ALTERNATIVE OPEN SLOPE REVETMENTS, summarizing report A2.96.04

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A = revetment constant in a design relation for wave load 

a = revetment constant in a design relation for flow load 

b = width of a sand sausage 

b = thickness of the filter layer 

c = coefficient 

C = Chézy’s resistance coefficient 

D = thickness of the revetment 
   for riprap: D = Dₐ (nominal diameter) 
   remaining: D = effective thickness 

Dₐ = maximum diameter of the sand sausage 

Dₑq = equivalent diameter 

Dₛ = nominal diameter of riprap; 

Dₙₐ = depth of gully under the top layer 

Fr = Froude number of the flow; 

Fr = Froude number of the flow; 

Fr = gravitational acceleration; g = 9.81 

H = wave height of regular waves 

Hₛ = significant wave height 

h = water depth 

hₑq = equivalent wash-over height 

hₑw = wash-over height (upper water level relative to the crest) 

Kₐ = damage coefficient (Hudson) 

Kₑ = depth factor 

Kₛ = slope parameter 

Kᵣ = turbulence factor 

kₛ = permeability coefficient of the top layer 

kₑ = permeability coefficient of the filter layer 

kₑ = permeability coefficient of the geotextile 

kᵣ = equivalent roughness according to Nikuradse 

kₑq = permeability coefficient of the top layer 

L₀ = distance between the top and the valley of the front, measured 
   along the slope 

L₁ = distance along the slope between the valley of the front of the rise 
   and the phreatic line 

Lₒ = deep-water wavelength for regular waves; 

Lₒ = deep-water wavelength at the peak period; 

l = length of a sand sausage 

n = pore ratio 

M₅₀ = mass of a piece of stone which is exceeded by 50% 

m = draining coefficient in 

\[ q = m \frac{2}{3} \sqrt{\frac{2}{3} g H^{1.5}} \]
List of symbols (continued)

- \( q \) = specific discharge \( (m^2/s) \)
- \( q_{cr} \) = critical specific discharge (beginning of loss of stability) \( (m^2/s) \)
- \( q_{eq} \) = equivalent specific discharge \( (m^2/s) \)
- \( s \) = wave steepness; \( s = H/L_o \) (-)
- \( s_{op} \) = wave steepness; \( s_{op} = H_p/L_{op} \) (-)
- \( T \) = wave period of regular waves (s)
- \( T_g \) = thickness of the geotextile (m)
- \( T_p \) = wave period at the peak of the spectrum (s)
- \( u \) = vertically-averaged flow velocity (m/s)
- \( u_{cr} \) = critical vertically-averaged flow velocity (m/s)
- \( \alpha \) = slope angle \( (°) \)
- \( \alpha_r \) = relative density; \( \Delta M = (\rho_r - \rho_s) / \rho_s \) (-)
- \( \Delta \) = relative density including water-filled pores \( (kg/m^3) \)
- \( \rho_s \) = density of the protection material \( (kg/m^3) \)
- \( \rho_w \) = density of water \( (kg/m^3) \)
- \( \Theta \) = stability parameter (-)
- \( \phi \) = critical Shields parameter (-)
- \( T \) = leakage length (m)
- \( \theta \) = angle of internal friction \( (°) \)
- \( \gamma \) = gradient of the wave front \( (°) \)
- \( \xi \) = breaker parameter for regular waves; \( \xi = \tan \alpha \) (-)
- \( \xi_{op} \) = breaker parameter; \( \xi_{op} = \frac{\tan \alpha \sqrt{H/L_o}}{\sqrt{H_s/L_{op}}} \) (-)
1 Introduction

1.1 Background

Within the scope of the research on the stability of open slope revetments, much knowledge has been developed about the stability of packed stone revetments under wave load. This development of knowledge has lead to a design methodology that has been laid down in writing in the "Handboek voor dimensionering van gezette taludbekledingen" ("Handbook for the dimensioning of packed slope revetments") (CUR/TAW, 1992).

The above-mentioned types of construction and load have constantly been given the highest priority, because of their significance to the Dutch water-control structures. Until recently, no or unsatisfactory design tools were available for a number of other (open) types of revetment and for stability aspects. This is why the design methodology for packed stone revetments has recently been extended in applicability by means of a number of monographs.

This extension concerns both other types of construction, such as:

- interlock systems and block mats;
- gabions;
- concrete mattresses;
- geosystems, such as sandbags and sand sausages

and other stability aspects, such as:

- flow-load stability;
- soil-mechanical stability;
- residual strength.

The report in question aims at giving a summary of the increased knowledge, especially that concerning the design tools that have been made available. The reports behind it have been included as appendices in a separate volume.

1.2 Project Information

The study was conducted by order of The Road and Hydraulic Engineering Division (DWW) of Rijkswaterstaat, which acts on behalf of the Technical Advisory Committee on Water Defences (TAW-A2). The DWW contract, number DWW-726, dd. 22 July 1993, was used to commission this study. The abstract was written by ir J.P. de Waal of the Hydraulics Laboratory "De Voorst".
2 Discussion of Stability

2.1 Wave-load stability

2.1.1 Introduction

This section deals with the stability of a revetment system, as far as the damage mechanism consists of the top layer being pushed up by wave load.

With the research on packed stone revetments, much knowledge has been developed about the wave conditions that, depending on the constructional properties, lead to initial damage. This especially applies to packed revetments on a granular filter.

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 to the strength, depending on the type of wave attack:

\[
\left( \frac{H_s}{\Delta D} \right)_{cr} = \text{function of } \xi_{op}
\]

In which:
- \( H_s \) = significant wave height (m)
- \( \Delta \) = relative density (-)
- \( D \) = thickness of the revetment (m)
- \( \xi_{op} \) = breaker parameter (-)

The relative density is defined as follows:

\[
\Delta = \frac{\rho_s - \rho_w}{\rho_w}
\]

with:
- \( \rho_s \) = density of the protection material (kg/m³)
- \( \rho_w \) = density of water (kg/m³)

For porous top layers, such as sand mattresses and gabions, the relative density of the top layer must be determined, including the water-filled pores:

\[
\Delta_m = (1 - n) \cdot \Delta
\]

In which:
- \( \Delta_m \) = relative density including pores (-)
- \( n \) = porosity of the top layer material (-)

The breaker parameter is defined as follows:
\[
\xi_{op} = \frac{\tan \alpha}{\sqrt{H_s/L_{op}}}
\]  
\begin{equation}
L_{op} = \frac{g}{2 \pi} T_p^2
\end{equation}

\[\alpha\] = slope angle (°)  
\[T_p\] = wave period at the peak of the spectrum (s)

**2.1.2 The Black-Box Model**

The (old) black-box model consists of the design diagrams as shown in the Figures 104 inclusive 113 in (CUR/TAW, 1992). This methodology starts from a rough division into types of construction under which packed revetments on sand, clay and a granular filter fall. Per type of construction, the tendency in the observed critical relation between load and strength has been translated into a design relation. The basic form of this relation is:

\[
\left( \frac{H_s}{\Delta D} \right)_{cr}^{2/3} = A \left( \frac{H_s}{\Delta D} \right)_{cr} = 8.0
\]

In which:

\[A\] = revetment constant (-)

Table 2.1 gives indicative values of some standard types of revetment.

The advantage of this old black-box design formula is its simplicity. The disadvantage, however, is that the value of \(A\) is known only very roughly for many types of construction. In this report the term "old black-box model" is used to make a distinction with the updated black-box model as presented in (Klein Breteler, 1994). This updated black-box model is a kind of hybrid of the old model and the analytical model.

**2.1.3 The Analytical Model**

The analytical model is based on packed stone revetments on a granular filter. In this calculation model, a large number of physical aspects are taken into account. Three important differences from the old black-box model are:

- The wave characteristics are translated into an indicative pressure distribution on the revetment.
- The type of construction is characterized by the leakage length \(A\). (This is used to determine the pressure distribution under the top layer, so that the upward load is known).
- An increase factor \(\Gamma\) is applied to the strength of the revetment.

In short, in the analytical model descriptions of the physical aspects that are relevant to the
stability have been incorporated. The end result of the analytical model may, for that matter, again be presented as a relation such as formula (2.4). These descriptions have been used in the so-called updated black-box model (Klein Breteler, 1994).

To be able to apply the design method for packed stone revetments under wave load to other systems, the following items may be adapted:

- the revetment parameter \( A \);
- the (representative) strength parameters \( \Delta \) and \( D \);
- the design wave height \( H_d \);
- the (representative) leakage length \( \Lambda \);
- the increase factor \( \Gamma \) on the strength.

Only suchlike adaptations are presented in this summarizing report. The basic formulas of the analytical model are not repeated here. For these, one is referred to (CUR/TAW, 1992) and (De Waal et al., 1995). Due to the extent of the analytical model, calculation examples using this method are not worked out in this report.

After initial damage has occurred, the revetment can generally offer resistance to the wave attack for some time yet. The duration between initial damage occurring and the moment the core material of the dike is exposed to wave attack is called the residual strength. Calculation models to quantify this residual strength are being developed. For a number of construction types interesting observations are already available.

### 2.2 Flow-load stability

#### 2.2.1 Introduction

This section deals with the aspect of heavy flow attack on open slope revetments. Severe flow attack may in practice occur in many situations, such as with flow over a steep slope and flow attack near many kinds of constructions (downstream of sills, gates, discharge constructions and the like). With constructions, the flow is often specifically determined by the geometry and the boundary conditions. With flow over a steep slope, such as on the downstream slope of a washed-over dam or dike, the situation is less ambiguous. This situation is worked out further here.

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, that is whether the discharge or the flow velocity can be determined most accurately.

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

For background information, see Annexe 1.

#### 2.2.2 Design formula based on flow velocity
The flow velocity can generally be determined well when the water flows downwards over a slope gentler than approximately 1:20. The flow velocity is then of importance to the soil protection and the dike revetments along the flow.

