DESIGN OF ALTERNATIVE REVETMENTS

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Abstract

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 (CUR/TAW 1995) and stability of rock under wave and current load (CUR/CIRIA 1991).
Until recently, no or unsatisfactory design tools were available for a number of other (open) types of revetment and for other stability aspects. This is why the design methodology for placed block revetments has recently been extended in applicability by means of a number of desk-studies for other (open) revetments:

- 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 paper 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 (Pilarczyk et al 1998).

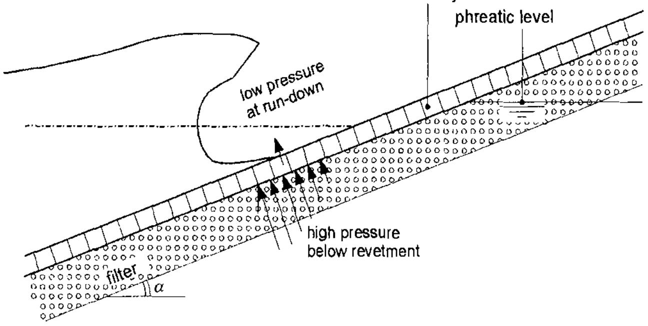

Figure 1, Pressure development in a revetment structure

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1 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 sliding.
- During maximum wave run-down there will be an incoming wave that a moment later will cause a wave impact. Just before impact there is a 'wall' of water giving a high pressure under the point of maximum run-down. Above the run-down point the surface of the revetment is almost dry and therefore there is a low pressure on the structure. The high pressure front will lead to an upward flow in the filter or a gabion. This flow will meet the downward flow in the run-down region. The result is an outward flow and uplift pressure near the point of maximum wave run-down (Figure 1).

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

$$\frac{\partial^2 \phi}{\partial y^2} + \frac{\partial^2 \phi}{\partial z^2} = 0 \tag{1}$$

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

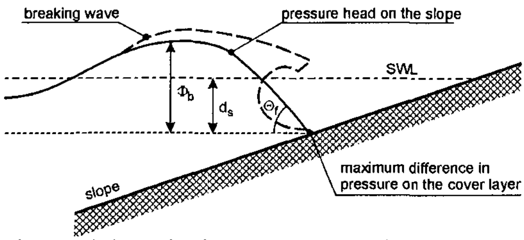

Figure 2 Schematization of pressure head on a slope

After complicated calculations the uplift pressure in the filter or a gabions can be derived. The uplift pressure is dependent on the steepness and height of the pressure front on the cover layer (which is dependent on the wave height, period and slope angle, see Figure 2), the thickness of the cover layer and the level of the 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 (Pilarczyk et al 1998):

$$\frac{H_{scr}}{\Delta D} = f\left(\frac{D}{\Lambda \xi_{op}}\right)^{0.67} \quad \text{with} \quad \Lambda = \sqrt{\frac{bDk}{k'}} \tag{2a}$$

or

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

or $$\frac{H_{scr}}{\Delta D} = F\,\xi_{op}^{-0.67} \tag{2c}$$

where H_{scr} = significant wave height at which blocks will be lifted out [m]; ξ_{op} = $\tan\alpha/\sqrt{(H_s/(1.56T_p^2))}$ = breaker parameter [-]; T_p = wave period at the peak of the spectrum [s]; Λ = leakage length [m], $\Delta = (\rho_s - \rho)/\rho$ = relative volumetric mass of cover layer [-]; 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, $\tan\alpha$ and friction [-]; F = total (black-box) stability factor [-].
The leakage length (Λ) and stability coefficient are explained in detail in the next section.

2 STRUCTURAL RESPONSE

2.1 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:

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

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: $$L_{op} = \frac{g}{2\pi} T_p^2 \tag{7}$$

with: 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 (pitched blocks). 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": $\Lambda = \sqrt{(bDk/k')}$. The final result of the analytical model may, for that matter, again be presented as a relation such as Eq. 3 where $\mathbf{F} = f(\Lambda)$.
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.

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 summarising review. The basic formulas of the analytical model are not repeated here. For these, reader is referred to (CUR/TAW 1995).

