Wave Forces and Structural Response of Placed Block Revetments on Inclined Structures

K.W. Pilarczyk[1], M. Klein Breteler[2] and A. Bezuijen[3]

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Contents:

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1) Manager R & D, Rijkswaterstaat, Road and Hydraulic Engineering Division, P.O. Box 5044, 2600 GA Delft

2) Manager Coastal Structures Group, Delft Hydraulics, P.O. Box 152, 8300 AD Emmeloord

3) Senior researcher, Delft Geotechnics, P.O. Box 69, 2600 AB Delft, the Netherlands

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Abstract

A summary of the theoretical and empirical knowledge of the stability of block-revetment structures under wave attack is given. This has been accomplished by selecting the very essence of the design formulas from (CUR/TAW, 1992/1994) and completing it afterwards with a database of results of large-scale model studies from all over the world.

Furthermore, the additional filter criteria and geotechnical criteria will be presented. For each of these mechanisms easy to use design formulas are given. Finally, the specific design considerations related to block revetments will be discussed.

1 Introduction

Revetments of placed blocks or block-mats are often used as a protection of slopes of various coastal structures against wave attack. Extensive studies have been performed during the last decade on the stability of placed block-revetments, mainly in the Netherlands, but also abroad. The studies in the Netherlands have been carried out by DELFT HYDRAULICS and DELFT GEOTECHNICS in cooperation with and commissioned by the Netherlands Ministry of Transport and Public Works (Rijkswaterstaat). It has led to the establishment of a large database with results of model and prototype studies, and to the development of a set of design formulas. In addition, it also resulted in a practical design handbook (CUR/TAW, 1992/1994). Some of these results are also incorporated in the PIANC-report (1992).

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1) Manager R & D, Rijkswaterstaat, Road and Hydraulic Engineering Division, P.O. Box 5044, 2600 GA Delft

2) Manager Coastal Structures Group, Delft Hydraulics, P.O. Box 152, 8300 AD Emmeloord

3) Senior researcher, Delft Geotechnics, P.O. Box 69, 2600 AB Delft, the Netherlands

Examples of the structures considered in this paper are shown in Figure 1. Such structures are found on banks of rivers and lakes, sloping seawalls and seadikes, jetties, and in harbours. The main cause of damage of structures in this application is wave attack. Damage by current is only found when applied in spillways, steep canals, etc., but these are not considered here.

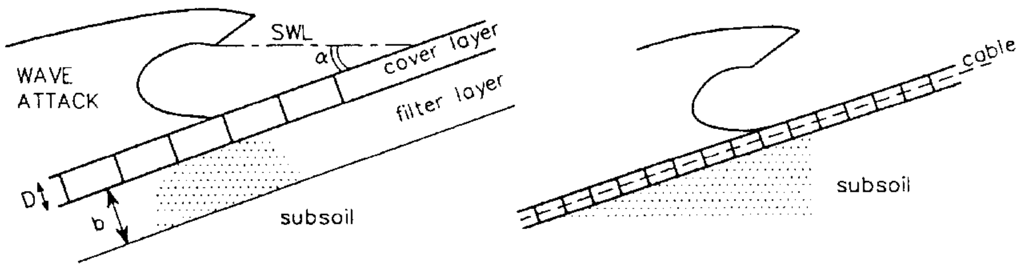


Figure 1. Examples of block-revetment structures (cross-sections)

Basalt columns are an example of natural stone that can be applied as placed blocks to form the cover layer of a revetment. Prefabrication of concrete blocks, however, is often a very common alternative for natural blocks.

The advantage of blocks precast on or close to the site is the independence of stone supply. This holds in particular when supply has to be arranged from distant sources. Other advantages are that specifications on material size, shape and density can be met easily through the fabrication process. By choosing the right aggregates the material characteristics can also be influenced (density, porosity, surface roughness).

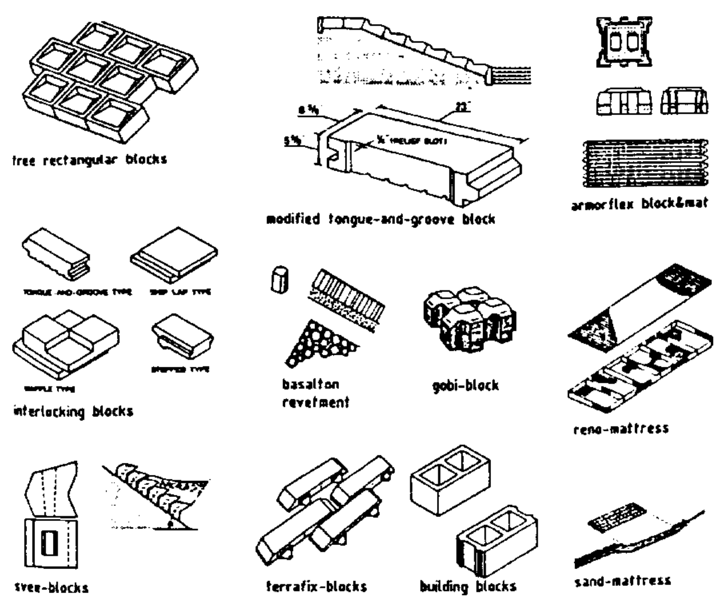


Figure 2. Examples of revetment blocks

Artificial blocks are often designed to provide for additional strength. This additional strength allows for reduction of the weight or size of the stones. The fabrication of specified elements permits the application of cables to connect individual elements, which improves the stability. These cables can be steel wires or polymer fibres. Even without special measures a mutual friction between (closely) placed blocks will provide for an extra stability against uplifting. In the design often no account is taken of frictional forces, which leads to a conservative design. However, in recently developed methods some friction can be taken into account.

Placed blocks can be applied together with appropriate sublayers. In case of repair a certain area has usually to be replaced. The placement of preconnected blocks or blockmats usually requires special, though rather simple, equipment. Working from floating equipment is limited due to wave-induced movements.

In general, a revetment system will consist of a number of layers, the principal of which are the cover layer, filter layer(s) and, as far as necessary, complementary sublayer(s). In the following it will be shown that a revetment system must be designed as an integrated system of cover layer, sublayers and subsoil (Figure 1) (PIANC, 1992 and CUR/TAW, 1992/94).

The failure mechanisms, which should be considered in designing block revetments, are among others:

1° uplifting of blocks
2° migration of subsoil particles through the granular filter and/or cover layer
3° geotechnical instability of sublayers/subsoil
4° sliding of the revetment

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

To determine the stability of the cover layer the $H_s/\Delta D$ ratio versus the breaker index ξ_{op} (surf-similarity parameter) can be used, with:

$$\xi_{op} = \frac{\tan\alpha}{\sqrt{H_s/L_{op}}} \qquad (1)$$

A theory concerning the uplift of individual blocks (1°) will be presented briefly in the following sections, resulting in a formula including all significant parameters that influence the stability. This theory will be simplified for practical application and then fitted to the results of large-scale model studies. Then, the additional filter criteria (2°) and geotechnical criteria (3°) and (4°) will be presented. Finally, the specific design considerations related to block revetments will be discussed.

2 Cover layer stability

2.1 External wave loading

Upon breaking on a slope, regular waves exert cyclic hydraulic loads. On the basis of physical model tests in wave tanks good knowledge has been obtained of the relevant load phenomena within a wave cycle. For different types of revetments, different moments or periods from the wave cycle are decisive for the stability of the cover layer.

The external loads can be quantified by way of physical model tests and with numerical methods (Petit et al. 1994; Van Gent et al. 1994). The practical use of the numerical simulations is still very limited, especially regarding the wave-induced pressure distribution on a slope (in space and time).

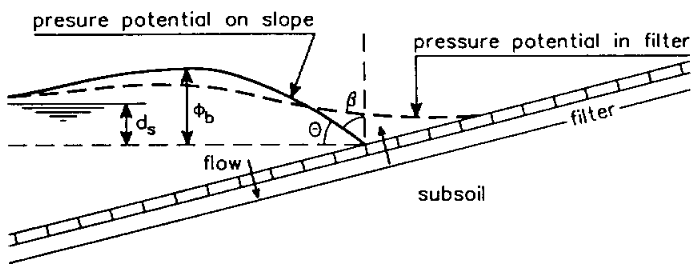


Figure 3. Schematized damage-mechanism caused by pressure potential (piezometric head) over the cover layer

A much simpler approach towards a computation of the relevant wave loads is to abandon a full description of time and place dependent wave pressures on the slope, and to concentrate only on the instant of critical wave loads. For placed block revetments the most critical load situation occurs at the moment of maximum wave run-down. This proved to have general validity for the structures considered here. The critical pressure front is schematized sufficiently by the parameters θ (maximum gradient), ϕ_b (maximum piezometric head) and d_s as shown in Figure 3. Empirical formulas for ϕ_b and β, based on wave pressure measurements in a small-scale model with slopes between $1/2 < \tan\alpha < 1/4$ and wave steepness between $0.01 < H/L_0 < 0.07$ (regular waves), are given in (Burger et al., 1990):

$$\frac{\phi_b}{H} = \min\left(\frac{\xi_o}{\sqrt{\tan\alpha}}\ ;\ 2.2\right) \tag{2}$$

$$\tan\theta = \cot\beta = \frac{5.9\cdot\tan\alpha}{\xi_o} \tag{3}$$

$$\frac{d_s}{H} = \min\left(\frac{0.11\cdot\tan\alpha}{(H/L_o)^{0.8}}\ ;\ 1.5\right) \tag{4}$$

Model tests have shown that the same formulas can be applied with reasonable accuracy for oblique wave attack up to 45° as for perpendicular wave attack.

2.2 Internal loading

The internal hydraulic loading on the revetment can be split up into two items:

1) The pressure under the cover layer, relative to the pressure on top of the cover layer, causing uplift of the blocks.
2) The hydraulic gradients under the cover layer (mainly parallel along the slope), which can cause the migration of subsoil particles.

