Geosynthetics: Applications, Design and Construction, De Groot, Den Hoedt & Termaat (eds)
© 1996 Balkema, Rotterdam. ISBN 90 5410 836 3

Filter and revetment design of water imposed embankments induced by wave and draw-down loadings

A. Bezuijen
Delft Geotechnics, Netherlands

H.-J. Köhler
Federal Waterway Engineering and Research Institute (BAW), Germany

ABSTRACT: Design procedures for flexible revetment structures (rip-rap and placed block revetments), including geotextiles are presented. The loading can be wind waves, as well as ship induced wave loading (waterlevel draw-down and secondary waves). The stability of these revetments is governed by the interaction between pore water and cover layer, filter layer and subsoil of the structure. The different functions of these layers are elaborated. The design procedure for rip-rap is supported by an example. Special attention will be paid how the necessary input parameters for the subsoil can be obtained.

1 INTRODUCTION

Flexible revetments are the most common type of revetment used in estuaries and inland waterways. The function of a revetment is to protect the subsoil against the wave loading.

In rigid revetments as sheet piles and concrete slab the subsoil is often protected by creating in impermeable layer in front of the subsoil. In such a rigid revètment the impermeable layer (the sheet pile or the concrete slab) takes all the loading.

In flexible revetments the situation is often more complicated. The cover layer of the revetment is permeable in most cases. This cover layer has two functions. It reduces the loading and it increases by its weight the strength of the subsoil. This paper deals only with permeable cover layers. Flexible revetments with impermeable cover layers, as asphalt revetments and revetments including a geomembrane will not be dealt with.

In a revetment with a permeable cover layer the interaction between cover layer, filter layer (if present between the cover layer and the subsoil) and subsoil has to be taken into account in the design.

Due to the permeable cover layer the wave induced loading will partly be present in the subsoil. Design methods for these type of revetments, as presented by Bezuijen et al. (1990), CUR/TAW (1993), Köhler (1995) therefore include calculation methods for the loading on the filter layer and subsoil.

In this paper the wave induced loading will be described, followed by a section describing the characteristics of the materials involved. Next section deals with the theory how wave induced loading is transferred through cover layer, filter layer and subsoil. This section therefore presents the design philosophy used throughout the paper. From this description the calculation methods are explained. These include cover layer stability, filter stability and subsoil stability. Most attention will be paid to the cover layer thickness necessary for stabilisation of the subsoil. This clearly shows the interaction between the various layers. The last chapter will present some design examples.

2 HYDRAULIC LOADING

2.1 Wind induced loading

At the coast, along estuaries and lakes wind induced wave loading is the dominant loading on the revetment. Wave height and wave period can be calculated with the Bretschneider formula (Shore Protection Manual, 1984), or numerical programs if wind speed and water depth during design conditions are known.

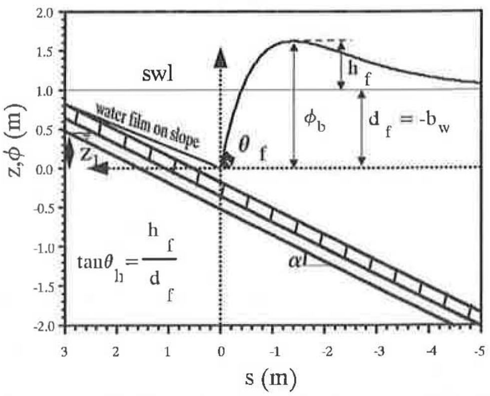

Fig. 1: Definition sketch critical wave pressure distribution on slope.

The wave height is not the important loading on the revetment. The loading is the water velocity, in case erosion of stones from the cover layer is the possible failure mechanism, and the water pressures, in case of a block revetment and if the strength of the layers below the cover layers is studied. This means in principle that water velocity and water pressures has to be derived from wave height and period. However, it is not necessary to derive such a relation for the water velocity to calculate the possibility of stone erosion. Up to now all relations presented in literature describing the stone erosion are empiric relations based on flume tests. In these relations the wave height and period is directly coupled to the possibility of stone erosion.

For the wave pressures relations between wave characteristics and wave pressures are presented by Bezuijen and Klein Breteler (1996): It was found that the critical wave pressure distribution on a block revetment with a granular filter underneath the cover layer can be written as:

$$\phi_t=(a_w s+b_w)e^{c_w s}-b_w \tag{1}$$

Where: ϕ_t is the piezometric head on the slope, s the distance from the wave front as is shown in Fig. 1. The formula is valid for negative s only. a_w, b_w and c_w are constants that can be linked with the parameters presented in Fig. 1:

$$a_w=-\tan(\theta_f) \tag{2}$$

$$b_w=-\frac{\phi_b}{1+\tan\theta_h} \tag{3}$$

$$c_w=\frac{a_w}{b_w}\frac{1}{e\ \tan\theta_h+1} \tag{4}$$

The parameters presented in Fig. 1 were measured in flume tests with irregular waves. The following empiric relations were found for a loading exceeded by 2% of the waves only:

$$\frac{\phi_b}{H_s}=\min\left(\frac{0.27\xi_{op}}{(\tan\alpha)^{0.75}},\ 2.5\right) \tag{5}$$

$$\tan\theta_f=2.25 \tag{6}$$

and:

$$\theta_h=\frac{45^0}{\sqrt{\xi_{op}}} \tag{7}$$

The relation for θ_f is valid if $\xi_{op}< 2.5$. Where $\xi_{op}=\tan\alpha/\sqrt{(H_s/l_p)}$ with H_s the significant wave height and l_p the wave length at the peak of the energy wave spectrum. As is clear from Fig. 1, the critical moment in the wave loading on a revetment is not the wave impact, but the moment just before breaking of the wave. At that moment the pressures on the revetment are relatively low for s>0 and blocks can be lifted in that area.

In case the subsoil consists of fine sand the pressure amplitude and period is of importance. For the amplitude equation (5) can be used as maximum amplitude on the slope and for the period the wave period can be taken. Often a sine form is used in calculations for the course of pressure in time. This is not true but the error that is introduced is small compared with other uncertainties, as will be dealt with later on in this paper.

For some revetments the loading at wave impact is of importance. Determining the loading at wave impact is rather difficult. Very large pressures, up to 9 times the pressure corresponding to the wave height, are reported using small pressure gauges. However, for revetments the pressure peaks that exist over larger areas are of importance. In such a case the following relations can be used (Bezuijen et al. 1990):

$$P_i=3.\rho g H_s \tag{8}$$

and:

$$b_i=0.4H_s \tag{9}$$

where P_i and b_i are the pressure peak and the width of the peak on the slope respectively.

