Pilarczyk, K. (editor), 1998, Dikes and Revetments, A.A. Balkema (publisher)
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CHAPTER 16

Alternative revetments

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1 INTRODUCTION

Within the scope of the research on the stability of open slope revetments, much knowledge has been developed about the stability of placed (pitched) stone revetments under wave load and stability of rock under wave and current load. This development of knowledge has lead to a design methodology that has been laid down in writing in the " Design Manual for Pitched Slope Protection" (CUR/RWS, 1995a), and the "Manual on the Use of Rock in Hydraulic Engineering" (CUR/RWS, 1995b).

The above-mentioned types of structures and loads have constantly been given the highest priority, because of their significance to the Dutch water-retaining 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 placed stone revetments has recently been extended in applicability by means of a number of desk-studies for other (open) revetments.

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, and residual strength.

This contribution aims at giving a summary of the increased knowledge, especially that concerning the design tools that have been made available. The details behind it can be found in Chapter 12 and in the original reports (Klein Breteler, 1996, Stoutjesdijk, 1996, Pilarczyk, 1997).

2 THEORETICAL BACKGROUND OF WAVE LOADING

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

Wave run-down will lead to two important mechanisms:

·The downward flowing water will exert a drag force on the cover layer and the decreasing freatic level will coincide with a downward flow gradient in the filter (or in a gabion). The first mechanism can be schematised by a free flow in the filter or gabion with a typical gradient equalling the slope angle. It may result in geotechnical instabilities such as subsoil erosion and sliding of a revetment. In case of steep slopes this mechanism can often be decisive for the design.

·During maximum wave run-down there will be an incoming wave that a moment later will cause a wave impact. Just before impact there is a 'wall' of water giving a high pressure under the point of maximum run-down. Above the run-down point the surface of the revetment is almost dry and therefore there is a low pressure on the structure. The interaction of high and low pressure is shown in Figure 1.

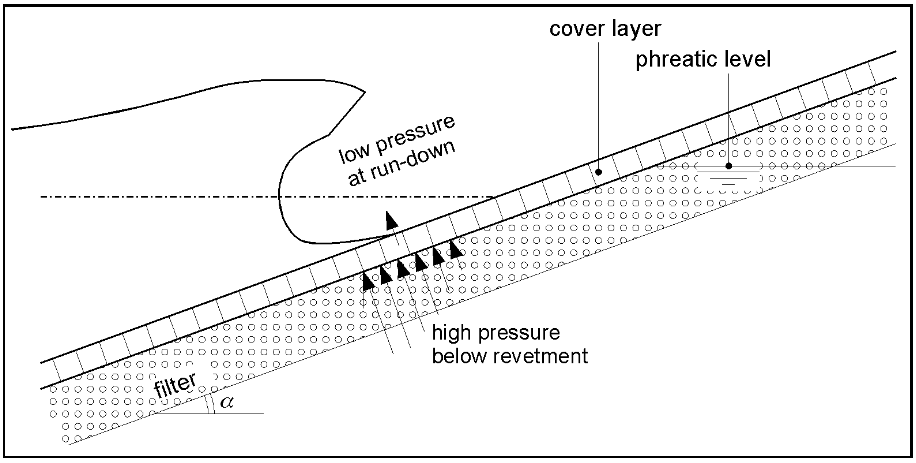

Figure 1 Pressure development in a revetment structure

The high pressure front will lead to an upward flow in the filter or a gabion. This flow will meet the downward flow in the run-down region. The result is an outward flow and uplift pressure near the point of maximum wave run-down. The situation is presented in Figures 1 and 2.

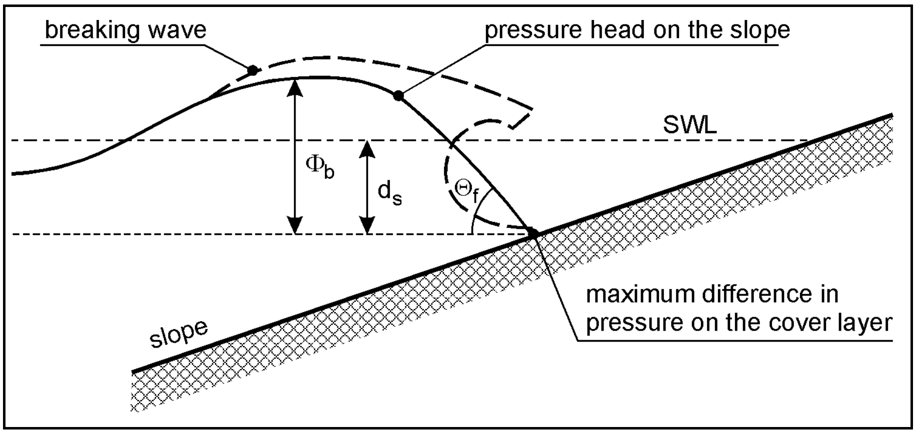

Figure 2 Schematization of pressure head on a slope

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

$$\frac{\partial^2 \phi}{\partial y^2}+\frac{\partial^2 \phi}{\partial z^2}=0 \qquad (1)$$

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

After complicated calculations the uplift pressure in the filter or a gabions can be derived. The uplift pressure is dependent on the steepness and height of the pressure front on the cover layer (which is dependent on the wave height, period and slope angle), the thickness of the cover layer and the level of the freatic line in the filter or a gabion. In case of gabions, it is not dependent on the permeability of the gabions, if the permeability is larger then the subsoil. The equilibrium of uplift forces and gravity forces leads to the following (approximate) design formula (see also Chapter 12 and CUR/RWS, 1995a):

$$\frac{H_{scr}}{\Delta D}=f\left(\frac{D}{\Lambda\,\xi_{op}}\right)^{0.67} \qquad (2a)$$

or $$\frac{H_{scr}}{\Delta D}=f\left(\frac{D}{b}\frac{k'}{k}\right)^{0.33}\xi_{op}^{-0.67} \qquad (2b)$$

or $$\frac{H_{scr}}{\Delta D}=F\;\xi_{op}^{-0.67} \qquad (2c)$$

where H_{scr} = significant wave height at which blocks will be lifted out [m]; $\xi_{op} = \tan\alpha/\sqrt{(H_s/\text{-}(1.56T_p^2))}$ = breaker parameter [-]; T_p = wave period at the peak of the spectrum [s]; Λ = leakage length [m], Δ = relative volumetric mass of cover layer [-] = $(\rho_s - \rho)/\rho$; b = thickness of a sublayer [m], D = thickness of a top layer [m], k = permeability of a sublayer [m/s], k'= permeability of a top layer [m/s], f = stability coefficient, mainly dependent on structure type and with minor infl-uence of Δ, tanα and friction [-]; F = total (black-box) stability factor [-]. The leakage length (Λ) is explained in detail in the next sub-section.

It is not expected that instability will occur at once if the uplift pressure exceeds the gravity forces. Therefore, the value of 'f' is in the range of 5 for static stability of loose elements (no friction between the units), 7.5 for tolerable/acceptable movement of loose units or for static stability of systems with considerable friction between the units, and up to 10 to 15 for acceptable movement of interlocked or cabled systems. In most cases the permeabilities of the cover layer and sublayer(s) are not exactly known and the coefficient 'f' and the parameter 'Λ' are combined to one 'black-box' stability factor '**F**'. On the other hand, the above result turns out to be in good agreement with the experimental results. Therefore, the factor 'F' will be used in the further discussion. These equations indicate the general trends and have been used together with measured data to set up the general calculation model (*cur/rws*, 1995a).

3 STRUCTURAL RESPONSE

3.1 Wave-load approach

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

$$\left(\frac{H_s}{\Delta D}\right)_{cr} = \text{function of } \xi_{op} \tag{3a}$$

For revetments, the basic form of this relation is:

$$\left(\frac{H_s}{\Delta D}\right)_{cr} = \frac{F}{\xi_{op}^{2/3}} \text{ with maximum } \left(\frac{H_s}{\Delta D}\right)_{cr} = 8.0 \tag{3b}$$

In which: F = revetment (stability) constant (-), H_s = (local) significant wave height (m), Δ = relative density (-), D = thickness of the top layer (m), and ξ_{op} = breaker parameter (-).
The relative density is defined as follows:

$$\Delta = \frac{\rho_s - \rho_w}{\rho_w} \tag{4a}$$

with: ρ_s = density of the protection material and ρ_w = density of water (kg/m^3). 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:

$$\Delta_t = (1 - n) \bullet \Delta \tag{4b}$$

In which: Δ_t = relative density including pores (-) and n = porosity of the top layer material (-).
The breaker parameter is defined as follows:

$$\xi_{op} = \frac{\tan\alpha}{\sqrt{H_s / L_{op}}} \tag{5}$$

The wave steepness S_{op} is defined as:

$$S_{op} = \frac{H_s}{L_o} = \frac{2\pi H_s}{g T^2} \tag{6}$$

In which: $\mathbf{L_{op}} = \frac{g}{2\pi} T_p^2$ (7)

α = slope angle (°), L_{op} = deep-water wavelength at the peak period (m), and T_p = wave period at the peak of the spectrum (s).
The advantage of this black-box design formula is its simplicity. The disadvantage, however, is that the value of F is known only very roughly for many types of structures.

The analytical model is based on the theory for placed stone revetments on a granular filter. In this calculation model, a large number of physical aspects are taken into account. In short, in the analytical model nearly all physical parameters that are relevant to the stability have been incorporated in the "leakage length" factor. The final result of the analytical model may, for that matter, again be presented as a relation such as Eq. 3 where **F** = f(Λ). For systems on a filter layer, the leakage length is given as:

$$\Lambda = \sqrt{\frac{b_f\, D\, k_f}{k'}} \tag{8}$$

with: b_f = thickness of the filter layer (m), k_f = permeability of the filter layer or subsoil (m/s), and 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 (eventually with gullies/surface channels) is filled in. For the thickness of the filter layer, b_f, it is examined to which depth changes at the surface affect the subsoil. One can fill in 0.50 m for sand and 0.10 m for clay. The values for D and Δ depend on the type of revetment. When schematically representing a block on a geotextile on a gully in sand or clay, 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:

$$\Lambda = \sqrt{\frac{(k_f\, d_g + k_g\, T_g)\, D}{k'}} \tag{9}$$

with: k_f = permeability of the filter layer (gully) (m/s), d_g = gully depth (m), k_g = permeability of the geotextile (m/s), T_g = thickness of the geotextile (m), D = thickness of the top layer (m), and k' = permeability of the top layer (m/s).

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

·the revetment parameter **F**;
·the (representative) strength parameters Δ and D;
·the design wave height H_s;
·the (representative) leakage length Λ;
·the increase factor Γ on the strength.

Only suchlike adaptations are presented in this summarizing review. The basic formulas of the analytical model are not repeated here. For these, reader is referred to (CUR/RWS, 1995a).

3.2 Flow-load stability

Severe flow attack may in practice occur on revetments, such as with flow over a steep slope and flow attack near many kinds of structures (downstream of sills, gates, discharge structures and the like). At these structures, 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 over-flow dam or dike, the situation is less ambiguous.

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, for example:

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

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

$$\Delta D = 0.035 \frac{\Phi}{\Psi} \frac{K_T K_h}{K_s} \frac{u_{cr}^2}{2g} \tag{10}$$

in which: Δ = relative density (-), D = characteristic thickness (m), g = acceleration of gravity (g=9.81 m/s^2), u_{cr} = critical vertically-averaged flow velocity (m/s), Φ = stability parameter (-), Ψ = critical Shields parameter (-), K_T = turbulence factor (-), K_h = depth parameter (-), and K_s = slope parameter (-).

* The stability parameter Φ depends on the application. Some guide values are given below.

	Continuous toplayer	Edges and transitions
Riprap and placed blocks	1.0	1.5
Blockmats, gabions, washed-in blocks, geobags, and geomattresses	0.5	0.75

* With the critical Shields parameter Ψ the type of material can be taken into account. Some guide values are given below.

Revetment type:	Ψ (-)
riprap, small bags	0.035
placed blocks, geobags	0.05
blockmats	0.07
gabions	0.07 (to 0.10)
geomattresses	0.07

* The degree of turbulence can be taken into account with the turbulence factor K_T. Some guide values for K_T are given below.

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

* 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:

developed profile: $K_h = \dfrac{2}{\left(\log\left(\dfrac{12\,h}{k_s}\right)\right)^2}$ (11a)

non-developed profile: $K_h = \left(\dfrac{h}{k_s}\right)^{-0.2}$ (11b)

very rough flow ($h/k_s < 5$): $K_h = 1.0$ (11c)

In which: h = water depth (m) and k_s = equivalent roughness according to Nikuradse (m).

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

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

$$K_s = \sqrt{1 - \left(\frac{\sin\alpha}{\sin\theta}\right)^2} = \cos\alpha\sqrt{1 - \left(\frac{\tan\alpha}{\tan\theta}\right)^2} \qquad (12a)$$

or

$K_s = \cos\alpha_b$ (12b)

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

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

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

With a downward flow along a steep slope it is difficult to determine or predict the flow velocity exactly, 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 that case a design formula based on the discharge is preferable. For downward flow along a slope steeper than approximately 1:10, the following relation is recommended:

$$\frac{q_{cr}}{g^{0.5}(\Delta D)^{1.5}} = \frac{a}{(\sin\alpha)^{1.17}} \qquad (13a)$$

Or, expressed in ΔD:

$$\Delta D = \frac{(\sin\alpha)^{0.78}}{a^{0.67}} \frac{q_{cr}^{0.67}}{g^{0.33}} \tag{13b}$$

In which:

Δ = (representative) relative density (-)
D= (representative) thickness of slope protection (m)
q_{cr}= critical specific discharge (m^2/s)
α= 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:

$$q = m \frac{2}{3}\sqrt{\frac{2}{3} g}\, h_{ov}^{1.5} \approx 1.7\, m\, h_{ov}^{1.5} \tag{13c}$$

In which:

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

The discharge coefficient 'm' depends on the shape of the overflow. Assuming a complete overflow, for most dam forms 0.8 ≤ m ≤ 1.0 applies. For more details one is referred to Annexe 1 in (Klein Breteler, 1996).

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:

$$h_{eq} = h_{ov} + \tfrac{1}{3} H_s \tag{13d}$$

In which:

h_{eq} =equivalent wash-over height (m)
H_s = significant wave height (m)

By substituting h_{ov} by h_{eq} in equation (13c), an "equivalent" discharge is found:

$$q_{eq} \approx 1.7\, m\, (h_{ov} + \tfrac{1}{3} H_s)^{1.5} \tag{13e}$$

The (critical) value of this discharge can be used in the design formula (11c). One comment is that the coefficient 1/3 was derived for a case of heavy overflow on which wave attack is superimposed, with reference to the stability of riprap dams of the Storm-Surge Barrier Eastern

Scheldt.

In conclusion, 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_{cr} or a discharge q_{cr}. In both design formulas constants are found which depend on the type of revetment. These constants are:

· the (representative) strength ΔD;
· the stability parameter Φ;
· the critical Shields parameter Ψ;
· the angle of internal friction θ;
· the equivalent roughness according to Nikuradse k_s;
· the revetment parameter a.

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

3.3 Soil-mechanical stability

The water movement on a revetment structure 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 structure (Figure 3).