We consider two main types of structures: (1) with granular filter between the cover layer and subsoil, and (2) without granular filter.
Quite often a placed-block revetment has a configuration as shown in Figure 1. A cover layer is placed on a filter layer of limited thickness lying on the base material. Usually the permeability of the filter layer is much larger than the permeability of both the base and the cover layer. The situation of a cover layer which is less permeable than the underlying filter layer can also be present in revetment structures with a block mattress, gabions, sand bags or gravel-filled geotextile cover layers.
The hydraulic loading on this type of structure can be quantified analytically.

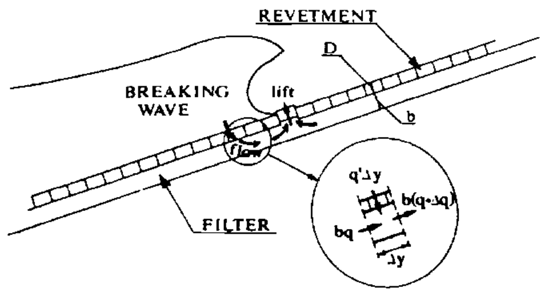
The internal loading of revetments without granular filter has to be described with more sophisticated solution methods. For example, the STEENZET/2 program of the Delft Geotechnics, a finite element program specially developed to calculate the pore pressure response in the filter layer(s) and subsoil below a placed block revetment. This program can use measured wave pressures as a boundary condition and it can handle laminar as well as turbulent flow (Hjortnæs-Pedersen et al., 1987).

In the following we will first consider revetments on a granular filter and secondly extent the derived formula to revetments without granular filter.

REVETMENTS WITH GRANULAR FILTER

During the wave rundown there is a large piezometric head gradient on top of the revetment (see Figure 3), caused by the simultaneous occurrence of rundown of the preceding wave and the arrival of the present wave. The piezometric head underneath the cover layer is a damped representation of the potential on top of the revetment, causing an uplift pressure at the location of maximum wave rundown. The extend of the damping is influenced by the permeability ratio of the cover layer and the filter layer and also by the compressibility of the air/-water-mixture in the filter. The latter is important for very fine granular filters (D_{50} < 3 mm) and will not be considered here (see for example Bezuijen et al., 1986).

The piezometric head over the cover layer during wave rundown can be quantified by considering the mass balance of the water in the filter and the Darcy flow equation (Fig. 4).



$$\Rightarrow \quad \frac{\delta^2\phi}{\delta y^2} = \frac{1}{\Lambda^2}(\phi - \phi')$$

Figure 4. Mass balance in filter

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

$$\frac{d^2\phi}{dy^2} = \frac{\phi - \phi_T}{\Lambda^2} \tag{5}$$

The leakage length Λ is defined as the length of a piece of protection, in which the flow resistance through cover layer and filter layer are the same. This parameter is a measure of the pressure head difference on the cover layer for given wave forces:

$$\Lambda = \sqrt{\frac{bDk}{k'}} \quad \text{or} \quad \frac{\Lambda}{D} = \sqrt{\frac{b}{D}\frac{k}{k'}} \tag{6}$$

A solution of equation (5) for schematized boundary conditions (Figure 3 and 4) was presented by Wolsink (see Burger et al., 1990):

$$\phi_w = \left(\frac{1}{2}\Lambda\cdot\cos\alpha\cdot\tan\theta\cdot\left(1 - \exp\left[-\frac{\phi_b}{\Lambda\cdot\cos\alpha\cdot\tan\theta}\right]\right) + \frac{1}{2}\Lambda\sin\alpha\right)\left(1 - \exp\left[\frac{-2z_1}{\Lambda\sin\alpha}\right]\right) \tag{7}$$

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

1) Maximum downward gradient:

$$i = \sin\alpha \tag{8}$$

2) Maximum upward gradient:

$$i = \cos\alpha\tan\theta\left(1 - \exp\left[\frac{-\phi_b}{2\Lambda\cos^2\alpha\tan\theta}\right]\right) - \frac{\sin\alpha}{2}\exp\left[\frac{-\phi_b}{2\Lambda\cos^2\alpha\tan\theta}\right] \tag{9}$$

Equation (7) and (9) are presented in Figure 5 and 6. It is clear that the uplift pressure over the cover layer increases as the leakage length (Λ) increases and the steepnes of the wave front (θ) increases. But the larger Λ, the smaller is the maximum upward gradient in the filter (i).

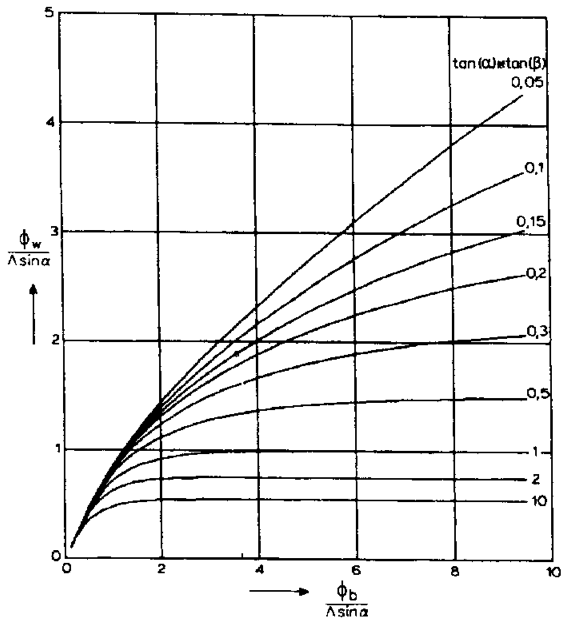


Figure 5. Uplift

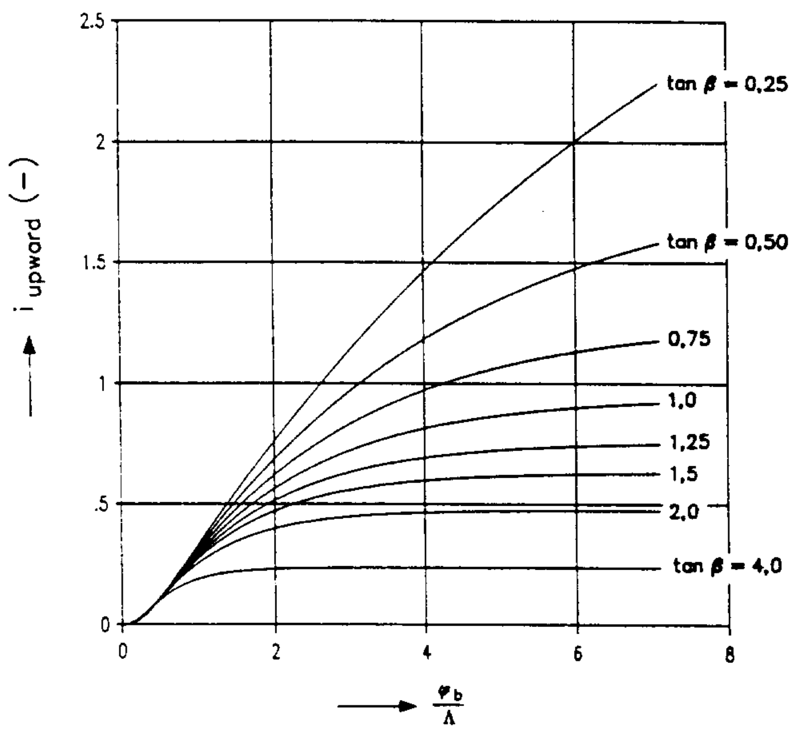


Figure 6. Maximum upward gradient

The above formulas for the loads are derived for regular wave attack. Unfortunately, little is known about the load ratio of regular and irregular waves. Experiments show that especially the large waves cause instability and that the number of waves during a storm plays a minor role. On comparing the piezometric head on the slope under regular and irregular wave attack the following is concluded for the wave height at threshold of damage:

$$\left(\frac{H}{H_s}\right)_{damage} = 1.4 \tag{10}$$

The displacement of a block occurs if the uplift pressure exceeds the weight of the block added with the additional forces, such as friction and inertia. The limit state is:

$$\phi_w = \Gamma \Delta D \cos\alpha \tag{11}$$

Introducing the coefficient for irregular wave attack, $H/H_s = 1.4$, into formula (7) yields a complicated stability formula, that can be approximated by (Klein Breteler, 1991):

$$\frac{H_{scr}}{\Delta D} = f\left(\frac{D}{b}\frac{k'}{k}\right)^{0.33} \xi_{op}^{-0.67} \tag{12}$$

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

From these equations, assuming by approximation that f is constant, it appears that:

- An increase in the volumetric mass, Δ, produces a proportional increase in the critical wave height. If ρ_s is increased from 2300 to 2600 kg/m³, H_{scr} is increased by about 23%,
- The breaker parameter, ξ_{op}, comprises the slope angle ($\tan\alpha$) and the wave steepness (H_s/L_{op}). If the slope angle is reduced from 1:3 to 1:4, H_{scr} is increased by about 20%,
- An increase of 20% in the thickness of the cover layer, D, increases H_{scr} by about 27%,
- A 30% reduction in the leakage length ,Λ, increases H_{scr} by about 20%. This can generally be achieved by halving the thickness of the filter layer or by doubling the k'/k value. The latter can be achieved by approximation, by:
  - reducing the grain size of the filter by about 50%, or
  - by doubling the number of holes in (between) the blocks, or
  - by making hole sizes 1.5 times larger,or
  - by doubling joint-width between blocks.

Changing the structural parameters changes the coefficient f slightly; the effect of these parameters can only be evaluated by approximation. It should be noted that changing the structural geometry can mean that failure mechanisms other than blocks being lifted out may govern the stability.