2.2 Ship induced loading

Ship induced loading has two different components:

1. the waterlevel draw-down. When the bow of the ship passes a certain point along the embankment the water level decreases and remains at a lower level as long as the ship passes. The duration of the waterlevel draw-down depends on the dimensions of the ship and its velocity. The decrease of the waterlevel and the velocity of the decrease depend on the dimensions of the channel and the dimensions of the ship. For class IV inland waterways a design waterlevel draw-down of 0.6 m is used in Germany. The draw-down time is between 3.3 and 5 s. The 3.3 s is a rather short draw-down time but it takes into account that the draw-down caused by one ship and the secondary waves of another ship can interfere.
2. the secondary waves. These waves propagate from the stern of the ship. These can result in breaking waves on the revetment. For the loading on placed block revetments or for the loading on the subsoil, as will be described in the next section, the secondary waves are not the design load. However, they are for the stone erosion in a rip-rap revetment. Experiments have shown that more stone displacements in a rip-rap cover layer are caused by ships producing high secondary waves (as tugboats) than by ships with a considerable waterlevel depression (loaded class IV ships). No general design rules can be presented for these secondary waves, because these depend strongly on the distance between the vessel and the revetment. However, secondary waves of more than 1 m wave height have been measured in experiments with a strong tugboat. Wave period can be determined from the length of the ship and its velocity.

3. CHARACTERISTICS OF LAYERS

3.1 Cover layer

In a flexible revetment structure the functions of the cover layer are:

1. withstand the loading without stone erosion or block movement.
2. provide, by its weight, stability for the filter layers and subsoil.

The first function requires that the elements in the cover layer have minimum dimensions and a certain internal strength to prevent falling apart. The strength of the cover layer elements is hardly ever a problem in case of flexible revetments. The strength can be critical in revetments with concrete or asphalt slabs as a cover layer, but these types are not dealt with in this paper. The minimum dimensions of the revetment are determined by various calculation methods, as will be dealt with in the next chapter.

The second function requires that the cover layer as a whole has a certain weight, that also the under water weight is sufficient and that its permeability is sufficient to prevent lifting of this layer at wave run-down. These aspects will also be dealt with in the next chapter when the stability of sublayers is calculated.

3.2 Filter layer

The functions of the filter layer are:

1. prevention that fines from the subsoil are washed out.
2. reduction of the cyclic gradients that reach the subsoil.
3. adding weight to the cover layer to increase the stability of the subsoil.

When the filter layer is composed out of granular material, the minimum size is determined by the requirement that the grains has to be stabilized by the cover layer. The maximum size is determined by the first function. Sometimes it is not possible to fulfil these two requirements with one layer and two or more layers are used with decreasing grain diameters for the layers closer to the subsoil.

A very important parameter for the filter layers is the permeability. The best way to determine the permeability is by measurement. However, especially for the layers with coarser grains this is rather complicated and a correlation formula can be used. The flow in the filter layer will have a laminar and a turbulent component, as presented in the Forchheimer equation:

$$H = av + bv^2 \tag{10}$$

where H is the piezometric head, v the filter velocity and a and b are constants. These constants can be correlated with grain size and porosity according to the relations:

$$a = 160 \frac{\nu}{g} \frac{(1-n)^2}{n^3 d_{15}^2} \tag{11}$$

and

$$b = \frac{2.2}{g n^2 d_{15}} \tag{12}$$

where υ is the kinematic viscosity (m^2/s), n the porosity, g the acceleration due to gravity and d_{15} the diameter of the grains with 15% of the grain material being smaller. The resulting permeability is shown in Fig. 2.

3.3 Geotextiles

The prime function of a geotextile in a revetment structure is in most cases the filter function. Secondary functions can be separation and reinforcement. Geotextiles replace the relative costly procedure of applying various layers of granular filters.

Looking at the functions of a filter layer in a revetment, as mentioned in the last section, it appeared that a geotextile only takes over the first of the 3 functions mentioned. Applying a geotextile will not lead to a significant reduction of the cyclic gradients, unless it has a permeability that is smaller

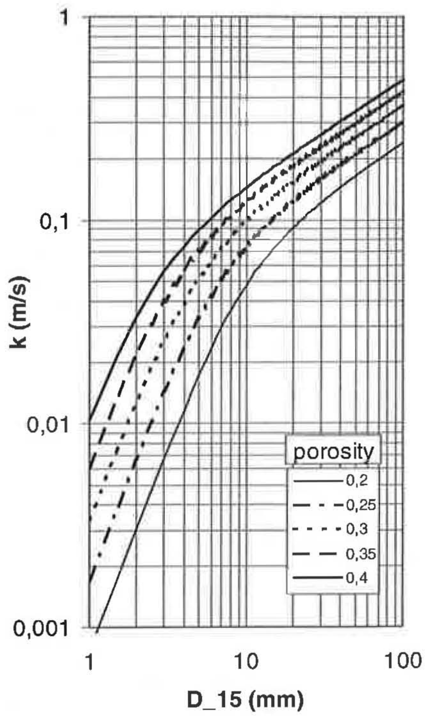

Fig 2: Permeability of granular material (i=0.3).

than the surrounding granular layers. Such a small permeability is not desirable, because of excess pore pressures. It is clear that a geotextile hardly adds any stabilizing weight to the subsoil. Replacing a granular filter by a geotextile is therefore only possible if the last 2 filter functions are taken over by the other layers and it is not always possible to reduce the overall thickness of cover layer and filter layer by applying a geotextile.

Since filtering is the prime function the opening size is the most important parameter. It should be large enough to ensure a certain permeability, but prevents the fines in the subsoil from being washed out. In most cases a geometric closed filter is chosen, according to the relation (Ogink, 1976):

$$O_{98}/d_{85}<1 \qquad (13)$$

Where the O_{98} is the apparent opening size of the geotextile corresponding with the average diameter of a standardized sand fraction of which 98% remains on the fabric and d_{85} the diameter of the grains with 85% of the grain material being smaller. Using this criterion a natural filter will be built in the subsoil below the revetment, but this will not influence the revetment itself.

3.4 Geotextile requirements according to CEN

In the specific requirements for the use of geotextiles in various applications, that are drafted at the moment in CEN/TC189, there are also specific requirements on "Erosion control". These specific requirements has to be fulfilled when a geotextile is used in a revetment. In the requirements 3 types of tests are distinguished:

1. Harmonised tests (H). These test are necessary to obtain a CE label. Each geotextile to be used in an erosion control application has to be tested according to these tests and the manufacturer has to prove that the product meets the results mentioned on the label.
2. Applicable tests (A). These tests are, according to the CEN, recommended in any case a geotextile is used in an erosion control application.
3. Additional tests (S). Tests that are only recommended in special cases.

The various tests for "Erosion control" are summarized in Table 1. Some of the tests are presented in italic. This means that these tests are not yet work items in CEN/TC 189. For these tests it is still uncertain if these tests will be standardized and if so, in what time. The tests with no letter are not used for this application. The harmonised tests are related with the prime function of the geotextile in this application: retaining particles and allowing flow. The tensile test is an exception. However, it is clear that a certain strength is necessary for the geotextile to fulfil its function.

All harmonised tests are index tests. This means that a characteristic of a geotextile is tested without taking into account the influence of the surrounding soil in an application. This should always be taken into account when using the results of these tests. The most striking example is the permittivity test. Clogging or blocking by fine particles can lead to a much lower permittivity in real soil than measured. However, even without clogging and without fine particles the permittivity of a geotextile in soil can be much smaller (a reduction with more than a factor 5 was measured, Köhler and Bezuijen, 1994), because the openings in geotextile are partly covered by granular material below and above. Fig. 3 shows that if a geotextile follows large grains that the area through with water can flow and thus the permittivity is considerably reduced.

