.33 (more diagrams and details: see Klein Breteler and Pilarczyk, 1998). In these 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}. If the revetment construction consists of a top layer on a filter layer, the thickness of the filter layer may in these diagrams be partially or completely (depending on the type of revetment) added to the thickness of the top layer. The equivalent thickness D_{eq} is defined as:

$$D_{eq} = D + \frac{b}{\Delta_t} \tag{2.70}$$

where

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_t	= $(1-n).\rho_s + n.\ \rho_w$	[kg/m^3]
ρ_s	= density of the sand	[kg/m^3]

Δ_t is about 0.9 – 1.0 for sand-filled systems and 1.2 – 1.4 for block mats and concrete-filled systems.

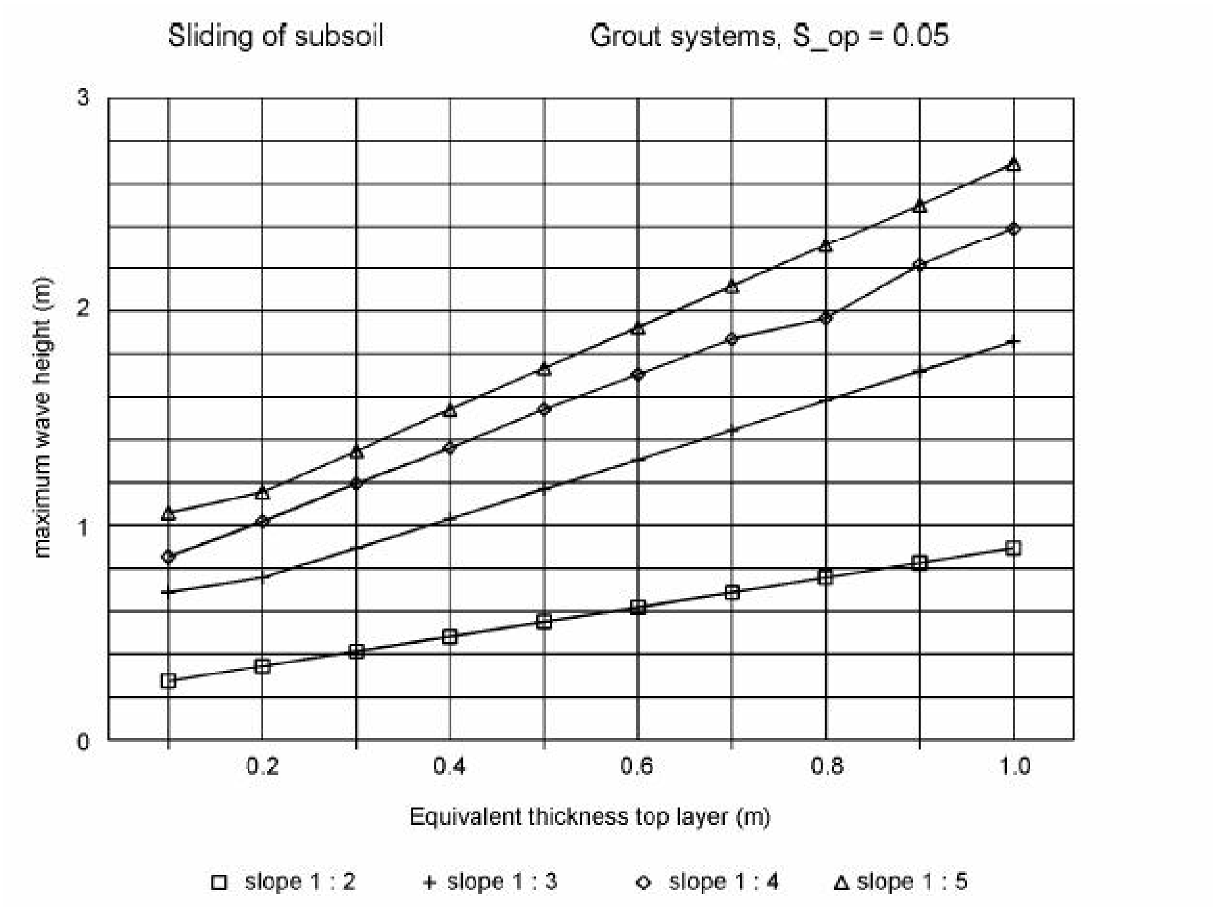

Figure 2.33 Geotechical stability, design diagram for grouted systems and H_s/L_{0p} = 0.05 (Source: Klein Breteler & Pilarczyk, 1998)

In case of a system placed on a geotextile on a clay layer (with sand underneath) the effect on the stability depends on the thickness of the clay layer b_c (= additional weight). However, the thin layers of clay may have a negative effect on the hydraulic gradients at the interface of clay and sand. This effect is accounted for in the equivalent thickness as follows:

$$D_{eq} = D + 0.8\frac{b_c}{\Delta_t} - 0.5 \tag{2.71}$$

In that case all failure mechanisms should be considered using the equivalent thickness as a reference parameter.

Liquefaction

A cyclic variable load causes compaction to occur in a layer of sand. This leads to a decreasing pore volume. The water in the pores is subjected to pressure and will start to run off. At first, water overtension occurs. This causes a decrease in the contact pressure between the grains and with this the resistance is sliding. Finally, the water overtension might become so large that the contact pressure between the grains falls away completely. This is called liquefaction or softening. The difference between liquefaction and elastic storage is that with liquefaction water overtension is connected with a plastic deformation of a grain skeleton instead of an elastic deformation. Water overtension through liquefaction occurs when the grain skeleton deforms plastically to a denser packing. From which follows that the dangers connected with liquefaction are smaller as the the subsoil is compacted better during construction.