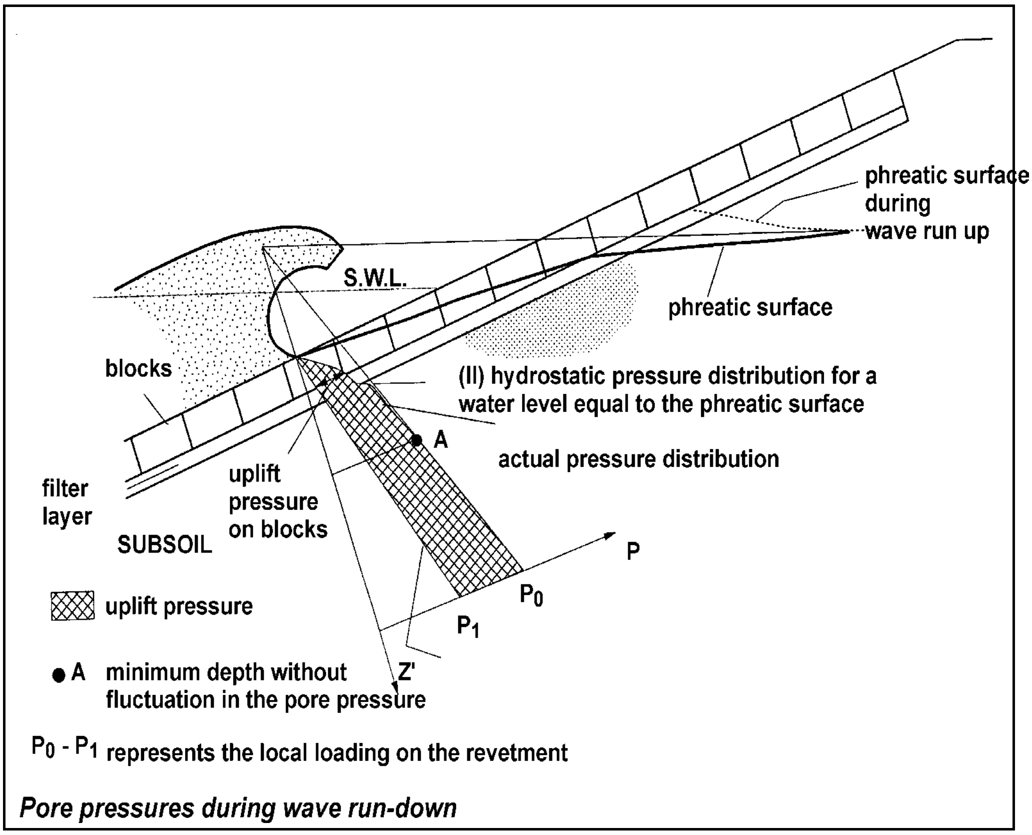

Figure 3 Pore pressure in the subsoil due to wave attack

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

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

These aspects and the accompanying damage mechanisms en design methods are discussed in detail below. Background information can be found CUR/RWS (1995a).

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 (Stoutjesdijk, 1996):

- ·lifting of the top layer;
- ·partial sliding of the top layer;
- ·sliding of the top layer;
- ·sliding of the subsoil (Figure 4).

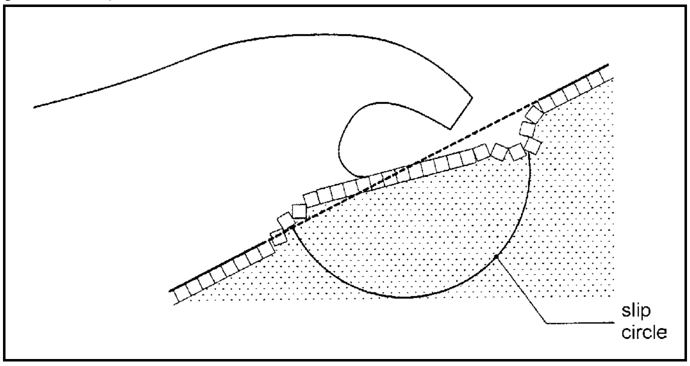

Figure 4 Development of S-profile and possible local sliding in the base (sand)

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

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. The design method with regard to the different damage mechanisms connected with elastic storage are presented in the form of design diagrams (see Diagrams 1 to 4 in Figure 6). In these diagrams the permissible wave height is plotted against the thickness of the top layer and the slope gradient for a certain wave steepness S_{op}. If the revetment construction consists of a top layer on a filter layer, the thickness of the filter layer may in these diagrams be partially or completely (depending on the type of revetment) added to the thickness of the top layer. The equivalent thickness is defined as:

$$D_{eq} = D + \frac{b}{\Delta_t} \tag{14a}$$

where D_{eq} is the equivalent thickness of the top layer, D is the real thickness of the top layer, b is the thickness of the filter layer and Δ_t is the relative mass (weight) under water of the top layer.

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

$$D_{eq} = D + 0.8 \frac{b_c}{\Delta_t} - 0.5 \qquad (14b)$$

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

The relative mass (weight) under water of the top layer is defined as:

$$\Delta_t = \frac{\rho_t - \rho_w}{\rho_w} \qquad (15a)$$

with: ρ_t = volumetric mass of top layer (kg/m^3), ρ_w = volumetric mass of water (kg/m^3).

For sand-filled systems ρ_t is equal to:

$$\rho_t = (1 - n)\,\rho_s + n\,\rho_w \qquad (15b)$$

ρ_s= volumetric mass of sand (kg/m^3)
n= porosity of sand (-)

(Δ_t is about 0.9 to 1.0 for sand-filled systems and 1.2 to 1.4 for blockmats and concrete-filled systems).

The Diagrams 1 to 4 are basically developed for the concrete-filled systems. For the sand-filled systems and gabions the maximum allowed wave height for a certain failure mechanisms will be a factor 1.1 lower due to the less integrity and stiffness of the system. For the blockmats the allowed wave height will be a factor 1.2 higher. In case of systems placed on a filter layer the diagrams on the lifting and partial/total sliding of the top layer can be neglected. In that case the stability of the top layer must be treated in conjunction with the filter layer; the sliding of the subsoil will be the determinant factor.

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. In sand, these water tensions can be calculated using the MCYCLE-program developed by Delft Geotechnics. As the top layer becomes more impermeable, the water tension occurs closer to the surface of the slope. In the case of a very permeable top layer this is exactly the opposite. Softening (liquefaction) can be defined as follows:
A cyclic variable load causes compaction to occur in a layer of sand. This leads to a decreasing pore volume. The water in the pores is subjected to pressure and will start to run off. At first, water overtension occurs. This causes a decrease in the contact pressure between the grains and with this the resistance to sliding. Finally, the water overtension might become so large that the contact pressure between the grains falls away completely. This is called softening or liquefaction.

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

With regard to liquefaction, the following design rules are suggested for constructions with a reasonably compacted subsoil:

·With a top layer on sand there is no danger of liquefaction, if:
— the slope gradient is gentler than or equal to 1:3,
— the slope gradient is gentler than 1:2 and the wave height H_s is smaller than 2 m,
— 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 liquefaction.
·With a top layer on a granular filter there is generally no danger of liquefaction.

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

Through a drop in the water level a difference in the rise over the top layer may occur. A drop in the water level may occur as a result of tide or a ship passing through a waterway or canal. As with placed stone revetments, the resulting uplift is especially dangerous when the top layer is sanded up due to which the permeability of the top layer may decrease in time.

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

$$\Lambda \sin\alpha/2 \leq \Delta D \cos\alpha \tag{16}$$

in which: Λ = leakage length (m), α = slope angle (°), Δ = (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. Should the application of formula (16) show that calculations on the phenomenon should be made, one is referred to the original reports.

Anchoring

Assuming that in the uplift zone, defined by the wave run-down (Figure 5), a blockmat or a mattress can be lifted during or when the design conditions are exceeded, then the upper part of a revetment fulfills the anchoring function. For such a case, Pilarczyk proposes the following formula for minimum length of anchoring, $\mathbf{L_{a,min}}$, measured along the slope above the design water level (Figure 5), which is needed to compensate the loss of friction in a case of uplifting of the units in the uplift area:

$$L_{a,min} = 0.5\, H_s\, (f_o \cos\alpha - \sin\alpha)^{-1} = 0.5\, H_s \cos\alpha\, (f_o - \tan\alpha)^{-1} \qquad (17)$$

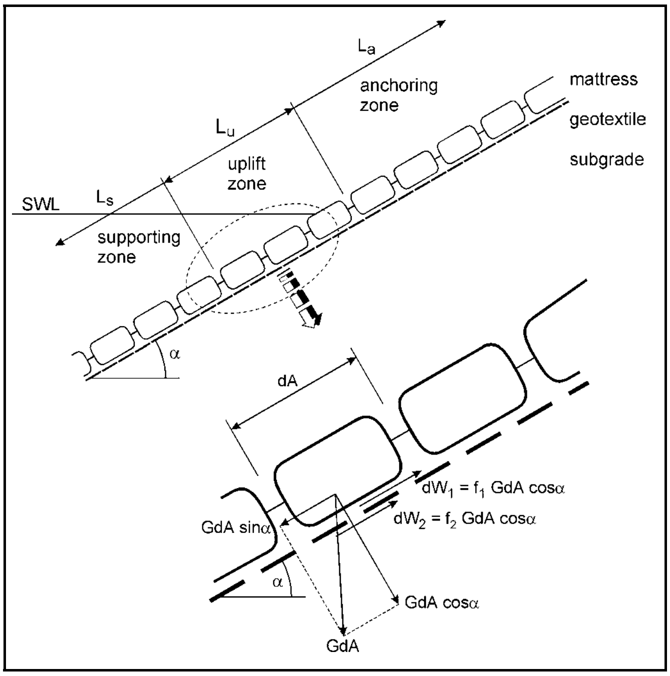

Figure 5 Schematization of uplifting and anchoring zones

where: α = angle of slope and $f_o = f_2$ = 0.70 to 0.75 for mattress on sand or granular filter, and 0.60 on a (wet) clay; when blockmat connected to geotextile or geomattress are lying on a geotextile, $f_o = f_1$ = friction factor between two geotextiles and equals to about 0.30 for relatively smooth mattresses and 0.35 for more articulated ones. When $\tan\alpha > f_o$, a milder slope should be applied.

From this criterion we see that a steep slope will easily lead to the exceeding of the friction forces, and furthermore a steep slope is shorter then a gentle slope and will give less support to the section that tends to slide down.

We can conclude that the stability against sliding increases if we decrease the slope angle (or increase the friction of revetment on the subsoil). The thickness of the revetment will not contribute much to the stability.

Additional information and design charts for local sliding of subsoil, sliding of revetments, and anchor forces are presented in (Spraque & Koutsourais, 1992 and CUR/RWS, 1995a,b). The lower and upper limits of extent of protection are discussed in Dikes&Revetments, Chapter 18.