The only applicable test (with an A) is the test damage during installation. It is in all cases important that the geotextile survives the installation and it is known from experience that this is not always the case. The only reason that this test has not a "H" is because the CE norm is a product norm and cannot tell anything about the application of the product.

static punct. GTX	
static punct. GMB	
dynamic perforation	H
tensile test	H
permittivity	H
opening size	H
transittivity	
damage during installation	A
friction d. shear /incl.plane	S
water penetration res.	
tensile creep	
compressive creep	
permeability under load	S
filter stab. stretched con.	S
long term filter stability	S
impact resistance	
abrasion damage	S
multiaxial tensile strength	
long term puncture resist.	
tensile on seams & joints	S
hydrolisis	S
microbiological degradation	S
chemical oxidation	S
weathering	S
in soil tensile strength	
long term drainage	

Table 1: Tests to be used for the application "Erosion control". The letters refer to the type of test.

From the tests with an s, the friction test is performed if the product is applied on a slope, and the stability against sliding can be critical. This will quite often be the case when a geotextile is applied in a revetment. The next 4 tests with an s will not be performed very often. Tensile strength on seams and joints is rather important if geotextiles are seamed together. Then the last 3 tests with an s are the durability tests. They will be performed when a permanent structure is designed or the water is expected to contain chemical components.

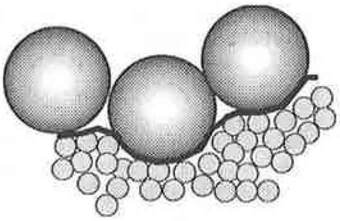

Fig. 3: Geotextile bent along larger grains reducing its permittivity.

According to Dutch experience oxidation can be a dangerous in a revetment application due to the change of conditions at the shore line.

The table presented here is only a concept version and is likely to be changed in the coming CEN meetings or during the inquiry in the European countries on the final draft. It is presented here to give an idea what requirements can be expected in the (near?) future.

4. CALCULATION METHODS

4.1 Theory

In revetment design the hydraulic loading is the most important loading. As mentioned before most of the loading is a pressure loading. Wave run-down or water level depression by passing ships leads to a temporary reduction of the pressure on top of the revetment. Deep in the subsoil the pressure will remain constant. This means that the total pressure loading on the revetment is always equal to the reduction of the wave pressure. The calculation methods only determine how this loading is divided between the various layers of the revetment. Two situations will be treated as an example, see also Fig. 4 and 5. Fig. 4 shows schematically the distribution of the piezometric head in a block revetment at minimum wave pressure and water level. Usually the permeability of the blocks is limited resulting in a rather high difference in piezometric head over the blocks. This means that the difference in piezometric head over the other layers, and the resulting vertical gradients are relatively limited. The rip-rap revetment shown in Fig. 5 has a relatively large cover layer permeability. The loading on the rip-rap is therefore limited. In this case the largest loading can be found in the subsoil, where very large vertical gradients will be found.

These figures show where damage can be expected. For a block revetment this will be on the cover layer (blocks will be lifted out of the revetment), for a rip-rap revetment movement of the subsoil or sliding is the most likely failure mechanism. The calculation methods to be dealt with in the next paragraphs will therefore calculate the piezometric head underneath the blocks for the block

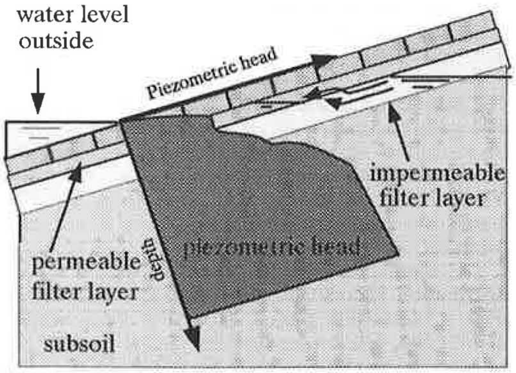

Fig. 4: Distribution of piezometric head in a block revetment at minimum water level (schemed).

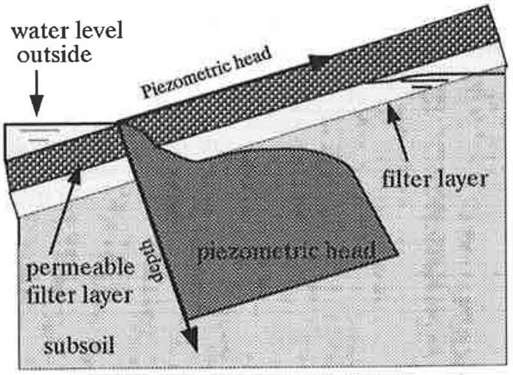

Fig. 5: Distribution of piezometric head in a rip-rap revetment at minimum water level (schemed).

revetments and the piezometric head in the subsoil for the rip-rap revetment.

The division presented here indicates the general trend. However, it is possible to have a permeable block revetment and/or impermeable subsoil, where the loading on the subsoil becomes critical (Bezuijen et al.,1986). It is also possible that in a rip-rap revetment the loading on the rip-rap becomes critical. Especially when a geotextile is applied between the rip-rap and the filter layer (Köhler and Bezuijen, 1994).

It will be clear from the description presented above and the Figures 4 and 5 that there will be no hydrostatic pressure distribution in the subsoil underneath a wave loaded revetment. The changes from the hydrostatic pressure distribution will be calculated, because these cause the hydraulic loading on the revetment.

4.2 Cover layer displacements

Block lifting

To estimate the possibility of block lifting the pressure distribution in the filter layer is calculated and compared with the strength of the revetment. The flow in the filter layer is assumed to be quasi-static This means that at every moment the pressure distribution in the filter is determined by the pressure distribution on the revetment at the same moment and the position of the phreatic line in the filter layer at that moment. This will be the case if the filter layer has a minimum permeability. For most block revetments placed on a filter layer this condition will be fulfilled. In these cases the filter layer will be much more permeable than the subsoil and the flow in the subsoil can be neglected for calculation of the loading on the revetment. The permeability of the cover layer is also less than the flow in the filter layer which results in a semi-confined flow in the filter layer. Such a flow can be described with the differential equation:

Where x is the distance along the slope and Λ is the leakage length defined as: $\sqrt{(kbD/k')}$ with k and k' the

$$\frac{d^2\phi}{dx^2}=\frac{\phi-\phi_t}{\Lambda^2} \quad (14)$$

permeability of filter layer and cover layer respectively and b and D the thickness of these layers. For the wave distribution on the slope shown in Fig. 1 and described in eq. (1) the solution of eq. (14) at maximum uplift pressure (at s = 0 in Fig. 1) reads:

$$\phi_w = 0.5\Lambda'[\sin\alpha - \frac{a_w}{(1+c_w\Lambda')^2} - \frac{b_w c_w}{1+c_w\Lambda'}] \\ [1-\exp(-\frac{2z_1}{\Lambda\sin\alpha})] \quad (15)$$

Where $\Lambda'=\Lambda\cos\alpha$ and z_1 is the position of the phreatic surface in the filter, as indicated in Fig. 1. The mean loading over the block with maximum loading will be less, because the maximum loading is present at one point of the revetment only. For blocks with a length L and a maximum loading determined by equation (15) the mean value can be approximated by:

$$\phi_m=\phi_w\frac{2\Lambda}{L}[1-\exp(-\frac{L}{2\Lambda})] \quad (16)$$

Equation (15 and 16) determines the loading on the revetment. This loading has to be compared with the strength. The strength of the cover layer is a combination of the weight of the blocks and friction between the blocks. The lower bound can be written as:

$$\phi_{st}=\Delta D(\cos\alpha+f_b\sin\alpha) \quad (17)$$

Where: ϕ_{st} is the difference in piezometric head over the blocks that leads to failure, $\Delta=(\rho_b-\rho)/\rho$, with ρ_b the density of the blocks and ρ the density of water, and f_b is the friction between the blocks (appr. 0.2).