The following design rules are suggested for constructions with a reasonably compacted subsoil:

- with a top layer on sand there is no danger of liquefaction if:
  - the slope gradient is less than or equal to 1:3
  - the slope gradient is less than 1:2 and the wave height H_s is smaller than 2 m.
  - the slope gradient is less than 1:2 and the subsoil is well-compacted.
- with a top layer on clay there is no danger of liquefaction
- with a top layer on a granular filter there is generally no danger of liquefaction

Drop in the water level

Through a drop in the water level a difference in the rise over the top layer may occur. A drop in the water level may occur as a result of tide or a ship passing through a waterway or canal. As with placed stone revetments, the resulting uplift is especially dangerous when the top layer is sanded up due to which the permeability of the top layer may decrease in time. No calculations need to be made on this phenomenon if the following relation holds:

$$\frac{\Lambda \sin\alpha}{2} \leq \Delta D \cos\alpha \qquad (2.72)$$

where

Δ	= (representative) relative density of the top layer	[kg/m^3]
D	= (representative) thickness of the top layer	[m]

Fault tree

The revetments considered here are subject to three categories of failure mechanisms: instability of the revetment itself, instability of the subsoil and durability problems. With regard to the first, this mechanism is mainly determined by the wave and flow loading. The second mechanism consists of the mechanisms mentioned above. The third mechanism is especially relevant for the gabions and geosystems.

The fault tree can now be formulated as such:

{failure} = { instability of the revetment OR instability of the revetment OR durability problems }

with
{ instability of revetment } = { lifting OR sliding }

Note that not all mechanisms are relevant for all types of revetments considered here. For block mats, *e.g.*, the normative mechanism is instability of the subsoil. There is relatively little danger of the top layer lifting or sliding.