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

\[ \Delta D = 0.035 \frac{\Phi}{\Psi} \frac{K_T K_h}{K_s} \frac{u_{cr}^2}{2g} \]  

(2.7)

in which:

- \( \Delta \) = relative density
- \( D \) = characteristic dimension
- \( g \) = acceleration of gravity \((g = 9.81)\)
- \( u_{cr} \) = critical vertically-averaged flow velocity
- \( \Phi \) = stability parameter
- \( \Psi \) = critical Shields parameter
- \( K_T \) = turbulence factor
- \( K_h \) = depth parameter
- \( K_s \) = slope parameter

The coefficients in this formula are discussed in more detail below.

**Stability parameter \( \Phi \)**

The stability parameter \( \Phi \) depends on the application. Some guide values are given in Table 2.2.

**Critical Shields parameter \( \Psi \)**

With the critical Shields parameter \( \Psi \) the type of material can be taken into account. Some guide values are given in Table 2.3.

**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 given in Table 2.4.

**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 measure of development of the flow profile and the roughness of the revetment. The following formulas are recommended:
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When the flow velocity is known well, or can be calculated reasonably accurately, Pilarczyk's relation (1990) is applicable:

\[ \Delta \Omega = 0.035 \frac{\Phi}{\Psi} \frac{K_T K_h}{K_s} \frac{u_{cr}^2}{2g} \]  

(2.7)

in which:

- \( \Delta \) = relative density
- \( \Omega \) = characteristic dimension
- \( g \) = acceleration of gravity \( (g = 9.81) \)
- \( u_{cr} \) = critical vertically-averaged flow velocity
- \( \Phi \) = stability parameter
- \( K_T \) = critical Shields parameter
- \( K_h \) = depth parameter
- \( K_s \) = slope parameter
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developed profile : \( K_n = \frac{2}{\log \left( \frac{12h}{k_s} \right)^2} \)  
(2.8)

nondeveloped profile : \( K_n = \left( \frac{h}{k_s} \right)^{0.2} \)  
(2.9)

very rough flow (\( h/k_s < 5 \)) : \( K_n = 1.0 \)  
(2.10)

In which:

\( h \) = water depth (m)
\( k_s \) = equivalent roughness according to Nikuradse (m)

Formula (2.9) is an adapted version of Pilarczyk’s formula (1990).

In the case of dimensioning the revetment on a slope, the water level at the toe of the slope must be filled in for \( h \). The equivalent roughness according to Nikuradse depends on the type of revetment and is given for several types in Chapter 3. For riprap, \( k_s \) is equal to twice the nominal diameter of the stones.

Slope parameter \( K_s \)

The stability of revetment elements also depends on the 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:

\[
K_s = \sqrt{1 - \left( \frac{\sin \alpha}{\sin \theta} \right)^2}
\]  
(2.11)

with:

\( \theta \) = angle of internal friction of the revetment material (°)
\( \alpha \) = transversal slope of the bank (°)

The advantage of this general design formula of Pilarczyk (2.5) is that it can be applied in numerous situations. The disadvantage is that the distribution in results, as a result of the large margin in some parameters, can be rather wide.

2.2.3 Design formula based on the discharge

With a downward flow along a steep slope it is difficult to determine or predict the flow velocity and the water depth unambiguously, because the flow is very irregular (high turbulence, inclusion of air as a result of which the water level cannot be determined very well, etcetera). One is confronted with this when dimensioning the revetment of (the crest and) the inner slope of a dike in the case of flooding. In which case a design formula based on the discharge is better used.

For downward flow along a slope steeper than approximately 1:10, the following relation is recommended:
developed profile : \[ K_h = \frac{2}{\left(1.06g \left(\frac{0.12h}{k_s}\right)^2\right)} \] (2.8)

nondeveloped profile : \[ K_h = \left(\frac{1.06}{g^2}\right)^{0.5} \] (2.9)

very rough flow \((h/k_s < 5)\) : \[ K_h = \left(\frac{1.06}{g^2}\right)^{0.5} \] (2.10)

In which:

\[ h = \text{water depth} \quad (m) \]
\[ k_s = \text{equivalent roughness according to Nikuradse} \quad (m) \]

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\[ K_s = \sqrt{1 - \left(\frac{\sin\alpha}{\sin\theta}\right)^2} \] (2.11)

with:

\[ \alpha = \text{angle of internal friction of the revetment material} \quad (\degree) \]
\[ \theta = \text{transversal slope of the bank} \quad (\degree) \]

The advantage of this general design formula of Pilarczyk (2.5) is that it can be applied in numerous situations. The disadvantage is that the distribution in results, as a result of the large margin in some parameters, can be rather wide.

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For downward flow along a slope steeper than approximately 1:10, the following relation is recommended:
\[
\frac{q_{cr}}{g^{0.5} (\Delta D)^{1.5}} = \frac{a}{(\sin \alpha)^{1.17}}
\]  
\[\text{(2.12)}\]

Or, expressed in \(\Delta D\):

\[
\Delta D = \frac{(\sin \alpha)^{0.78}}{a^{0.67}} \frac{q_{cr}^{0.67}}{g^{0.33}}
\]  
\[\text{(2.13)}\]

In which:

- \(\Delta\) = (representative) relative density (-)
- \(D\) = (representative) thickness of the slope protection (m)
- \(q_{cr}\) = critical specific discharge \((m^3/s)\)
- \(\alpha\) = slope angle (°)
- \(a\) = coefficient of the slope revetment (-)

The value of \(a\) depends on the type of revetment. For riprap it applies that: \(a = 0.11\).

The major advantage of this design formula is its simplicity. The disadvantage is that the revetment parameter \(a\) has not been sufficiently investigated for many types of revetments, whereas from the available measurement data it can be derived that the value of \(a\) has a wide range.

For the design of a revetment in the case of a combination of (heavy) washing over with some wave action, a representative value for the discharge can be determined in the following way.

There is a relationship between the discharge and the outer water level in relation to the crest of a dam. This water level is called the wash-over height \(h_{ov}\). The relationship between the wash-over height and the discharge is given by:

\[
q = m \frac{2}{3} \sqrt[3]{2 g h_{ov}^{1.5}} = 1.7 m h_{ov}^{1.5}
\]  
\[\text{(2.14)}\]

In which:

- \(m\) = draining coefficient (-)
- \(h_{ov}\) = wash-over height; outer water level relative to the crest (m)

The draining coefficient \(m\) depends on the shape of the overflow. Assuming a complete overflow, for most dam forms \(0.8 \leq m \leq 1.0\) applies. For more details one is referred to Annexe 1.

For the design of a revetment in the case of a combination of (heavy) washing over with some wave action, an equivalent wash-over height can be defined:

\[
h_{eq} = h_{ov} + \frac{1}{3} H_s
\]  
\[\text{(2.15)}\]

In which:
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\[
\frac{q_{cr}}{g^{0.5} (\Delta D)^{1.5}} = \frac{a}{(\sin \alpha)^{1.17}}
\]

(2.12)

Or, expressed in \( \Delta D \):

\[
\Delta D = \left( \frac{\sin \alpha}{a} \right)^{0.78} \frac{q_{cr}}{g^{0.33}} a^{0.67}
\]

(2.13)

In which:

- \( A \) = (representative) relative density (-)
- \( h \) = (representative) thickness of the slope protection (m)
- \( q_{cr} \) = critical specific discharge (m²/s)
- \( \alpha \) = slope angle (°)
- \( a \) = coefficient of the slope revetment (-)

The value of \( a \) depends on the type of revetment. For riprap it applies that: \( a = 0.11 \).

The major advantage of this design formula is its simplicity. The disadvantage is that the revetment parameter \( a \) has not been sufficiently investigated for many types of revetments, whereas from the available measurement data it can be derived that the value of \( a \) has a wide range.

For the design of a revetment in the case of a combination of (heavy) washing over with some wave action, a representative value for the discharge can be determined in the following way.

There is a relationship between the discharge and the outer water level in relation to the crest of a dam. This water level is called the wash-over height \( h_{ov} \). The relationship between the wash-over height and the discharge is given by:

\[
q = m \left( \frac{2}{3} g h_{ov}^{1.5} \right) \approx 1.7 m h_{ov}^{1.5}
\]

(2.14)

In which:

- \( m \) = draining coefficient (-)
- \( h_{ov} \) = wash-over height; outer water level relative to the crest (m)

The draining coefficient \( m \) depends on the shape of the overflow. Assuming a complete overflow, for most dam forms \( 0.8 \leq m \leq 1.0 \) applies. For more details one is referred to Annex 1.

For the design of a revetment in the case of a combination of (heavy) washing over with some wave action, an equivalent wash-over height can be defined:

\[
h_{eq} = h_{ov} + \frac{1}{3} H_{s}
\]

(2.15)

In which:
By substituting $h_w$ by $h_{eq}$ in equation (2.12), an "equivalent" discharge is found:

$$q_{eq} \approx 1.7 \ m \ (h_{ov} + \frac{1}{3} H)^{1.5}$$  \hspace{1cm} (2.16)

The (critical) value of this discharge can be used in the design formula (2.10). One comment is that the coefficient 1/3 was derived for a case of heavy washing over on which wave attack is superimposed, with reference to the stability of riprap dams of the Stormvloedkering Oosterschelde (Storm-Surge Barrier Eastern Scheldt) which are under construction.

2.2.4 Summary

On the basis of the flow situation, a design formula can be selected for flow load which is expressed in a (critical) flow velocity $u_c$ or a (critical) discharge $q_c$. In both design formulas constants are found which depend on the type of revetment. These constants are:

- the (representative) strength $\Delta D$;
- the stability parameter $\Phi$
- the critical Shields parameter $\Psi$
- the angle of internal friction $\theta$
- the equivalent roughness according to Nikuradse $k_e$
- the revetment parameter $a$

The recommended values for these parameters are given (if available) with the system concerned, in Chapter 3.

2.3 Soil-Mechanical Stability

2.3.1 Introduction

The water movement on a revetment construction can also affect the subsoil, especially when this consists of sand. This effect is treated within the framework of the soil-mechanical aspects and can be of importance to the stability of the construction.