Diagram 1a

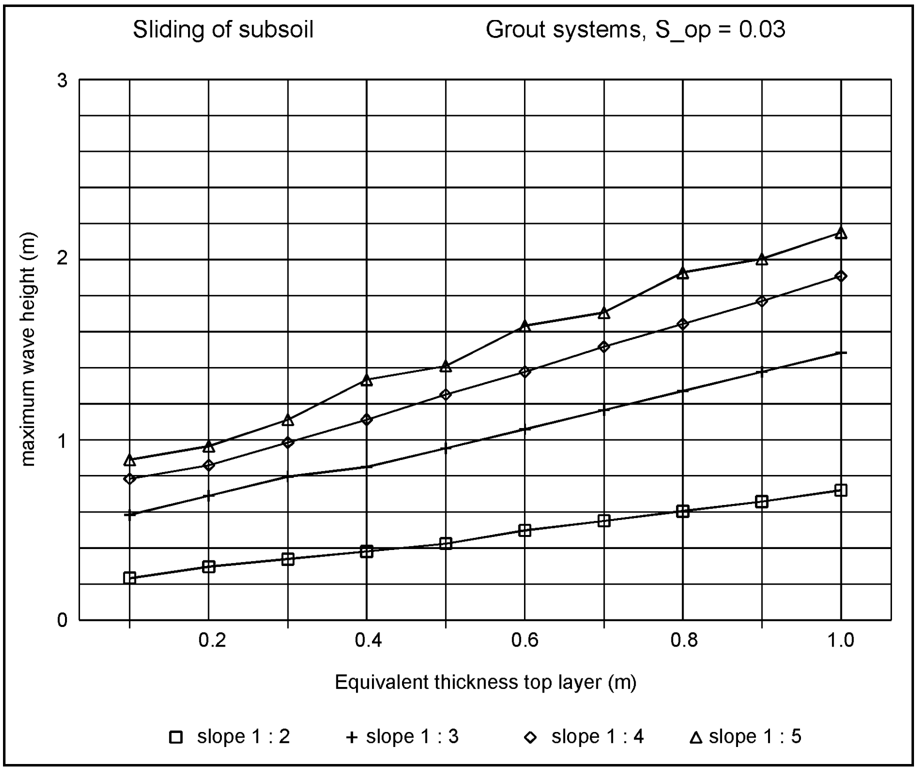

Diagram 1b

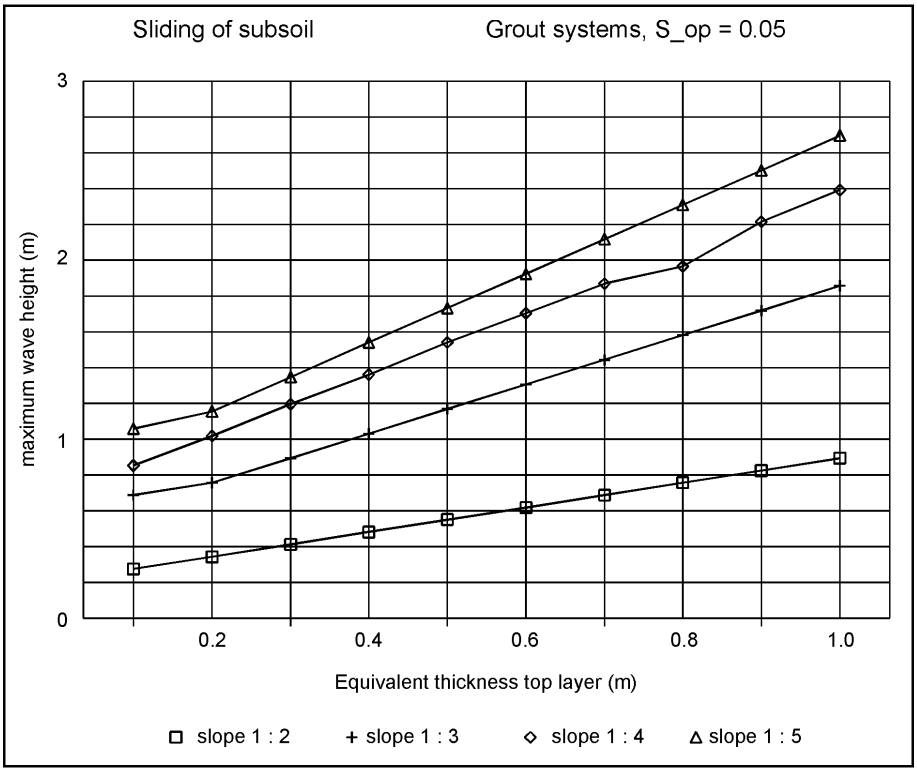

Figure 6a Design diagrams for sliding of subsoil

Diagram 2a

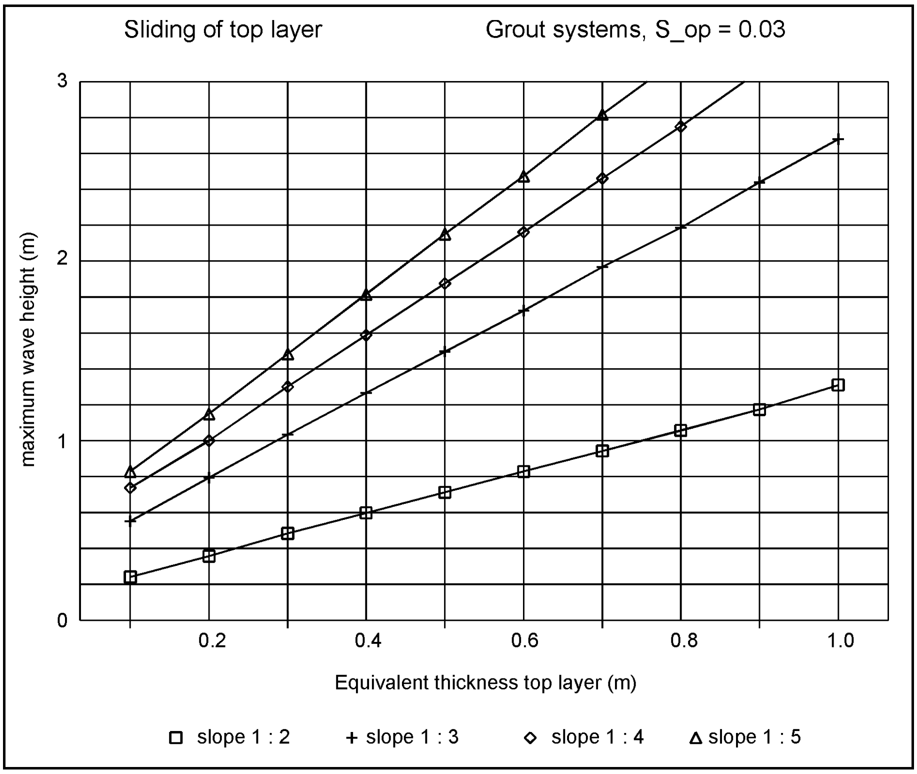

Diagram 2b

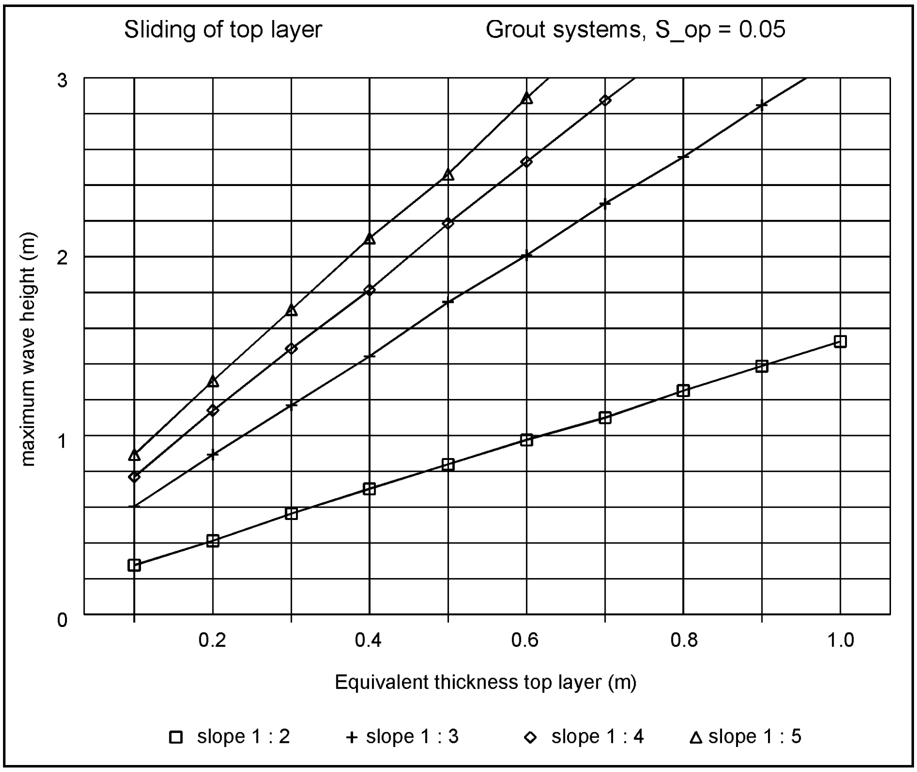

Figure 6b Design diagrams for sliding of top layer

Diagram 3a

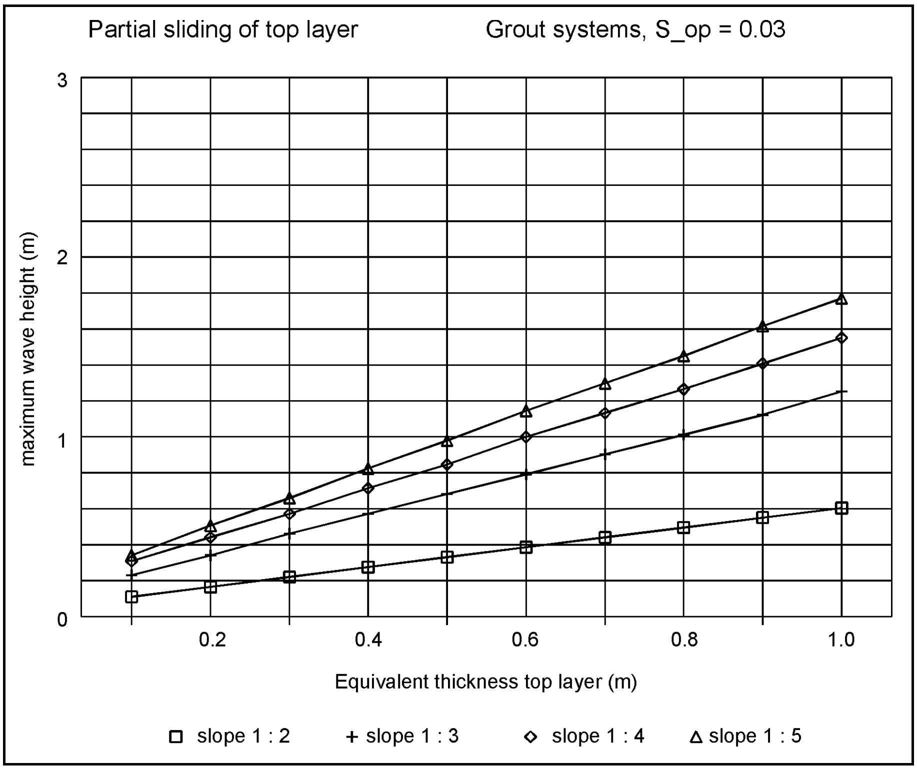

Diagram 3b

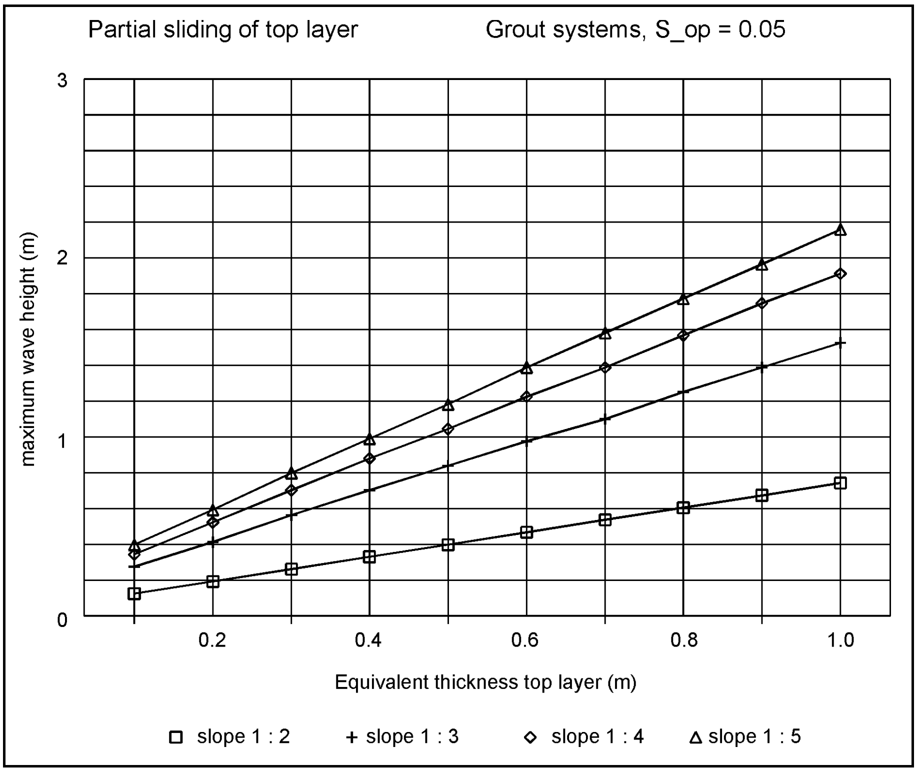

Figure 6c Design diagrams for partial sliding of top layer

Diagram 4a

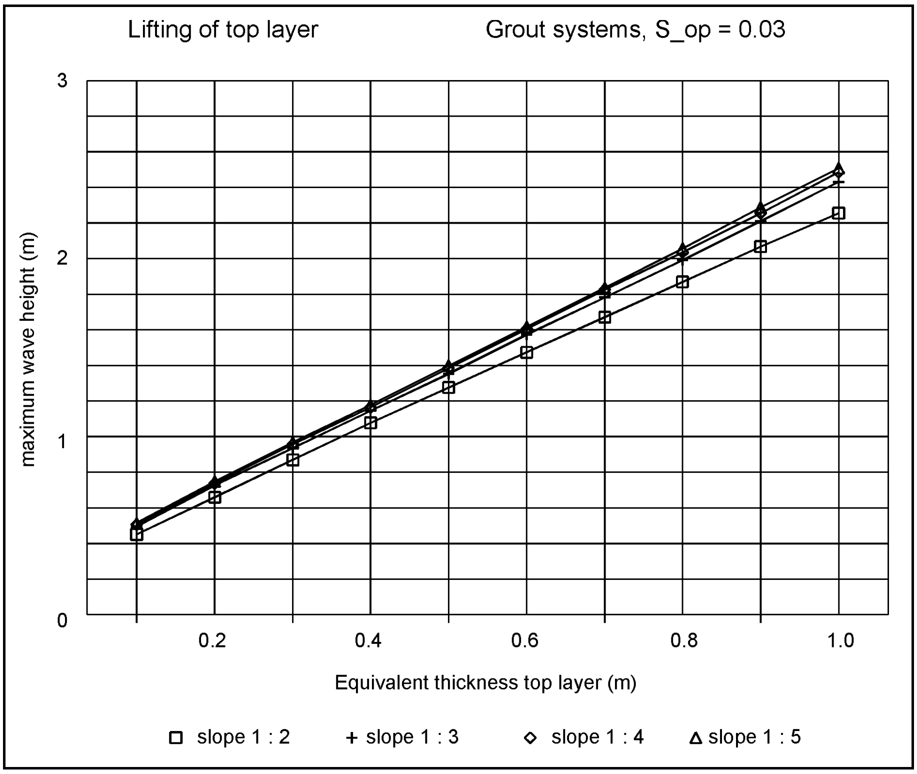

Diagram 4b

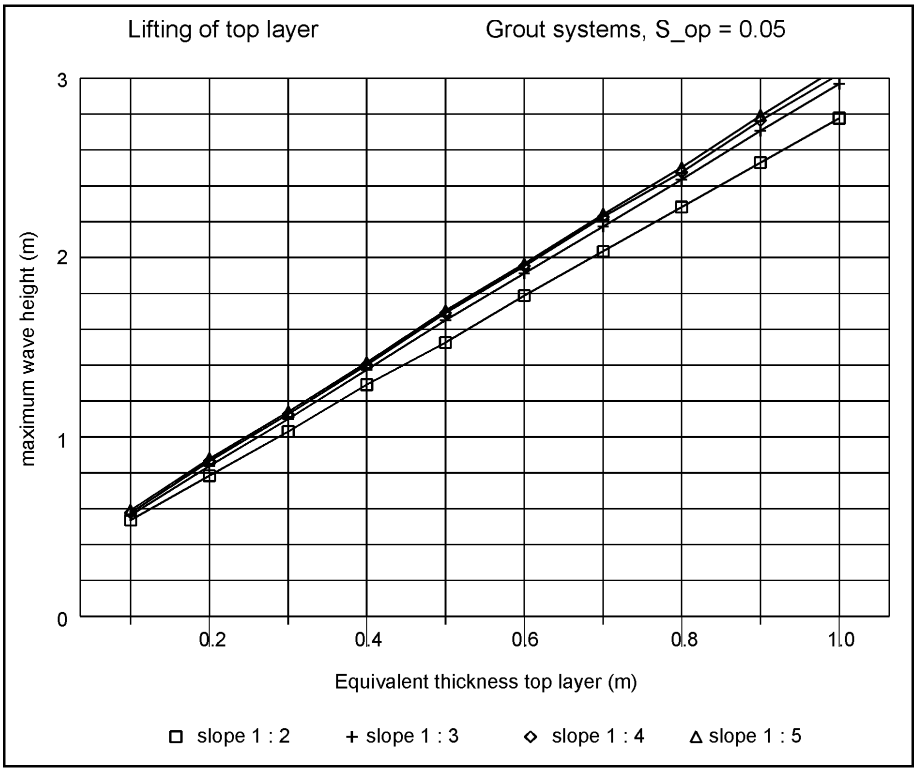

Figure 6d Design diagrams for lifting of top layer

4 STABILITY CRITERIA FOR BLOCK MATS

4.1 System description

A (concrete) block mat is a slope revetment made of (concrete) blocks that are joined together to form a "mat", see Figure 7. 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.

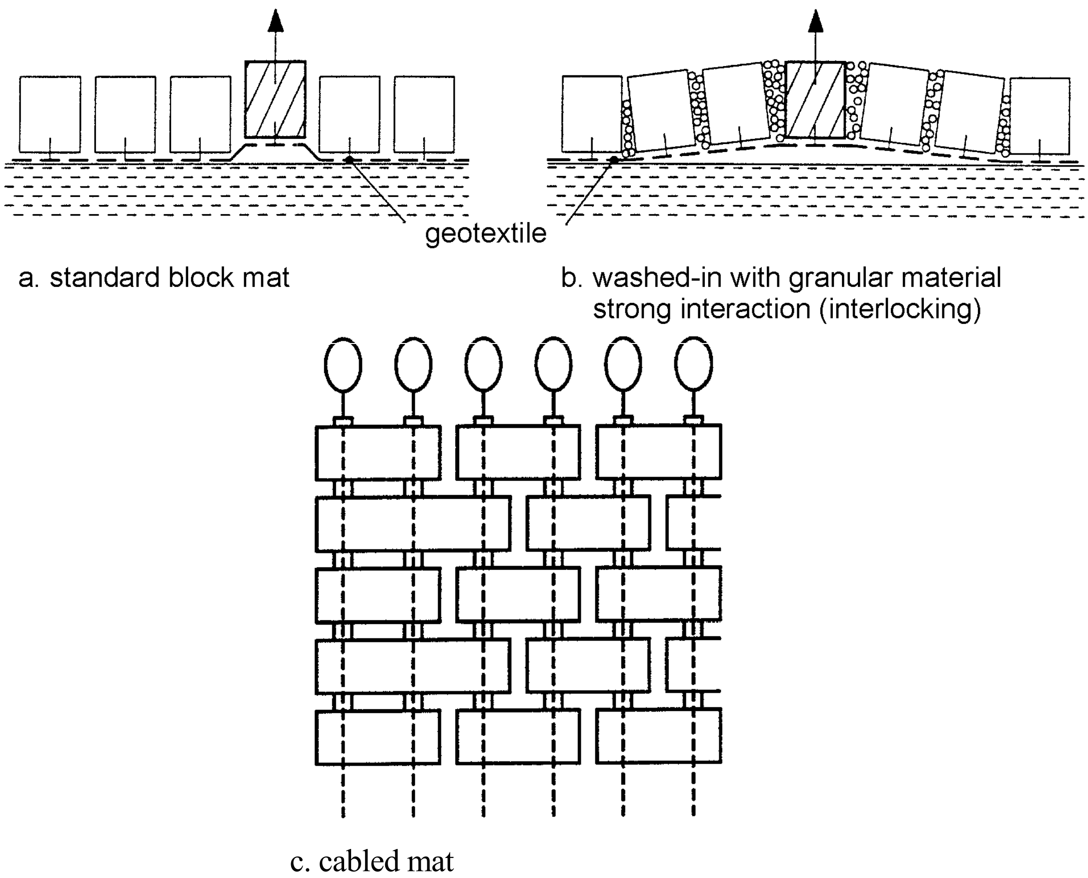

Figure 7 Examples of block mats

Two major advantages of block mats are their properties of being able to be laid quickly and efficiently and partly under water. Block mats 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 placed 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 it being washed in well.

The weakness of block mats 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 blocks is of major importance. The materials used for this purpose (steel or nylon cables, geotextile) should be able to withstand in the long term the effect of (sea) water, sunlight, plants, animals, vandalism, etc. An example are the synthetic pins, which connect the blocks to the geotextile and which may become brittle at low 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 placed 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 blockmat 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 anchored at the top.

As for the possibilities of reuse and repair, block mats and interlock systems are at a disadvantage compared with placed blocks/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 (see Figure 4). 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.

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 0.5 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, blockmats may have a residual strength of the duration of several storms.

Repair of damage to blockmats 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.