2.2 Flow-load stability

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

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

When the flow velocity is known, or can be calculated reasonably accurately, Pilarczyk's relation (Pilarczyk et al 1998) is applicable:

$$\Delta D = 0.035 \frac{\Phi}{\Psi} \frac{K_T K_h}{K_s} \frac{u_{cr}^2}{2g} \tag{8}$$

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 (-).
These parameters are explained below.

Stability parameter Φ :
The stability parameter Φ depends on the application. Some guide values are:

Revetment type	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

Shields parameter Ψ :
With the critical Shields parameter Ψ the type of material can be taken into account:

- riprap, small bags $\Psi \approx 0.035$
- placed blocks, geobags $\Psi \approx 0.05$
- blockmats $\Psi \approx 0.07$
- gabions $\Psi \approx 0.07$ (to 0.10)
- geomattresses $\Psi \approx 0.07$

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

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

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

fully developed velocity profile: $$K_h = \frac{2}{\left(\log\left(\frac{12h}{k_s}\right)\right)^2} \tag{9a}$$

non-developed profile: $$K_h = \left(\frac{h}{k_s}\right)^{-0.2} \tag{9b}$$

very rough flow ($h/k_s < 5$): $$K_h = 1.0 \tag{9c}$$

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 used 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 height of the rib for articulating mats.

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

$$K_s = \sqrt{1-\left(\frac{\sin\alpha}{\sin\theta}\right)^2} = \cos\alpha\sqrt{1-\left(\frac{\tan\alpha}{\tan\theta}\right)^2} \tag{10a}$$

or

$$K_s = \cos\alpha_b \tag{10b}$$

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 = cosα). 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, because the flow is very irregular. In such case formulas based on the discharge are developed (Pilarczyk et al 1998).

2.3 Soil-mechanical stability

The water motion on a revetment structure can also affect the subsoil, especially when this consists of sand.
Geotechnical stability is dependent on 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). Wave pressures on the top layer are passed on delayed and damped to the subsoil under the revetment structure 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. It can lead to sliding or slip circle failure, see Figure 3.

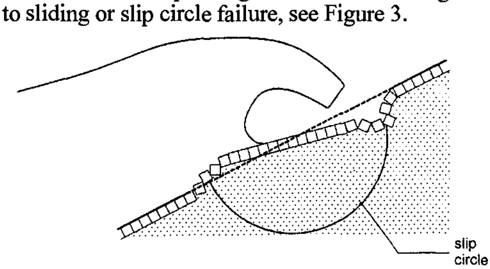

Figure 3 Schematised development of S-profile and possible local sliding in the base (sand)

The design method with regard to geotechnical instability is presented in the form of design diagrams. An example is given in Figure 4 (more diagrams and details: see Pilarczyk et al, 1998). The maximum wave height is a function of the sum of the cover layer weight (ΔD) and filter thickness (b_f).

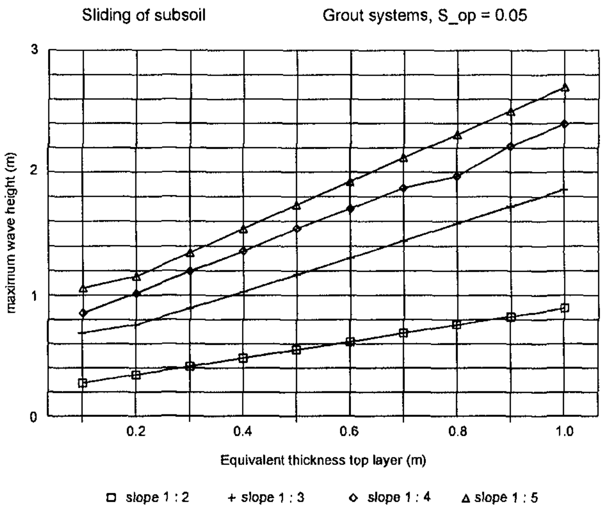

Figure 4, Geotechnical stability, design diagram for grouted systems and H_s/L_{op} = 0,05

3 STABILITY CRITERIA FOR BLOCKMATS

3.1 System description

A (concrete) blockmat is a slope revetment made of (concrete) blocks that are joined together to form a "mat", see Figure 5. The interconnection may consist of cables from block to block, of hooks connecting the blocks, or of a geotextile on which the blocks are attached with pins, glue or other means. The spaces between the blocks are usually filled with rubble, gravel or slag.