This stability formula contains all our theoretical understanding of the physical phenomena involved. However, the practical applicability is limited:
- it is only useful for block-revetments with a granular filter layer underneath the blocks.
- the permeability of the cover layer can be calculated with the formulas in Klein Breteler et al. (1988), but they are very complicated.
- a lower boundary for Γ has been given by Burger et al. (1990), but a good quantification is still not possible.
- the theoretical and empirical bases for the assumption $H/H_s = 1.4$ is poor.

Most of these problems can be avoided by the application of only the most essential parts of the stability formula and to complete it with empirical data from large-scale model studies. Partly based on the general trends in the results of model tests, we have selected the very essence of eq. 12:

$$\frac{H_{scr}}{\Delta D} = F \cdot \xi_{op}^{-0.67} \tag{13}$$

The value of F depends on the type of structure, characterised:
a) Low stability: $(k'/k)\cdot(D/b) < 0.05 - 0.1$
b) Normal stability: $0.5 - 1 > (k'/k)\cdot(D/b) > 0.05 - 0.1$
c) High stability: $(k'/k)\cdot(D/b) > 0.5 - 1$

The conditions for high stability are very difficult to meet and it turns out that there are no model studies performed with this type of structure. Therefore, it has been left out in the following, leaving only two types of structures. With the formulas from Klein Breteler et al. (1988) they can be defined as follows (D_{15} = grain size of filter exceeded by 85% of weight, (m)):
- low stability, if:
  * thick filter layer, namely : $b/D > 0.5$ and
  * course filter material, namely : $D_{15} > 4$ mm and
  * closed cover layer, namely:
    . solid blocks with small joints: open area $\Omega < 2\%$
    . blocks with holes with a spacing less than 0.3 m: $\Omega < 5\%$
    . blocks with holes with a spacing wider than 0.3 m: $\Omega < 10\%$
- normal stability (structures other than defined as "low stability")

EXTENSION TO OTHER TYPES OF REVETMENTS WITH MODEL TESTS

Up to now we have dealt with block revetments on a granular filter layer only. However, there are also structures with a cover layer directly placed on clay, or with a geotextile on sand. Furthermore there are block-mats and interlocking cover layers. For these structures there is no such a theory as for the blocks on a granular filter. Therefore, we can merely assume that eq. (13) also is valid for these structures. In the next chapter this assumption proves not to be contradictory to the available test results.

We can conclude that the theory has led to a simple stability formula (eq. 13) and a subdivision into 8 types of structures:

a) cover layer with loose blocks (without linkage or interlocking):
   - a1) cover layer on granular filter, low stability
   - a2) cover layer on granular filter, normal stability
   - a3) cover layer on geotextile on sand
   - a4) cover layer on clay

b) cover layer with linked blocks (for example with cables or interlocking, such as block-mattresses):
   - b1) cover layer on granular filter, low stability
   - b2) cover layer on granular filter, normal stability
   - b3) cover layer on geotextile on sand
   - b4) cover layer on clay

The theory presented in the previous chapter is fitted to the results of a large collection of results of model studies from all over the world. Only large-scale studies are used because both the waves and the wave induced flow in the filter should be well represented in the model.

In the following Data of all available tests are summarised in Figures and for each type of structure a lower and upper boundary for the value of F is given. The lower boundary gives with eq. (13) a stability curve, below which stability is guaranteed. Between the upper and lower boundary the stability is uncertain. It depends on various unpredictable influences if the structure will be stable or not. The upper boundary gives a curve above which instability is (almost) certain.

In some of the reports the exact information about cover layer or filter was lacking. In those cases the open area percentage, Ω, and/or the characteristic grain size was estimated from photos, drawings or descriptions.

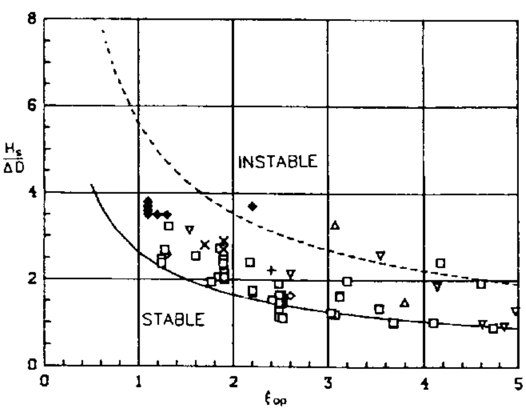


Figure 7. Test results on type a1 (loose blocks on granular filter, low stability)

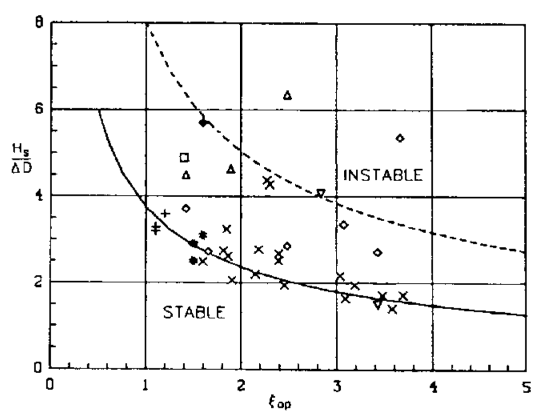


Figure 8. Test results type a2 (loose blocks on granular filter, normal stability)

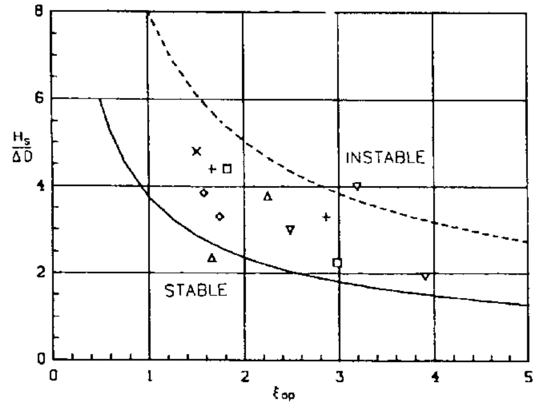


Figure 9. Test results type a3
(loose blocks on geotextile on sand)

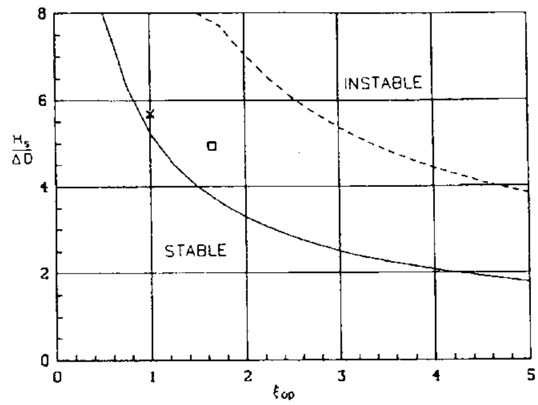


Figure 10. Test results type a4
(loose blocks on clay)

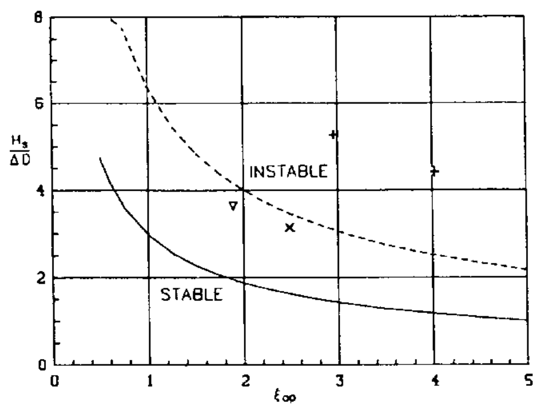


Figure 11. Test results on type b1
(linked blocks on granular filter, low stability)

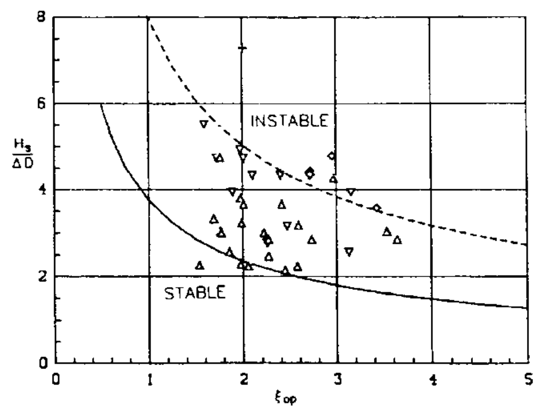


Figure 12. Test results type b2
(linked blocks on granular filter, normal stability)

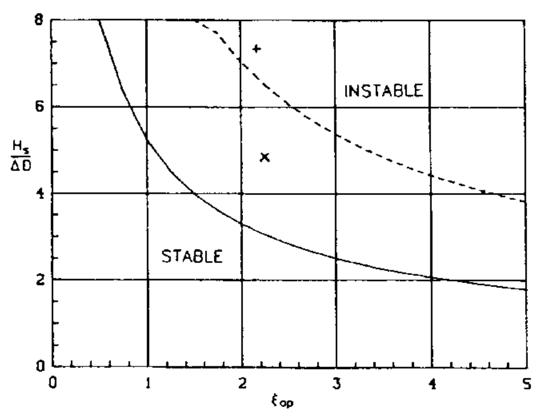


Figure 13. Test results type b3
(linked blocks on geotextile on sand)

Comment to the figures and discussion:

1° Most of the tests have been performed without a berm in the slope, except the studies reported in Oesterdam (1982) and Tekmarine (1982) and most of Tekmarine (1985).

2° The studies concerning structure type b2 and b3 were very scarce. Therefore, also the tests with a rather small scale are used.

3° There are no test data available on structures with linked blocks on clay (b4).

4° The results for structure type a3 and b3 (blocks on geotextile on sand) may only be applied if $H_s < 1 - 1.5$ m or to structures with course sand ($d_{50} >$ 0.3 mm) and gentile slope ($\tan\alpha < 0.25$), because the uplift of blocks is assumed to be the dominant damage-mechanism (instead of soil mechanical failure).