There are three aspects that will be discussed within the framework of soil-mechanical aspects:

- elastic storage;
- softening;
- drop in the water level.

These aspects and the accompanying damage mechanisms and design methods are discussed in detail below. Background information can be found in Annexe 2.

2.3.2 Elastic storage

Elastic storage in the subsoil is connected 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:

- lifting of the top layer;
- sliding of the top layer;
- sliding of the subsoil.

For the stability of the top layer, elastic storage could particularly be of importance if the top layer is placed directly on the subsoil without there being small gullies under the top layer. These conditions imply that the leakage-length approach according to the analytical model for the stability under wave load cannot be applied.

The stability 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.

Relevant concepts are:

- **Elastic storage**
  The wave at the surface continues in the subsoil. The signal is damped and delayed because the water in the pore space is elastically compressible. This results in different signals on the surface and in the subsoil, and in the subsoil alternately in water overtension and undertension.

- **Elastic storage length**
  Elastic storage causes alternating water overtension and undertension in the subsoil. The magnitude of these water tensions varies according to the depth. At a depth equal to the elastic storage length, these water tensions are the most dangerous to the stability.

The design method with regard to the different damage mechanisms connected with elastic storage are presented in the form of design diagrams. In these diagrams the permissible wave height is plotted against the thickness of the top layer and the slope gradient. 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.

**2.3.3 Softening**
Also through cyclic generation of water tension, water overtensions may occur in the subsoil, but with impermeable top layers also directly under the top layer. These water tensions can be calculated using the MCYCLE program. As the top layer becomes more impermeable, the water tension manifests itself closer to the surface of the slope. In the case of a very permeable top layer this is exactly the opposite.

Softening can be defined as follows:

- **Softening**
  A cyclic variable load causes compaction to occur in a pack of sand. This leads to a decrease in the pore space. The water in the pores is subjected to pressure and will want to run off. At first, water overtension occurs. This causes a decrease in the contact pressure between the grains and with this the resistance to sliding. Finally, the water overtension becomes so great that the contact pressure between the grains falls away completely. This is called softening.

The difference between softening and elastic storage is that with softening, water overtension is connected with a plastic deformation of the grain skeleton instead of an elastic deformation. Water overtension through softening occurs when the grain skeleton deforms plastically to a denser packing. From which follows that the dangers connected with softening are smaller as the subsoil is compacted better during construction.

With regard to softening, the following design rules are applicable to constructions with a reasonably compacted subsoil:

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

In these design rules hardly any distinction is made between types of revetment.

### 2.3.4 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 packed stone revetments, this is only a problem if any possible filter layer and the top layer are sanded up and because of this obtain a low permeability.

No calculations need to be made on this phenomenon if applies:

$$\frac{\Lambda \sin \alpha}{2} \leq \Delta D \cos \alpha$$

(2.17)

in which:

- $\Lambda = \text{leakage length}$

(m)
alternatieve open taludbekledingen: samenvattend verslag h 1930
maart 1996

\( \alpha = \) slope angle \( (\degree) \)

\( \Delta = \) (representative) relative density of the top layer \( (-) \)

\( D = \) (representative) thickness of the top layer \( (m) \)

The leakage length is determined in the same way as is done for the wave load. For systems on a filter layer, the leakage length is given through:

\[
\Lambda = \sqrt{\frac{b_f D k_f}{k'}}
\]

with:

\( b_f = \) thickness of the filter layer \( (m) \)

\( k_f = \) permeability of the filter layer or subsoil \( (m/s) \)

\( k'/ = \) permeability of the top layer \( (m/s) \)

With a system without a filter layer (directly on sand or clay, without gullies being formed under the top layer) not the permeability of the filter layer, but the permeability of the subsoil is filled in. For the thickness of the filter layer it is examined to which depth changes affect the surface. One can fill in 0.3 m for sand and 0.03 m for clay.

The representative values for \( D \) en \( \Delta \) depend on the type of revetment.

Should the application of formula (2.15) show that calculations on the phenomenon should be made, one is referred to the Chapters 6.3 and 7.4 of Annexe 2 (report "Grondmechanisch ontwerp van alternatieve systemen" ["Soil-Mechanical Design of Alternative Systems"], part "oplichten van de bekleding" ["lifting the revetment"]).
3 Types of Construction

3.1 Blocks on sand

3.1.1 System description

A revetment of blocks on a geotextile on sand is, in theory, very stable under wave attack, because the permeability of the sand is small, as a result of which the leakage length and with this the difference in pressure is small. However, in practice it is found that this type of construction is less stable than according to this theory. The explanation for this is found in the presence of cavities and gullies directly under the blocks, as a result of which the porosity of water under the blocks becomes larger than that of the sand itself.

With blocks on sand, testing the soil-mechanical stability is also important.

Damage mechanisms

The collapse of a slope with blocks on a (sand-tight) geotextile on sand can take place in two ways:

1. Lifting of a block:
   Because of differential pressures over the top layer during wave attack, one or more blocks may be pushed out of the revetment. For which not only wave fronts, but also wave impacts are of importance.

2. Deformation of the slope through soil-mechanical instability:
   Through water tensions in the sand, the grain tension may become so low that deformation of the slope into an S-profile occurs.

Background information can be found in Annexe 3.

3.1.2 Design rules with regard to wave load

Black-box model

The revetment parameter $A$ for blocks on sand lies between 3.7 and 8.0. This means that the stability relation according to the old black-box model becomes:

$$
\left( \frac{H_s}{\Delta D} \right)_{cr,\text{black-box}} \text{ with as maximum } \frac{2}{3}, \quad \left( \frac{H_s}{\Delta D} \right)_{cr,\text{black-box}} = 8.0
$$

In this formula the strength parameters $\xi$ and $D$ can be determined according to the standard definition.

Analytical model

For a more advanced determination of stability, the analytical model for blocks on sand is made applicable by:
• adaptation of the determination of the leakage length, in connection with the
difference between the permeability of a granular filter and small gullies;
• adaptation of the design wave characteristics, in connection with the importance of
wave impacts.

Adapted leakage length

When schematically representing a block on a geotextile on a gully on sand, the block should
be regarded as the top layer and the combination of the geotextile and the small gully as the
filter layer. The leakage length can be calculated using:

$$\Lambda = \sqrt{\frac{(k_f d_g + k_g T_g) D}{k'}}$$

with:

- $k_f = \text{permeability of the filter layer (gully), (m/s)}$
- $d_g = \text{gully depth, (m)}$
- $k_g = \text{permeability of the geotextile, (m/s)}$
- $T_g = \text{thickness of the geotextile, (m)}$
- $D = \text{thickness of the top layer, (m)}$
- $k' = \text{permeability of the top layer, without resistance to the approaching flow, (m/s)}$

For the moment, it is recommended to determine the top layer on the basis of a leakage-
length calculation with:

a If the top layer is perfectly level and there is hardly any gully formation or
subsidence of the subsoil:
  - $d_g = 2 \text{ mm}$
  - $k_f = 0.6 \text{ m/s}$

b If the top layer is not completely level and there might be some gully formation:
  - $d_g = 5 \text{ mm}$
  - $k_f = 0.9 \text{ m/s}$

c If the top layer is clearly undermined by gullies:
\[ d_c = 20 \text{ mm} \]
\[ k_f = 1.8 \text{ m/s} \]

For the determination of the value of \( k \) in this formula, one is referred to Annexe 3. (blocks on sand).

**Adapted design wave height**

In order to implicitly take into account the effect of wave impacts, it is recommended for the time being to dimension the construction on:

- \( H_{15\%} \), with which a block movement of 0.1D is allowed (instead of \( H_{25\%} \)); and
- \( H_{10\%} \), with which no block movement is allowed (instead of \( H_0 \)).

For deep water applies:

\[
\begin{align*}
H_{15\%} &= 1.5 H_s \\
H_{10\%} &= 1.1 H_s
\end{align*}
\]  

(3.3)  
(3.4)

**3.1.3 Design rules with regard to flow load**

The following constants are applicable in the design formulas for flow load (see section 2.2.2):

**(representative) Strength \( \Delta D \)**

The strength parameters \( \Delta \) and \( D \) follow from the standard definitions.

**Stability parameter \( \Phi \)**

The stability parameter follows from the recommended values for revetments without interaction between the elements:

\[
\begin{align*}
\Phi &= 1.0 \quad \text{with a flat bottom, without transitions} \\
\Phi &= 1.5 \quad \text{with protruding edges, a rough bottom or transitions}
\end{align*}
\]

**Critical Shields Parameter \( \Psi \)**

The recommended value for the critical Shields parameter is:

\[ \Psi = 0.05 \]

**Revetment parameter \( a \)**

The revetment parameter for blocks on sand is estimated at:

\[ a = 2.6 \]

**Equivalent roughness according to Nikuradse \( k_s \)**
The roughness of a revetment of blocks on sand is relatively small. A suitable value for the equivalent roughness according to Nikuradse is therefore:

\[ k_e \approx 0.01 \text{ m} \]

Angle of internal friction \( \theta \)

For the top layer an angle of internal friction is not applicable. In the calculation one can use:

\[ \theta = 90^\circ \]

### 3.1.4 Design rules with regard to soil-mechanical stability

#### Elastic storage

For the soil-mechanical stability, one assumes a sound toe anchoring, which is the most common situation. For a revetment of blocks on sand with toe anchoring, the design diagrams of Figures 1 through 4 can be used for elastic storage. In the design diagrams a distinction between two types of load is made. When using the diagrams, both types of load must be tested to determine the normative value. For background information on these types of load, one is referred to Annexe 3.

#### Softening

The permeability of the top layer will generally be at least as large as that of the subsoil. Which is why the resulting water tensions are not very large, although the top layer is stiff compared to the subsoil. The recommended design rules for softening do not differ from those presented in section 2.3.3.

#### Drop in the water level

In the initial situation, the permeability of the top layer is larger than the permeability of the subsoil. The leakage length is small. This means that no large differences in rise occur under the top layer as a result of a drop in the water level. In general, a drop in the water level can only be a danger if the top layer is sanded up or if there is gully formation under the top layer.