2.7.3 Analysis of failure modes and derivation of limit states

Block mats

Stability of the block mat revetment

Because of the interaction between the revetment elements of block mats and interlock systems, and assuming a sound anchoring and toe construction, there is relatively little danger of the top layer lifting or sliding. For the soil-mechanical design of these systems, a shallow sliding in the subsoil is therefore normative (see Figure 2.32). The deformation of the slope into an S-profile is caused by the tranportation of underlying material during the lifting of (parts of) the block mat. When the interconnection of the mats is inadequate, the edges may turn back. When the anchoring is inadequate, sliding of the mat may occur. The most important difference from a conventional setting is that the subsoil may seriously deform, whereas the mat itself can still be reasonably intact. This means that the mat, even when seriously overloaded, will still provide significant protection to the subsoil, as a result of which the development of damage goes relatively slowly. Practical tests have shown that, even with serious overloading, block mats may have a residual strength of the duration of several storms.

For block mats the following values of F in the black-box formula (Eq. 2.3b) are recommended (Klein Breteler & Pilarczyk, 1998):

Table 2.10 Recommended values for the revetment parameter F for block mats

Type of revetment		F [-]
Linked blocks on sand		5 – 6
Linked blocks on clay	good clay	5 – 6
	mediocre clay	4.5 – 5
Linked blocks on a granular filter	favourable construction	5 – 6
	normal construction	4 – 5
	unfavourable construction	3 – 4

Soil mechanical stability of block mats

Elastic storage:

The soil mechanical is usually expressed in terms of design diagrams (*e.g.* as in Figure 2.33; for more diagrams see CUR/TAW, 1995). In a fresh clay subsoil the maximum water tension occurs so closely beneath the surface that there is no danger of sliding of the subsoil. If it is a matter of so-called “structured” clay (clay that as a result of drying out shows a structure of lumps and small cracks), the permeability becomes larger and the effects of the water tensions are felt deeper. It can be assumed that clay will show an increasing degree of structuring during the life of the revetment. For this situation, the specific studies should be performed.

Liquefaction:

The permeability of interlock systems and block mats will in general be at least as large as that of the subsoil. The resulting water tensions are therefore not large, although the systems are stiff compared to the subsoil. The recommended design rules for liquefaction do not deviate from those presented in the previous section.

Drop in the water level:

The danger as a result of a drop in the water level depends on the leakage length, *i.e.*, the characteristics of the top layer and the sublayer. With a block mat on sand, the leakage length is generally small and the danger due to drop in the water level is also small. The leakage length can be determined using Equation (2.2a). The strength parameters Δ and D follow from the standard definitions.

Durability

Naturally, the durability of the interconnection of linked blocks is of major importance. The materials used for this purpose (steel or nylon cables, geotextile) should be able to withstand in the long term the effect of (sea) water, sunlight, plants, animals, vandalism, *etc.* An example are the synthetic pins, which connect the blocks to the geotextile and which may become brittle at low temperature. This synthetic material must be sufficiently tough.

Concrete-filled mattresses

Stability of the concrete-filled mattress revetment

The permeability of the mattress is one of the factors that determine the stability. It is found that the permeability given by the suppliers is often the permeability of the geotextile, or of the so-called Filter Points. In both cases, the permeability of the whole mattress is much smaller. A high permeability of the

mattress ensures that any possible pressure build-up under the mattress can flow away, as a result of which the uplift pressures across the mattress remain smaller.
In general, with a subsoil of clay and silty sand the permeability of the mattress will be higher than the permeability of the subsoil. Therefore the water under the mattress can usually be discharged without excessive lifting pressures on the mattress.
The permeability of the mattress will be lower than the permeability of the subsoil or sub layers if a granular filter is applied, or with a sand or clay subsoil having an irregular surface (gullies/cavities between the soil and the mattress). This will result in excessive lifting pressures on the mattress during wave attack.