5° The results for structure type a4 can be applied on the condition that clay of high quality is used (erosion resistant). If there is no such clay present, then a geotextile is recommended to prevent erosion during (lengthy) wave loading. The stability is than equal to that of structure type a3.

The combination of the stability formula, that was derived from theory, together with the results of many large-scale model studies from all over the world has produced a reliable design tool for the preliminary design of placed block-revetments. Its reliability is only influenced by the fact that most tests have been performed with regular waves. An inaccuracy is introduced by the transformation of the regular wave load to an equivalent irregular wave load. Further studies are undertaken to improve the transformation method.

3 Migration of subsoil through filter or cover layer

The migration of the subsoil particles through the filter layer or through the cover layer leads to local erosion of the subsoil near the water level and will result in a local settlement of the filter and cover layer. This damage mechanism shows as some stones that are sunk compared to adjacent stones, or as a gradually increasing S-profile develops. Some minor settlement is hardly effecting the stability against wave action, but it must warn us that it will get worse every serious wave attack (storm). Loss of coherence of the cover layer is the final stage and then failure is at hand.

No problems will arise if the granular filter or geotextile on the subsoil is geometrically sandtight (D_x is grain size of filter and d_x that of subsoil):

- granular filters on sand: $D_{15}/d_{50} < 5$ (14)
- geotextiles on sand: $O_{90}/d_{90} < 1$ (15)
- geotextiles on clay or silt: $O_{90}/d_{90} < 1$ and $O_{90} < 100\mu m$ (16)

Unfortunately these criteria are often difficult to meet.

A more advanced requirement is based on hydrodynamic sandtightness, viz. the internal flow must not be capable of washing out the subsoil material (even though the openings of the geotextile are much larger than the subsoil grains). This arises from:

- the hydrodynamic forces on the subsoil are greatly reduced by the geotextile.
- the cohesion forces of the particles do not allow small particles to be washed away.

The hydrodynamical sandtightness criteria can be applied in the majority of structures because hydraulic loads usually are low in the vicinity of the subsoil (see Figure 15). Only in some cases, in which the geotextile or subsoil-filter interface is very close to the surface of the structure and, provided the hydraulic loads are heavy (for example breaking waves), the geotextiles or filter should be geometrically sandtight (see Figure 14).

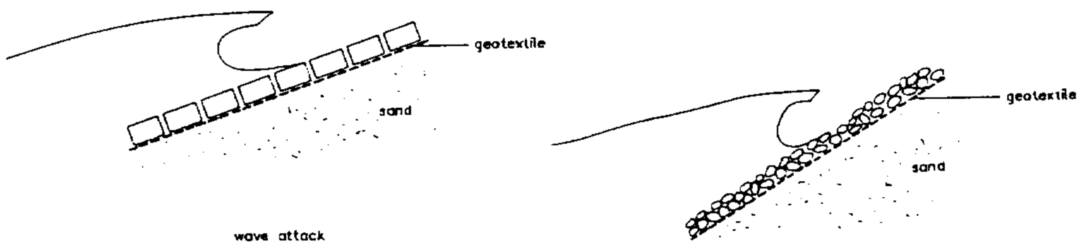


Figure 14. Examples of structures in which geometrically sandtight geotextiles are necessary

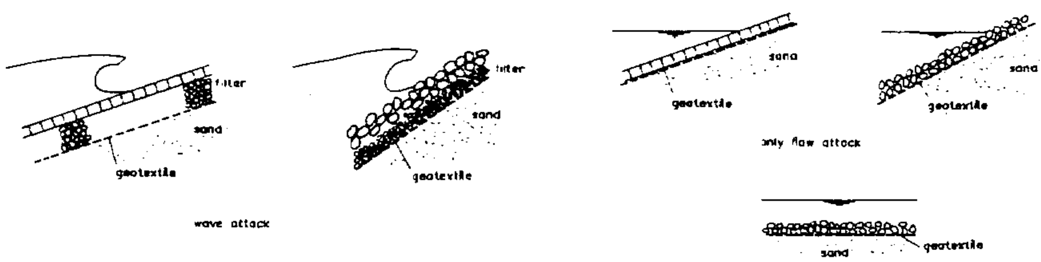


Figure 15. Examples of structures in which hydrodynamically sandtight geotextiles can be applied

The critical hydraulic gradient for granular filters on a sand subsoil can be read from Figure 16.

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

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

$$i_{cr} = \frac{0.03}{n^2 D_{15}} \tag{17}$$

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

$$i_{cr} = \frac{0.01}{n^2 D_{15}} \tag{18}$$

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

$$i_{cr} = \frac{0.001}{n^2 D_{15}} \tag{19}$$

Where n is the porosity of the filter layer (usually 0.3 < n < 0.4) and D_{15} is the grain size of the granular material on the geotextile (m).

The value of i can be calculated with formula (8) and (9).

If gradients larger than i_{cr} can be expected (structures like in Figure 14), then a geometrically sandtight geotextile or filter is recommended.

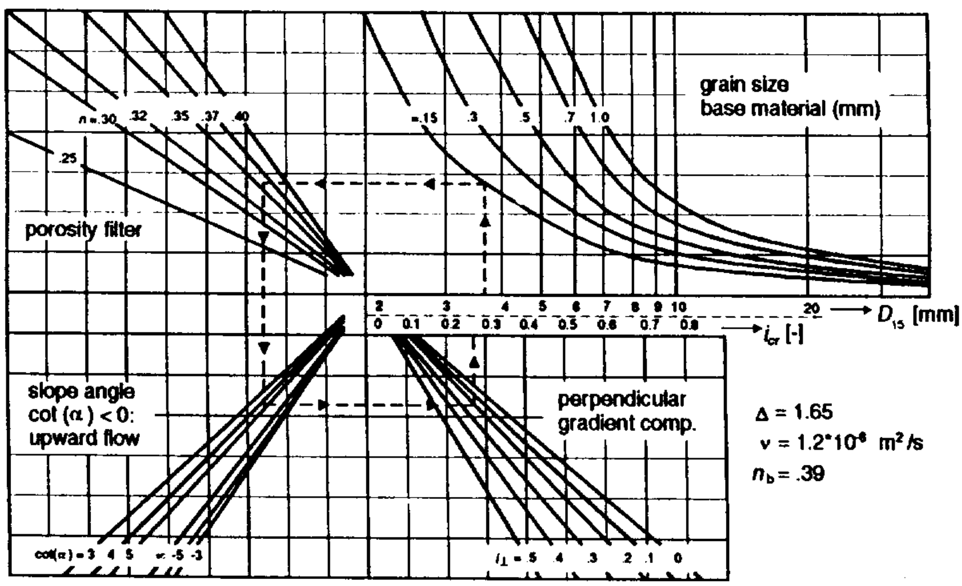


Figure 16. Diagram for critical gradient for granular filter on sand

4 Geotechnical stability

4.1 Introduction

Apart from instability of the cover layer (chapter 2) and migration of fines (chapter 3) instability of the subsoil (Kohler and Schulz 1986, Bezuijen et al. 1990) is also a failure mechanism. An example of such a failure is shown in photo in Figure 17. The cover layer (in this case a blockmattress) did not fail, but the deformation of the subsoil lead to damage of the revetment.

Figure 17/Photo 1. Block mattress being damaged by subsoil failure

This failure mechanism can be the dominant failure mechanism in case of a permeable revetment (rip-rap revetment, block mattresses with holes in the blocks) on a granular subsoil. In case of a relatively impermeable revetment like asphalt revetments, or placed blocks on a highly permeable filter layer, lifting of the cover layer will precede geotechnical instability. As a result a design method based on lifting of the blocks, as described in chapter 2, has to be used for an impermeable revetments.

Firstly a qualitative description of the wave induced pore pressures is given and the influence on the stability of the revetment is discussed. Thereafter stability formulas are derived and examples are presented.

Comparison with the results of large scale model tests have shown that there is reasonable agreement; the calculated results are on the safe side.

4.2 Pore pressures in the subsoil

The general structure of a revetment is shown schematically in figure 1.
A permeable revetment means that going from the subsoil to the cover layer the permeability of each layer increases. The failure mechanism of such a revetment is governed by the water movement caused by wave attack.