The leakage length can be determined using the formulas in section 3.1.2. The strength parameters \( \Delta \) and \( D \) follow from the standard definitions. See also section 2.3.4.

### 3.2 Block mats and interlock systems

#### 3.2.1 System description

A (concrete) block mat is a slope revetment made of (concrete) blocks that are joined together to form a "mat", see Figure 5. The interconnection may consist of cables from block to block, 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.

Two major advantages of block mats are their properties of being able to be laid quickly and efficiently and partly under water.

With interlock systems, the blocks interlock somehow or other because of their shape. In contrast to block mats, there is no physical connection between the blocks.

Block mats and interlock systems are more stable than a setting of loose blocks, because a single stone cannot be moved in the direction perpendicular to the slope without moving other nearby stones. This is the most important difference from a setting of clamped stones, where the presence of loose stones must always be taken into account.

The revetment system functions optimally if no movement whatsoever of an individual stone is possible without the adjacent stones being moved also. It is, however, sufficient to demand that already with a small movement of an individual stone a significant interactive force with the surrounding stones is mobilized. Large movements of individual blocks are not permissible, because transport of filter material may occur. After some time, this leads to a serious deformation of the surface of the slope.

The above can be translated into the following requirement: at a movement of 5 to 10% of the thickness of the top layer, an interactive force equal to the weight of the block should occur. If the system cannot meet this requirement, the top layer should be dimensioned as if "loose" stones occur, such as with a top layer with clamped stones.

A mat with a cable connection basically has a greater rigidity than a mat with a geotextile connection. Because a geotextile connection can only exert a significant interactive force after some displacement of an individual block, the system does not function without being washed in well.

The weakness of block mats and interlock systems is an edge, as between two mats and with transitional constructions. If mats are not joined together in a way that allows for the major hydraulic forces that act on the construction, the edges may turn back. Then the stability is hardly larger than that of separate, loose stones. The stability of the edges of a mat may be increased by using extra wide blocks at the edge. The edge of a mat with a half-brick bond can therefore better be made of alternating whole and one-and-a-half blocks than of alternating whole and half blocks.

Naturally, the durability of the interconnection of linked stones is of major importance. The materials used for this purpose (steel 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 temperatures. This synthetic material must be sufficiently tough.

Construction/repair

A major advantage of block mats is that they can be laid both above and under water.

Just as with clamped stones, a good compaction of the slope is important, as well as making
the surface sufficiently flat, so that a sound connection of the mat on the subsoil is guaranteed. With application on banks, it is highly recommended to place the mat on undisturbed soil and not filling up too deep excavations without reason.

When a geotextile is applied, the surface of the slope must first be cleaned of foreign bodies, to prevent piercing. Geotextiles which do not form a part of the mat, should be secured to prevent rising or shifting during the laying of the mat. At the seams, the geotextile must overlap at least 0.5 to 1.0 m, because of the possible scouring of the subsoil. This is especially important if the mat is laid directly on sand or clay.

The laying of the block mat is done using a crane and a pointer. In general, they can be placed very accurately, so that the gap between the blocks of adjoining mats can be limited to 1 to 2 cm, provided that at least part of the mat can be laid above the water line. Placing the mat completely under water is much more difficult. The gap between the blocks of adjoining mats may nowhere be more than 3 cm.

After laying the mats, they can be interlocked so that the edges and corners cannot turn back. Without interlocking, particularly the corners of the mat are vulnerable. Moreover, the mat should be anchored at the top, and the toe can be reinforced with a toe beaching. A toe construction which provides support against sliding is not necessary with mats.

As for the possibilities of reuse and repair, block mats and interlock systems are at a disadvantage compared with clamped stones, because:

- the mat connections break when the slope is broken up;
- for repair, a relatively large part of the slope must be broken away and mended.

**Damage mechanisms**

With block mats, the damage mechanism usually consists of the deformation of the slope into an S-profile. This is caused by the transportation 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.

For more information one is referred to Annexe 4.

**Residual strength**

A slope revetment which consists of block mats shows other damage mechanisms than a conventional setting. For practical tests have shown that particularly the formation of an S-profile is an important damage mechanism. Especially with mats on sand, soil-mechanical instability can play a part.

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.

The differences with conventional settings result in the definitions of initial damage and
collapse of the revetment having to be adjusted slightly. Initial damage is now an irreversible deviation of the mat compared to the laid surface. The revetment is said to have collapsed when the scour depth under the top layer is larger than or equal to 1.0 m. The residual strength is the time between initial damage occurring and the revetment collapsing.

Also, practical tests have shown that, even with serious overloading, block mats may have a residual strength of the duration of several storms.

In a model for damage development, the scour depth is related to the number of waves. In the function regulation the extent of overloading plays an important part. The properties of the underlying material are probably hardly relevant (apart from the criteria of soil-mechanical instability). A start for the development of the model is partly based on the prototype data of some test sections. There are, however, as yet not sufficient data to work out the model further and to verify it. The development of the model is initially best concentrated on the most important damage mechanism, the formation of an S-profile.

Mending damage to block mats and mattresses requires more attention than to conventional settings, particularly because of the necessary deployment of special equipment and the requirements for mending the couplings. It is therefore advisable to develop a strategy for mending damage, especially if the used design criterion allows some damage.

Background information can be found in Annexe 5.

3.2.2 Design rules with regard to wave load

Black-box Model

Table 3.1 gives an overview of usable values for the revetment constant A in the black-box model for linked blocks (block mats and interlock systems).
Analytical Model

The stability of a block mat or interlock system is calculated by applying an increase factor ($\Gamma_{\text{block mat}}$) to the strength of a comparable setting of loose blocks. The value of $\Gamma_{\text{block mat}}$ depends on the width of the block, the leakage length, the slope gradient and the height and angle of the front of the rise. The formulas for $\Gamma_{\text{block mat}}$ are not very practical. For application in practice, therefore, the result has been graphically represented in Figures 6 and 7.

The size of $\Gamma_{\text{block mat}}$ depends on:

\[
\frac{B}{\Lambda} \frac{l_2}{\Lambda}
\]

with:

- $B$ = block width (m)
- $\Lambda$ = leakage length (m)
- $l_2$ = \[
\min \left\{ \frac{0.19 \sqrt{H_s L_{\text{op}}}}{\cos \alpha}, \frac{0.061 L_{\text{op}} \sqrt{\tan \alpha}}{\cos \alpha} \right\}
\]
- $\alpha$ = slope angle (°)
- $H_s$ = significant wave height (m)
- $L_{\text{op}}$ = wavelength on deep water (m)

In the above notation, "min{a;b}" means the choice of the smallest value of $a$ and $b$.

In which the effect of the wash-in material on the strength is always neglected, because this effect is probably only noticeable with large displacements, which are generally not acceptable.

Roughly speaking, the value of $\Gamma_{\text{block mat}}$ lies between 1.0 and 1.2, which is fairly low. It does, however, correspond to observations that the extra strength of block mats and interlock systems is small or not worth mentioning if only small movement is allowed.

The extra strength with larger allowed movement is probably caused by the bending stiffness of the connection between the elements. This bending stiffness has for the time being been left out of consideration. Should the design criterion be formulated more broadly than the current small movement, this bending stiffness should be quantified. However, no calculation method is available for this yet.

3.2.3 Design rules with regard to flow load

The following constants should be used in the design formulas for flow load (see section 2.2.2):

\[
(\text{representative}) \text{ } \text{Strength } \Delta D
\]

The strength parameters $\Delta$ and $D$ follow from the standard definitions.

**Stability Parameter $\Phi$**

- $\Phi = 0.50$ for a flat bottom, without transitions
- $\Phi = 0.75$ for protruding edges, a rough bottom or transitions
Critical Shields Parameter $\Psi$

$\Psi = 0.07$

Revetment parameter $a$

For block mats through which vegetation can grow (Dycel, Petraflex, Armorflex), and which are approximately 10 cm thick, with a well-developed vegetation it applies that $a \approx 2.5$. For other systems insufficient information is available. It can be mentioned, however, that the spread in $a$ can be large. Thus, the value of $a$ for systems with a relatively large distance between the blocks and weak interconnections can be found to lie close to that for riprap ($a \approx 0.11$).

Equivalent roughness according to Nikuradse $k_s$

De roughness of the revetment strongly depends on the type of block mat (blocks with or without holes, grown through with grass or filled with riprap, etc.). The value of $k_s$, therefore, can lie between approximately 0.01 for flat or well grown-through revetments to approximately 0.10 m for very rough revetments.

Angle of internal friction $\theta$

For the top layer, an angle of internal friction does not apply. In the calculation one can use:

$\theta = 90^\circ$

3.2.4 Design rules with regard to soil-mechanical stability

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.

Elastic storage

Figure 8 shows the design diagram for the stability of the systems if these are placed on sand. In this diagram, moderately packed sand is assumed (the angle of internal friction is $35^\circ$). Should there be a sound compaction of the whole pack of sand, it is also possible to use the diagrams on the pages 180/181 in (CUR/TAW, 1992). They concern tightly packed sand with an angle of internal friction of $40^\circ$.

If the system is placed on a filter layer, in the diagrams one can allow for a contribution of the filter layer thickness $b$ to the thickness of the top layer. One can fill in $D + b/1.2$ for the total thickness of the top layer.

In a fresh clay subsoil the maximum water tension occurs so closely beneath the surface that there is no danger of sliding in 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 structure during the life of the revetment. For this situation, the design diagram in Figure 9 is given.

**Softening**

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 softening do not deviate from those presented in section 2.3.3.

**Drop in the water level**

The danger as a result of drop in the water level depends on the leakage length, i.e. the characteristics of the top layer and underlayer, see section 2.3.4. 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 the formulas in section 3.1.2. The strength parameters $\Delta$ and $D$ follow from the standard definitions. See also section 2.3.4.

### 3.3 Concrete Mattresses

#### 3.3.1 System Description

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 each mattress system having its own appearance and properties. In figure 10, a number of examples are given.