Equation (15 and 16) together with the expected wave height according to equations (5, 6 and 7) can be combined with equation (17) to obtain a design chart, as shown in Fig. 6. From this chart it is clear that a higher value of ξ_{op} and a higher value of Λ lead to a lower stability of the revetment.

In case of waterlevel depression by passing ships, the water level outside the revetment will remain more or less horizontal. In such a situation a_w and c_w in equation (1) are zero, and equation (15) simplifies to:

$$\phi_w=0.5\Lambda'\sin\alpha[1-\exp(-\frac{2z_1}{\Lambda\sin\alpha})] \quad (18)$$

Assuming that the position of the phreatic line hardly changes during the water level depression $z_1=-b_w$ and in case $2z_1>>\Lambda\sin\alpha$ the expression simplifies even further to:

$$\phi_w=0.5\Lambda'\sin\alpha \quad (19)$$

Stone erosion

As mentioned in the introduction, the equations for stone erosion are empirically based. Most well known formula is the Hudson formula. Nowadays, more accurate formulas are available. Based on 385 tests in a wave flume with irregular waves, van der Meer (1988) derived the following formulas:

for plunging waves:

$$\frac{H_s}{\Delta D_{n50}}=6.2\frac{p^{0.18}(S/\sqrt{N})^{0.2}}{\sqrt{\xi_m}} \quad (20)$$

and for surging waves:

$$\frac{H_s}{\Delta D_{n50}}=p^{-0.13}(S/\sqrt{N})^{-0.2}\sqrt{\cot\alpha}\,\xi_m^p \quad (21)$$

Where:

H_s	: the significant wave height	[m]
ξ_m	$=\tan\alpha/(H_s/L_m)$ the surf similarity parameter	[-]
L_m	: the average wave length	[m]
D_{n50}	: the nominal diameter of the stones	[m]
p	: the permeability factor (see text)	[-]
N	: the number of waves	[-]
S	: the damage level	[-]
α	: the slope angle	[deg]

The formulas are derived for wind waves. They are not valid for ship induced water level depression. However, the secondary waves usually compose the design load for stone erosion. These secondary waves have not exactly the same form as wind waves, but there is enough resemblance to use these formulas.

The significant wave height can be used if a wave distribution is available from wave measurements over a reasonable period. Normally such a wave distribution will not be available and it is suggested to use the maximum wave height to be expected from a ship as the significant wave height.

The formulas show that also the number of waves

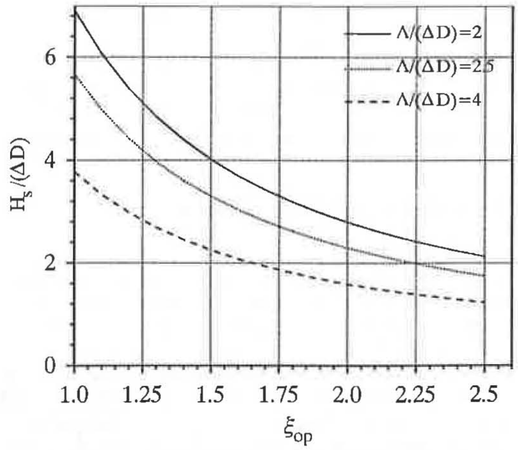

Fig. 6: Stability factor $H_s/\Delta D$ as a function of ξ_{op} and the leakage length Λ.

Table 2: Permeability factor p in various structures.

filter diam.	d filter	core diameter	p
0.22	0.5	impermeable	0.1
0.5	1.5	0.125	0.4
-	-	0.31	0.5
-	-	-	0.6

have an influence, altough small, as can be expected.

A difficulty in these formulas is the permeability parameter p. This is an empirically based parameter. The parameter is only determined for certain structures. For others an "engineering guess" is necessary. The values mentioned by van der Meer are summarized in Table 2.

In this table the nominal diameter of the filter layer and core are presented for various structures as a function of the nominal diameter of the cover layer stones. Also the thickness of the filter (d filter) is presented as a function of the nominal diameter of the cover layer material. In the third structure there was no filter layer. The cover layer was placed on a relatively permeable core. In the last structure in the table there was no filter and no subsoil. The value p=0.6 is valid for a dam made of only cover layer material. In all other cases the thickness of the cover layer was 2 times the nominal diameter of the stones. In rip-rap revetments in inland water ways the thickness of the cover layer will be larger than 2 times the nominal diameter of the stones and in most cases there will be an impermeable subsoil. This means that likely values for p vary between 0.1 and 0.4.

In this case there is only a limited interaction between the cover layer, filter layer and subsoil and therefore an empirical approach was possible. Although even in this case the permeability factor presents already some complications. In the cases of block lifting, dealt with before, and loading on the subsoil there is more interaction between the layers and the number of parameters becomes too large for such an empirical approach. For these last two cases the physical phenomena that lead to failure (pore pressure, sliding plane) has to be taken into account.

4.3 Loading on and strength of the subsoil

Unsteady pore pressure distribution

The flow in the filter layer underneath a placed block revetment could be described using what is called "quasi static flow". The parameter t (the time) is not included in the differential equation (14). This means that the pressure distribution at a certain moment is independent from the pressure distribution before and after this moment. Experiments (Bezuijen et al., 1987) have shown that this is a valid assumption to describe the flow in filter layers underneath placed block revetments with a permeability higher than $1*10^{-3}$ m/s. However, model experiments (Bezuijen et al. 1986) and field measurements Köhler (1996) have shown that this is not a valid assumption to describe the flow in the subsoil. Due to the low permeability the compressibility of the subsoil becomes of importance.

This means that another differential equation becomes the governing equation:

$$k\frac{\partial^2\phi}{\partial z^2}=n\beta' g\frac{\partial\phi}{\partial t}+\frac{\partial e}{\partial t} \tag{22}$$

Where:

k	: the permeability of the subsoil	[m/s]
ϕ	: the piezometric head	[m]
n	: the porosity	[-]
β'	: the compressibility of the pore water	[1/Pa]
g	: acceleration of gravity	[m/s^2]
t	: time	[s]
e	: volumetric strain in the soil	[-]

This equation contains derivatives to the time. This means, that to come to a solution for the time (t_0), information about the solution at previous time steps is necessary. The second term in this equation describes the compressibility of the groundwater, the last term the compressibility of the soil skeleton.