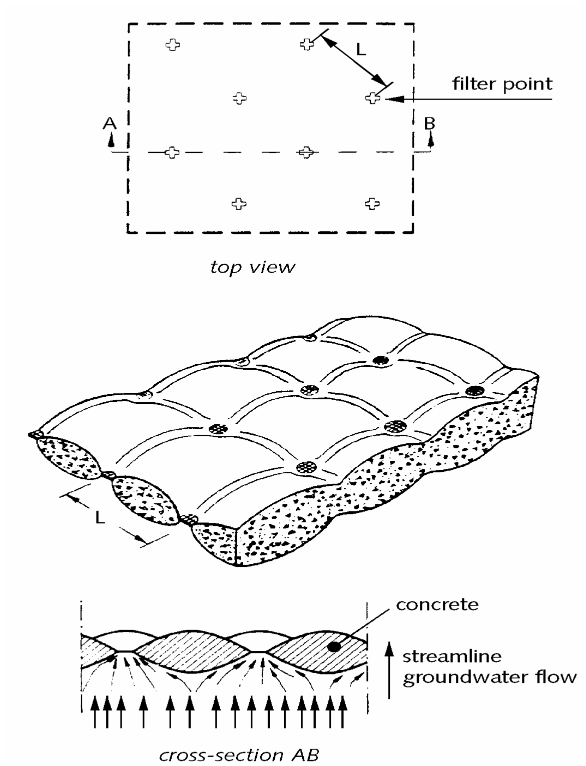

Figure 2.34 Principles of permeability of Filter Point Mattress (Source: Klein Breteler & Pilarczyk, 1998)

In the design of rules for conrete mattresses with regard to wave load, an adapted damage mechanism is assumed. Moreover, the calculation of the leakage length is adapted. During wave attack the mattress will be exposed to a differential pressure which is directed upwards, as is also the case with placed block revetments (see Figure 2.34). This takes place the moment the wave has drawn back, just before the wave impact. Just as with placed block revetments, the leakage length for this differential pressure is the most important construction-descriptive parameter. Under normal conditions the leakage length is computed according Eq. 2.2a. However, in case of cavities underneath the mattresses (*i.e.*, surface irregularities and/or erosion channels) the leakage length is calculated according to (Luth, 1993):

$$\Lambda = \sqrt{\frac{dDk}{k'}} \qquad (2.73a)$$

with:

$$k = 5.75\sqrt{g} \cdot \sqrt{\frac{d}{0.6}} \cdot \log\left(\frac{6d}{k_s}\right) \tag{2.73b}$$

where

d	= depth of cavity	[m]
k	= permeability of cavity	[m/s]
k_s	= Nikuradse roughness; for cavities about 0.5 mm	[m]

The leakage length of various mattresses based on Equation 2.73, are shown in Table 2.11.

Table 2.11 Leakage length of various concrete-filled mattresses [m]

Mattress	Leakge length Λ [m]		
	on sand*)	on sand**)	on filter
Standard – FP	≤ 1.0	4.0	3.0 – 4.0
FPM	≈ 1.5	10	6.0
Slab	≤ 1.0	2.0	3.0
Articulated (Crib)	≤ 0.5	≤ 1.0	≤ 0.5

*) good contact of mattress with sublayer (no gullies/cavities underneath).
**) pessimistic assumption: poor compaction of subsoil and presence of cavities under the mattress.

The failure mechanism of the concrete mattress is probably as follows:
- At first, cavities will form under the mattress as a result of uneven subsidence of the subsoil. The mattress is rigid and spans the cavities.
- With large spans, wave impacts may cause the concrete to crack and spans to collapse. This results in a mattress consisting of concrete slabs which are coupled by means of geotextile.
- With sufficiently high waves, an upward pressure difference over the mattress will occur during wave run-down, which lifts the mattress.
- The pumping action of these moments will cause the subsoil to migrate, as a result of which an S-profile will form and the revetment will collapse completely.