4.2 Design rules with regard to wave load

Table 1 gives an overview of usable values for the revetment constant **F** in the black-box model for linked blocks (block mats).

Table 1Recommended values for the revetment parameter F for block mats

Type of revetment		**F (-)**
Linked blocks on sand		5 to 6
Linked blocks on clay	good clay	5 to 6
	mediocre clay	4.5 to 5
Linked blocks on a granular filter	favourable construction	5 to 6
	normal construction	4 to 5
	unfavourable construction	3 to 4

N.B.: the lower values refer to blocks connected to geotextile while the higher ones refer to cabled blocks.

The terms "favourable", "normal" and "unfavourable" refer to the composition of the granular

filter and the permeability-ratio of the top layer and the filter layer (see *cur/rws*, 1995a). In a case of fine granular filter and relatively permeable top layer the total composition can be defined as "favourable'. In a case of very coarse granular layer and less permeable top layer the composition can be defined as "unfavorable". In a case of blocks connected to a geotextile and concrete-filled mattresses on a filter layer the construction can be usually defined as between "unfavourable" and "normal", and the stability factor **F** = 3.0 to 3.5 (max. 4.0) can be applied. For block mats and permeable mattresses on sand **F** = 5 (max. 6.0) can be applied. The higher values can also be used in cases that the extreme design loading is not very frequent or when the system is (repeatedly) washed in by coarse material providing additional interlocking.
This wide range of recommended values for **F** only gives a first indication of a suitable choice. The stability of a block mat might also be determined more accurately using the analytical model. However, this analytical model is still not including all necessary components and is not verified properly. The analytical model can be treated as a useful tool for qualitative sensitivity analysis concerning the influence of various input parameters. Therefore, for the time being, the stability calculation based on the black-box approach seems to be a reasonable choice.

4.3 Design rules with regard to flow load

The resistance of block mats against current can be calculated according to Eq. 10. The constants Φ and Ψ which should be used in the design formula are defined in Section 3.2. The strength parameters Δ and D follow from the standard definitions.

Revetment parameter 'a' (Eq. 13)
For block mats through which vegetation can grow, 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 scatter 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
The 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 θ
For the top layer, an angle of internal friction $\theta = 90°$ and $K_s = \cos\alpha$ can be used for rigid (cabled and anchored block mats, eventually washed-in with a granular material). In other cases $\theta = \delta$ = an angle of friction at interface between a block mat and a subgrade (see also Section 3.2).

4.4 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

Diagrams 5a,b in Figure 8 show the design diagrams for the stability of the systems if these are placed on sand. In these diagrams, moderately packed sand is assumed (the angle of internal friction is 35°). Should there be a sound compaction of the whole pack of sand, it is also possible to use the diagrams in (CUR/TAW, 1995a). They concern tightly packed sand with an angle of internal friction of 40°. These diagrams can be also treated as an upper limit for concrete-filled mattresses.

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 $D + b/\Delta_t$ for the equivalent (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 structuring during the life of the revetment. For this situation, the specific studies should be performed.

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 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 3.1. 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. The strength parameters Δ and D follow from the standard definitions.

As an indication, a system with D = 0.1 m, Δ_t = 1.2, and slope 1 on 2 placed on the sandy subsoil must be studied more in detail when the leakage length is larger than 0.2 m. This is a case when the permeability of the top layer is equal or lower than 7.5 10^{-5} m/s.

Example of geotechnical calculation

Input:

Thickness of top layer D = 0.15 m and slope 1 on 4. Block mats are placed directly on geotextile on compacted sand or on 0.5 m thick minestone sublayer (Δ_t = 1.35).

Question: wat is the difference in the stability concerning the elastic storage phenomena?

Answer:

We use Diagrams 5 in Figure 8. $D_{eq.}$ is 0.15 m for sand and (0.15 m + 0.5m/1.35) = 0.52 m when placed on minestone. The acceptable wave heights for given thickness of sand and minestone are presented in Table 2.

Table 2 Permissible wave heights

D_{eq}	$S_{op} = 0.03$	$S_{op} = 0.05$
0.15 m	0.85 m	1.0 m
0.52 m	1.4 m	1.8 m

The results show that the block mat/mattress of D = 0.15 m placed directly on sand is geotechnically stable for wave heights of 0.85 to 1.0 m while, when placed on 0.50 m of minestone layer (or fine granular filter), the stability increases up to wave heights of 1.4 to 1.8 m. Liquefaction does not need to be considered because the slope is milder than 1 on 3 and sand is compacted.

Assuming that the permeability of mattress is nearly the same as sand or minestone the effect of the drop in a water level can be treated as follow:
For D_{eq} = 0.5 m, b = 0.3 m (the effective depth in minestone), and k/k' = 1 the leakage length is about 0.40 m. For these conditions (leakage length, mattress thickness and slope gradient) Eq. 16 is satisfied and the influence of the drop in water level can be neglected.

4.5 Sample calculations of block mat

4.5.1 *Block mat on geotextile 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_s = 0.9 m
- Wave period T_p = 4.0 s
- The water is deep.

Construction:
- Slope gradient $\cot\alpha$ = 4.0
- Relative density Δ = 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} = 0.15 mm. There is a good toe anchoring.

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

Wavelength	L_{op}	= 25.0 m	from Formula (7)
Wave steepness	H_s/L_{op}	= 0.036	from Formula (6)
Breaker parameter	ξ_{op} 2	= 1.32	from Formula (5)
Revetment parameter	F	= 5 to 6.0	from Table 1

Diagram 5a

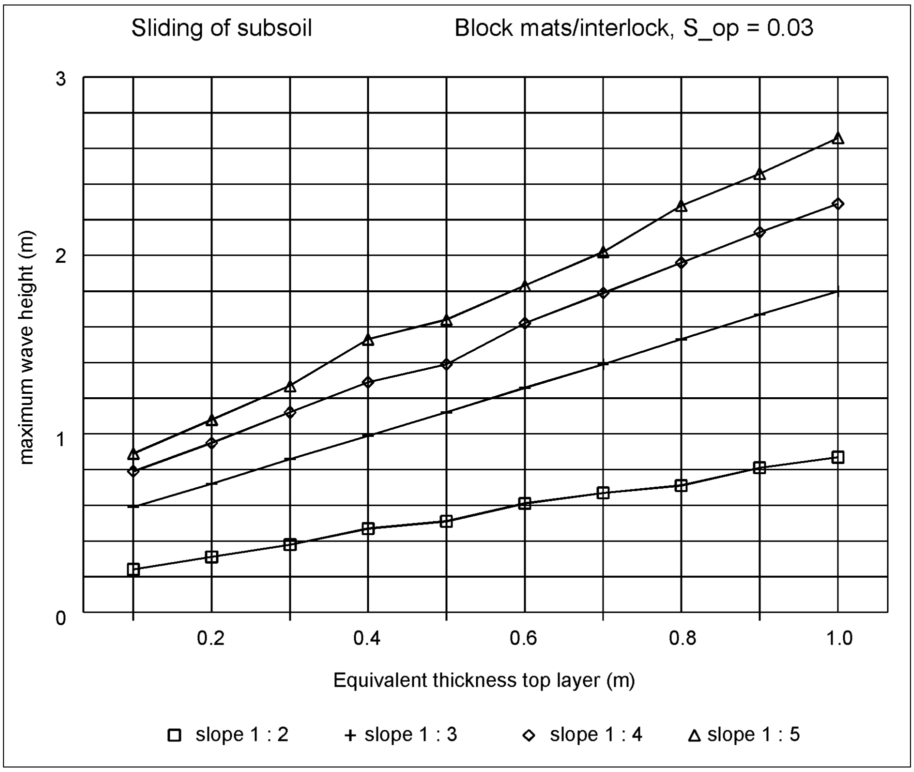

Diagram 5b

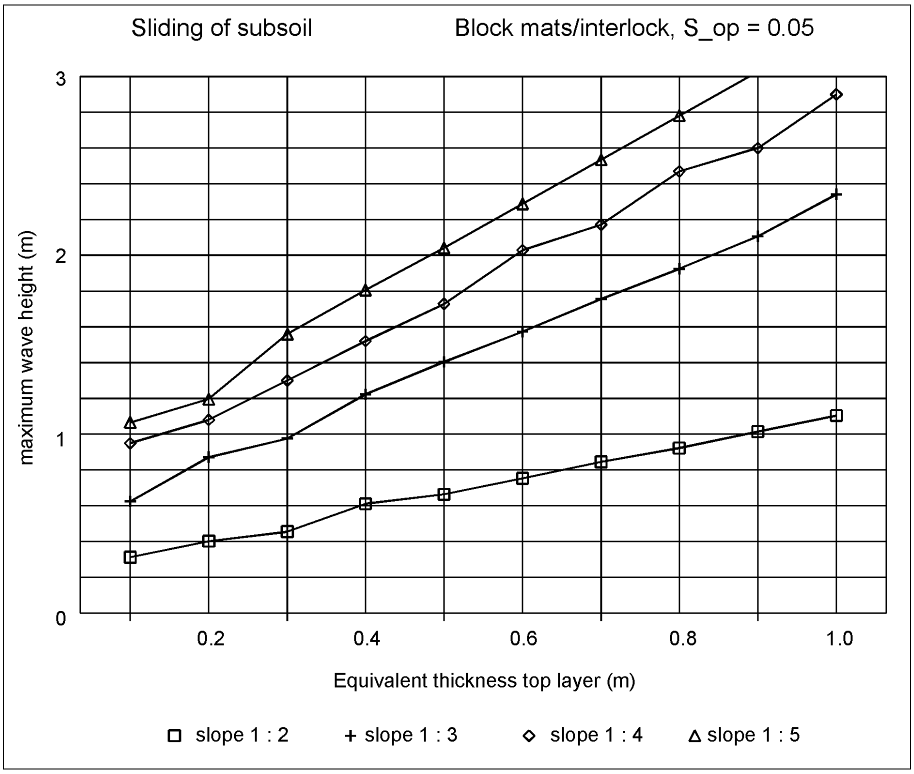

Figure 8 Design diagrams for sliding of subsoil (for block mats)

Critical load $\left(\frac{H_s}{\Delta D}\right)_{cr}$ = $(5 \text{ to } 6.0)\xi_{op}^{-2/3}$ =

= 4.15 to 5.0 from Formula (3)

Required block thickness D = 0.14 to 0.17 m

This 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 0.15m 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 Diagrams 5a and 5b in Figure 8. The permissible wave height is H_{cr} = 0.9 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 3.3, this means that there is no danger of softening.

Drop in the water level
Substitution of the known variables in Formula (16) 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.

4.5.2 *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 0.10m on a geotextile on sand is stable with regard to the flow load, and whether the anchoring at the top is needed or not.

Flow conditions:

Water depth	h	=	1.5 m
Normative flow velocity	u_{cr}	=	3.0 m/s

The turbulence is heavy and the flow is non-developed.

Construction:

Slope gradient	$\cot\alpha$	=	2.0
Relative density	Δ	=	1.3

There is a uniform slope, without transitions;
Sand (D_{b50} = 0.15 mm) is well compacted; $\varphi = 40^\circ$.

Below, the required block thickness is determined on the basis of the flow velocity, see Formula (10).

Turbulence factor K_T = 2.0 from Section 3.2

Water depth parameter K_h = 0.31 from Formula (11b)
Angle of internal friction θ = 90° for anchored system, and
$\theta = \delta = 3/4\varphi = 30°$ without anchoring
Slope parameter $K_s = \cos\alpha = 0.89$ for anchored system
$K_s = 0.445$ from Formula (12a)
Stability parameter Φ = 0.50 and 0.75 for edges of mattress
Critical Shields parameter Ψ = 0.07

Substitution of these data and the values for Δ and u_{cr} in the design formula (10) produces the following required thickness of top layer:
- for anchored system: D = 0.06m (0.09 m for free edges of mattress), and
- for non-anchored system: D = 0.12 m (0.18 m for free edges of mattress).

The chosen block thickness of 0.10 m is therefore satisfactory for anchored system only.
There are no waves and therefore there is no danger of soil-mechanical instability occurring.

5 STABILITY CRITERIA FOR CONCRETE-FILLED MATTRESSES

5.1 Concrete Mattresses

Characteristic of concrete mattresses are the two geotextiles with concrete or cement between them. The geotextiles can be connected to each other in many patterns, which results in each mattress system having its own appearance and properties. Some examples are given in Figure 9. Some construction aspects relating to anchoring and transitions of mattresses are shown in Figure 10.

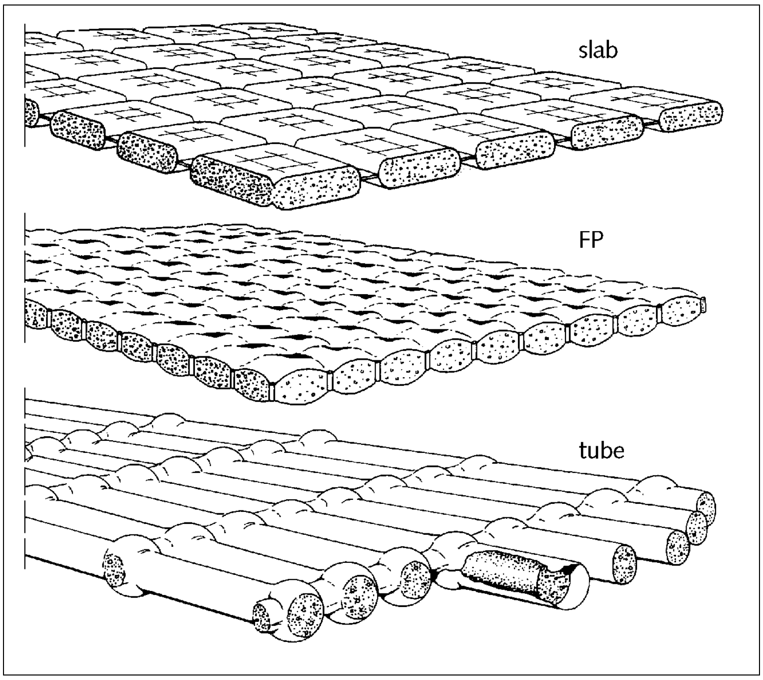

Figure 9 Examples of concrete-filled mattresses

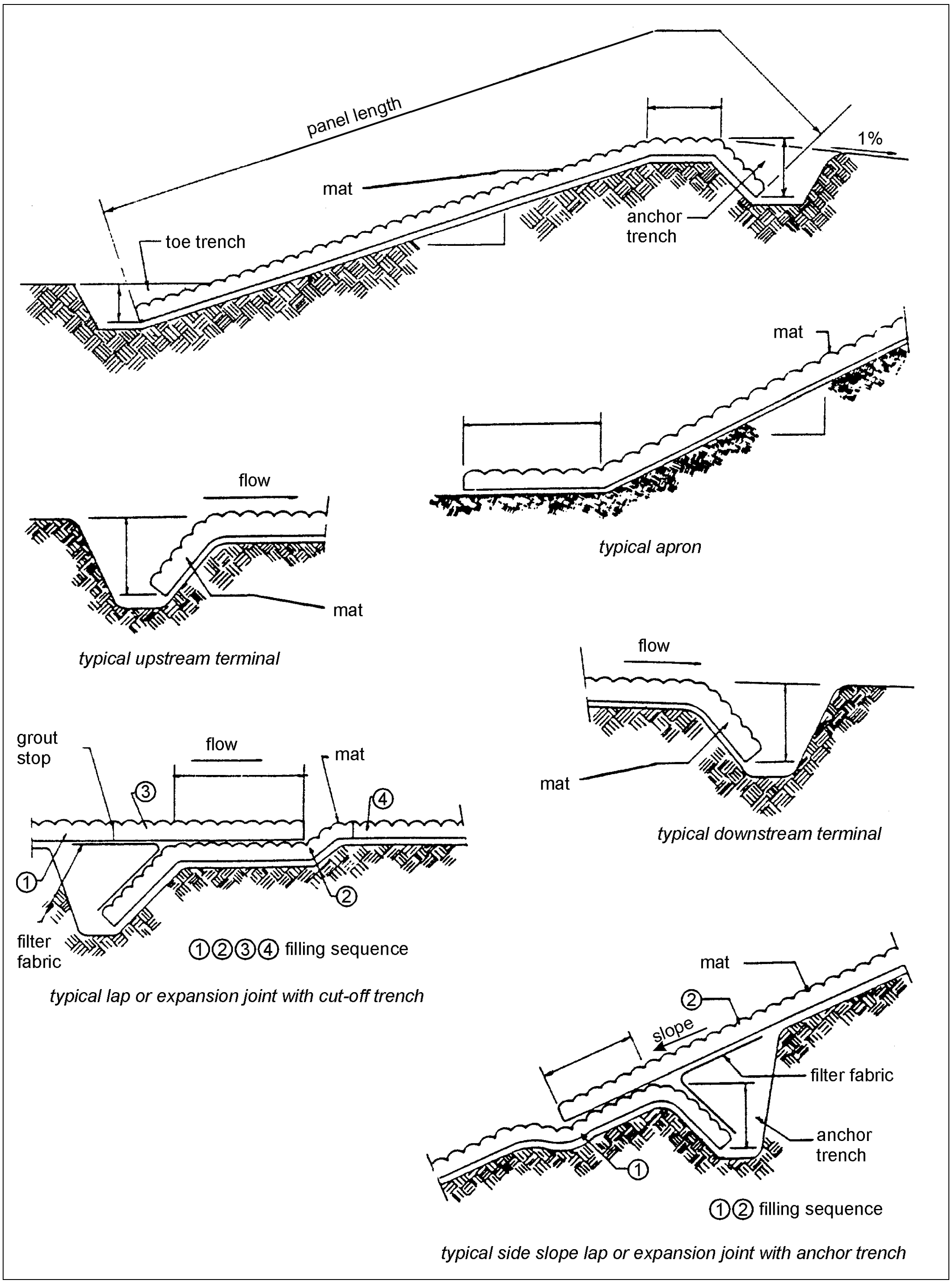

Figure 10 Construction aspects of mattresses

On long term, especially when no UV-protection for geotextile is applied, the surface of a geomattress will deteriorate and the concrete filled-mat will function as a block mat; a block mat with concrete units connected to the lower sheet of geotextile by existing binders, which normally are used as spacers to provide a required thickness. These binders should have a proper strength to compensate the weight of the concrete element. Therefore, the stability of geomattresses should also be controlled accordingly to design criteria provided for blockmats (Section 4).