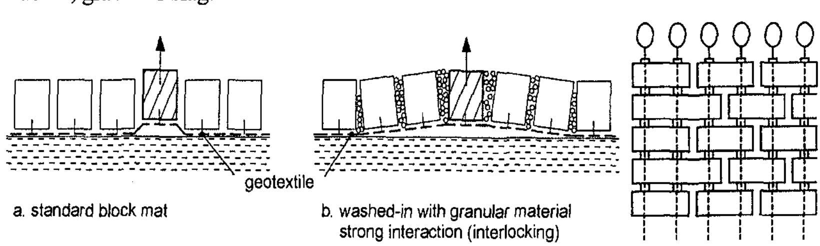

Figure 5 Examples of blockmats

The major advantage of blockmats is that they can to be laid quickly and efficiently and partly under water. Blockmats 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. It is essential to demand that already with a small movement of an individual stone a significant interactive force with the surrounding stones is mobilised. Large movements of individual blocks are not acceptable, because transport of filter material may occur. After some time, this leads to a serious deformation of the surface of the slope.

The blockmats are vulnerable at edges and corners. If two adjacent mats are not joined together, then the stability is hardly larger than that of pitched loose stones.

3.2 Design rules with regard to wave load

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

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

Table 1 Recommended values for the revetment parameter F for blockmats (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/CIRIA, 1991). 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 "unfavourable". 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 blockmats 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.

Furthermore it is essential to check the geotechnical stability with the design diagrams (see for example Figure 4 and for a full set of diagrams see Pilarczyk et al 1998).

4 STABILITY CRITERIA FOR CONCRETE-FILLED MATTRESSES

4.1 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 a variety of mattress systems, each having its own appearance and properties. Some examples are given in Figure 6.

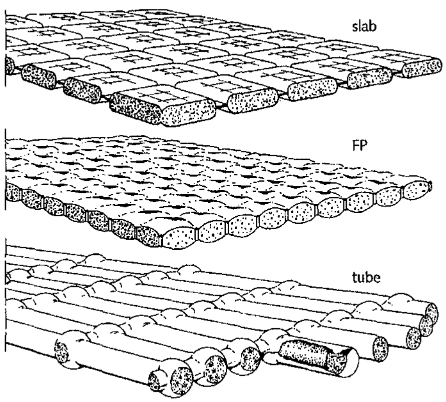

Figure 6 Examples of concrete-filled mattresses

The permeability of the mattress is one of the factors that determine the stability. It is found that the permeability given by the suppliers is often the permeability of the geotextile, or of the so-called Filter Points. In both cases, the permeability of the whole mattress is much smaller. A high permeability of the mattress ensures that any possible pressure build-up under the mattress can flow away, as a result of which the uplift pressures across the mattress remain smaller.
In general, with a subsoil of clay and silty sand the permeability of the mattress will be higher than the permeability of the subsoil. Therefore the water under the mattress can usually be discharged without excessive lifting pressures on the mattress.
The permeability of the mattress will be lower than the permeability of the subsoil or sub layers if a granular filter is applied, or with a sand or clay subsoil having an irregular surface (gullies/cavities between the soil and the mattress). This will result in excessive lifting pressures on the mattress during wave attack.

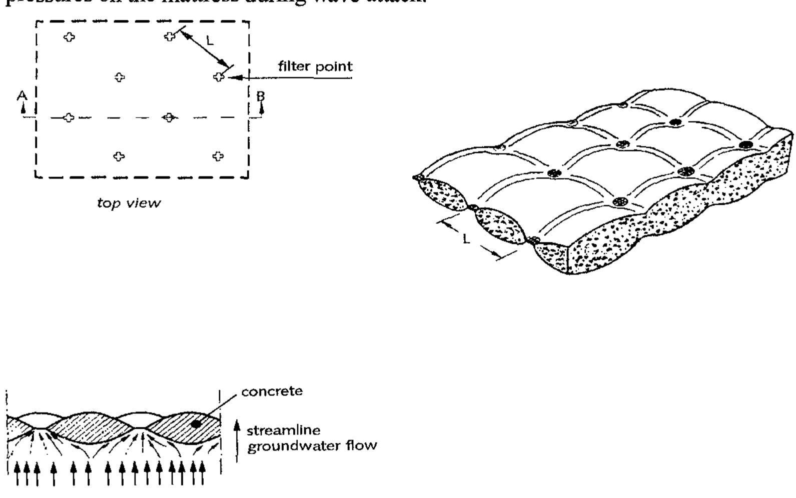

Figure 7, Principles of permeability of Filter Point Mattress

4.2 Design rules with regard to wave load

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 (Figure 1).
- 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.