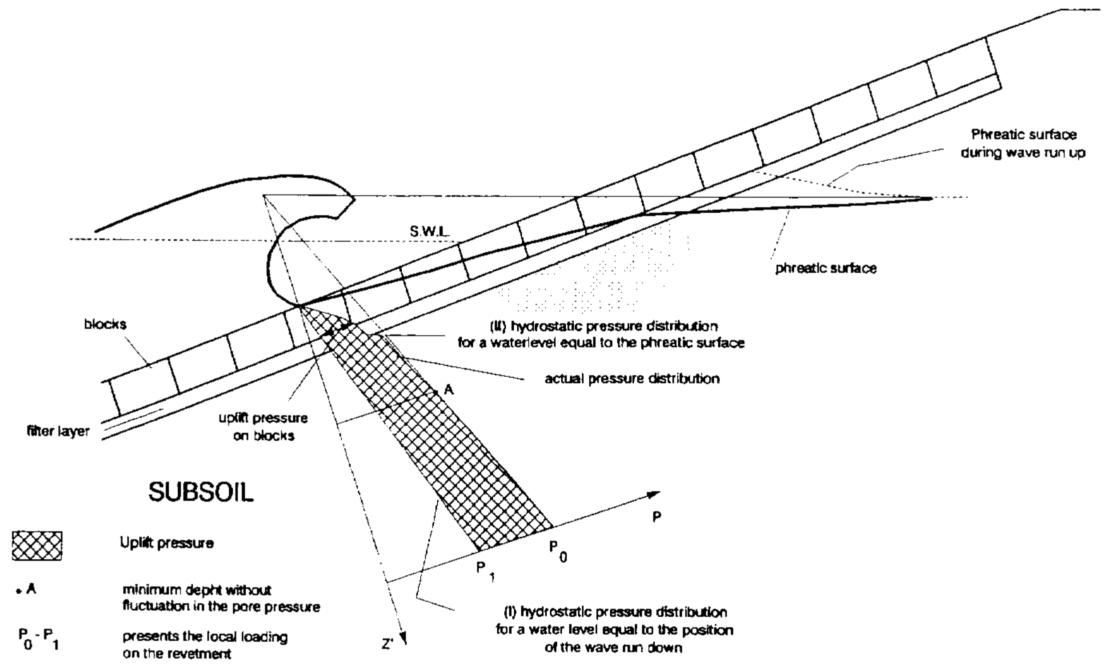


Figure 18. Pore pressures during wave run-down

The overall loading on the revetment by the wave action and the pore pressures in both the revetment layers as well as the subsoil is elucidated in figure 18. At the outside of the revetment there is a fluctuating water level caused by the wave attack. Deep down in the subsoil the phreatic surface is hardly influenced by the fluctuations in wave action. This surface is located somewhat higher than the mean water level (SWL). During wave run-down the water pressure on the revetment is reduced. This reduction in water pressure on the revetment would not induce any loading on the revetment if the pore pressures were equal to the hydrostatic pressures corresponding with the reduced water level (line I in figure 18). However, somewhere in the subsoil (e.g. at point A and deeper inside the subsoil) the fluctuations of the wave attack are damped and the pore pressure remains nearly constant. At that location the pore pressures will be equal to a hydrostatic pressure distribution corresponding to the phreatic surface (line II in figure 18). From point A upwards to the top of the revetment the real pore pressure will be somewhere in between these two lines. This leads to a pressure difference over the revetment (cover layer and sublayer) as is shown in figure 18 by the hatched area. The total loading on the revetment is the pressure difference (p_0-p_1). This pressure difference can cause instability of the revetment resulting in lifting or sliding of parts of the revetment. As can be seen in figure 18 this pressure difference is proportional to the difference between the nearly constant phreatic surface and the maximum run-down.

At the location where a revetment has to be built the design wave is known. The only way to influence the total load on the revetment is to minimize wave

run-down. This can be done by changing the slope of the revetment.
When the wave run-down is fixed, the total load on the revetment is also fixed, because this is simply the pressure difference caused by the difference in water head in- and outside. Therefore the total loading (p_0-p_1) is independent of the design of the structure. However, the distribution of the load can be influenced by the designer. A small permeability of the cover layer will lead to a large pressure difference over the cover layer and thus a considerable loading over that layer. On the other hand, for permeable revetment the loading will be concentrated on the subsoil.

In this chapter it is concentrated on the loading on and the strength of the subsoil.

4.3 Strength of subsoil

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

$$\frac{\tau}{\sigma} < \tan\Phi \tag{20}$$

Without any water movement the calculation of the normal and shear stress in a plane parallel to a slope is straight forward, leading to the well known relation that the slope angle cannot exceed the friction angle. In case of water movement in the subsoil and thus a non-hydrostatic pressure distribution, the influence of the pore pressure on the normal and shear stress has to be included in the calculation. This can lead to a failure surface that is different from the plane parallel to the slope. Therefore generally the stability has to be evaluated by a slip circle analysis or finite element calculation.

Generally the pore pressure distribution in the subsoil underneath a revetment under wave attack has to be calculated by numerical methods. For example Hjortnæs-Pedersen et al. (1987), have described a finite element method to simulate the pore pressure distribution underneath a placed block revetment. In this method it is possible to use measured wave pressures as a boundary condition. The advantage of a finite element method is that the influence of layers with different permeabilities and different geometries can be evaluated. A disadvantage is that such methods are generally rather complicated.

Bezuijen (1991) has develloped a simplified procedure for permeable revetments that leads to a minimum revetment weight per square meter to prevent subsoil instability, including the influence of the pore pressure distribution. This method will be described in the next section.

4.4 Stability calculations

In the stability calculation a slip surface parallel to the slope is assumed at a depth z in the subsoil, see figure 18. The value of z will be determined later. Two different situations have been taken into account:

1. the cover layer has a good toe structure or anchoring (in case of a block mattress) giving only normal forces to the slope. If also the filter layer (if present) is locked up in a way that this will cause no shearing forces, then the only shear stress that can exist is caused by the subsoil layer with thickness z_0,

2. there is no adequate toe structure or anchoring. In that case the revetment (cover layer and the filter layer if present) will also contribute to the shear stress at $z = z_0$.

From the calculations presented by Bezuijen (1991) it appears that the revetment thickness will be impractically high in the latter case (up to 1 m thickness for cover layer and filter layer for a 1:3 slope loaded with a 1 m significant wave height) and therefore in this paper it is concentrated on the situation with an adequate toe structure or anchoring.

The normal stress on a slip surface in the subsoil is composed of the weight of the revetment and the weight of the subsoil with a thickness z_0 above the slip surface (for both the weight under water). This stress has to be reduced by the difference in piezometric head at depth z_0 and just below the revetment (Figure 19).

In this calculation method the water flow in the subsoil is supposed to be perpendicular to the slope of the revetment. Finite element calculations has shown that this is a reasonable assumption in case of a permeable revetment placed on a relatively impermeable subsoil. Such a flow does not influence the shear stress. The revetment does not contribute to the shear stress because the component of the weight parallel to the slope is counter balanced by the toe structure.

The shear stress is difficult to determine exactly. The weight component parallel to the slope leads to a shear stress in the plane of the slip surf ace, but this component can also be partly balanced by the friction between the cover layer and the subsoil. Since the friction coefficient between the revetment and the subsoil is unknown and since it depends on the material just above the subsoil, it was decided to neglect this influence. This means that in figure 19 the shear stress along the slip surface has to balance the weight component of the subsoil parallel to the slope. With these assumptions the following relations for the normal and vertical stress at the slip surface z_0 are found:

$$\sigma = \left[\sigma_b + (1-n_s)(\rho_s - \rho)gz_0\right]\cos\alpha - \rho g\cdot\Delta\phi \qquad (21)$$

$$\tau = \left[(1-n_s)(\rho_s-\rho)gz_0\right]\sin\alpha \tag{22}$$

$$\sigma_b = \left\{(\rho_c-\rho)D + [1-n](\rho_f-\rho)b\right\} \tag{23}$$

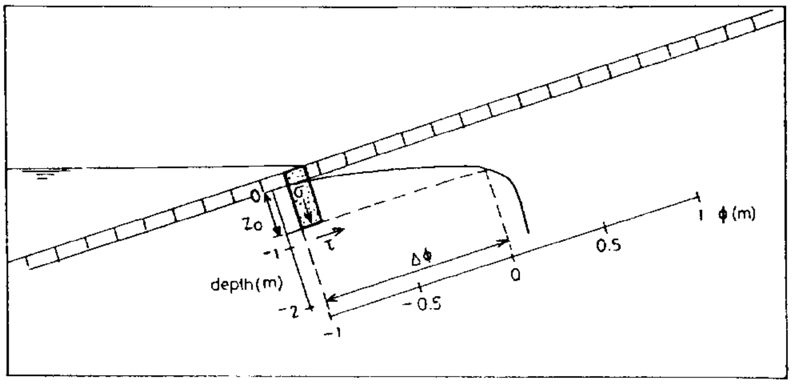


Figure 19. Definition sketch for stability calculation

4.5 Stability criterion

It is now possible with equations (20) and (21) and (22) to derive a minimum value of the weight of the revetment and filter layer (σ_b) necessary to achieve a stable revetment. With a toe protection or anchoring this is:

$$\sigma_b \geq \frac{\rho g \cdot \Delta\phi}{\cos\alpha} - (1-n_s)\cdot(\rho_s-\rho)\cdot g\cdot z_0\cdot\left(1-\frac{\tan\alpha}{\tan\Phi}\right) \tag{24}$$

To use this relation it is necessary to define $\Delta\phi$ and the critical depth (z_0) at which the slip surface occurs.

In this paper $\Delta\phi$ is assumed to be equal to the run-down value. According to CUR/CIRIA (1991) the following relation can be used for the run-down ($R_{d2\%}$) for irregular waves:

$$\frac{R_{d,2\%}}{H_s} = -0.33\cdot\xi_{op} \qquad \textit{for } \xi_{op} < 4.5$$
$$\frac{R_{d,2\%}}{H_s} = -1.50 \qquad \textit{for } \xi_{op} \geq 4.5 \tag{25}$$

z_0 can be determined with consolidation theory, see (Bezuijen, 1991):

$$z_0 = \frac{1}{2}\cdot L_{es}\cdot\sqrt{(\pi)} \tag{26}$$

The permeability of the subsoil is most accurately determined from permeability tests. A first approximation can be obtained, based on the grainsize and porosity of the soil (den Adel, 1989):

$$k = \frac{g}{160 \cdot \nu} \cdot \frac{n_s^3 \cdot d_{15}^2}{(1 - n_s)^2} \tag{27}$$

As relation for w' can be used (Verruijt, 1969):

$$w' = w + \frac{s}{p_a} \tag{28}$$

Equations (24), (25) and (26) are used to prepare the plots shown in figure 20, and 21. The following parameters are used in the calculations for these plots:

ρ_s = 2650 kg/m^3		ρ = 1000 kg/m^3
n_s = 0.45		Φ = 35°
s = 10%		

The air content in the water, s, is seldom known. From simulations of large scale experiments values between 5 and 10% were found. Lindenberg (1986), has measured this value in fine densified sand (135 μm) and found a value of 6% and in very loose sand 12%. In a normal situation with some compaction 10% is assumed to be a safe value (the larger the air content, the lower the stability)

Figure 20 shows the calculated weight as a function of the wave height for different slope angles, and grain sizes. The difference in the figures is the wave steepness, which has a significant influence on the result. From the results it is clear that for steep slopes with a subsoil of small grains geotechnical instability becomes a dangerous failure mechanism.