5.2 Permeability

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. The susceptability for blocking can be reduced by increasing the gradation of the subsoil. To reduce the susceptibility for clogging it is recommended to reduce the sludge content of the subsoil. Background information can be found in Klein Breteler (1996, Annex 6).

In this section the stability aspects for the following (selected) types of concrete mattresses are discussed:

· Standard FP-mat: a standard mat consists of two geotextile sheets stitched together at the locations of the Filter Points (square Filter Points of about 4 × 4 or 5 × 5 cm);
· FPM: the mattress consists of two geotextile sheets kept on constant distance (= thickness) by a wires and equipped with small tubes as Filter Points (the type of the mat is defined by the distance between the filter points);
· Slab-mat: the mattress consists of slabs interconnected by thin sections (width of 5 to 10 cm) equipped with square Filter Points of about 10 × 10 cm);
· Articulated tube-mat (Crib): the mattress consists of interconnected (concrete-filled) geotextile cylinders with relatively much open geotextile-space in between (i.e. 30 × 30 cm).

The main properties of the entire mattress (besides properties of geotextiles and applied concrete for filling as specified by the manufacturer) are thickness, permeability, hydraulic roughness, and resistance to shear stress of the mattress. Especially, the proper specification of permeability is very essential for estimation of the stability. Stability increases with the permeability of the mattress, however, the critical conditions for the soil stability may be the limiting factor. In general, with a subsoil of clay and silty sand the permeability of the mattress will be higher than the permeability of the subsoil. Therefore the groundwater reaching the mattress can usually be discharged without excessive lifting pressures on the mattress. With a subsoil of coarse sand or gravel (or granular filter), or with irregular surface of fine soils (gullies/cavities between the soil and the mattress) the permeability of the mattress will be lower than the permeability of the subsoil/sublayer. In that case the groundwater reaching the mattress will result in excessive lifting pressures on the mattress. A proper preparation of the sublayer is thus of importance; good compaction and avoiding of gullies under the mattress.

As it was already mentioned above there is a lack on proper information on the total permeability of the mattresses; 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.

Based on the knowledge of placed block revetments (see Chapter 12) and the collected information from the literature and company informations, the following indicative permeabilities have been calculated (Table 3).

Table 3Calculated permeabilities for concrete mattresses
(it represents the best judgement due to lack on information)

Mattress	**A (cm^2)**	**Ω (-)**	**D (m)**	**k_g (mm/s)**	**T_g (mm)**	**D_{f15} (mm)**	**k' (mm/s)**
Standard-FP 'square' filter points:4x4 cm	20x20	0.04	0.25	0.4	1	cavity	0.3
						5	0.3
						1	0.3
Standard-FP 'square' filter points:5x5 cm	25x25	0.04	0.15	0.8	0.6	cavity	0.5
						5	0.5
						1	0.5
FPM 'tube' filter points: 2.5 cm^2	25x25	0.004	0.15	0.8	0.6	cavity	0.05
						5	0.05
						1	0.05
FPM 'tube' filter points: 2.5 cm^2	25x25	0.004	0.15	**4**	0.6	cavity	0.25
						5	0.20
						1	0.20
Slab 'square' filter points:10x10 cm	60x60	0.28	0.25	5	1.4	cavity	1.9
						5	1.7
						1	1.1
Slab 'square' filter points:10x10 cm	60x60	0.028	0.25	0.4	1.4	cavity	0.1
						5	0.1
						1	0.1
Crib	-	0.2-0.5	0.1	5	1.4	-	5

Notation:

k'=linearised permeability of the mattress (m/s)

k_g= linearised permeability of geotextile (m/s)

Ω = A_p/A=ratio of the area through which water can pass to the total area (-)

A= area of top layer per Filter Point (m^2)
A_p=permeable area of the Filter Point (m^2)
D= average thickness of top layer (m)
T_g= thickness of geotextile (m)
D_{f15} =grain size of the filter (lying directly under the mattress),
15% by weight of which is less than the stated size (m)

N.B. k_g is defined as the permeability coefficient in the relation: $v = k_g\ i^{0.5}$, where **i** is the hydraulic gradient (ratio of the hydraulic head, H and the thickness of a geotextile, T_g).

5.3 Design rules with regard to wave load

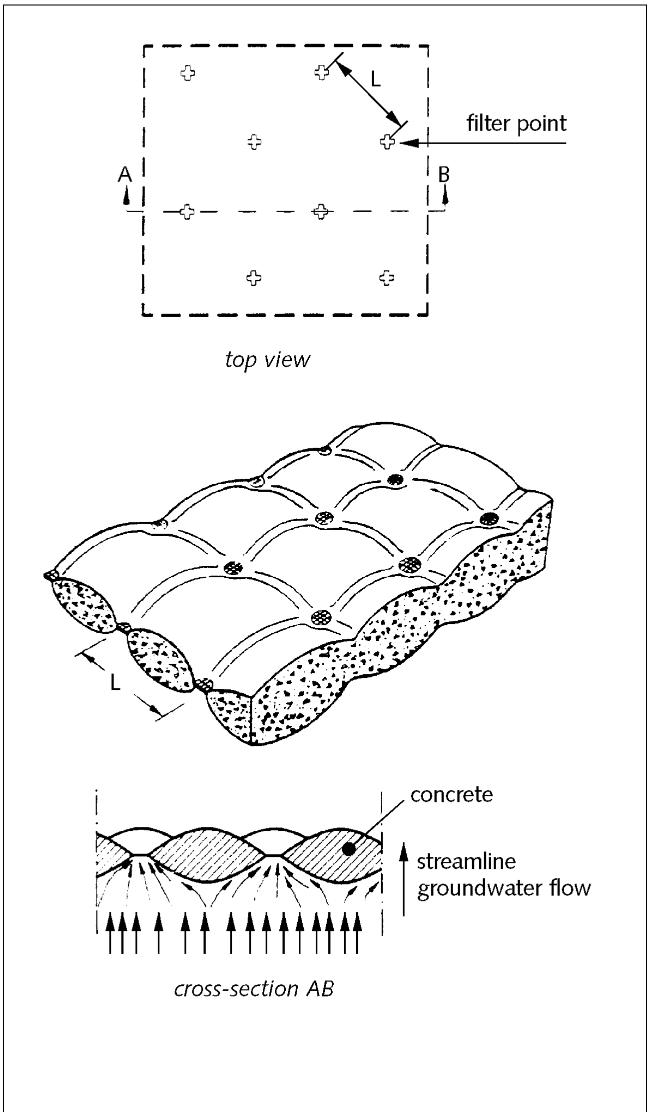

Figure 11 Principles of Filter Points mattress

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 placed block revetments (Figure 11). This takes place the moment the wave has drawn back, just before the wave impact. Just as with placed block revetments, the leakage length for this differential pressure is the most important construction-descriptive parameter. The leakage length Λ for concrete mattresses is defined as (Eq. 8):

$$\Lambda = \sqrt{\frac{b_f\, D\, k_f}{k'}}$$

with: Λ = leakage length (m) and b_f = thickness of the filter layer (m).

In case of cavities underneath the mattresses (= surface irregularities and/or erosion channels) the leakage length is calculated according to (Luth, 1993):

$$\Lambda = \sqrt{\frac{dDk}{k'}} \qquad (18a)$$

$$k = 5.75\sqrt{g}\,\sqrt{\frac{d}{0.6}}\,\log\left(\frac{6\,d}{k_s}\right) \qquad (18b)$$

with: d = depth of cavity (m), k = permeability of the cavity (m/s), and k_{nik} = Nikuradse roughness of the cavities (about 0.5 mm).

Based on the previous calculations (Table 3) some values of the calculated leakage length are given in Table 4 for selected types of mattresses. The thickness of the filter layer is assumed to be 20 cm; the depth of the cavity is assumed 1 cm.

Table 4Calculated leakage lengths based on data from Table 3

Mattress	b_f (m)	D (m)	D_{f15} (mm)	k (mm/s)	k' (mm/s)	Λ (m)
Standard-FP 'square' filter points: 4x4 cm	0.2	0.25	cavity	4800	0.3	6.9
			5	67	0.3	3.6
			1	5	0.3	1.0
Standard-FP 'square' filter points: 5x5 cm	0.2	0.15	cavity	4800	0.5	3.9
			5	67	0.5	2.0
			1	5	0.5	0.6
FPM 'tube' filter points: 2.5 cm^2	0.2	0.15	cavity	4800	0.05	12.0
			5	67	0.05	6.4
			1	5	0.05	1.8
FPM 'tube' filter points: 2.5 cm^2	0.2	0.15	cavity	4800	0.25	5.4
			5	67	0.20	2.9
			1	5	0.20	0.9
Slab 'square' filter points: 10x10 cm	0.2	0.25	cavity	4800	1.9	2.6
			5	67	1.7	1.4
			1	5	1.1	0.5
Slab 'square' filter points: 10x10 cm	0.2	0.25	cavity	4800	0.1	9.0

			5	67	0.1	4.7
			1	5	0.1	1.3
Crib	0.2	0.1	cavity	4800	5	≈ 1.0
			1-5	5-67	5	≤ 0.5

The calculations show that, in general, the leakage length for concrete mattresses is rather large, thus, the uplift pressure can be high. Only the Crib-mat is more permeable providing lower values of the leakage lengths even in case of the presence of cavities.

Based on this kind of calculations the representative/characteristic values of the leakage length for various mattresses can be assumed as follow:

Leakage length Λ (m)

Mattress	on sand*)	on sand**)	on filter
Standard – FP	≤1.0	4.0	3.0 to 4.0
FPM	≈1.5	10	6.0
Slab	≤1.0	2.0	3.0
Articulated (Crib)	≤0.5	≤1.0	≤0.5

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

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

· 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 coupled by means of the geotextile.
· With sufficiently high waves, an upward pressure difference over the mattress will occur during wave run-down, which lifts the mattress.
· The pumping action of these movements will cause the subsoil to migrate, as a result of which an S-profile will form and the revetment will collapse completely.

Taking into the consideration the above failure mechanisms the following design (stability) formula has been derived for the mattresses (Eq. 3b):

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

with:

$$D = \frac{\text{mass per } m^2}{\rho_s}$$ (which can be called $D_{effective}$ or $D_{average}$)

Δ= relative volumetric mass of the mattress (-) = $(\rho_s - \rho)/\rho$
ρ_s= volumetric mass of concrete (kg/m^3)
F=stability factor (-)

Applying this formula the stability factor **F** has been calculated for concrete mattresses on sand (with cavities) and on filter layer. The results are presented in Table 5 and in Figure 12. The value of **F** in the design formula of the black-box model depends on the leakage length and the subsoil/sublayer: **F** = 2 to 4. A permeable mattress on sand has a medium-sized or small leakage length and then the value of F is 3 to 4. A low-permeable mattress on a filter has a large leakage length and therefore an **F**-value of 2 to 3 (Figure 12).

Table 5 Values of F as function of leakage length (Λ) and structural conditions ($0.2 < \tan\alpha < 0.4$; $\sigma_b = 5$ MPa; $H_s < 1.5$ m; $\rho_s = 2300$ kg/m^3)

Leakage length	Mattress on sand	Mattress on filter
Λ = 0.5 to 0.65 m	F = 4.0	F = 4.0
Λ = 1.0 m	F = 3.5	F = 3.3
Λ = 2.4 m	F = 2.9	F = 2.5
Λ = 8.0 m	F = 2.7	F = 2.2

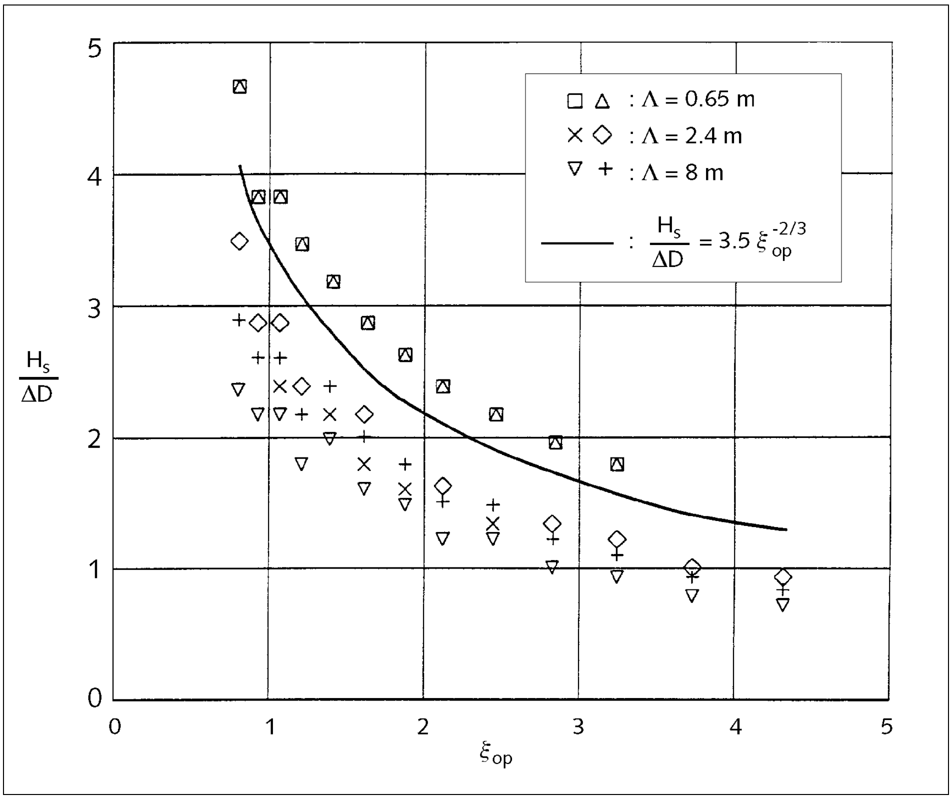

Figure 12 Calculation results for concrete mattresses

The accuracy of F depends on the accuracy of estimation of the permeabilities and the resulting leakage length. For the more precisely determination of the leakage length, one is referred to the analytical model. However, besides the mattresses of a type as, for example, the tube mat (Crib) with relative large permeables areas, the other types are not very sensitive to the exact value of the leakage length. That also means that the stability factor **F** for the most standard concrete mattresses will be only little affected by the accuracy of the estimated permeability of mattress. The representative relative density Δ follows from the standard definition. For the representative thickness D, the average thickness should be filled in.