Taking into consideration the above failure mechanisms, the following design (stability) formula has been derived for the mattresses:

$$\frac{H_s}{\Delta D} = \frac{F}{\xi_{op}^{2/3}} \quad \textit{with:} \left[\frac{H_s}{\Delta D}\right]_{max} = 4 \tag{2.74}$$

with:

D	=	$\dfrac{\text{mass per } m^2}{\rho_s}$ (which can be called $D_{effective}$ or $D_{average}$)	[m]
Δ	=	relative volumetric mass of the mattress = $(\rho_s - \rho)/\rho$	[-]
ρ_s	=	volumetric mass of concrete	[kg/m^3]
F	=	stability factor, see table	[-]

Table 2.12 shows the values of F for a structure with the following conditions:
- $0.2 < \tan\alpha < 0.4$
- $\sigma_b = 5$ MPa
- $H_s < 1.5$ m
- $\rho_s = 2300$ kg/m^3.

Table 2.12 Values of stability factor F as a function of Leakage length and structural conditions

Leakge length Λ [m]	Mattress on sand	Mattress on filter
Λ = 0.5 – 0.65 m	F = 4.0	F = 4.0
Λ = 1.0 m	F = 3.5	F = 3.3
Λ = 2.4 m	F = 2.9	F = 2.5
Λ = 8.0 m	F = 2.7	F = 2.2

For an exact determination of the leakage length, one is referred to the analytical model (Klein Breteler *et al.*; 1998). However, besides the mattresses of a type as, for example, the tube mat (Crib) with relative large permeable areas, the other types are not very sensitive to the exact value of the leakage length. It can be recommended to use the following values of F in design calculations:

- F = 2.5 or (≤ 3) - for low-permeable mattresses on (fine) granular filter,
- F = 3.5 or (≤ 4) - for low-permeable mattress on compacted sand,
- F = 4.0 or (≤ 5) - for permeable mattress on sand or fine filter (D_{f15} < 2 mm).

The higher values can be applied for temporary applications or when the soil is more resistant to erosion (i.e. clay), and the mattresses are properly anchored.

The accuracy of F depends on the accuracy of the estimation of the permeabilities and the resulting leakage length. This does only hold for mattresses with relative large permeable areas. For mattresses with smaller permeable areas, the value of the stability factor F will be less affected by the accuracy of the estimated permeability of the mattress. The representative relative density Δ follows from the standard definition. For the representative thickness D, the average thickness should be used.

Soil mechanical stability of concrete-filled mattresses

The flow through a concrete mattress is concentrated in the filter points. The permeability of the systems filled with concrete lies between approximately 1.10^{-4} and 1.10^{-5} m/s. A concrete mattress is less flexible than a sand mattress and does not connect to the subsoil as well as a sand mattress. In contrast with sand mattresses, it is assumed that only the sliding of the whole mattress can occur and not just part of it.

Elastic storage:
With regard to elastic storage, the following design example for a wave height H = 1 m and a slope 1:3 is given. The required thickness of the mattress on sand for various failure mechanisms and wave steepness (S_{op}) can be found in Table 2.13.

Table 2.13 Required thickness of mattress for wave height H = 1 and slope 1:3

Failure type	S_{op} = 0.03	S_{op} = 0.05
Lifting of the top layer	0.35 m	0.25 m
Sliding of the top layer	0.30 m	0.25 m
Sliding of the subsoil	0.55 m	0.40 m

Concrete mattresses are mostly stiff and anchored at the top. Therefore, not the sliding and/or uplifting of the top layer but the sliding of the subsoil is the most dangerous. If the systems are placed on a filter, one can take into account an increase in the stability with regard to elastic storage (see Equations 2.70 – 2.71).

Liquefaction:
The design rules with regard to liquefaction do not differ from those presented in the previous section.

Drop in the water level:
The danger of a drop in the water level depends on the leakage length, *i.e.*, the characteristics of the top layer and sublayer. With a concrete mattress on sand, the leakage length is generally small and the danger of due to a drop in the water level is also small. The representative relative density Δ follows

from the standard definition. For the representative thickness D one should fill in the (over the surface) averaged thickness.

Durability

The weakest point of concrete-filled mattresses are the filter points. In the long term, pollution or the clogging of the geotextile can cause a decrease in the permeability. The susceptibility for blocking can be reduced by increasing the gradiation of the subsoil. To reduce the susceptibility for clogging it is recommended to reduce the sludge content of the subsoil.