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 differential pressures across the mattress remain smaller. The stability is therefore the largest with a large mattress permeability. In the long term, however, pollution of the Filter Points or the clogging of the geotextile can cause a decrease in the permeability.

Background information can be found in Annexe 6.

#### 3.3.2 Design rules with regard to wave load

In the design rules for concrete mattresses with regard to wave load, an adapted damage mechanism is assumed. Moreover, the calculation of the leakage length is adapted. This consideration, which is closely related to a consideration in accordance with the analytical model, results in a design formula in the form of the black-box formula.
During wave attack, the mattress will be exposed to a differential pressure which is directed upwards, as also is the case with packed stone revetments. This takes place the moment the wave has drawn back, just before the wave impact. Just as with packed stone revetments, the leakage length for this differential pressure is the most important construction-descriptive parameter. Some characteristic values are given in Table 3.2.

The failure mechanism of the concrete mattress is probably as follows:

- First, cavities under the mattress will form 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 the spans to collapse. This results in a mattress consisting of concrete slabs which are somewhat coupled by means of the geotextile.
- With sufficiently high waves, an upward difference in rise over the mattress will occur during wave run-down, which lifts the mattress.
- The pumping action of these movements will cause the subsoil to migrate, as a result of which an S-profile will form and the revetment will collapse completely.

The value of A in the design formula of the black-box model depends on the leakage length and the subsoil: A = 2 to 4. A permeable mattress on sand has a medium-sized or small leakage length and then the value of A is 3 to 4. A low-permeable mattress on a filter has a large leakage length and therefore an A-value of 2 to 3. For the determination of the leakage length, one is referred to the analytical model.

The representative relative density A follows from the standard definition. For the representative thickness D, the (over the surface) average thickness should be filled in.

It can be concluded that, compared to the available data in literature, the derived stability relations give a safe estimation of the stability. Because the relations have not been verified sufficiently yet, it is not recommended to decrease the existing safety.

In the long run, the permeability of the top layer may diminish as a result of accretion and silting-up. This will have a negative effect on the stability, especially with systems with a leakage length smaller than approximately 2 m. If the leakage length is larger than 2 m, the effect of the permeability of the top layer on the stability is rather small.

### 3.3.3 Design rules with regard to flow load

Table 3.3 gives a number of characteristic values for the critical flow velocity. For the application of the design formulas, guide values for the constants are given below.

<table>
<thead>
<tr>
<th>(representative) Strength ΔD</th>
</tr>
</thead>
</table>

The representative relative density A follows from the standard definition. For the representative thickness D, one should fill in the (over the surface) average thickness.

Stability parameter $\Phi$

$\Phi = 0.50$ for a flat bottom, without transitions

$\Phi = 0.75$ for protruding edges, a rough bottom or transitions
Critical Shields parameter $\Psi$

$\Psi = 0.07$

Revetment parameter $a$

For concrete mattresses, only data on longitudinal flow velocities are known, from which no values for $a$ can be derived.

Equivalent roughness according to Nikuradse $k_e$

The roughness of the revetment depends on the type of mattress. The value of $k_e$ probably lies around 0.05 m.

Angle of internal friction $\theta$

$\theta = 90^\circ$

3.3.4 Design rules with regard to soil-mechanical stability

The flow through a concrete mattress is concentrated in the Filter Points. The permeability of the systems filled with concrete lies approximately between $1 \times 10^4$ en $5 \times 10^3$ 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 diagrams are given:

- lifting of (a part of) the revetment (Figures 11 and 12);
- sliding of the whole revetment (Figures 13 and 14);
- sliding of a part of the subsoil, including the revetment (Figures 15 and 16).

If the systems are placed on a filter, one can take into account an increase in the stability with regard to elastic storage. For the total thickness of a concrete mattress on a filter, $D + b/1.1$ can be filled in.

Softening

The design rules with regard to softening do not differ from those presented in section 2.3.3.

Drop in the water level

The danger as a result of drop in the water level depends on the leakage length, i.e. the characteristics of the top layer and underlayer, see section 2.3.4. With a concrete mattress on sand, the leakage length is generally small and the danger due to a drop in the water level is also small.

The representative relative density $\Delta$ follows from the standard definition. For the
representative thickness $D$ one should fill in the (over the surface) average thickness. See also section 2.3.4.

### 3.4 Sand Mattresses

#### 3.4.1 System description

A sand mattress consists of two geotextiles attached onto each other, between which sand is interposed, see Figure 17. This way, a mattress is formed of sausages lying next to each other which run from the top to the bottom of the slope and which are interconnected. The lower geotextile is usually flat and the upper geotextile lies on top of it, in arches.

Sand mattresses cannot be used in situations where the significant wave height is higher than 1.0 m or where the flow velocity is larger than 1.5 m/s.

**Construction/design/repair**

The edges and connections of sand mattresses are vulnerable and must therefore be finished carefully. Mattresses lying next to each other can be sown together and the ends can be secured with for example ground anchors.

In actual practice, mattresses are not only threatened by the hydraulic load. The possibility of vandalism occurring, limits sand mattresses to being applied in places where unauthorized persons do not have access to. The system is, however, also vulnerable to collision, (drifting) ice, floating bulky refuse, sunlight and chemical degradation.

Background information can be found in Annexe 6.

#### 3.4.2 Design rules with regard to wave load

Sand mattresses cannot be used when the significant wave height $H_s$ is larger than 1.0 m.

Unfortunately, not much research has been conducted into the stability of sand mattresses. Besides Pilarczyk's design formula (1990), a small-scale model investigation, a desk study and a prototype experiment have been found. Based on these, the following value for $A$ in the design formula is recommended according to the black-box model: $A = 4$ to $5$, so that:

$$\frac{H_s}{\Delta D} = \frac{4^{1/5} 0.5^{1.0 \text{ m}}}{5^{2/3}}$$

In which the relative density including pores $\Delta_r$ should be filled in for the representative relative density.

For the representative thickness $D$ of the mattress, the average thickness should be filled in:
The above design formula matches the experience gained in the Hartelkanaal (Verheij et al., 1984) reasonably well and is conservative compared to the results of the small-scale model investigation (Van Hijum, 1975), but suggests a clearly higher stability than according to Pilarczyk (1990). The maximum permissible wave height is derived from the supplier’s experience.

To derive a design formula on the basis of the assumption that a great deal of water transmission between the mattress and the sand occurs, as with blocks on a geotextile on sand, is probably not sensible here. The large flexibility leads to a rather good connection between mattress and subsoil.

It should be mentioned that the above stability relation concerns a construction that cannot collapse because of instability of the edges and interconnections.

### 3.4.3 Design rules with regard to flow load

Above a flow velocity of 1.5 m/s, the sand in the mattresses is no longer internally stable, as is more or less the case for all geosystems filled with fine material. The design formulas are given in section 2.2.2.

(\text{representative}) \textbf{Strength} \Delta D

For the representative relative density, the relative density including the pores $\Delta_m$ should be filled in and for the representative thickness $D$ of the mattress, the average thickness should be filled in.

\begin{itemize}
  \item \textbf{Stability parameter} $\phi$
    \begin{align*}
      \phi &= 0.50 \quad \text{for a flat bottom, without transitions} \\
      \phi &= 0.75 \quad \text{for protruding edges, a rough bottom or transitions}
    \end{align*}
  \\
  \textbf{Critical Shields parameter} $\psi$
    \begin{align*}
      \psi &= 0.07
    \end{align*}
  \\
  \textbf{Revetment parameter} $a$
    \begin{align*}
      \text{No data with regard to the value of } a \text{ are known for sand mattresses.}
    \end{align*}
\end{itemize}
Equivalent roughness according to Nikuradse $k_s$

The roughness of the revetment depends on the type of sand mattress. The value of $k_s$ probably lies around 0.05 m.

Angle of internal friction

The angle of internal friction for sand mattresses probably lies around the value for sand:

$$\theta = 30 \text{ to } 40^\circ$$

3.4.4 Design rules with regard to soil-mechanical stability

A sand mattress is relatively flexible and connects closely to the subsoil. The flow through a sand mattress occurs more or less equally divided over the surface. The permeability of the sand-filled systems, for example with sand of 400 $\mu$m, is approximately $2 \cdot 10^{-3}$ m/s. In the sand mattress, damping of the pressure variations occurs, just as in the subsoil. This means that the load on the subsoil decreases as the thickness used for the sand mattress is larger.

Elastic storage

With regard to elastic storage the following design diagrams are given:

- lifting of (a part of) the revetment (Figures 18 and 19);
- sliding of a part of the revetment (Figures 20 and 21);
- sliding of the whole revetment (Figures 22 and 23);
- sliding of a part of the subsoil, including the revetment (Figures 24 and 25).

If the systems are placed on a filter, in these diagrams one can allow for an increase in the stability with regard to elastic storage. $D + b/0.9$ can be filled in for the total thickness of a sand mattress on a filter. A sand mattress on a filter is seldom used, though.

Softening

For sand mattresses there is no danger of instability as a result of softening.

Drop in the water level

In the initial situation, the permeability of the top layer is larger than that of the subsoil. The leakage length is small. As a result of this, no large differences in rise due to tide occur directly under the top layer. Only when the slope is steep (1:2, for example) and the mattress is filled with fine sand, need calculations be made on the drop in the water level.

For the representative relative density, the relative density including pores $\Delta_m$ should be filled in here, and for the representative thickness $D$ of the mattress, the average thickness.

3.5 Geosystems
3.5.1 System description

Geosystems in hydraulic engineering are bags filled with sand or gravel (or cement, perhaps). The bags may have different shapes and sizes, varying from the well-known sandbags for emergency dikes to large flat shapes or elongated "sausages", see Figure 26.

The most common use for sandbags in hydraulic engineering is that for temporary constructions. The reasons why sandbags are not or hardly used for permanent constructions are as follows:

- the resistance against flow load and wave load is relatively small;
- because the geosystems are prone to vandalism and the effect of sunlight, for example, the durability is relatively small;
- good design formulas are lacking;
- a construction made of sandbags looks ugly.