In measurements the influence of the compressibility can be seen on the pore pressure measurements. During a decrease in water level the pressure in the subsoil remains at a certain level and slowly drops to the pressure that corresponds to the new water level.

The formula is written down in one dimension only. Experiments and calculations have shown that the flow in the subsoil is predominantly perpendicular to the slope and therefore a 1-dimensional calculation method perpendicular to the slope can be used. 2-dimensional calculation methods are available (see for example Hjortnæs-Pedersen et al., 1987), but will not be dealt with in this paper. The Figures 4 and 5 show how the pressure distribution in the subsoil will be according to this equation for minimum water both level for wind waves and water level depression by ships. The critical situation is the moment of lowest water level outside. In this situation there will be a rapid increase of the piezometric head with depth in the subsoil, see also Fig. 5. As a consequence there will be a high upward directed vertical gradient and an excess pore pressure relative to the low water level.

Analytical solutions are available for homogeneous soil in a 1-dimensional situation. Loaded with a sine wave and assuming that the deformation of the grain skeleton can be neglected,the solution for piezometric head in the subsoil, $\phi(z,t)$ reads:

$$\phi(z,t)=A\ e^{-\frac{z}{L_{es}}\sqrt{\pi}}\cos\left(2\pi\frac{t}{T}+\frac{z}{L_{es}}\sqrt{\pi}\right) \tag{23}$$

where:

A	: the amplitude of the sine on the subsoil	[m]
z	: the depth	[m]
L_{es}	= $\sqrt{(T.c_v)}$ the consolidation length	[m]

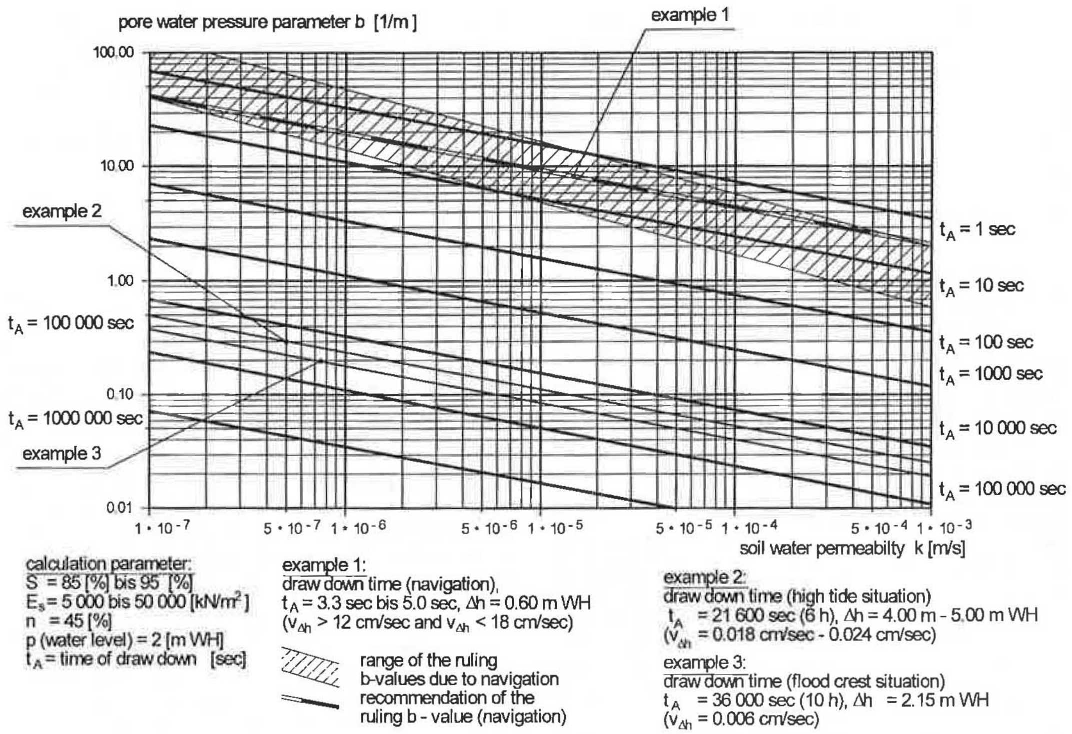

Fig. 7: Values for b (the design value of b(t) in eq. (24)) for different draw-down times to determine the pore pressure distribution in subsoils as a function of permeability.

c_v $= k/(\rho g n \beta')$ the consolidation coefficient [m²/s]
ρ : the density of the pore water [kg/m³]
T : the wave period [s]

The solution for water level draw-down is more complicated. It appeared from measurements that the solution can be approximated by an exponential function:

$$\phi(z,t) = z_A(1 - a(t)e^{-b(t)z}) \tag{24}$$

Where z_a [m] is the water level depression; and a(t) [-] and b(t) [1/m] are empirical constants. In most cases the parameter a(t) equals 1. As can be seen from equation (23) the parameter b(t) is comparable with the parameter $\sqrt{\pi/L_s}$, or $\sqrt{(\pi/(T.c_v))}$ in the solution for loading with a sine wave. Due to the different loading there will be some differences, but b(t) will also depend on the wave period (here the velocity of the draw-down) and the soil parameters. Although b(t) is a function of time, only the critical value b, that leads to the highest excess pore pressures, is of importance for design purposes. Values of b are derived based on field measurements and tests. The results are summarized in Figure 7.

As can be seen from Figure 7 the method has a much wider applicability than only for ship induced water level depression in inland navigation channels. Example 1, mentioned in this figure will be dealt with in chapter 5.

Stability of subsoil

Two failure mechanism are important for the stability of the subsoil:

1. Filter stability

In a lot of cases the hydraulic gradient in the upper layers of the subsoil will be higher than one. In such a situation the particles of the subsoil will not be stable unless protected by an adequate filter layer. This can be a granular filter or a geotextile. An inadequate filter is shown in practice by a slowly deterioration of the revetment in time. Fines will be washed out, some settlement occurs and this settlement will slowly increase.

2. stability against slip circle sliding

The excess pore pressures reduces the effective stress increasing the possibility of slip circle failure. This is a dangerous failure mechanism, since it can lead to a complete destruction of the revetment in a very short period.

The next sections will present these failure mechanisms more in detail.

4.4 Filter stability

Four main influencing factors on the actual filter performance may be:

- the ability of holding back soil particles of the adjacent soil treated under specified load conditions (mechanical aspects of the filtration process)

- the ability to act as a drain against seepage and groundwater effects (hydraulic aspects of the filter process)
- the ability of flexible reaction to follow underground distortions without leading to damage of the filter layer
-the ability of safe reactions against shear stress and normal pressure.

Without reference to other than the four influencing factors mentioned above, the mechanical differences between granular and geotextile filters have a great influence on the appropriate filter performance. Whereas a granular filter of a certain thickness, generally more than five cm and up to one, two or more decimeters, is spread out by dumping in place, the geotextile is placed on the soil. In comparison to the granular filter it is in practice a rather thin layer in the range of one to ten or even up to 20 mm thickness.