Gabions

Wave attack on gabions will lead to a complex flow over the gabions and through the gabions. During wave run-up the resulting forces by the waves will be directed opposite to the gravity forces. Therefore the run-up is less hazardous then the wave run-down.
Wave run-down, as it was already mentioned in Section 2.2.1, will lead to two important mechanisms: The downward flowing water will exert a drag force on top of the gabions and the decreasing freatic level will coincide with a downward flow gradient in the gabions.
During maximum wave run-down there will be an incoming wave that a moment later will cause a wave impact. Just before impact there is a 'wall' of water giving a high pressure under the point of maximum run-down. Above the run-down point the surface of the gabions is almost dry and therefore there is a low pressure on the gabions. The interaction of high pressure and low pressure is shown in Figure 2.1.

A simple equilibrium of forces leads to the conclusion that the section from the run-down point to the freatic line in the filter will slide down if:

- if there is insufficient support from gabions below this section
- if the downward forces exceed the friction forces: (roughly) $f < 2 \cdot \tan\alpha$

with:
f = friction of gabion on subsoil [-]
α = slope angle [-]
From this criterion we see that a steep slope will easily lead to the exceeding of the friction forces, and furthermore a steep slope is shorter then a gentle slope and will give less support to the section that tends to slide down.

Hydrodynamic forces, such as wave attack and current, can lead to various damage mechanisms. The damage mechanisms fall into three categories:

1. Instability of the gabions
   a) The gabions can slide downwards, compressing the down slope mattresses
   b) The gabions can slide downwards, leading to upward buckling of the down slope mattresses
   c) All gabions can slide downwards
   d) Individual gabions can be lifted out due to uplift pressures
2. Instability of the subsoil
   a) A local slip circle can occur, resulting in a S-profile
   b) The subsoil can wash away through the gabions
3. Durability problems
   a) Moving stones can cut through the mesh
   b) Corrosion of the mesh
   c) Rupture of the mesh by mechanical forces (vandalism, stranding of ship, etc.).

Stability of the gabion revetment

An analytical approach of the development of the uplift pressure in the gabions can be obtained by applying the formulas for the uplift pressure under an ordinary pitched block revetment, with as leakage length: Λ=0.77 D, with D the thickness of the gabion1. With this relation the stability relations according to the analytical model are also applicable to gabions. Substitution of values, which are

reasonable for gabions, in the stability relations according to (CUR/CIRIA 1991) provides stability relations which indeed match the a line through the measured points.
After complicated calculations the uplift pressure in the gabions can be derived (Klein Breteler et al, 1998). The uplift pressure is dependent on the steepness and height of the pressure front on the gabions (which is dependent on the wave height, period and slope angle), the thickness of the gabions and the level of the freatic line in the gabions. It is not dependent on the permeability of the gabions, if the permeability is larger then the subsoil. The equilibrium of uplift forces and gravity forces leads to the following (approximate) design formula:

$$\frac{H_s}{\Delta D} = F \cdot \xi_{op}^{-2/3} \tag{2.75}$$

with:
6 < F < 9 and slope of 1:3 ($\tan\alpha = 0.33$)
and
Δ = relative density of the gabions (usually: $\Delta \approx 1$)

It is not expected that instability will occur at once if the uplift pressure exceeds the gravity forces. On the other hand, the above result turns out to be in good agreement with the experimental results.