The results of this calculation method have been compared with the results of large scale model tests. Measurements on the pore pressure distribution in the sand has shown that this is comparable to the assumed pore pressure distribution for a block revetment placed on sand (Bezuijen, 1991). Since the method neglects friction and clamping forces between the blocks, the resulting block thickness appeared to be conservative.

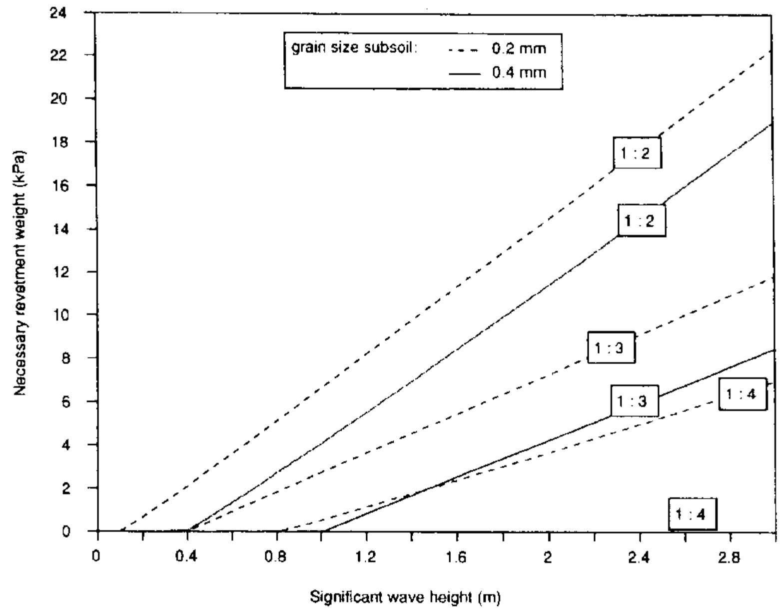


Figure 20. Result of stability calculation. Weight necessary to prevent instability of the subsoil for different slopes and grain sizes of the subsoil as a function of the wave height (wave steepness 5%)

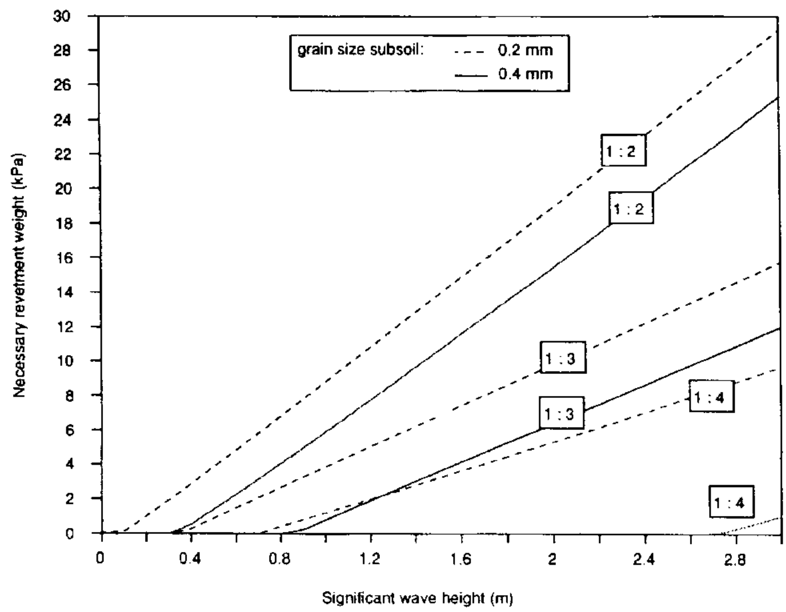


Figure 21. Result of stability calculation. Weight necessary to prevent instability of the subsoil for different slopes and grain sizes of the subsoil as a function of the wave height (wave steepness 3%)

4.6 Sliding of a revetment

Apart from lifting of parts of the cover layer and geotechnical instability, also sliding of the cover layer can be a dangerous failure mechanism, especially on steep slopes and for cover layers which give only little horizontal support (for example, rip-rap revetments or revetments with bags), and for cover layers with a poor toe structure or anchoring. As for lifting of the blocks, sliding is most likely in case of an impermeable cover layer placed on a permeable filter layer. A situation which lead to a flow in the filter layer as is shown in figure 4.

For the revetments with little horizontal support only the stability of a single element from the cover layer against sliding has to be evaluated. For revetments with horizontal support the sliding forces and stabilizing forces of a larger part of the cover layer have to be evaluated. This section presents a simple procedure for the stability of a part of a block revetment against sliding.

The loading on the revetment by wave attack can cause sliding of the cover layer. Assume a revetment without any toe protection or anchoring of the blocks. Stability of such a revetment has to be guaranteed by the friction force between the revetment and the subsoil. The maximum friction force is assumed to be related to the normal force between the soil and the subsoil. A linear relation without cohesion is assumed:

$$F_f = F_n \tan\Phi' \qquad (29)$$

Without wave attack F_n is determined by the underwater weight of a block multiplied by the cosine of the slope angle. The driving force for sliding is the sine component of the block weight. The driving force will be smaller than the maximum friction force; otherwise it would be impossible to build the revetment without toe structure or anchoring.

During wave attack the driving force remains the same. The normal stress changes due to the uplift pressures. The situation for a placed block revetment is shown in Figure 22. The distribution of the difference in piezometric head over the revetment is sketched during wave run-down. At the upper end of the revetment the difference is negative, meaning that there are no hydrostatic uplift pressures. The weight of the block is still nearly the weight above water and the normal stress between blocks and subsoil is hardly influenced by the wave attack. Lower on the revetment there is a positive difference in piezometric head which reduces the grain stress and can lead to sliding of the revetment. Below the wave front there is again a negative difference in piezometric head. The blocks are pushed on the subsoil and have an apparent weight larger than the weight below the water line, but the driving force is reduced (compared with the situation without wave attack), because the blocks are situated below the water line. These blocks will even have more stability against sliding than in the situation without wave attack.

Suppose sliding of the blocks just before the wave front occurs. Then this will not influence the blocks on the upper part of the revetment unless there is some connection by cables or interlocking, but it will influence the lower blocks.

Sliding will only occur when the total driving force of all the blocks, below the highest block for which the driving force exceeds the friction force, is higher than the total maximum possible friction force.

In the case of a block mattress connected with cables the situation will be the other way around. When there is a large joint between the blocks, the blocks in the most heavily loaded area cannot be stabilized by the blocks lower on the revetment and should be stabilized by the cables between the blocks. The cable forces are transmitted to the blocks higher on the revetment. This means that for the lower part, or the upper part of the revetment (depending on the type of revetment, placed blocks or block mattresses) the maximum possible friction force has to be summed and compared with the maximum driving force. If the driving force is higher than the maximum possible friction force then stability has to be guaranteed by a sufficiently strong toe structure or anchorage of a block mattress.

The method described here can be used qualitatively with different methods to calculate the uplift pressure. Bakker & Meijers (1988) have elaborated this method for the calculation method described in section 2.2. They suggested a value for the friction angle: $\Phi' = 2/3\ \Phi_f$, with Φ_f the friction angle of the filter material. Further they concluded on the basis of their results that toe structures and/or anchorages will often be indispensable.

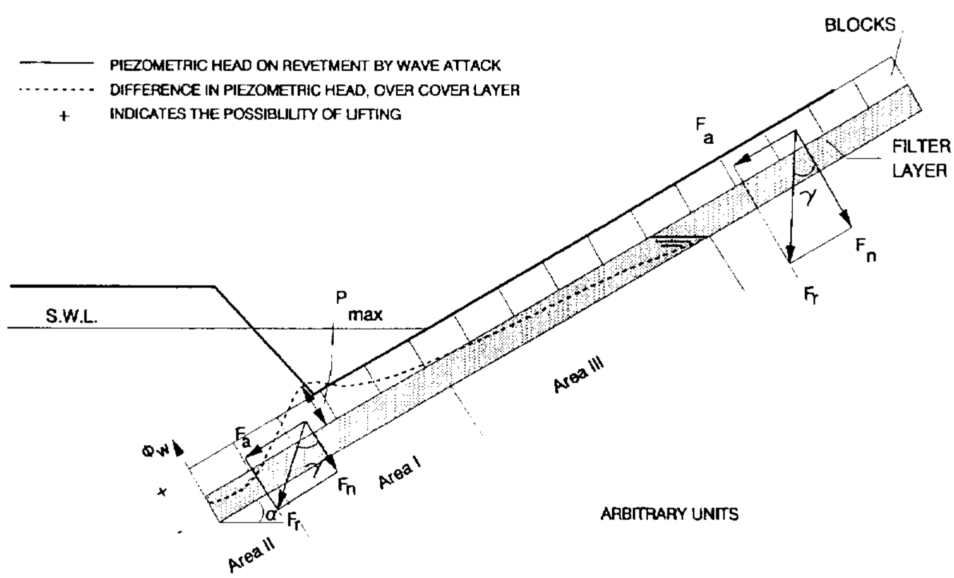


Figure 22. Possible distribution of piezometric head just before a breaking wave and its influence on the direction of the resultant force of block on the subsoil

Bezuijen et al. (1990) presented a simple method to get a first indication how large the forces on toe structures can be. In this method it was assumed that over the height equal to one wave height there is no friction between the cover layer and the subsoil, see Figure 22. This is be a pessimistic assumption for a revetment with a short leakage length. On the other parts of the revetment the maximum possible friction force is comparable with the situation without wave attack. Additional strength due to a negative pressure gradient below the water line is neglected. Above the area of wave attack it is assumed that the influence of the water can be neglected.