Major advantages of sandbags as construction material are:

- low costs;
- simple processing;
- the elements can be tailor-made.

Uses

Uses for sand- or cement-filled bags are, among other things:

- revetments of relatively gentle slopes;
- temporary toe constructions in places where in due course vegetable toe constructions should develop;
- (temporary) training walls
- temporary or permanent offshore breakwaters
- temporary dikes surrounding dredged material containment areas.

Sand containers and sand sausages, in particular, lend themselves to the last three uses. These long "sausages" ("tubes") filled with sand are a specific form of a retaining wall. They are mainly used when the retaining height is not too large. Sand sausages can also be used for retaining constructions for beach nourishments.

Because this material is easy to use and cheap, it is extremely suitable for temporary constructions. A training wall is a good example. The working of a training wall is difficult to predict in advance. That is why it is a good procedure to make such a construction using a relatively cheap product first, to see how one thing and another works out, and subsequently either make improvements or, after some time, a permanent construction.

Above a flow velocity of 1.5 m/s, the geosystems cannot be used because the sand in the systems is no longer internally stable.

Construction/design
Sandbags can be placed as follows:

1. **As a blanket:** One or two layers of bags placed directly on the slope. An "interlocking" problem arises if the bags are filled completely. The bags are then too round. A solution is not to fill the bags completely, so that the sides flatten out somewhat, as a result of which the contact area becomes larger.

2. **As a stack:** Bags stacked up in the shape of a pyramid. The bags lie half-overlapping with the long side parallel to the shoreline.

When installing geosystems, one should see to it that this does not take place on a rough foundation. Sharp projections may easily damage the casing of the element.

Geosystems must not be filled completely. With a fill ratio of approximately 75% an optimal stability of the elements is reached.

A sound soil protection is necessary if gravel (sand) sausages are used in circumstances where they are under attack of flow or waves.

When using gravel or sand-filled bags as slope protection, regular inspections should be conducted into damage of the bags. When damage is detected, it should be repaired.

Background information can be found in Annexe 7.

### 3.5.2 Design rules with regard to wave load

The stability relation of sand, gravel or cement bags which are used as protection elements on a slope appears to deviate somewhat from the formula according to the black-box model. For regular waves the recommended formula is as follows:

\[
\left( \frac{H}{\Delta D} \right)_{cr} = \frac{3.5}{\sqrt{\frac{\Delta \rho}{\Delta \rho}}}
\]

In which \(\Delta \rho\) is the relative density if the pores are completely filled with water (\(\Delta \rho_w\)). The representative thickness \(D\) is the average thickness of the top layer, measured perpendicularly to the slope.

If this stability relation is combined with the relation found between \(H_s\) and \(H\), (significant wave height with irregular waves and the wave height with regular waves) this results in the following stability relation:

\[
\left( \frac{H_s}{\Delta D} \right)_{cr} = \frac{2.5}{\sqrt{\frac{\Delta \rho}{\Delta \rho_{on}}}}
\]

For concrete sausages used as a protection element on the crest of a low or underwater breakwater, it is found that the following stability relation for regular waves can be used:

\[
\left( \frac{H}{\Delta b} \right)_{cr} = 3.2 \left( \frac{H}{L_0} \right)^{1/3}
\]
In which $b$ is the width of the sausage. Should two sausages be connected, the widths of both sausages together can be filled in for $b$. If the sausage is placed with its longitudinal direction perpendicularly to the axis of the breakwater, the following stability relation applies:

$$ \left( \frac{H}{\Delta l} \right)_{cr} = 1.0 $$

In which $l$ is the length of the sausage.

3.5.3 Design rules with regard to flow load

Above a flow velocity of 1.5 m/s, the sand in the systems is no longer internally stable, as is more or less the case for all geosystems filled with fine material. This value serves as the upper limit in the design. The following constants can be used for the application of the design formulas (see section 2.2.2).

(representative) Strength $\Delta D$

The relative density should be filled in for $\Delta$ if the pores are completely filled with water ($\Delta_w$). The representative thickness $D$ is the thickness of the top layer measured perpendicularly to the slope.

<table>
<thead>
<tr>
<th>Stability parameter $\Phi$</th>
<th>Value</th>
<th>Condition</th>
</tr>
</thead>
<tbody>
<tr>
<td>$\Phi_f = 0.50$</td>
<td></td>
<td>for a flat bottom, without transitions</td>
</tr>
<tr>
<td>$\Phi_f = 0.75$</td>
<td></td>
<td>for protruding edges, a rough bottom or transitions</td>
</tr>
</tbody>
</table>

Critical Shields parameter $\Psi$

$\Psi = 0.07$

Revetment parameter $a$

No data with regard to the value of $a$ are known for geosystems.

Equivalent roughness according to Nikuradse $k_r$

The roughness of the revetment strongly depends on the type of geosystem. For which no guide value can be given.

Angle of internal friction $\theta$

For sand mattresses, the angle of internal friction probably lies around that for sand:

$\theta = 30$ to $40^\circ$

For cement-filled systems, the value is probably higher.

3.5.4 Design rules with regard to soil-mechanical stability
The interaction between hydraulic load, the revetment and the subsoil may play a part in the stability of geosystems, especially if the top layer is less permeable than the subsoil. Sandbags on gravel and cement-filled bags on coarse sand are examples of such situations.

In which way the soil-mechanical stability can be an issue strongly depends on the shape of the construction. So-called "Longard tubes" or sand sausages are used as dune-toe protection, for example. For this purpose, it is particularly important to use filtering cloth to protect the subsoil against erosion. The failure mechanism with this type of use is usually the eroding of the earth on both sides of and under the sausage, or the tearing of the geotextile. Perhaps wave impacts on the sausage cause the sand under the sausage to tend to soften (to become quicksand), but it is more likely that the large thickness of the sausage has a damping effect on the load on the subsoil. Which is why this chapter pays no further attention to this type of construction.

Attention is focused on the use of sand- and cement-filled bags on a slope gradient used as protection against wave attack and/or flow. This often concerns temporary solutions. If the bags are designed as a (semi)permanent solution, a geotextile or cloth should be applied under the bags to prevent erosion of the subsoil.

**Elastic storage**

In Figures 27 through 38, the design diagrams are given for the stability with regard to elastic storage. The design diagrams are for the use of the systems on sand or clay. If the systems are placed on a filter, the thickness of the filter layer $b$ can be included in the thickness of the top layer in the above-mentioned design diagrams, as follows:

- For cement-filled systems on a filter, $D + b/1.1$ can be filled in for the thickness of the top layer;
- For sand-filled systems on a filter, $D + b/0.9$ can be filled in for the thickness of the top layer.

**Softening**

Because sandbags have a permeability which is just as large or larger than that of the subsoil, there is no cause for pressure build-up under the top layer. The same goes for the cement-filled systems, although locally under the cement the water cannot flow away easily. However, averaged over the top layer, the proposition also applies to cement-filled systems.

When formulating design rules with regard to softening, the permeability of the sandbags and the cement bags are equated with that of the subsoil. For cement-filled bags, the stiffness is chosen a factor 10 larger.

The design rules for cement-filled systems do not differ from the rules presented in section 2.3.3. For sand-filled systems there is no danger of the subsoil softening.

**Drop in the water level**

In the initial situation, the permeability of the top layer is larger than that of the subsoil. The leakage length is small. As a result of this no large differences in rise occur directly under
the top layer as a result of tide. Calculations on the drop in the water level only have to be made if the slope is steep (1:2 for example) and the system is filled with fine sand.

For $\Delta$, one should fill in the relative density in the case of the pores being completely filled with water ($\Delta_w$). The representative thickness $D$ is the thickness of the top layer measured perpendicularly to the slope.
3.6 Gabions and Reno Mattresses

3.6.1 System description

A slope protection made of gabions consists of a layer of wire mesh baskets linked together and filled with graded stone. The idea behind it can be compared to that for geosystems, i.e. stabilizing lightly erodible material by making packages of it. The materials used for gabions and reno mattresses are much coarser than those used for geosystems. Moreover, the gabion and reno mattress packages are usually interconnected.

Gabions are used in several ways and for various purposes. From a constructive point of view, gabions can be stacked or laid on the slope, see Figure 39. In the first case, the gabions are often block-shaped and the construction also has an earth-retaining function. In the second case, they are shaped like a mattress and chiefly function as a slope revetment. In this study, attention is mainly paid to the use of gabions as mattresses under wave attack.

Gabions used as slope revetment can have various sizes. The length is often 3 to 4 m, the width 1 to 3 m and the thickness 0.5 to 1 m. The gabion is usually subdivided into separate cells. Gabions with a thickness smaller than 0.5 m are generally called "reno mattresses". A reno mattress is not as thick as a gabion and is larger both in length and width. The name is derived from the river Reno in Italy where they are used as bank protection. In the following, reno mattresses will also be called gabions.

To prevent erosion of the subsoil, a granular filter or a geotextile is often applied under the gabion. In salty environments, the wire mesh of the basket is often plasticized and/or made of stainless steel.

In most cases, the filling material will move before the gabion itself becomes unstable. As the filling material moves more, wear of the wire mesh and the possible breaking open of the gabions will play a more important part.

Construction/repair

Gabion revetments are often made on site by first installing the empty baskets and interconnecting them, after which the baskets are filled and the wire mesh lid is attached to the edges and the partitions between the cells. However, it is also possible to use prefabricated gabions.

Damage mechanisms

It is found that the following damage mechanisms can occur:

a. **Lifting the gabion/lifting the gabion out of the layer**
   - due to an upward pressure gradient in the gabion due to wave load,
   - due to the bending upwards under pressure of the gabions lying on top.

b. **Slipping of the gabion**
   - due to use on a slope that is too steep,
   - due to insufficient compaction of the filter layer.

c. **Breaking open of the gabion**
— due to wear (physical or chemical),
— due to damage during construction,
— due to vandalism,
— due to ageing.

d Deformation of the subsoil
— due to a bad filter construction,
— due to transport of sand along the slope during lifting,
— due to soil-mechanical instability.