Depending on the textile tensile strength a geotextile can withstand a tensile force in the axis of length or width under weak or strong circumstances with appropriate elongations. A granular filter may not withstand elongation forces because it has no cohesion.

The modelling principle of transporting soil particles through an effective conduit opening size is based also on the velocity of the through passing fluid and therefore dependent on the hydraulic gradient occurring in place under steady and/or changing hydraulic conditions.

The amount of water passing through a soil/filter system is ruled by the law of continuity ($q = v \cdot F =$ const). It is important, that water must be permitted to drain out of the filter without obstruction, for instance caused by blocking or clogging phenomena, or simply by disregarding the permeability ratio between adjacent filter or soil layers in accordance to the prevailing or even temporary local working hydraulic gradients. On the other hand if the requested permeability ratio is too large between the layers this may establish a too high gradient in the adjacent sub layer initiating movements of endangered soil particles which may be entrapped by the filter layer which has less permeability resulting in clogging. Especially from these requests arises the well known difficulties in ascertaining a satisfactory working soil/filter system.

Geotextiles act in a very different way compared with granular systems. Geotextiles are able to take over tensile forces in the axes of length and width. Fig. 8 shows the way of performance of a geotextile. From the occurring k - ratios between filter layer and adjacent soil base, the pressure acting underneath the filter with hydraulic load changes. This leads, using the continuity equation, to the acting mechanical stresses and strains inside the soil/filter system according to a geotextile filter. The effective stress condition should be ascertained by the protection layer system, to ensure that sliding or uplifting of the soil skeleton or soil base does not happen, during upward directed hydraulic loading.

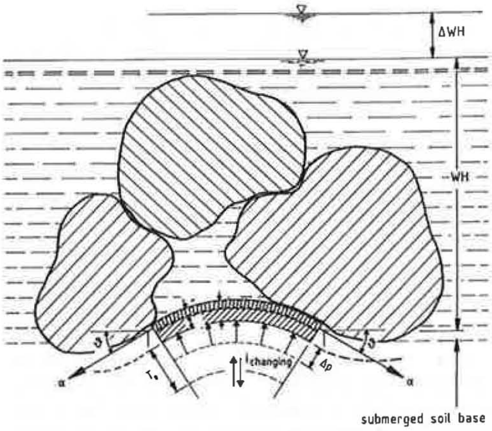

Fig. 8: Principle of a geotextile acting as an 'armour' layer.

As is shown in Fig. 8, a geotextile filter, initially laid out smoothly, had been forced to plastic elongation even due to small values of tensile forces inside the geotextile caused by temporary pressure changes Δp. The underlying soil may follow this displacement, due to loss of contact confining pressure in the adjacent soil base. Critical gradients $i > 1$ lead to distortion of the soil base, which ought to be protected against erosion. The displacement of the soil widens the pore structure (breathing of soil) and mobile soil particles are encouraged to wander inside the widened conduits, being transported by the water flow. As long as these particles may pass through or intrude the filter layer out of the adjacent soil base, the filter system has a chance to reach equilibrium state of satisfying filter performance in adopting the subsoil base as an active part of an efficiently working filter layer. In case the wandering fine soil particles get trapped inside or in front of the filter layer, causing remarkable reductions of permeability, the filter performance tends to change into that of a lining system.

Even a badly designed loose granular filter with grain sizes smaller than the sand fraction could withstand increasing pore pressures and rising hydraulic gradients without remarkable distortion of the filter layer. It would react with the occurrence of piping, immediately reducing the temporary working excess pressure gradients.

A geotextile filter can cause more trouble, if the accumulation of fine filter particles remain underneath the filter, establishing a soil layer with much smaller k - values than the desired one. This phenomenon is not only found in a suffosive soil base, it also may happen in a more or less uniform silty sand. The mobile silt fraction could cause a quite similar effect. The question as to which filter is to be required, i.e. a more open or a more strict geometrically designed filter, will vary according to the prevailing field and load conditions. In the case of water imposed embankments, where small soil deformations and settlements are usually acceptable, an open filter would be the appropriate design.

4.5 Stability against sliding

Tree possible waves to calculate the possibility of sliding will be described:

1. sliding parallel to the revetment.
2. slip circle sliding.
3. finite element calculation.

The first method can be used as an engineering tool. The second and third method are presented to show the validity of the first method. Evaluation of the possiblity of sliding depents on the type or revetment. Here sliding is elaborated for a rip-rap revetment which a relatively open cover layer and filter layer. The method described is in use by BAW to design the revetment in inland waterways in Germany.

Sliding parallel to the revetment

The stability of the revetment is evaluated by calculation of the driving and resisting force for a sliding plane at a critical depth in the subsoil, see Figs. 9 and 10. Here the expression for water level depression will be presented. Comparable expressions for wind waves without toe support are derived by Bezuijen (1991). Due to the rapid increase in piezometric head in the subsoil with depth during the water level depression the critical sliding plane will not be in the cover layer or filter layer, but in the subsoil. Using the exponential function to describe the piezometric head in the subsoil all driving and resisting forces can be calculated. These forces are calculated for the part of the revetment below the water line. It is assumed that the part above the water line will be stable. If a geotextile is used as a filter layer, then the anchor force of this geotextile from the layers above is included, see detail B in Fig. 10. Also the toe support at the lower end of the revetment is included.

The design formula for the requested thickness of the revetment is written:

$$d_c = \frac{\tan\phi'_S \cdot \gamma_w \cdot \Delta u(z,t) - c'_S - \tau_G - \tau_A - \tau_F}{\cos\beta \cdot \gamma'_C \cdot (\tan\phi'_S - \eta \cdot \sin\beta)} - \frac{d_{s\,crit.} \cdot \gamma'_S + d_F \cdot \gamma'_F}{\gamma'_C} \qquad (25)$$

with the critical depth $d_{S\,crit.}$ of soil layer :

$$d_{S\,crit} = \frac{-\ln(\gamma'_s \cos\beta(\tan\phi'_s - \eta\tan\beta))}{b(t)} + \frac{\ln(\gamma_w z_A a(t) b(t) \tan\phi'_s)}{b(t)} \qquad (26)$$

and the excess pore water pressure $\Delta u(z,t)$, acting temporally in the critical sliding plane in the depth