It is assumed that local settlement of the subsoil will lead to free spans of the concrete mattress. Then, the wave impact can cause the breaking of these spans, if the ratio of H_s/D is too large for a certain span length. A calculation method is derived on the basis of an empirical formula for the maximum wave impact pressure and the theory of simply supported beams. The collapsing of small spans (less then 1 or 2 m) is not acceptable, since these will lead to too many cracks.
The empirical formula for the wave impact is (Klein Breteler et al 1998):

$$\frac{F_{impact}}{\rho g} = 7.2\ H_s^2\ \tan\alpha \tag{11}$$

With: F_{impact} = impact force per m revetment (N).

Calculation has resulted in an average distance between cracks of only 10 to 20 cm for a 10 cm thick mattress and wave height of 2 m. This means that at such a ratio of H_s/D the wave impacts will chop the mattress to pieces. For a mattress of 15 cm thick and a wave height of 1,5 m the crack distance will be in the order of 1 m.
Apart from the cracks due to wave impacts, the mattress should also withstand the uplift pressures due to wave attack. These uplift pressures are calculated in the same way as for block revetments. For this damage mechanism the leakage length is important.
In most cases the damage mechanism by uplift pressures is more important then the damage mechanism by impact.

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	1.5	3.9	2.3
FPM	1.0	3.9	2.0
Slab	3.0	9.0	4.7
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

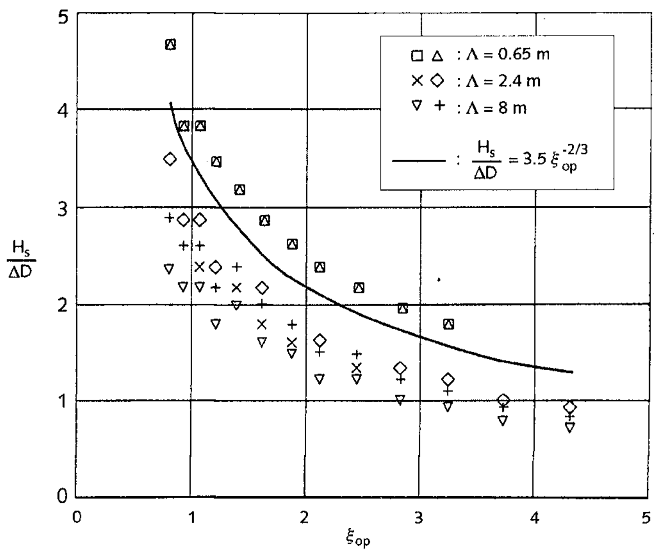

Figure 8 Calculation results for concrete mattresses ($H_s/\Delta D < 4$ because of acceptable crack distance due to impacts on spans).

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

$$\frac{H_s}{\Delta D} = \frac{F}{\xi_{op}^{2/3}} \quad \textit{with:} \left[\frac{H_s}{\Delta D}\right]_{max} = 4 \tag{12}$$

with:

$D = \dfrac{\text{mass per m}^2}{\rho_s}$ (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, see table (-)

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

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

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

5 STABILITY OF GABIONS

5.1 Introduction

Gabions are made of rectangular baskets of wire mesh, which are filled with stones. The idea of the protection system is to hold the rather small stones together with the wire mesh. Waves and currents would have easily washed away the small stones, but the wire mesh prevents this. A typical length of gabions is 3 to 4 m, a width of 1 to 3 m and a thickness of 0.3 to 1 m. The gabions with small thickness (less then 0.5 m) and large length and width are usually called Reno-mattresses.
An important problem of this protection system is the durability. Frequent wave or current attack can lead to a failure of the wire mesh because of the continuously moving grains along the wires, finally cutting through. Another problem is the corrosion of the mesh. Therefore meshes with plastic coating or corrosion resistant steel are used. On the other hand the system is less suitable where waves and currents frequently lead to grain motion.