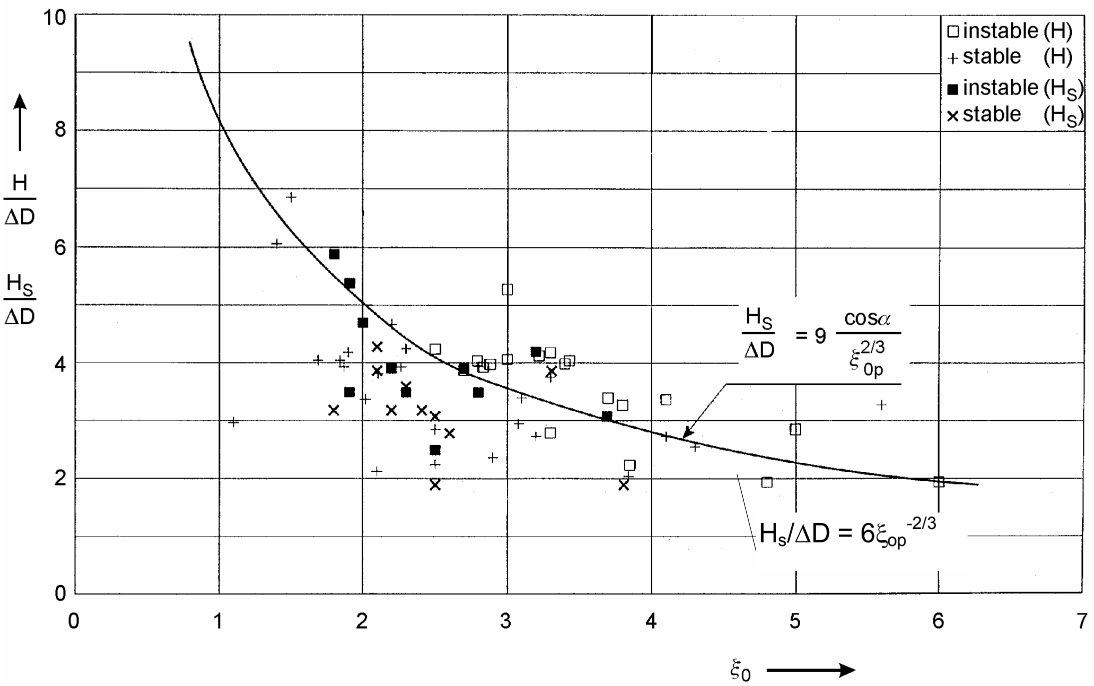

Figure 2.35 Summary of test results ((Ashe 1975) and (Brown 1979)) and design curves (Source: Klein Breteler & Pilarczyk, 1998)

The experimental verification of stability of gabions is rather limited. Small scale model tests have been performed by Brown (1979) and Ashe (1975), see Figure 2.35.

Soil mechanical stability of gabions

Erosion of the subsoil:
The grains in the gabions are usually quite course in relation to the grain size of the subsoil. Therefore, erosion of the subsoil and migration of sand through the gabions will occur, unless appropriate measures are taken.

The erosion can easily be prevented by using a geotextile under the gabions. If the characteristic opening size O_{90} is smaller than the D_{50} of the subsoil, no erosion through the geotextile will take place. Another solution is to use a granular filter under the gabions. Of course a much smaller grain size is used than the grains in the gabions. It is recommended to design the filter as follows:

- no migration of filter through gabion: $D_{90,filter} > D_{15,gabion}/4$
- no migration of subsoil through filter: $D_{15,filter} > 10.D_{50,\,subsoil}/4$ if $D_{50,subsoil} < 0.3$ mm
  $D_{15,filter} > 5.D_{50,subsoil}/4$ if $D_{50,subsoil} > 0.3$ mm
- internal stability and de-mixing: $D_{90,filter}/D_{15,filter} < 4$

Geotechnical stability:
Wave attack on gabions will give large pressure fluctuations in the pore water in the sand subsoil. These pressure fluctuations can lead to local slip circle failure. The slope will then have a S-profile. To avoid this damage mechanism either thicker gabions or a gentles slope can be selected.