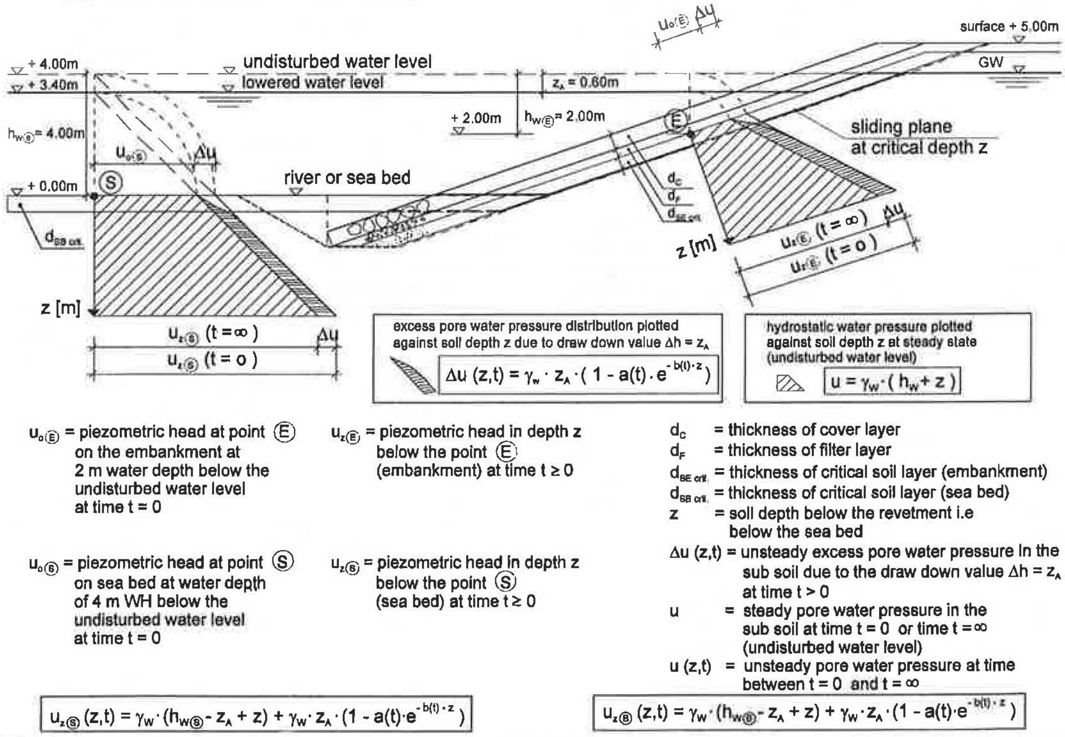

Fig. 9: Definition sketch pore pressures along a sliding plane parallel to the slope for a rip-rap revetment.

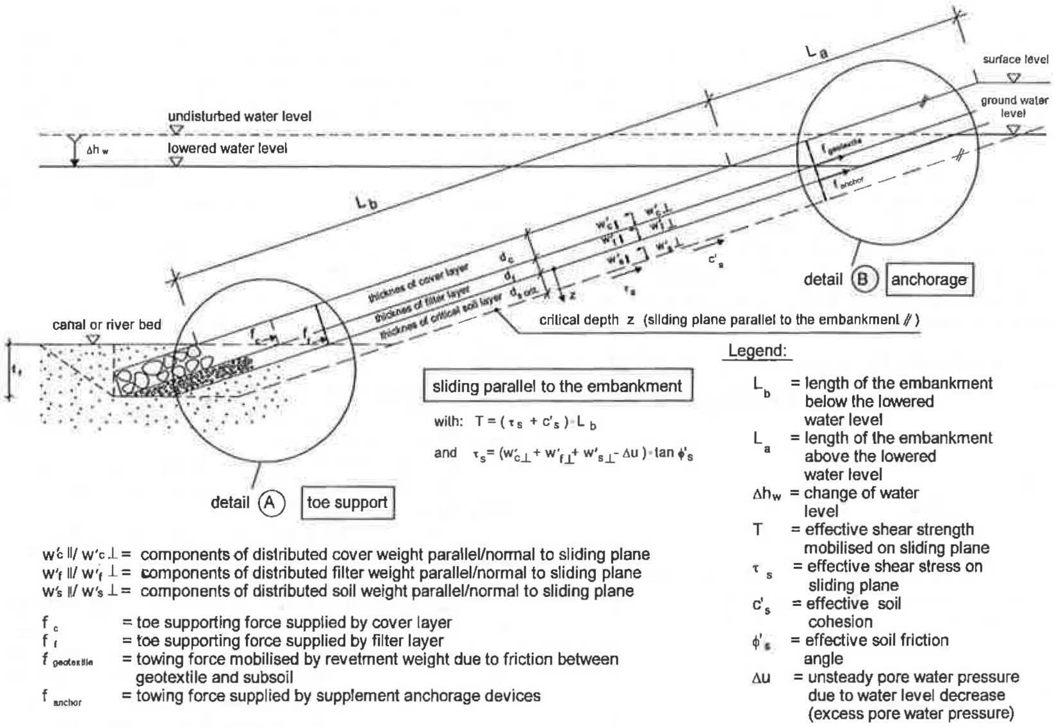

Fig. 10: Forces along sliding plane parallel to slope, rip-rap revetment.

$d_{S\,crit}$.

$$\Delta u(z,t) = \gamma_w z_a \left(1 - a(t) e^{-b(t) d_{S\,crit}}\right) \qquad (27)$$

The design parameters for the calculation of revetment cover thickness are:

- ϕ'_S[°] effective angle of internal friction of the subsoil
- γ'_S[kN/m³] submerged unit weight of the subsoil (below water level)
- c'_S[kN/m²] effective cohesion of the subsoil
- ϕ'_C[kN/m³] effective angle of internal friction of the cover layer
- γ'_C[kN/m³] submerged unit weight of the cover layer (below water level)
- γ_C[kN/m³] unit weight of the cover layer (above water level)
- c'_C[kN/m²] effective cohesion of the cover layer
- ϕ'_F[kN/m³] effective angle of internal friction of the filter layer
- γ'_F[kN/m³] submerged unit weight of the filter layer (below water level)
- γ_F[kN/m³] unit weight of the filter layer (above water level)
- c'_F[kN/m²] effective cohesion of the filter layer
- α_{Fg}[kN/m] geotextile force of serviceable limit state (allowable strain e = 1 %)
- α_{Fa}[kN/m] anchoring force of serviceable limit state (allowable strain e = 1 %)
- δ'_{FS}[°] angle of sliding friction between subsoil and filter layer
- δ'_{FC}[°] angle of sliding friction between filter layer and cover layer
- z_a draw down value
- h[-] factor of safety
- β[°] slope angle
- b (t)[1/m] pore water pressure parameter
- a (t)[-] pore water pressure parameter
- γ_w[kN/m³] unit weight of the water
- τ_G [kN/m²] allowed added shear stress acting from geotextile filter
- τ_A [kN/m²] allowed added shear stress acting from anchorage
- τ_F[kN/m²] allowed added shear stress acting from toe support

where: $\tau_G = G_{limit} / L_b$ [kN/m²]
$\tau_A = A_{limit} / L_b$ [kN/m²]
$\tau_F = F_{limit} / L_b$ [kN/m²]

- A_{limit}[kN/m] limited anchoring force in the revetment per unit length of the canal
- F_{limit}[kN/m] limited force of toe support in the revetment per unit length of the canal
- G_{limit}[kN/m] limited pulling force of the geotextile filter in the revetment (anchorage) per unit length of the canal

where: $G_{limit} = \tau_{limit} \cdot L_b$ [kN/m] or
$G_{limit} = \tau_{mob} \cdot L_b$ [kN/m]

and the smallest of:

$\tau_{mob} = d_C \, \gamma_c \tan(\delta'_{FS} - b) \; L_a / L_b \cos\beta$ or
$\tau_{limit} = \alpha_{Fg} / L_b$ [kN/m²]
$\alpha_{Fg} = 0$ (if a granular filter will be used)

Ticknesses and lengths are presented in Figs. 9 and 10.