Each block in the area of wave attack (area I in Figure 22) contributed to the total driving force for sliding:

$$F_a = L \cdot B \cdot D \cdot \rho_c g \,\sin\alpha \tag{30}$$

A block in the area without wave attack will be stable, even if it is loaded to some extent with the driving force from the area with wave attack. With F_a the driving force for sliding and F_f the friction force, the maximum possible loading from area I and III (F_t) on the area without wave attack (area II in Figure 22) before sliding occurs can be written as :

$$F_t = F_f - F_a = L_{II} B D \cdot (\rho_c - \rho)\, g \left[\cos\alpha \tan\Phi' - \sin\alpha\right] \tag{31}$$

and above the water line (area III in Figure 22) :

$$F_t = F_f - F_a = L_{III} B D \cdot \rho_c\, g \left[\cos\alpha \tan\Phi' - \sin\alpha\right] \tag{32}$$

With these relations the force on a toe structure or anchorage can be calculated as will be done in the following example.

A placed block revetment on a slope 1:3 is loaded with a wave height of 1 m. The toe structure is placed 1.5 m below the area of wave attack. The revetment consists of blocks with a dimension of 0.5 x 0.5 x 0.25 m. The friction angle between the blocks and the filter layer is 25°.

In the loaded area each block contributes with a loading force of 445 N (using a value of 2300 kg/m^3 for the density of concrete). The stabilizing area is below the water line, therefore Equation 31 has to be used leading to a stabilizing force of 100 N for each block. There are approximately 6 blocks in the loaded area and 9 blocks in the area without wave attack. The total force that will load the toe structure is therefore 1770 N for each row of blocks. Since the length of the blocks is 0.5 m this result leads to a loading of the toe structure of 3540 N/m.

5 Design considerations

The review of the key elements that must be considered in the design (dimensioning) of revetment structures is illustrated in Figure 23. More detailed design methods for these structures are discussed in (PIANC 1992 and CUR/TAW 1992/1994).

The most critical structural design elements are :

(1) the stability of the cover layer,
(2) the security of the foundation,
(3) the minimization of settlement and sliding
(4) the toe protection to prevent undermining.

All of these are potential causes of failure of coastal structures.

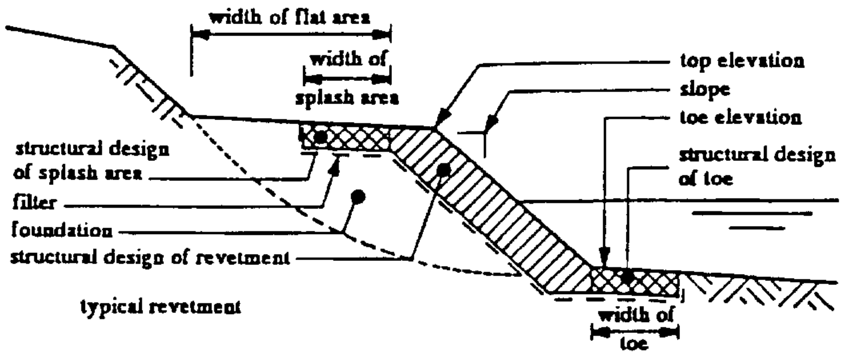


Figure 23. Design components of typical revetment structure

The usual steps needed to develop an adequate structure design are :

1. General aspects
   - 1.1 Formulate purpose of the project
   - 1.2 Formulate functional requirements
   - 1.3 Formulate project restraints (materials, labour, equipment, financing, time, risk, etc)
   - 1.4 Formulate global description of boundary conditions.

2. Alternative general solutions
   (items 2 and 3 are reversible)

3. Boundary conditions
   - 3.1 Determine hydraulic boundary conditions and loads (water levels, wave climate, currents, morphology, beach variations, etc)
   - 3.2 Determine geotechnical boundary conditions (soil types and relevant parameters)
   - 3.3 Determine other relevant conditions and loads (ice, earthquake, vegetation, etc).

4. Feasibility studies for generated alternatives
   4.1 Determine suitable structure configurations (geometry), crest, toe and/or berm elevation related to the allowed run-up and overtopping
   4.2 Review the possible failure mechanisms
   4.3 Select a suitable armour alternative and size of armour units
   4.4 Make a preliminary feasibility analysis of alternatives
      - develop cost estimate for each alternative
      - evaluate possible construction requirements and limitations for the alternative solutions
   4.5 Select the final solution.

5. Final design
   5.1 Consider the use of models (improving boundary conditions)
   5.2 Consider the probabilistic approach
   5.3 Make final estimation and evaluation of the structure geometry (slope, berm, crest, toe) and check the allowable figures on run-up and/or overtopping
   5.4 Design the final dimensions of the revetment ; cover layer, filter and/or underlayers
   5.5 Make prediction of scour and design toe protection
   5.6 Design transitions and crest protection (splash area)
   5.7 Design other structure-related elements
      - drainage features (if required)
      - surface run-off and overtopping run-off facilities
      - flanking of the structure
   5.8 Check for possible failure mechanisms of the final design and corresponding damage and risk level
   5.9 Go through the overall project and checklist and finally decide whether alternative geometries should be considered
   5.10 Prepare specifications for materials, equipment, execution and cost, including quality control and motivation of choices.

The primary function of any slope protection, flexible or not, is to protect the edge of land and water against hydraulic loads by waves, tides and currents. The determination of the hydraulic design conditions is the result of a quantification of the local conditions in combination with a certain level of safety. In this way the design conditions are defined and presented in the form of a water level, a wave height and a wave period, usually completed with some expectation for the form of the energy density spectrum and sometimes even completed with an estimate for the duration of the selected design condition.

However, the fact that the design conditions are fixed does not at all mean that the loads on the slope revetment structure are also fixed. Within certain limits, of course, it is possible for the designer to influence and consequently to choose the size, the sort and the place of attack of the hydraulic loads, by a proper selection of the geometry, lay-out and materials for the structure. In previous sections the mathematical formulas for the calculation of external and internal hydraulic loads were presented. The parameters in the formulas can be manipulated by the designer to control the performance and effectiveness of structures.

With reference to the design of block revetments as one of various other alternatives and in view of design optimization the following design considerations can be used during the design process:

(1) Steeper or milder slope gradient.
- Steeper slope makes the protective length (revetment) shorter ; as a first approximation, the slope length (L) is related to the height of slope to be protected (h) by $L = h/\sin\alpha$.
- For breaking waves ($\xi_{op} < 2.5$) the run-up ($R_{u2\%}$) on the steeper slope will increase proportionally to $\tan\alpha$, namely: $R_{u2\%} \approx 8H_s\tan\alpha$; this yields a higher crest position and eventually, a larger volume of the dike.
- The run-down on the steeper slopes also increases, possibly leading to higher overpressures and thus, thicker protective elements.
- For steeper slopes of loosely placed blocks, the friction between the blocks increases with $\sin\alpha$. However, it is difficult to quantify the consequences of this effect exactly.
- For steeper slopes the internal gradients increase, leading to more severe requirements concerning the sublayers.
- A steeper slope imposes more severe requirements for the support by a toe-protection.
- The damage progress after an initial damage is more rapid for the steep slopes, thus providing more dangers of scouring.
- Steep slopes are more easily damaged by ice, especially when using slopes steeper than 1 on 3, the above considerations should be taken into account for a proper design.
- For steeper slopes the risk of geotechnical instability increases.

(2) Berm or no berm.
- Application of a berm reduces the run-up, making possible a lower crest elevation.
- A berm can serve as a maintenance road.
- A berm creates a discontinuity in a protection (weak point).
- A berm reduces the phreatic level in a dike with a positive effect in case of low permeable or impermeable revetments.
- A berm reduces ice ride-up.

(3) High or low permeability of the cover layer.
- High permeability, in combination with a proper sublayer, reduces the uplift pressure and leads to thinner units. It is however important that the permeability does not decrease during the life time (aging).

- When the high permeability is created by large openings in or between blocks, washing out of the sublayers can take place ; to avoid this the following measures can be taken:
  (a) coarser filter; however this sometimes leads to increase of the hydraulic gradients across the cover layer and thus to thicker units,
  (b) geotextile underneath the cover layer elements. Attention should be paid to a sufficiently low hydraulic resistance normal to the slope, which should not increase the uplift pressure again.
  (c) another solution can be the use of bounded filters (sand-bitumen, sand-cement, etc).
    * To reduce these disadvantages the permeability should be distributed over the units instead of being concentrated (e.g. in one big hole).
- High permeability of the cover layer may increase the hydraulic gradients at the sublayer-subsoil interface or in the subsoil; proper care should be exercised in adequate sublayer design.
- High permeability of cover layer reduces the run-up somewhat.
- In the case of a very high permeability of block revetments created by large holes the drag forces along the slope may increase considerably, leading to large forces on the units and thus larger dimensions.

(4) Rough or smooth surface.
  - A rough surface (can also be obtained by using blocks of various height) reduces the run-up and thus it reduces the crest elevation and eventually the volume of a dike. This effect is evident mainly when the whole run-up zone is equipped with roughness elements. When the upper slope is protected by a grass-mat the application of the roughness elements on the lower part of a slope will have a limited effect.
  - High roughness elements introduce high drag-forces which should be incorporated in the stability calculations.
  - Rough surface is unfavourable under ice conditions.

(5) High or low permeability of sublayers (filter).
  - Decreasing of sublayer-permeability reduces the uplift forces on the cover layer. In the case of a cover layer of low permeability this may lead to reduction of the thickness of the cover layer. However, it should be checked whether this lower permeability of the sublayer and the corresponding reduction of weight is acceptable with respect to the stability of the sublayers.
  - For non-cohesive (granular) materials a decrease of the permeability can be obtained by :
    * finer granular material (however, washing out through the cover layer should be avoided and the geotechnical (in-) stability should be checked),
    * wide-graded material (the internal stability should be examined).