It can be recommended to use the following values of F in design calculations:
F = 2.5or (≤ 3) - for low-permeable mattresses on (fine) granular filter,
F = 3.5or (≤ 4) - for low-permeable mattress on compacted sand,
F = 4.0or (≤ 5) - for permeable mattress on sand or fine filter (D_{f15} < 2 mm).

The higher values can be applied for temporary applications or when soil is more resistant to erosion (i.e. clay), and the mattresses are properly anchored. It should be noted that according to these calculations the tube (Crib) type mattress has relatively a high stability.
However, this calculation method is not suitable for this kind of mattresses. The large open area of geotextile will in reality be directly exposed to wave action leading to local geotechnical instability of soil under the geotextile. This type of mattress is recommended for bank protection with a limited wave attack (say, wave height lower than 0.5 m). On longer term this open area can be protected by vegetation growing through the geotextile.

The impermeable concrete mattress (i.e. standard Uniform Section Mat/USM) can be designed in respect to the criterion of allowable cracking due to wave impact. Assuming crack-distance 0.5 to 1.0 m and low (normal) position of the phreatic surface under the revetment, the approximate stability criterion will be (Klein Breteler, 1966, Annex 6):

$$\frac{H_s}{\Delta D} = 4 \text{ to } 6 \qquad (19)$$

It can be recommended to use the lower value for (properly reinforced) mattresses placed on granular filters and a higher one when placed on compacted sand or fine filter (D_{f15} < 2 mm). However, when no local uplifting of the mattress is allowed (i.e. because, on long term, it can lead to the deformation of the slope) the lower stability factors are recommended, namely: the value of 2 for mattresses on granular filter and the value of 3 to 4 for mattresses on compacted sand or fine filter. In a case of high position of the phreatic surface under the revetment (i.e. due to the long duration of the high water level) followed by a rapid drop in the external water level, a high static uplift pressure can occur leading possibly to the sliding of the mattress. This sliding stability should be controlled using design criteria mentioned in Chapter 13.

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.

The anchoring of mattresses is described in (Sprague & Koutsourais, 1992).

5.4 Design rules with regard to flow load

A number of characteristic values from literature for the critical flow velocity is given below.

Mattress	Thickness	on slope 1 on 2	on bottom

FP*	100 mm	4.1 m/s	-
USM*)	50 mm	2.7 m/s	3.3 m/s
	100 mm	3.9 m/s	4.7 m/s
	200 mm	5.5 m/s	6.4 m/s

*) FP = Filter Points mat;
**)USM = Uniform Section Mat; similar to an ordinary concrete slab (no Filter Points)

The design formulas are given in Section 3.2.

5.5 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 $1{\cdot}10^{-4}$ and $5{\cdot}10^{-3}$ 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. The design diagrams are presented in Figure 6.

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

Failure type	$S_{op} = 0.03$	$S_{op} = 0.05$
Lifting of toplayer	0.35 m	0.25 m
Partial sliding toplayer	0.80 m	0.60 m
Sliding of toplayer	0.30 m	0.25 m
Sliding of subsoil	0.55 m	0.40 m

Concrete mattresses are mostly stiff and anchored at the top. Therefore, not the sliding and/or uplifting of the toplayer but the sliding of the subsoil is the most dangerous (for H = 1 m and S_{op} = 0.03 the required thickness is 0.55 m). 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, $D + b/\Delta_t$ can be filled in, where b is the thickness of the filter.

Liquefaction
The design rules with regard to liquefaction do not differ from those presented in Section 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. 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 Δ follows from the standard definition. For the representative thickness D one should fill in the (over the surface) averaged thickness.

5.6 Design case for concrete mattress under 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 $H_s/Lop = 0.040$

The water is deep.

Construction:

Slope gradient	$\cot\alpha$	=	3.0
Thickness of top layer	D	=	0.15 m
Relative density	Δ	=	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 ξ_{op} =1.68 from Formula (5)

Revetment parameter **F** = 2.0 to 3.0 (see comment Table 5)

Critical load $\left(\frac{H_s}{\Delta D}\right)_{cr}$ =1.42 to 2.13 from Formula (3b)

From this follows that the maximum permissible wave height H_s is 0.28 to 0.42m.

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

Sliding due to elastic storage

The permissible wave height for the chosen block thickness follows from the design diagrams. For the top layer, D_{eq} = 0.32m can be used because of the existing filter layer.

Lifting of a part of the revetment	H_s	=	1.05 m from Diagram 3
Sliding of the whole revetment	H_s	=	1.20 m from Diagram 4
Sliding of the subsoil	H_s	=	0.85 m from Diagram 1

The critical wave height with regard to elastic storage is found to be 0.85m. For a wave height of 0.40m there is no danger for instability.

Softening

The sand is reasonably well compacted and the slope gradient is 1:3. According to the design rules in Section 3.3, this means that there is no danger of softening occurring.

Drop in the water level

Substitution of the known variables in Formula (16) 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 the original reports (Annex 2 in Klein Breteler, 1996).

6 STABILITY OF SAND-FILLED MATTRESSES

6.1 General

A sand mattress consists of two geotextiles attached onto each other, between which sand is interposed. These mattresses are designed to be laid flat on a prepared slope, joined together, and then filled. They form a large mass of sausage-like or pillow-like sand sections. They should be installed according to the manufacturer's recommendations.

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 also vulnerable to collision, (drifting) ice, floating bulky debris, sunlight and chemical degradation.

The number of types/products concerning the sand-filled mattresses is rather limited.

6.2 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 (max. 1.5 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 only (Klein Breteler, 1996). Based on these, the following value for F in the design formula is recommended according to the black-box model: F = 4 to 5, so that:

$$\frac{H_s}{\Delta D} = \frac{4 \text{ to } 5}{\xi_{op}^{2/3}} \tag{20}$$

for $H_s \leq 1.0$ m (< 1.5 m as an upper limit of application).

In this formula the relative density including pores Δ_t (Eqs. 15a,b) should be filled in for the representative relative density. For the representative thickness D of the mattress, the average thickness should be filled in:

100% filled: $D/D_d = 0.7$ to 0.8

90% filled: $D/D_d = 0.6$ to 0.7

with: D = average thickness of the mattress (m) and D_d = maximum diameter of the sausages (m).

The above design formula matches the experience gained during the prototype tests in the Hartel canal reasonably well and is conservative compared to the results of the small-scale model investigation (DELFT HYDRAULICS, 1975), but suggests a higher stability than according to Pilarczyk (1990). 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 due to instability of the edges and interconnections.

6.3 Design rules with regard to flow load

Above a flow velocity of 1.5m/s (max. 2 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 3.2. The detailed design specifications and stability criteria for sand-filled sausages mattress (Profix-mat) can be found in (Tutuarima & van Wijk, 1984).

6.4 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 μm, 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.

The stability diagrams are presented in Figure 6. These diagrams are basically developed for the concrete-filled systems. For sand-filled systems the maximum allowed wave height must be reduced by factor 1.1. The geotechnical response of sand mattresses is similar to those of sandbags (see Section 7.4 and Tables 6a,b).

Note:
Stability of the sand-filled (pocket) mattress as shown in Figure 13 (Harris, 1987,1989) can be calculated in the same way as it is done for single placed bags (see Section 7).

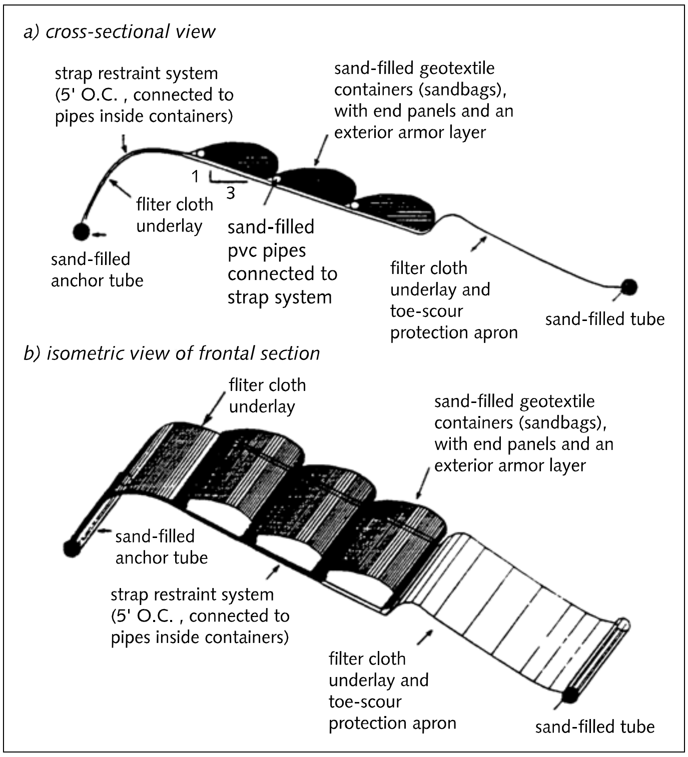

Figure 13 Sand-filled container (pocket) mattress

7 STABILITY OF GEOBAGS

7.1 General

Geobags or tubes can be 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 14).

The most common use for sandbags in hydraulic engineering is for temporary structures. The reasons why sandbags are not or hardly used for permanent structures 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.

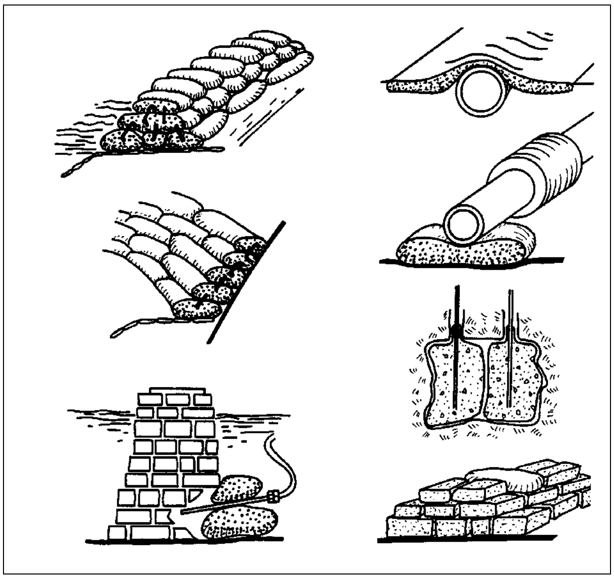

Figure 14 Application of geobags and containers

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 vegetation 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 structures for beach nourishments.

Because this material is easy to use and cheap, it is extremely suitable for temporary structures. 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 structure.

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

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 halfoverlapping 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 elements may easily damage the casing of the element. Geosystems must not be filled completely. With a fill ratio of approximately 75% an optimum 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 on geobags can be found in Pilarczyk (1995) and Wouters (1995).

New developments concern the large hydraulically filled geotubes and hydraulically or mechanically filled geocontainers (in combination with a split barge). Information on these systems can be found in Chapter 17 and in Leshchinsky (1995), Pilarczyk (1996, 1997), and Den Adel et al (1996).

7.2 Slope protection

In general, based on experience, the sand-filled structure can be used as temporary structures (i.e. to learn the natural interactions/responses), or as permanent structures at locations with relatively low wave attack ($H < 1.5$m), or as submerged structures where direct wave forces are reduced. The mortar-filled systems can resist much higher wave and current loading and, if necessary, can be interconnected by bars or by creating a special interlocking shape.

From the literature review (Wouters, 1995) it can be concluded that the stability of the coastal structures composed of geosystems (bags, geotubes, geocontainers) can usually be expressed in a similar way as for rock, namely in terms of the k_D factor in Hudson's formula or in terms of the parameter $H/\Delta D$. All relevant literature (model-) results (Jacobs, 1983, Tekmarine, 1982 and Porraz, 1979), expressed by the $H/\Delta D$ parameter, are presented as a function of $H/\Delta D$ vs. ξ_o in Figure 15 ($\Delta = \Delta_t$).

Based on the results summarized in Figure 15, the stability criteria have been defined Wouters, 1995). 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 (Eq. 3). The exponent of ξ_0 is 1/2 instead of 2/3.
(N.B.: exponent 2/3 provides some additional safety)

For regular waves the recommended formula is as follows:

$$\left(\frac{H}{\Delta D}\right)_{cr} = \frac{3.5}{\sqrt{\xi_o}} \qquad (21)$$

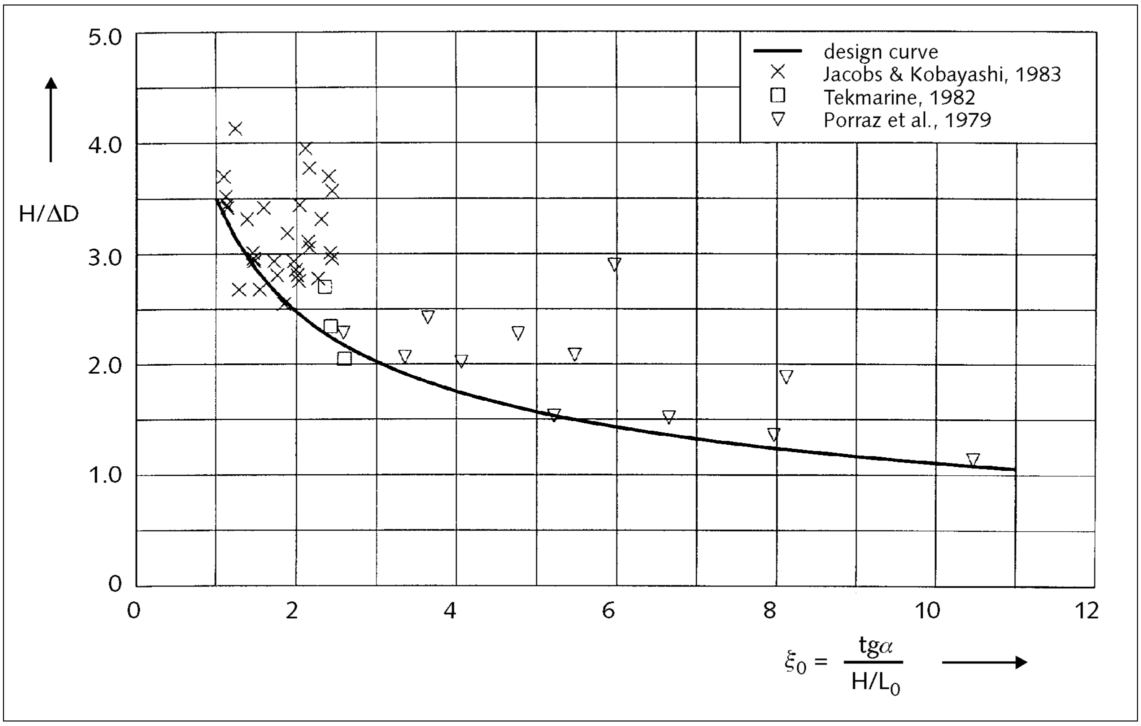

Figure 15 Summary of the stability test results for sand- and mortar-filled bags on slopes

In which Δ is the relative density if the pores are completely filled with water (Δ_t). 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, namely $H/H_s = 1.4$) this results in the following stability relation for random waves:

$$\left(\frac{H_s}{\Delta D}\right)_{cr} = \frac{2.5}{\sqrt{\xi_{op}}} \tag{22}$$

Stability of crest elements

For concrete units used as a protection element on the crest of a low or underwater breakwater the criteria based on the model results for geotubes (Waterloopkundig Laboratorium = Delft Hydraulics, 1973) can be applied as a first approximation (see Section 5.6).

For units lying parallel to the axis of a structure, it is found that the following stability relation for regular waves can be used:

$$\left(\frac{H}{\Delta b}\right)_{cr} = 3.2\left(\frac{H}{L_0}\right)^{1/3} \tag{23a}$$

or roughly,

$$\mathbf{H/\Delta b = 1} \tag{23b}$$

In which b is the width of the unit. Should two units be connected, the widths of both sausages together can be filled in for b.