5.2 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.

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

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

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

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

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

5.3 Stability of gabions under wave attack

An analytical approach of the development of the uplift pressure in the gabions can be obtained by applying the formulas for the uplift pressure under an ordinary pitched block revetment, with as leakage length: $\Lambda = 0.77\ D$.
With this relation the stability relations according to the analytical model are also applicable to gabions. Substitution of values, which are reasonable for gabions, in the stability relations according to (CUR/CIRIA 1991) provides stability relations which indeed match the a line through the measured points.
After complicated calculations the uplift pressure in the gabions can be derived (Klein Breteler et al, 1998). The uplift pressure is dependent on the steepness and height of the pressure front on the gabions (which is dependent on the wave height, period and slope angle), the thickness of the gabions and the level of the freatic line in the gabions. It is not dependent on the permeability of the gabions, if the permeability is larger then the subsoil. The equilibrium of uplift forces and gravity forces leads to the following (approximate) design formula (Eq. 3b):

$$\frac{H_s}{\Delta D} = F \cdot \xi_{op}^{-2/3} \text{ with } 6 < F < 9 \text{ and slope of 1:3 } (\tan\alpha = 0.33) \qquad (13)$$

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.

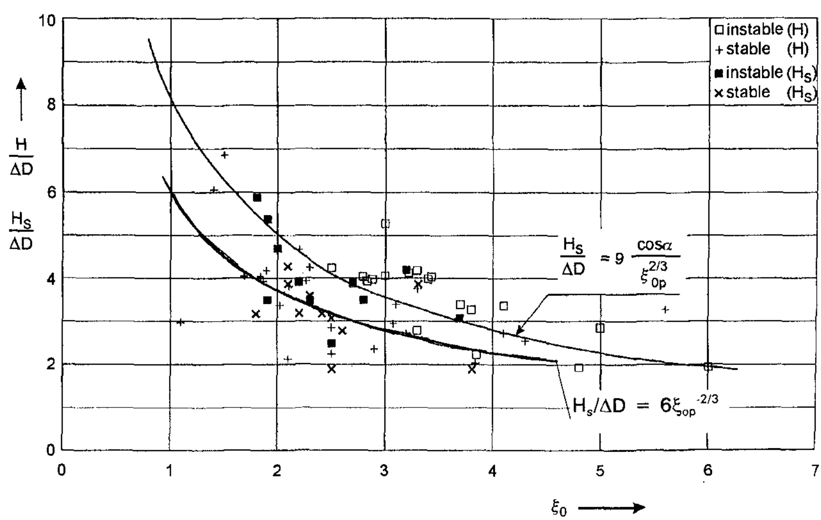

Figure 9: Summary of test results ((Ashe 1975) and (Brown 1979)) and design curves

The experimental verification of stability of gabions is rather limited. Small scale model tests have been performed by Brown (1979) and Ashe (1975), see Figure 9.

5.4 Motion of filling material

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

$$\frac{H_s}{\Delta_f D_f} = \frac{F}{\sqrt{\xi_{op}}} \qquad \text{with } 2 < F < 3 \tag{14}$$

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)

6 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.
The newly derived design methods and stability criteria will be of help in preparing the preliminary alternative designs with these systems. However, there are still many uncertainties in these design methods. Therefore, experimental verification and further improvement of design methods is necessary. Also more practical experience at various loading conditions is still needed.

REFERENCES

CUR/TAW 1995, Design Manual for Pitched Slope Protection, CUR report 155, A.A. Balkema (Publ.), Rotterdam.

CUR/CIRIA, 1991, Manual on use of rock in coastal engineering, CUR/CIRIA report 154 Gouda, Netherlands.

Pilarczyk, K.W. (editor), 1998, Dikes and revetments, Published by A. Balkema, Rotterdam, ISBN 90 5410 455 4.

Ashe, G.W.T., 1975, Beach erosion study, gabion shore protection, *Hydraulics Laboratory*, Ottawa, Canada

Brown, C., 1979, Some factors affecting the use of maccaferi gabions, *Water Research Lab.*, Australia, report 156.

Klein Breteler, M. et al, 1998, Alternatieve open taludbekledingen (Alternative open slope revetments, in Dutch), Delft Hydraulics, H1930.