The different damage mechanisms cannot be considered separately. The lifting mechanism of the gabion in particular probably plays an important part in other damage mechanisms, such as slipping and deformation of the subsoil.

Background information can be found in Annex 8.

3.6.2 Design rules with regard to wave load

Black-box model

The formula devised by Pilarczyk (1990) turned out to be very satisfactory as the black-box model for the stability of gabions. The adapted form of this formula is:

\[ \frac{H_s}{\Delta D} = \frac{9 \cos \alpha}{\xi^{2/3} \xi_{dp}} \]  (3.13)

The difference from the form of the black-box formula (2.4) used up to now is only the factor \( \cos \alpha \) in the right term. In black-box terms, therefore, \( A = 9 \cos \alpha \) is recommended in this case. Because formula (3.9) is generally recommended for slopes with gradients of 1:2 or slighter, \( \cos \alpha = 0.89 \) to 1.00 applies, so that \( A = 8 \) to 9. For the representative relative density, \( \Delta_m \) must be filled in.

This formula applies for irregular waves with wave height \( H_s \), whereas the model tests which confirm this relation were carried out with regular waves with wave height \( H \). This suggests that the transformation of regular waves to irregular waves is given by \( H_s = H \).

No unstable measuring results were found for \( \xi_o \)-values smaller than 2. This means that for \( \xi_o < 2 \), the stability relation according to the black-box model is not supported by measurement data. The presented black-box formula appears to give a safe value here.

It is especially in the above-mentioned area with \( \xi_o \)-values smaller than 2 where the lifting of the gabion determines the stability criterion. For larger values of \( \xi_o \), the aspect of slipping (stimulated by lifting) increasingly plays a part.

Analytical approach

A good analytical approach of the development of the rise under the gabions can be obtained by applying the formulas for the development of the rise in a filter under a packed stone revetment, with as leakage length:
$$\Lambda = 0.77 \, D$$  

(3.10)

With which 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 the Handboek (CUR/TAW, 1992) provides stability relations which indeed match the line according to the black-box model well.

In the above consideration, the effect of the roughness and the permeability of the gabions on the relation between the wave characteristics and the normative pressures on the slope are assumed to be negligible.

### 3.6.3 Design rules with regard to flow load

The design formulas were given in section 2.2.2.

**(representative) Strength \( \Delta D \)**

For the representative relative density, the relative density including pores should be filled in \( \Delta_n \). The representative thickness \( D \) is the thickness of the gabion measured perpendicularly to the slope.

**Stability parameter \( \Phi \)**

- \( \Phi = 0.50 \) for a flat bottom, without transitions
- \( \Phi = 0.75 \) for protruding edges, a rough bottom or transitions

**Critical Shields parameter \( \Psi \)**

- \( \Psi = 0.07 \)

**Revetment parameter \( a \)**

The value of \( a \) for gabions lies roughly between 0.28 and 0.42, depending on the thickness and the type of stone. An overview is given in table 3.4.

**Equivalent roughness according to Nikuradse \( k_e \)**

The roughness of the gabion depends on the stone sizes, among other things. For which no guide value can be given.

**Angle of internal friction \( \theta \)**

For the time being, one is recommended to use:

- \( \theta = 50^\circ \)

### 3.6.4 Design rules with regard to soil-mechanical stability

Characteristic of gabions is that, compared to the subsoil, these systems have a large
permeability. As a result of this, the stability of the top layer is determined by the differences in rise in the gabion itself and there is only a limited effect of the interaction with the subsoil. A failure mechanism such as the gabion sliding over the subsoil is determined by "slip forces" due to downward flowing water and differences in rise in the gabion.

A disadvantage of a very permeable system such as a gabion is that the hydraulic load on the subsoil is only damped to a limited extent. So the load acts rather directly on the subsoil. In connection with which the gabion must have a certain weight to be able to keep the subsoil stable.

**Elastic storage**

A gabion has a larger permeability than the subsoil. So a large difference in rise directly under the top layer will not occur quickly. This means that the mechanisms "lifting the top layer" and "sliding of the top layer", as far as they are connected with elastic storage, are not normative. There is, however, interaction between the subsoil and the flow in the gabion. Because of the large permeability of the gabion, the load on the subsoil is also relatively large. This is particularly important to the stability of the subsoil itself.

The Figures 40 and 41 give design diagrams for the stability of the subsoil. If the gabion is placed on a filter layer, the weight of the filter layer contributes positively to the stability. This way, the thickness of the filter layer may be added to the thickness of the gabion to obtain a "thickness of the top layer", as is used in the design diagrams.

**Softening**

The design rules with regard to softening do not differ from the rules presented in section 2.3.3.

**Drop in the water level**

In the initial situation, the permeability of the top layer (gabions) is larger than that of the subsoil. The leakage length is small. As a result of this, no large differences in rise due to tide occur directly under the top layer. A drop in the water level is not expected to present danger.
4 Sample Calculations

4.1 Blocks on sand under wave load

Wave load

A block revetment on a geotextile on sand is used on a dike under wave load. The following boundary conditions are given:

Wave conditions:

- Wave height $H_h = 0.9 \text{ m}$
- Wave period $T_p = 4.0 \text{ s}$
- The water is deep.

Construction:

- Slope gradient $\cot \alpha = 4.0$
- Relative density $\Delta = 1.3$
- It is possible that there is some gully formation. The leakage length is estimated to be 0.6 m. The core consists of reasonably well compacted sand with $D_{50} = 0.150 \text{ m}$. There is a good toe anchoring.

According to the black-box model, the required block thickness is found as follows:

<table>
<thead>
<tr>
<th>Property</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Wavelength</td>
<td>$L_{op} = 25.0 \text{ m}$ from Formula (2.5)</td>
</tr>
<tr>
<td>Wave steepness</td>
<td>$H_h/L_{op} = 0.036$</td>
</tr>
<tr>
<td>Breaker parameter</td>
<td>$\xi_{op} = 1.32$ from Formula (2.4)</td>
</tr>
<tr>
<td>Revetment parameter</td>
<td>$A = 3.7$ to $8.0$ from Table 2.1</td>
</tr>
<tr>
<td>Critical load</td>
<td>$H_s/\Delta D^c = 3.1$ to $6.7$ from Formula (3.1)</td>
</tr>
<tr>
<td>Required block thickness</td>
<td>$D = 0.10$ to $0.23 \text{ m}$</td>
</tr>
</tbody>
</table>

This wide range of recommended values for $D$ only gives a first indication of a suitable choice. The value of $D$ might also be determined more accurately using the analytical model. For the time being, a block thickness of 0.15 m seems to be a reasonable choice.

Soil-mechanical stability

Below, the above design will be checked with regard to the soil-mechanical stability. For this purpose, a chosen block thickness $D$ of 0.15 m is assumed.

Sliding due to elastic storage

There is a good toe anchoring. The permissible wave height for the block thickness that is chosen follows from interpolation of the results from Figures 1 through 4. The calculation procedure is represented in the table below:
The results for the load situations 1 and 2 are the same in this case. The permissible wave height is 2.6 m. The design wave height of 0.9 m, therefore, does not present danger as a result of elastic storage.

**Softening**

The sand is compacted reasonably well and the slope gradient is slighter than 1:3. According to the design rules in section 2.3.3, this means that there is no danger of softening.

**Drop in the water level**

Substitution of the known variables in Formula (2.17) leads to the conclusion that there is no danger as a result of a drop in the water level. No more calculations have to be made on the effect of a drop in the water level to the stability.

### 4.2 Block mat under load of water flowing longitudinally

A slope of 1:2 is under load of water flowing longitudinally. The slope forms the bank of a discharge canal and is situated near an outlet opening. It is being investigated if a block mat with a thickness of 0.10 m on a geotextile on sand is stable with regard to the flow load.

**Flow conditions:**

- Water depth \( h = 1.5 \) m
- Normative flow velocity \( u_{nr} = 3.0 \) m/s
- The turbulence is heavy and the flow is nondeveloped.

**Construction:**

- Slope gradient \( \cot \alpha = 2.0 \)
- Relative density \( \Delta = 1.3 \)

There is a flat slope, without transitions.
Below, the required block thickness is determined on the basis of the flow velocity, see Formula (2.7).

Turbulence factor \( K_r = 2.0 \) from Table 2.4
Water depth parameter \( K_h = 0.31 \) from Formula (2.9)
Angle of internal friction \( \theta = 90^\circ \)
Slope parameter \( K_s = 0.89 \) from Formula (2.11)
Stability parameter \( \phi = 0.5 \)
Critical Shields parameter \( \psi = 0.07 \)

Substitution of these data and the values for \( \Delta \) and \( u_\tau \) in the design formula (2.7) produces:

Required thickness of top layer \( D = 0.06 \) m

The chosen block thickness of 0.10 m is therefore satisfactory.

There are no waves and therefore there is no danger of soil-mechanical instability occurring.

### 4.3 Concrete mattress under wave load

#### Wave load

A 1:3 slope is protected by a concrete mattress on a granular filter. The mattress has an average thickness of 0.15 m. Below, the maximum permissible wave height is determined, for which the following conditions must be taken into account.

Wave conditions:

Wave steepness \( H_y/L_{opt} = 0.040 \)

The water is deep.

Construction:

Slope gradient \( \cot \alpha = 3.0 \)
Thickness of top layer \( D = 0.15 \) m
Relative density \( \Delta = 1.3 \)
Thickness of filter layer \( b_f = 0.20 \) m

The base material sand is reasonably well compacted. The mattress is low-permeable.

The maximum permissible wave height according to the black-box model is found as follows:

Breaker parameter \( \xi_{opt} = 1.68 \) from Formula (2.4)
Revetment parameter \( A = 2.0 \) to 3.0
Critical load \( \left( \frac{H_s}{\Delta D} \right)_{cr} = 1.42 \) to 2.13 from Formula (2.6)
From this follows that the maximum permissible wave height $H_s$ is 0.28 to 0.42 m.