If the unit is placed with its longitudinal direction perpendicularly to the axis of the breakwater, the following stability relation applies:

$$\left(\frac{H}{\Delta l}\right)_{cr} = 1.0 \tag{24}$$

In which l is the length of the unit (with maximum l = 3 times thickness).

Additional design considerations and stability criteria for various geobag structures, based on prototype tests, can be found in (Ray, 1977).

7.3 Design rules with regard to flow load

Above a flow velocity of 1.5m/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 of exposed systems. The stability can be calculated (approximately) with design formula (Eq. 10) and constants given in Section 3.2. Additional information can be found in Klein Breteler (1996, Annex 1).

7.4 Soil-mechanical stability of sandbags and sand mattresses

The soil-mechanical stability should be treated according to the criteria mentioned in Section 3.3. As an example, the equivalent thickness of sand-filled systems (sandbags or sand mattresses), required to avoid various failure mechanisms, is calculated below (Tables 6a,b) for the slope 1 on 3 and the wave height equal to 1 m and 2 m.

Table 6a Equivalent thickness of sand-filled systems for slope 1 on 3 and H = 1 m

Failure type	$S_{op} = 0.03$	$S_{op} = 0.05$
Lifting of toplayer	0.40 m	0.30 m
Partial sliding toplayer	0.90 m	0.70 m
Sliding of (total) toplayer	0.35 m	0.25 m
Sliding of subsoil	0.60 m	0.55 m

Table 6b Equivalent thickness of sand-filled systems for slope 1 on 3 and H = 2 m

Failure type	$S_{op} = 0,03$	$S_{op} = 0,05$
Lifting of toplayer	0.90 m	0.70 m
Partial sliding of toplayer	> 1 m	> 1 m
Sliding of (total) toplayer	0.85 m	0.65 m
Sliding of subsoil	> 1 m	> 1 m

For both cases the mechanism "partial sliding toplayer" is a determinant factor. In case of sand-mattress this danger can be avoid by a proper anchoring. However, due to the susceptibility to the sliding of the subsoil these systems are not suitable for heavy wave attack. In case of properly compacted soil the softening (liquefaction) of a subsoil is only of importance for slopes steeper than 1 on 3.

To check whether the sudden drop in the water level can play a role in the design, we assume that the permeability of the sand-filled system is equal to that of the subsoil. The criterion (Eq. 16) from Section 3.3 will be applied. To calculate the leakage length we assume D = 0.2 m, b = 0.3 m and k/k'= 1. Then, the leakage length is equal to 0.24 m. Using $\Delta_t = 0.9$ and slope 1 on 3 one may calculate that the leakage length must be not larger than 1.05 m. In our case this condition is satisfied.

7.5 Design case for 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 0.25m. 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	Δ_t	=	1.0

The underlayer consists of clay.

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

Wave length	L_{op}	=	11.4 m	from Formula (7)
Wave steepness	H_s/L_{op}	=	0.035	
Breaker parameter	ξ_{op}	=	1.78	from Formula (5)
Critical load	$\left(\frac{H_s}{\Delta D}\right)_{cr}$	=	1.87	from Formula (22)
Required block thickness	D	=	0.21 m	

The chosen sandbags (D = 0.25 m) prove to be stable.

The soil-mechanical stability of sandbags with D = 0.25 m is discussed below:

Lifting due to elastic storage:
From interpolation of Diagrams 4a,b in Figure 6 it follows that the permissible wave height is about 0.9m.

Sliding of the top layer due to elastic storage:
From Diagrams 2a,b it follows that the permissible wave height is about 0.95m.

Sliding of the subsoil due to elastic storage:

From Diagrams 1a,b it follows that the permissible wave height is 0.75m.

The above values for the permissible wave height prove that in this case no danger is expected as a result of elastic storage. However, the cover layer consists of individual bags, and in this case the partial sliding of top layer can be decesive (Diagram 3).
Diagram 3 provides the permissible wave height of only 0.45 m which is still larger than the design one. However, this last figure stresses the necessity of a proper (compacted) placing of sandbags in the zone of direct wave attack.

Softening:
The design guidelines in Section 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.

8 STABILITY OF GABIONS

8.1 Introduction

Gabions are used throughout the world to protect river banks, dikes and other slopes against the erosive forces of currents and waves. The gabions are made of rectangular baskets of wire mesh, which are filled with stones (Figure 16).
The idea of the protection system is to hold the rather small stones together with the wire mesh. Waves and currents would have easily washed away the small stones, but the wire mesh prevents this.

There are various construction methods. Some of them are presented in Figure 16. There are for example single cell gabions and systems with multiple cells. The individual cells (or gabions) can be placed adjacent to each other on the slope, or they can be staged to form a steep slope (stairs). The latter is also used along roads to give a steep slope the necessary geotechnical stability.

A typical length of gabions is 3 to 4 m, a width of 1 to 3 m and a thickness of 0.3 to 1 m. The gabions with small thickness (less then 0.5 m) and large length and width are usually called reno-mattresses. This name originates from the river Reno in Italy, where they are used as bank protection.

The slope protections are usually constructed by placing the empty cells on the slope and connect them. Then they are filled with gravel and closed.

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

The costs of a gabion protection of a slope, which is subjected to heavy wave attack, is not higher or lower then ordinary slope protection systems, such as asphalt, penetrated rip-rap, block revetments etc. (de Looff 1990). It depends on the availability of materials which system will be the best solution.

In this Section attention is focused on the stability of gabions under wave or current attack.

8.2 Hydraulic loading and damage mechanisms

Wave attack on gabions will lead to a complex flow over the gabions and through the gabions. During wave run-up the resulting forces by the waves will be directed opposite to the gravity forces. Therefore the run-up is less hazardous then the wave run-down.
Wave run-down, as it was already mentioned in Section 2, will lead to two important mechanisms:

· The downward flowing water will exert a drag force on top of the gabions and the decreasing freatic level will coincide with a downward flow gradient in the gabions.
· During maximum wave run-down there will be an incoming wave that a moment later will cause a wave impact. Just before impact there is a ‘wall’ of water giving a high pressure under the point of maximum run-down. Above the run-down point the surface of the gabions is almost dry and therefore there is a low pressure on the gabions. The interaction of high pressure and low pressure is shown in Figure 1.

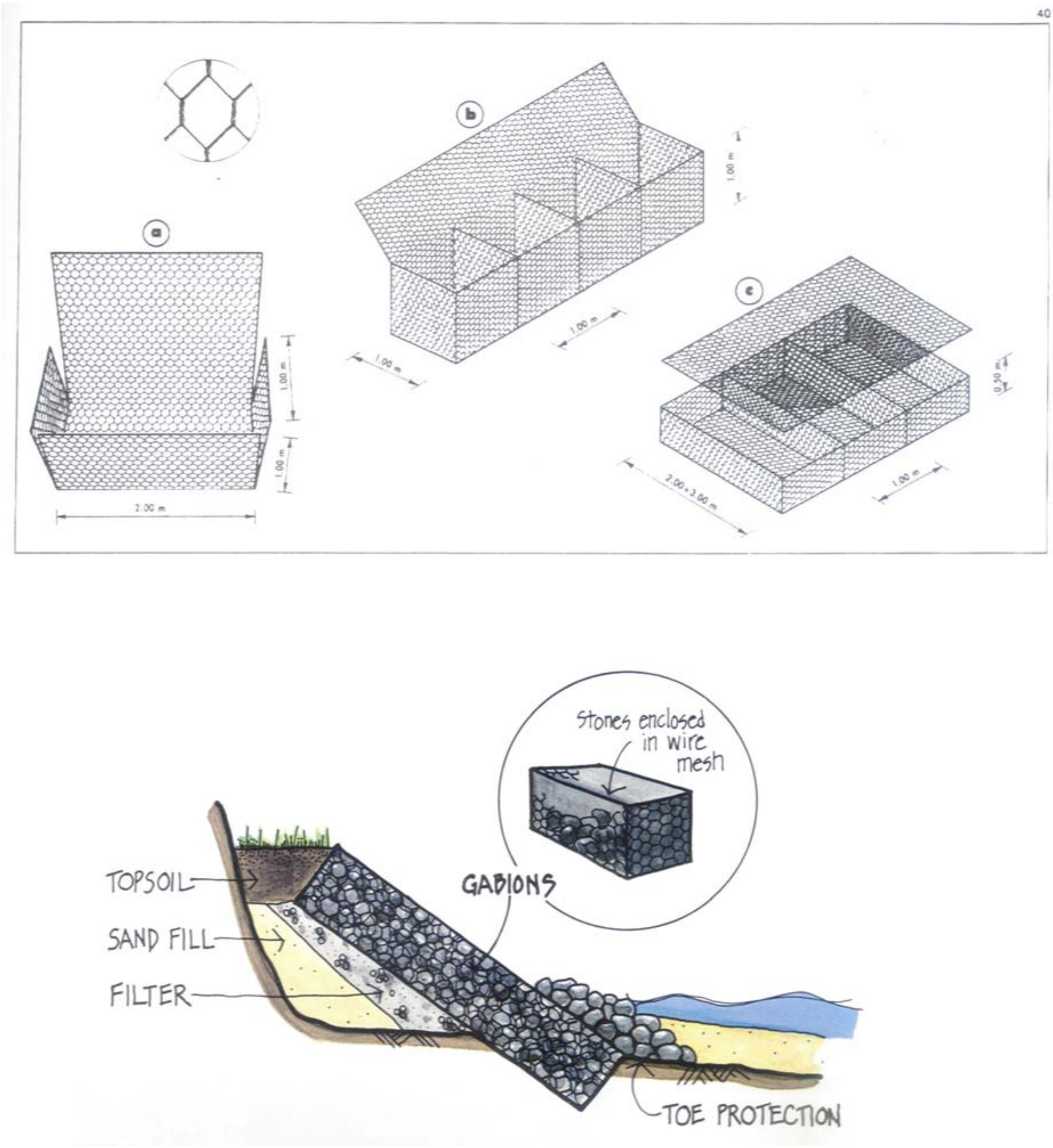

Figure 16 Examples of gabions and stone-filled mattresses

The first mechanism can be schematised by a free flow in the gabions with a typical gradient equalling the slope angle. This means for a slope of 1:4, for example, a gradient (I) of approximately 0.25, resulting in a force along the slope in downward direction.

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

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

with: f = friction of gabion on subsoil; α = slope angle.

From this criterion we see that a steep slope will easily lead to the exceeding of the friction forces, and furthermore a steep slope is shorter then a gentle slope and will give less support to the section that tends to slide down. Furthermore, we can conclude that the stability increases if we decrease the slope angle (or increase the friction of gabions on the subsoil). The thickness of the gabions will not contribute to the stability.

The high pressure front will lead to an upward flow in the gabions. This flow will meet the downward flow in the run-down region. The result is an outward flow and uplift pressure near the point of maximum wave run-down. The situation is given in Figures 1 .

Damage mechanisms

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

1.Instability of the gabions

a)The gabions can slide downwards, compressing the down slope mattresses

b)The gabions can slide downwards, leading to upward buckling of the down slope mattresses

c)All gabions can slide downwards

d)Individual gabions can be lifted out due to uplift pressures

2.Instability of the subsoil

a)A local slip circle can occur, resulting in a S-profile

b)The subsoil can wash away through the gabions

3.Durability problems

a)Moving stones can cut through the mesh

b)Corrosion of the mesh

c)Rupture of the mesh by mechanical forces (vandalism, stranding of ship, etc.).

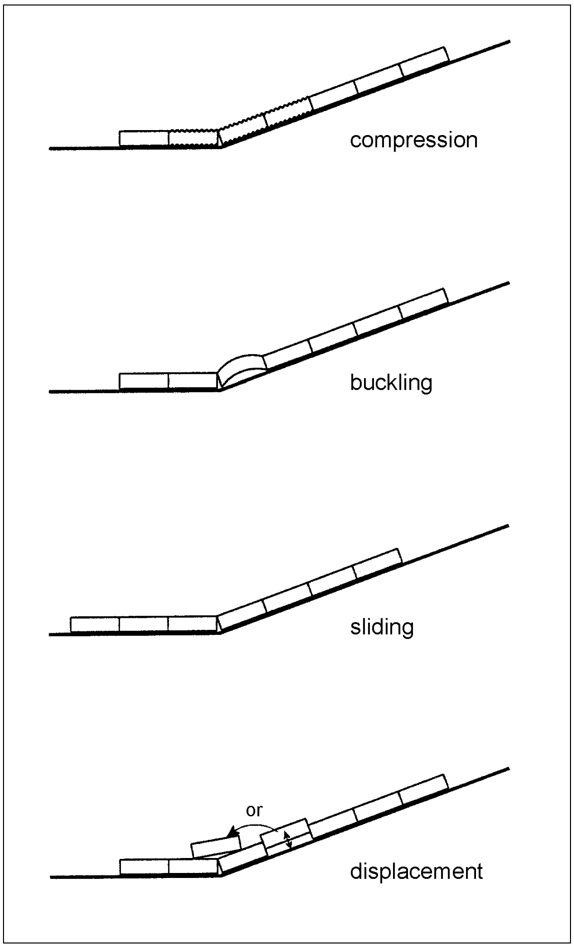

Figure 17 Damage mechanisms

The various modes of failure of the gabions itself are shown in Figure 17. However, the similar failure modes can also happend for geobags and some types of blockmats and geomattresses.

8.3 Stability of gabions under wave attack

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: $\Lambda = 0.77\ D$.

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 Manual (CUR/RWS, 1995a) provides stability relations, which indeed match the line according to the black-box model well.

After complicated calculations the uplift pressure in the gabions can be derived (Klein Breteler, 1996, Annex 8). The uplift pressure is dependent on the steepness and height of the pressure front on the gabions (which is dependent on the wave height, period and slope angle), the thickness of the gabions and the level of the freatic line in the gabions. It is not dependent on the permeability of the gabions, if the permeability is larger then the subsoil. The equilibrium of uplift forces and gravity forces leads to the following (approximate) design formula (Eq. 3b):

$$\frac{H_s}{\Delta D} = F \bullet \xi_{op}^{-2/3}$$ with $6 < F < 9$ and slope of 1:3 ($\tan\alpha = 0.33$).

with:
- H_s = significant wave height of incoming waves at the toe of the structure (m)
- Δ = relative density of the gabions (usually: $\Delta \approx 1$)
- D = thickness of the gabion (m)
- F = stability factor (-)
- ξ_{op} = breaker parameter (-) = $\tan\alpha/\sqrt{(H_s/(1.56T_p^2)}$
- T_p = wave period at the peak of the spectrum (s)

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

Results of small scale model tests

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

Brown used gabions of 30 cm long (upward along the slope), 20 cm wide (along the waterline) and a thickness varying between 1.8 cm and 4.1 cm in the model. The stone size was approximately 7 mm and the mesh width was 4 to 5 mm. The slope varied between 1:1.5 and 1:4. The subsoil was sand. Between the sand and the gabions he used a granular filter or a geotextile. The tests resulted in sliding of the gabions and upward buckling of the gabions near the toe of the structure. Sometimes the sliding was preceded by uplift of the gabions. Brown has summarised his results in two criteria:

Sliding $$\frac{H}{\Delta D} = \frac{2{,}8}{\tan\alpha} \qquad (25a)$$

(regular waves)

Uplifting $$\frac{H}{\Delta D} = \frac{7}{(\tan\alpha)^{1/3}} \qquad (25b)$$

Pilarczyk (see PIANC, 1987)) has transformed these results to criteria valid for irregular waves:

Sliding: $$\frac{H_s}{\Delta D} = \frac{2}{\tan\alpha}$$ for slopes steeper then 1:3 ($\tan\alpha > 0.33$) (26a)

Uplifting: $$\frac{H_s}{\Delta D} = \frac{4}{(\tan\alpha)^{1/3}}$$ for slopes up to 1:3 ($\tan\alpha \leq 0.33$) (26b)

Ashe (1975) used gabions of 7.5 cm long (upward along the slope), 30 cm wide (along the waterline) and a thickness of 3.8 cm in the model. The stone size was varied from approximately 6 to 16 mm and the mesh width was 6 mm. The slope varied between 1:1.5 and 1:3. The subsoil was sand. Between the sand and the gabions he used a granular filter or a geotextile. The tests resulted in sliding of the gabions and upward buckling of the gabions near the toe of the structure. Sometimes the sliding was preceded by uplift of the gabions.