- Applying clay as a cohesive sublayer needs formulation of proper specifications on clay properties to avoid erosion, piping or shrinkage. However, it should be checked whether an impermeable sublayer might cause other problems, e.g. malfunctioning of the toe.
- Lower permeability of sublayer/filter increases the hydraulic gradients at the interface with the subsoil or inside it. This can be coped with by increasing the thickness of the sublayer/filter or by applying a geotextile on top of the subsoil. Besides, the geotechnical stability should be evaluated.

(6) Shape of sublayer/filter-material.
- Rounded material is often cheaper than broken material ; however, in the case of insufficiently compacted grains, a slightly lower angle of internal friction may lead to geotechnical instability, more settlement, and forces on the toe-structure.

(7) Thick or thin sublayer/filter.
- In the case of block revetments of low permeability, reduction of the thickness of sublayer/filter leads to reduction of the up-lift forces but simultaneously it leads to increase of the hydraulic gradients along the interface with the subsoil or inside it.

(8) Shape of blocks.

Rectangular blocks	**Columns of irregular shape**
good alignment/joining	mostly nicer appearance
low permeability	higher permeability
easy mechanical placing	less easy mechanical placing
problems in bends	easier with bends
difficult to repair	easier repair
washing in grouting quite difficult	washing in/grouting possible
often cheaper	often more expensive
more rapid progress of damage	slower progress of damage

Note: In the case of blocks, the self-healing tendency as for riprap is absent; therefore the stability of blocks should be guaranteed under all design conditions.

(9) Concrete (or other artificial material) or natural stone.
- Natural stone, if available in respect to the required quality and quantity can often be a favourite solution.
- Concrete blocks (or asphaltic revetments) can often be a good alternative (especially when the natural stone is not locally available) because of:
  * often lower cost
  * good/constant quality

* uniform size
* mechanical execution
* more choice regarding composition, size, etc.

Note: The economical optimization including the availability of materials, equipment and skills is mostly decisive for the choice.

(10) Effect of ageing and/or wearing/fatigue.

During the lifetime of revetment structures their original specifications can change due to climatological effects (wind, rain, frost, abrasion, sedimentation due to waves, marine growth , etc). As far as possible the course of time should be taken into account in the design process.

However, it is not easy to quantify these effects. Some qualitative description is given below:

- Ageing of the cover layer
  * Due to the wave attack at various water levels the permeability and the interlocking may change with time. For small interspaces between the blocks the permeability can decrease due to siltation of sediment while the friction between the blocks may increase.
  * Vegetation in the interspaces may also increase the friction/interlocking; however, it is possible that in the case of a heavy wave attack, the silted and/or vegetated interspaces will be cleaned up again, thus providing no additional strength at the moment of design loading on the protective units.
- Ageing of the sublayers
  * In the case of alternative materials used as sublayers (minestone, slags, silex, etc) special attention should be paid to the changes of the physical properties of these materials under influence of air, wave shocks, varying humidity, frost, etc.
  * In the case of geotextiles special attention should be paid to the possibility of clogging and/or blocking (leading to drastic change of permeabilities and, thus, increase of uplift pressures).
  * The siltation of the sublayers/filter has in general a positive effect; due to the decrease of permeability the up-lift forces decrease.

(11) Residual strength of revetments.

Revetments should be designed in such a way that the chance of failure is acceptably low. The quantification of a risk is related to the type of revetment, especially regarding the progress of damage, for example:

- a very rough surface is more sensitive to damage than a smooth surface ;
- application of a strong geotextile retards the extension of damage to the subsoil;
- cohesive-(clay) or bounded-sublayers are primary measures to increase the secondary strength of revetment-structures if the permeability of those materials is not a disadvantage for the total stability, and if the cohesive material is of sufficient strength.

(12) Cost optimization

- The total costs of a revetment are related to :
  * capital costs (execution)
  * yearly maintenance
  * large/periodic maintenance
  * repair of damage
  * demolition (after a life-time)

  A total capitalization of the cost gives mostly the optimal result.
- In general, revetments with lower capital costs will be damaged more frequently and will need more maintenance. Local subsidy regulations may influence the choice. However, in case of sea-defences, especially along low shores, higher capital costs (stronger protection) should be preferred. In the case of land reclamation or bank protection the results of the capitalization of the costs can be applied directly.

6 Conclusions

1. The combination of the stability formula, that was derived from theory, together with the results of many large-scale model studies from all over the world has produced a reliable design tool for the preliminary design of placed block-revetments. Its reliability is only influenced by the fact that most tests have been performed with regular waves. An inaccuracy is introduced by the transformation of the regular wave load to an equivalent irregular wave load. Further studies are undertaken to improve the transformation method

2. Elaborating the geotechnical stability of the subsoil of granular material underneath a revetment under wave attack, the following conclusions can be made.
   * The calculation method developed in this paper can be used to investigate whether geotechnical instability is a dangerous failure mechanism.
   * If geotechnical instability according to this method appears to be critical it can be useful to apply more sophisticated (numerical) methods to analyse the stability.
   * A revetment needs to have a certain weight to prevent geotechnical instability of the subsoil. A very permeable cover layer (with respect to the subsoil) can prevent lifting of parts of the cover layer, but then geotechnical instability becomes a critical failure mechanism. This phenomenon should be considered when in a design a granular filter (which always has a certain weight) is replaced by a "weightless" geotextile.
   * In a revetment with a steep slope (I : 3 or more) it is absolutely necessary to have an adequate toe structure or anchoring of the blocks (the latter in case of a block mattress).
   * Geotechnical stability is most critical in case of a fine graded subsoil with a large air content in the pore water. Comparison of the calculated results

with the results of large scale model tests showed that the outcome of the method is on the safe side.

3. A good tuning of the permeabilities of the cover layer and sublayers (including geotextile) is an essential condition for balanced design. Geotextiles are only one of the components of structural design and must be considered in conjunction with, or as an alternative to granular filters/other options. The replacement of a granular filter by a geotextile means a much smaller weight on the subsoil and consequently a smaller stability against geotechnical failure.

4. Further prototype verification of developed dimensioning criteria is still needed. Careful evaluation of prototype failure-cases may provide useful information/data for verification purposes.

5. In all cases, experience and sound engineering judgement play an important role in applying these design rules, or else mathematical or physical testing can provide an optimum solution.

List of symbols

B = width of the block (m),
b = thickness of filter layer (m)
c_v = consolidation coefficient = $k/(\rho g.n_s w')$ (m^2/s)
d_x = grain size of subsoil corresponding to x% by weight of finer particles (m)
D = thickness of the cover layer [m]
D_x = grain size of filter corresponding to x% by weight of finer particles (m)
f = coefficient, dependent on Δ, $\tan\alpha$, friction, etc. (-)
F_a = the driving force for sliding (N),
F_f = the maximum friction force (N),
F_n = the normal force between cover layer and subsoil (N),
g = acceleration due to gravity (m/s^2),
H = incoming wave height of regular waves (m)
H_s = incoming significant wave height of irregular waves (m)
H_{scr} = significant wave height at which blocks are lifted (m)
i = maximum gradient in the filter (-)
k = permeability subsoil or filter (m/s)
k' = permeability of the cover layer [m/s]
L = length of the block (m),
L_{II} = the length of area II (m)
L_{III} = the length of area III (m)
L_{es} = consolidation length = $\sqrt{(T.c_v)}$ (m)
L_o = wave length of regular waves at deep water = $gT^2/2\pi$ (m)
L_{op} = wave length of irregular waves at deep water = $gT_p^2/2\pi$ (m)

n_f = the porosity of the filter layer (-)
n_s = the porosity of the subsoil (-)
O_{90} = average diameter of the standardized sand fraction, of which 90% remains on the geotextile after a sieve test under defined conditions (m)
p_a = atmospheric pressure (1.10^5 N/m^2)
$R_{d2\%}$ = wave run-down level (absolute value is exceeded by 2% of incoming waves) (m)
$R_{u2\%}$ = wave run-up level of irregular waves, exceeded by 2% of incoming waves (m)
s = air content (normally between 1 and 10%)
T_p = wave period at peek of spectrum (s)
T = average wave period of regular waves (s)
w = the compressibility of pure water (5.10^{-9} m^2/N)
w' = compressibility of the pore water with air (m^2/N)
y = length coordinate along the slope (m)
z_1 = phreatic level in filter relative to the point where the wave front meets the revetment (m) (usually $z_1 \approx \phi_b$).
z_0 = depth (m)

α = the slope angle (°).
β = angle between potential front of breaking wave and the verticle (°)
Δ = $(\rho_c - \rho)/\rho$ = relative density of blocks (-)
$\Delta\phi$ = the difference in piezometric head between the location at $z = z_o$ and just above the subsoil (m)
Γ = coefficient representing friction, inertia etc. (-)
τ = the shear stress on a plane in the subsoil (kN/m^2)
Λ = leakage length (m)
ν = kinematic viscosity ($1.2\ 10^{-6}$ m^2/s)
ξ_o = breaker parameter for regular waves = $\tan\alpha/\surd(H/L_o)$ (-)
ξ_{op} = breaker parameter for irregular waves = $\tan\alpha/\surd(H/L_{op})$ (-)
ρ = density of water (kg/m^3),
ρ_c = density of concrete (kg/m^3),
ρ_f = density of filter grains (kg/m^3),
ρ_s = the density of the grains of the subsoil (kg/m^3)
σ_b = the normal stress by the filter layer and/or cover layer (N/m^2)
σ = the normal stress on the same plane (kN/m^2)
θ = steepness angle of potential front of breaking wave (°)
ϕ = piezometric head in the filter layer (m)
ϕ_T = piezometric head on the cover layer (m)
ϕ_w = maximum piezometric head over cover layer (m)
Φ = the friction angle of the material (°)
Φ' = the friction angle between the blocks and the filter layer (-).
Ω = Relative open area of the revetment (joints between the blocks and gaps in the blocks) (-)

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