Motion of filling material:
It is important to know if the filling material will start to move during frequent environmental conditions, because it can lead to rupture of the wire mesh. Furthermore the integrity of the system will be effected if large quantities of filling material is moved.
During wave attack the motion of the filling material usually only occurs if $\xi_{op} < 3$ (plunging waves). Based on the Van der Meer's formula for the stability of loose rock (CUR/CIRIA, 1991) and the assumption that the filling of the gabion will be more stable then loose rock, the following criterion is derived from the Van der Meer formula (valid for the following range of parameters: permeability factor: $0.1 < P < 0.2$; number of waves: $2000 < N < 5000$; and damage level: $3 < S < 6$):

$$\frac{H_s}{\Delta_f D_f} = \frac{F}{\sqrt{\xi_{op}}} \tag{2.76}$$

with:
$2 < F < 3$
and
Δ_f = relative density of the grains in the gabions (usually: $\Delta_f \approx 1.65$)
D_f = diameter of grains in the gabion [m]

Motion of the grains in the gabions can be tolerated if it occurs seldom (less than once every decade). During frequently occuring wave attack (a few times a year) it is recommended to design the gabion such that no filling motion occurs.

Durability

An important problem of this protection system is the durability. Frequent wave or current attack can lead to a failure of the wire mesh because of the continuously moving grains along the wires, finally cutting through. Another problem is the corrosion of the mesh. Therefore meshes with plastic coating or corrosion resistant steel are used. On the other hand the system is less suitable where waves and currents frequently lead to grain motion.

Geosystems

Compared to other types of revetments there is not much known on the stability of geosystems. However, there are some, more general, conclusions that can be drawn with respect to the stability of geosystems agains wave attack (Wouters, 1995):

1. In nearly all publications the necessity of a proper filling grade is mentioned. With a filling grade of 75 to 80% the stable shape and optimal stability can be obtained.
2. A proper bed protection againts scour/undermining is needed when geosystems are exposed to current and wave attack.

3. In the case of sand or gravel-filled geosystems used as a slope protection, a regular inspection and monitoring is needed for an early discovery of the possible damage. The damage should be repaired as soon as possible to avoid a progressive damage development and loss of functional performance.
4. For sand, gravel and cement bags, which are appliedas slope protection, the following approximate stability criteria can be formulated (based on regular waves):

$$\frac{H}{\Delta D} = \frac{3.5}{\sqrt{\xi_{op}}} \tag{2.77}$$

with:
Δ = relative density (Δ = 1 for fully saturated fill material) [-]
D = thickness of the top layer measured perpendicular to the slope [m]
When substituting Hs = 1.4 H the stability criterion becomes:

$$\frac{H}{\Delta D} = \frac{3.5}{\sqrt{\xi_{op}}} \tag{2.78}$$

5. In the case of cement (mortar) filled geotubes or systems applied in the crests of low-crested breakwaters, the following criterion for regular waves can be applied:

$$\frac{H}{\Delta b} = 3.2\left(\frac{H}{L_0}\right)^{1/3} \tag{2.79}$$

where:
L = wave length [m]
B = width of the geotube [m]
In the case of geotubes lying with the longest dimension along the structure (parallel to the axis of the structure) the following criterion can be used:

$$H / \Delta l = 1 \tag{2.80}$$

where:
l = length of a tube [m]

With regard to the durability of geosystems the following remark can be made. On the long term, especially when no UV-protection for geotextile is applied, the surface of a geomattress will deteriorate and the concrete-filled mattress will function as a block mat; a block mat with concrete units connected to the lower sheet of geotextile by existing binders, which normally are used as spacers to provide a required thickness. These binders should have a proper strength to compensate the weight of the concrete element. Therefore, the stability of the geomattresses should also be controlled accordingly to the design criteria provided for block mats.

2.7.4 Identification of input parameters and uncertainties

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 these alternative open slope revetments. The permeability of the different layers is probably the parameter that is most difficult to estimate. However, good tuning of the permeabilities of the cover layer and the sublayers (including geotextile) is an essential condition for a balanced design of a revetment. The presence of transition structures can seriously change the permeability properties and hence the critical loading. This should be incorporated in a proper design of these alternative revetments.

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