Experiments with irregular waves are rather scarce. Van Dijk (1974) and Van Hijum (1975) presents some test results from small-scale tests performed for potential use of gabions as protection of artificial islands. When stability of gabions was evaluated by using the Hudson's formula the K_D-values vary from 3 to 4.5 for two layer system to 7.0 for one layer of gabions.

All test results and a formula of Pilarczyk (1990) are summarised in Figure 18 (the results are adjusted for the influence of wave breaking during some of the tests).

Black-box model by Pilarczyk

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:

$$\frac{H_s}{\Delta D} = \frac{9\cos\alpha}{\xi_{op}^{2/3}} \qquad (27)$$

The difference from the form of the black-box formula (3b) used up to now is only the factor $\cos\alpha$ in the right term. In black-box terms, therefore, $F = 9 \cdot \cos\alpha$ is recommended in this case. Because formula (3b) is generally recommended for slopes with gradients of 1:2 or milder, $\cos\alpha = 0.89$ to 1.00 applies, so that $F \approx 8$. Because of limited verification data it is recommended for the design to use $F = 7$ in Eq. 3b, or $F = 8 \cdot \cos\alpha$ in Eq. 27 (especially, for slopes steeper than 1 on 2). For the representative relative density, $\Delta = \Delta_t$ must be filled in (see Eq. 4b); usually, $\Delta_t = (1-n)\Delta \approx 1$.

No unstable measuring results were found for ξ_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. However, the presented black-box formula with $F = 7$ appears to give a safe value here.

It is especially in the above-mentioned area with ξ_o-values smaller than 2 where the lifting of the gabion determines the stability criterion. For larger values of ξ_o (i.e. steepre slopes), the aspect of sliding (stimulated by lifting) increasingly plays a part (Eq. 26a).

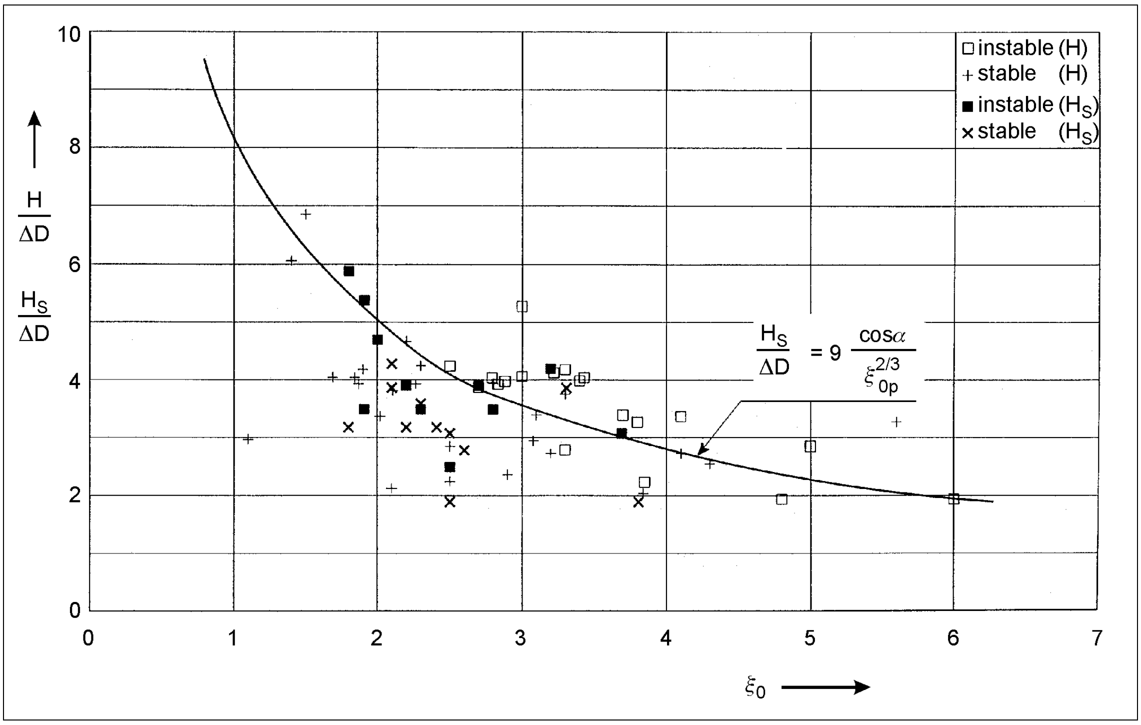

Figure 18 Summary of test results and design formulations

8.4 Stability of gabions under current attack

The stability of gabions on the bottom of gently sloping rivers and on river banks (steepness up to 1:2) can be approximated with the formula of Pilarczyk (1990), Eq. 10.

For gabions it is recommended to use a critical shear stress parameter Ψ_{cr} = 0.070. The stability factor for current, φ, is 1.0 for exposed edges of gabions and 0.75 for mattresses, and is 0.5 for continuous slope protections. For the representative relative density, the relative density including pores should be filled in (Δ_t). The representative thickness D is the thickness of the gabion measured perpendicularly to the slope.

The value of parameter '*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 7.

Table 7 Values of *a* for gabions

Thickness (m)	Gabions with small stones	Gabions with large stones
0.1	0.34	0.42
0.2	0.31	0.40
0.3	0.29	0.37
0.4	0.29	0.35
0.5	0.28	0.33

If velocity at gabion surface ,u_b, is known; the depth factor K_h = 1 and u = u_b. The roughness of the gabion depends on the stone sizes, among other things; roughly k_s/D_f = (2 to 5) where D_f = grain size of gabion filling (m). Angle of internal friction $\theta \approx 50°$.

The other influence factors are given in Section 3.2.

It should be mentioned that the above formulas give a first approximation of the dimensions of the gabions. For a final design it is recommended to ask for assistance of specialists, who can take into account a variability of a large section of the river.

8.5 Stability of subsoil under gabions

Erosion of subsoil

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

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

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

Geotechnical stability

Wave attack on gabions will give large pressure fluctuations in the pore-water in the sand subsoil. These pressure fluctuations can lead to local slip circle failure. The slope will then have a S-profile. To avoid this damage mechanism we can either select thick gabions or a gentle slope. For the design of gabions on a sand subsoil it is recommended to use Diagrams 6a,b in Figure 19. The use of this figure will give conservative (safe) results.

Motion of filling material

It is important to know if the filling material will start to move during frequent environmental conditions, because it can lead to rupture of the wire mesh. Furthermore the integrity of the system will be effected if large quantities of filling material is moved.

During wave attack the motion of the filling material usually only occurs if $\xi_{op} < 3$ (plunging waves). Based on the Van der Meer's formula for the stability of loose rock (CUR/RWS, 1995B) and the assumption that the filling of the gabion will be more stable then loose rock, the following criterion is derived (Van der Meer formula with permeability factor: $0.1 < P < 0.2$; number of waves: $2000 < N < 5000$; and damage level: $3 < S < 6$):

$$\frac{H_s}{\Delta_f D_f} = \frac{F}{\sqrt{\xi_{op}}} \qquad \text{with } 2 < F < 3 \tag{28}$$

with: H_s = significant wave height of incoming waves at the toe of the structure (m)
Δ_f = relative density of the grains in the gabions (usually: $\Delta \approx 1.65$)
D_f = diameter of grains in the gabion (m)
F = stability factor (-)
ξ_{op} = breaker parameter (-) = $\tan\alpha/\sqrt{(H_s/(1.56T_p^2)}$

T_p = wave period at the peak of the spectrum (s)

Diagram 6a

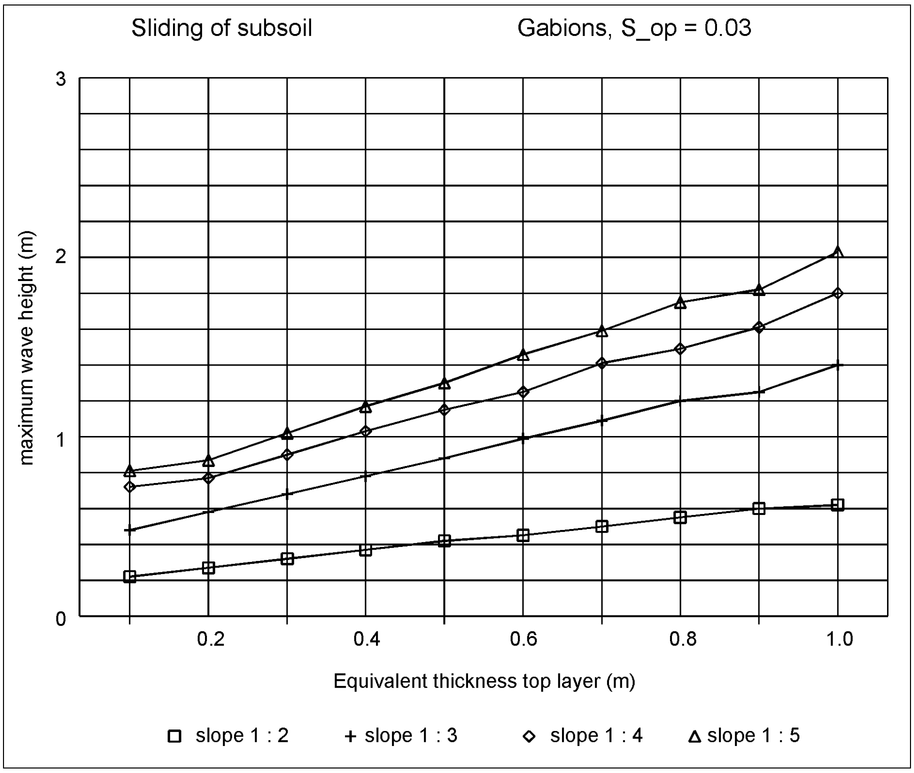

Diagram 6b

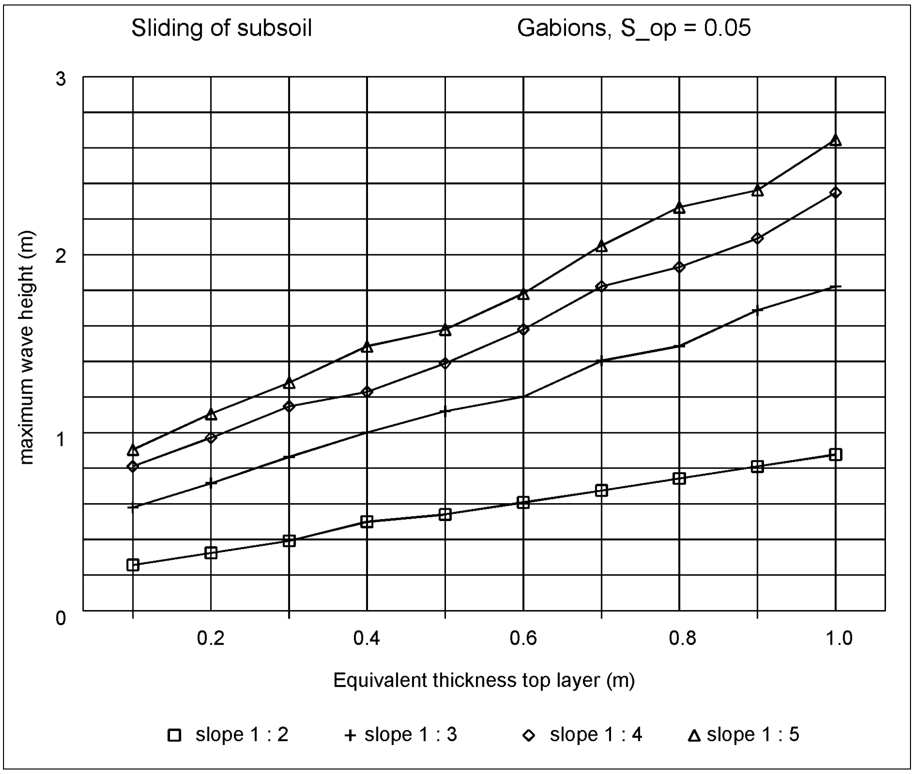

Figure 19 Design diagrams for sliding of subsoil (for gabions)

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

8.6 Calculation example (gabions loaded by 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 = 0.45m). 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 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	Δ_m	= 1.0

Below, the required block thickness is determined on the basis of the discharge, see Eq. 13.

Revetment parameter	a =	0.29	from Table 7
Equivalent wash-over height	h_{eq}	= 0.45m	from Formula (13d)
Equivalent discharge	q_{eq}	= 0.51m^2/s	from Formula (13e)

Substitution of these values (q_{cr} by q_{eq}) in the design formula (13b) provides:

Required thickness of the top layerD = 0.37m

The chosen thickness of 0.40 m is therefore satisfactory.

9 CONCLUSIONS

Alternative systems can be a good and mostly cheaper alternative for more traditional materials/systems. These new systems deserve to be applied on a larger scale.
Information presented on the stability criteria will be of help in preparing the preliminary alternative designs with these systems. However, there are still many uncertainties in the existing design methods. Therefore, further improvement of design methods and more practical experience at various loading conditions is still needed.

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The stability of gabions on the bottom of gently sloping rivers (up to 1:20) and on river banks (steepness up to 1:2) can be approximated with the formula of Pilarczyk (1990), Eq. 10:

$$\Delta D = \phi_c K_T \frac{0.035}{\Psi_{cr}} \frac{K_h}{K_s} \frac{u^2}{2g}$$

with: Δ = relative density of the gabions (usually: $\Delta \approx 1$)
- D = thickness of the gabion (m)
- φ_s = stability factor for current (-)
- K_T = turbulence and shear stress adjustment factor (-)
- Ψ_{cr} = critical shear stress parameter (-)
- K_h = depth factor (dependent on velocity profile) (-)
- K_s = slope factor (-)
- u = mean velocity (depth averaged) (m/s)
- g = acceleration of gravity (9.8 m/s^2)

In this equation we recognise at the left side of the =-sign the strength and on the right side the load.

For gabions it is recommended to use a critical shear stress parameter Ψ_{cr} = 0.070. The stability factor for current, φ_s,, is 1.2 for exposed edges and is 0.6 for continuous slope protections. The other influence factors are given in Section 3.2.

It should be mentioned that the above formulas give a first approximation of the dimensions of the gabions. For a final design it is highly recommended to ask for assistance of specialists, who can take into account a large section of the river.

The other influence factors can be calculated as follows:
turbulence and shear stress adjustment factor K_T:
Normal turbulence (quiet river): $K_T = 1.0$
Non uniform flow with increased turbulence (below stilling basins, in outer bends with r/B > 2): $K_T = 1.5$
High turbulence (under hydraulic jump, at local disturbances, in outer bends with r/B < 2): $K_T = 2.0$
Heavy circumstances (jet impact, screw race at small water depth): $K_T = 2.5$
depth factor, or velocity profile factor, K_h:
Developed velocity profile: $K_h = 2/(\log(10h/k_n))^2 = 628/C^2$
Non developed velocity profile: $K_h = (h/D_f)^{-0.2}$
Shallow water ($h/D_f < 5$): $K_h = 1$
If velocity at gabion surface is known: $K_h = 1$ and $u = u_b$
slope factor K_s:
river banks (flow parallel along water line): $K_s = \cos\alpha \sqrt{(1 - \tan^2\alpha/\tan^2\theta)}$
bottom protection: $K_s = \cos\alpha_b$

with: r = centre line radius of river bend (m)
- B = width of river (water surface width at upstream side of bend) (m)
- h = water depth (m)
- k_n = roughness = between $2D_f$ and $5D_f$
- C = Chezy coefficient (-)

D_f = grain size of gabion filling (m)
α = slope angle (perpendicular to flow direction) (^{o})
θ = angle of internal friction (if unknown: use approximately 50^o)
u_b = velocity at gabion surface (m/s)
α_b = slope angle of river bottom (parallel along flow direction) (^{o})