**Soil-mechanical stability**

Next, it is investigated if the soil-mechanical instability is jeopardized by a wave height of 0.40 m.

*Sliding due to elastic storage*

The permissible wave height for the chosen block thickness follows from the design diagrams. For the top layer, 0.33 m can be used because of the existing filter layer.

- Lifting of a part of the revetment $H_s = 0.84$ m from Figure 11
- Sliding of the whole revetment $H_s = 1.04$ m from Figure 13
- Sliding of the subsoil $H_s = 0.84$ m from Figure 15

The critical wave height with regard to elastic storage is found to be 0.84 m. For a wave height of 0.40 m there is no danger what this is concerned.

*Softening*

The sand is reasonably well compacted and the slope gradient is 1:3. According to the design rules in section 2.3.3, this means that there is no danger of softening occurring.

*Drop in the water level*

Substitution of the known variables in Formula (2.17) leads to the conclusion that there is no danger as a result of a drop in the water level if the leakage length is smaller than 1.17 m. In this case, this condition is probably not fulfilled. Further calculations should therefore be made, for which one is referred to Annexe 2.

**4.4 Sandbags under wave load**

A damaged part of a slope is temporarily protected against wave attack by sandbags. It is investigated whether the sandbags are stable under wave load if the average thickness of the top layer is 0.25 m. For this purpose, the following conditions should be taken into account:

Wave conditions:

- Wave height $H_s = 0.40$ m
- Wave period $T_p = 2.7$ s
- The water is deep.

Construction:

- Slope gradient $\cot \alpha = 3.0$
- Relative density $\Delta_m = 1.0$
- The underlayer consists of clay.

The required block thickness according to the black-box model is found as follows:

- Wavelength $L_{wp} = 11.4$ m from Formula (2.5)
Wave steepness \( \frac{H}{L_{op}} = 0.035 \)
Breaker parameter \( \xi = 1.78 \) from Formula (2.4)
Critical load \( \left( \frac{H_s}{\Delta D} \right)_{cr} = 1.87 \) from Formula (3.10)
Required block thickness \( t = 0.21 \text{ m} \)

The chosen sandbags prove to be stable.

The soil-mechanical stability is discussed below:

*Lifting due to elastic storage:*
From figure 29 it follows that the permissible wave height is 0.9 m.

*Sliding of the top layer due to elastic storage:*
From figure 33 it follows that the permissible wave height is 1.1 m.

*Sliding of the subsoil due to elastic storage:*
From figure 37 it follows that the permissible wave height is 0.6 m.

The above values for the permissible wave height prove that in this case no danger is expected as a result of elastic storage.

*Softening:*
The design guidelines in section 2.3.3 show that there is no danger of softening for the top layer on clay.

*Drop in the water level:*
The top layer is more permeable than the underlayer. Therefore, no danger as a result of a drop in the water level is expected.

Summarizing, it can be stated that no danger to the soil-mechanical stability is expected.

### 4.5 Load on gabions due to flooding water

A low dike with an inner and outer slope of 1:2 is loaded by flooding water, because the water level becomes 0.3 m higher than the crest. Moreover, there is some wave action \( (H_s = 0.45 \text{ m}) \). It is investigated whether gabions with a thickness of 0.40 m and filled with small stones form a stable revetment for this dike. For which the following conditions are taken into account:

**Load conditions:**

- Wave height \( H_s = 0.45 \)
- Wash-over height \( h_{ov} = 0.30 \text{ m} \)

**Construction:**

- Gradient of the inner slope \( \cot \alpha = 2.0 \)
Overflow coefficient \( m = 1.0 \)

Thickness of top layer \( D = 0.4 \) m

Relative density \( \Delta_m = 1.0 \)

Below, the required block thickness is determined on the basis of the discharge, see formula (2.13).

Revetment parameter \( a = 0.29 \) from Table 3.4

Equivalent wash-over height \( h_{eq} = 0.45 \) m from Formula (2.15)

Equivalent discharge \( q_{eq} = 0.51 \) m\(^2\)/s from Formula (2.16)

Substitution of these values (\( q_{eq} \) by \( q_{eq} \)) in the design formula (2.13) provides:

Required thickness of the top layer \( D = 0.37 \) m

The chosen block thickness of 0.40 m is therefore satisfactory.
References

<table>
<thead>
<tr>
<th>Type of revetment</th>
<th>A (-)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Blocks on sand</td>
<td>3.7 - 8.0</td>
</tr>
<tr>
<td>Blocks on clay, good clay</td>
<td>3.4 - 6.0</td>
</tr>
<tr>
<td>Blocks on clay, mediocre clay</td>
<td>3.4</td>
</tr>
<tr>
<td>Blocks on a granular filter, favourable construction</td>
<td>5.6 - 12</td>
</tr>
<tr>
<td>Blocks on a granular filter, normal construction</td>
<td>3.7 - 8.0</td>
</tr>
<tr>
<td>Blocks on a granular filter, unfavourable construction</td>
<td>2.6 - 5.7</td>
</tr>
</tbody>
</table>

Table 2.1 Recommended values for the revetment parameter A for a few standard types of revetment

<table>
<thead>
<tr>
<th>Situation: protruding edges, transitions rough bottom</th>
<th>continuous top layer, flat bottom</th>
</tr>
</thead>
<tbody>
<tr>
<td>without interaction: riprap, loosely-placed blocks</td>
<td>1.5</td>
</tr>
<tr>
<td></td>
<td>1.0</td>
</tr>
<tr>
<td>with interaction: washed-in blocks, block mats, gabions</td>
<td>0.75</td>
</tr>
<tr>
<td></td>
<td>0.5</td>
</tr>
</tbody>
</table>

Table 2.2 Recommended values for the stability parameter $\Phi$

<table>
<thead>
<tr>
<th>Revetment</th>
<th>$\Psi^{(-)}$</th>
</tr>
</thead>
<tbody>
<tr>
<td>riprap</td>
<td>0.035</td>
</tr>
<tr>
<td>loose, placed blocks</td>
<td>0.05</td>
</tr>
<tr>
<td>block mats</td>
<td>0.07</td>
</tr>
<tr>
<td>gabions</td>
<td>0.07</td>
</tr>
<tr>
<td>sand mattresses</td>
<td>0.07</td>
</tr>
</tbody>
</table>

Table 2.3 Recommended values for the critical Shields parameter $\Psi$
<table>
<thead>
<tr>
<th>Situation</th>
<th>( K_r ) (-)</th>
</tr>
</thead>
<tbody>
<tr>
<td>normal turbulence: abutment walls of rivers</td>
<td>1.0</td>
</tr>
<tr>
<td>increased turbulence: river bends downstream of stilling basins</td>
<td>1.5</td>
</tr>
<tr>
<td>heavy turbulence: hydraulic jumps sharp bends strong local disturbances</td>
<td>2.0</td>
</tr>
<tr>
<td>load due to water jet</td>
<td>3.0</td>
</tr>
</tbody>
</table>

Table 2.4 Recommended values for the turbulence factor \( K_r \)

<table>
<thead>
<tr>
<th>Type of revetment</th>
<th>( A ) (-)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Linked blocks on sand</td>
<td>5.3 - 11</td>
</tr>
<tr>
<td>Linked blocks on clay</td>
<td></td>
</tr>
<tr>
<td>good clay</td>
<td>6.4</td>
</tr>
<tr>
<td>mediocre clay</td>
<td>5.1</td>
</tr>
<tr>
<td>Linked blocks on a granular filter</td>
<td></td>
</tr>
<tr>
<td>favourable construction</td>
<td>5.7 - 12</td>
</tr>
<tr>
<td>normal construction</td>
<td>3.7 - 8.0</td>
</tr>
<tr>
<td>unfavourable construction</td>
<td>3.0 - 6.4</td>
</tr>
</tbody>
</table>

Table 3.1 Recommended values for the revetment parameter \( A \) for block mats and interlock systems

<table>
<thead>
<tr>
<th>Mattress</th>
<th>Leakage length ( A ) (m)</th>
<th>on sand</th>
<th>on a filter</th>
</tr>
</thead>
<tbody>
<tr>
<td>Incomat-FF</td>
<td></td>
<td>3.9</td>
<td>2.3</td>
</tr>
<tr>
<td>Incomat-Crib</td>
<td></td>
<td>1.0</td>
<td>0.5</td>
</tr>
<tr>
<td>Vecol</td>
<td></td>
<td>9.0</td>
<td>4.7</td>
</tr>
<tr>
<td>FPM</td>
<td></td>
<td>3.9</td>
<td>2.0</td>
</tr>
</tbody>
</table>

Table 3.2 Calculated leakage lengths (in m) of some concrete mattresses
<table>
<thead>
<tr>
<th>Type of concrete mattress</th>
<th>Thickness (mm)</th>
<th>Slope</th>
<th>Level bed</th>
</tr>
</thead>
<tbody>
<tr>
<td>PP</td>
<td>100</td>
<td>4.1</td>
<td>-</td>
</tr>
<tr>
<td></td>
<td>50</td>
<td>2.7</td>
<td>3.3</td>
</tr>
<tr>
<td>USM</td>
<td>100</td>
<td>3.9</td>
<td>4.7</td>
</tr>
<tr>
<td></td>
<td>200</td>
<td>5.5</td>
<td>6.4</td>
</tr>
</tbody>
</table>

Table 3.3 Characteristic values of the critical flow velocity (in m/s) for a few concrete mattresses

<table>
<thead>
<tr>
<th>Thickness (m)</th>
<th>Gabions with small stones</th>
<th>Gabions with large stones</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.1</td>
<td>0.34</td>
<td>0.42</td>
</tr>
<tr>
<td>0.2</td>
<td>0.31</td>
<td>0.40</td>
</tr>
<tr>
<td>0.3</td>
<td>0.29</td>
<td>0.37</td>
</tr>
<tr>
<td>0.4</td>
<td>0.29</td>
<td>0.35</td>
</tr>
<tr>
<td>0.5</td>
<td>0.28</td>
<td>0.33</td>
</tr>
</tbody>
</table>

Table 3.4 Values of a for gabions