Limitation of the toe supporting force is controlled by two failure mechanisms:

- the outer failure mechanism: sliding occurs below the embankment toe, caused by sea bed boiling or reduced earth pressure due to the temporally acting excess pore water pressure in the sea bed (the toe supporting forces need to be proved to withstand the design load before being introduced into the embankment sliding calculation).
- the inner failure mechanism: sliding occurs directly through the cover and filter layer, although sufficient toe support in the sea bed is provided (i.e. sheet pile wall, rock ect.). The limitation of the permissible toe supporting force F_{limit} is given by the following equation:

$$F_{limit} \leq \frac{\mu_F \cdot [(0{,}5 \cdot (d_C + d_F)^2 \cdot \gamma'_R \cdot \cos\beta \cdot \tan\phi'_R + (c'_C \cdot d_C))]}{\sin\beta \cdot (\cos\beta - \sin\beta \cdot \tan\phi'_R)} \quad (28)$$

with:

$$F_{limit} = f_c + f_f$$

where:

f_c = toe supporting force supplied by cover layer at any horizontal cross section of the revetment

f_f = toe supporting force supplied by filter layer at any horizontal cross section of the revetment

and:

μ_F [-] reduction factor for the toe supporting force in the revetment

γ'_R[kN/m^3] average submerged unit weight of the whole revetment (cover and filter layer)

ϕ'_R [°] average effective angle of internal friction of the whole revetment (cover and filter layer).

Optimization is achieved by superimposing all acting forces $T_{//}$ parallel to the embankment, supplied by the limited toe supporting force F_{limit}, permissible pulling force G_{limit} in the geotextile filter and the anchoring force A_{limit} by supplement anchoring devices, combining allowable strains in the whole structure for the requested state of equilibrium against slope sliding ($\sum T_{//} = 0$):

$$\sum T// = 0 \;:\; T + W_c + W'_c + F_{limit} + G_{limit} + A_{limit} = 0 \quad (29)$$

and the serviceability condition of all acting mobilised forces, which can be written as:

$$w'_{c\parallel} \tan\delta'_{FC} + w'_{c\perp} \tan\delta'_{FS} + w_{c\parallel} \tan\delta'_{FC} \cdot L_a/L_b \geq \tau_G + \tau_A + \tau_F \quad (30)$$

In these equations is:

T [kN/m]: effective shear strength per unit length of the canal, mobilised on the sliding plane below water level along the embankment length L_b, analogue to Coulomb's assumption

W_c [kN/m]: force of the weight component of cover layer above the water level, acting parallel to the embankment slope along the length L_a, mobilising friction between cover layer and filter layer or geotextile, which should be sufficiently provided, with the anchoring effect of the towing force in the geotextile for equilibrium against parallel embankment sliding

W'_c [kN/m]: force of the submerged weight component of cover layer below the water level, acting parallel to the embankment slope along the length L_b, mobilising friction between cover layer and filter layer (geotextile), which should be sufficiently provided, with the anchoring effect of the towing force in the geotextile, or supplemented with anchoring devices to prevent parallel embankment sliding

These parameters can be written as:

$T = (\tau_S + c'_S) \cdot L_b$

$W_c = w_{c\parallel} \cdot L_a$ (above water level)

$W'_c = w'_{c\parallel} \cdot L_a$ (below water level)

with:

$\tau_S = (w'_{c\perp} + w'_{f\perp} + w'_{s\perp} - \Delta u(z,t)) \cdot \tan \phi'_S$

The components of the distributed weight normal and parallel to the sliding plane resulting from the cover layer can be written as:

$w'_{c\perp} = d_C \cdot \gamma'_C \cdot \cos\beta$ and $w'_{c\parallel} = d_C \cdot \gamma'_C \cdot \sin\beta$

$w_{c\perp} = d_C \cdot \gamma_C \cdot \cos\beta$ and $w_{c\parallel} = d_C \cdot \gamma_C \cdot \sin\beta$

filter layer:

$w'_{f\perp} = d_F \cdot \gamma'_F \cdot \cos\beta$ and $w'_{f\parallel} = d_F \cdot \gamma'_F \cdot \sin\beta$

critical soil layer:

$w'_{s\perp} = d_{S\,crit} \cdot \gamma'_S \cdot \cos\beta$ and $w'_{s\parallel} = d_{S\,crit} \cdot \gamma'_S \cdot \sin\beta$

Even a small amount of soil cohesion prevents embankment sliding failure mechanisms. But soil cohesion should only be taken into consideration, when the serviceability can be guaranteed against the design load conditions, i.e. oscillating waves etc.. The following condition should be maintained:

$$c'_s + \tau_G + \tau_A + \tau_F < \Delta u(z,t) \tan\beta \quad (31)$$

Priority should been given to c'_s before τ_G, before τ_A and before τ_F.

The requested submerged weight mass g'_{req} per square meter of revetment is given by:

$$g'_{req} = (\gamma_C g'_C + \gamma_F g'_F) \cdot 100 \quad [kg/m^2] \quad (32)$$

Explanation to the outer failure mechanism

The limitation of the toe supporting force is determined by the type of chosen toe structure:

- toe support by *elongated toe revetment on to the sea bed*, mostly supplied by horizontal sliding friction immediately below the horizontally elongated bed revetment
- toe support by an *embedded revetment into the sea bed*, provided a combination of sliding friction below the revetment toe and a limited passive earth pressure controlled by the restriction of allowable embankment deformations in front of the embedded toe. In calculating the permissible earth pressure, the reduction of the embedded depth level by possible scour depth and a fluidized soil layer thickness up to the value of water level decrease z_A (draw down value) has to be taken into consideration. Therefore the toe support will decisively be determined by the anticipated size of the design depth, embedding the revetment into the sea bed.
- toe support by *sheet pile wall structure*, controlled by the allowed earth pressure in front of the sheet pile wall, taking into consideration scour as well as

fluidization effects. The anticipated design depth of a sheet pile wall in front of an water imposed embankment toe therefore has decisive influence on the necessary thickness of embankment revetment cover layer.

slip circle failure

To calculate the possibility of slip circle failure the same procedure has to be used as described for the sliding plane. Again the loading and the resistance against sliding of a part of the revetment has to be calculated. The only difference is the shape of the sliding plane. In case of slip circle failure this is a part of a circle as is shown in Figure 11. Calculation of the possibility of slip circle failure is the standard procedure to evaluate the stability of slopes. The only difference is that in this case the pore pressure distribution at minimum water level has to be used to calculate the effective stress in the subsoil, instead of a hydrostatic pressure distribution. Some of the programs to calculate slip circle failure commercially available can include all kind of pore pressure distributions. The result shown in figure 11, is the result of a tailor made program for evaluating the stability of a revetment. This calculation method will be dealt with more in detail in the example calculation.

Stability calculation using FEM

The finite element method (FEM), in geotechnical engineering used to calculate deformations, is in some cases also capable to perform limit state analysis, as it is done by slip circle analysis. FEM can give ever more information.

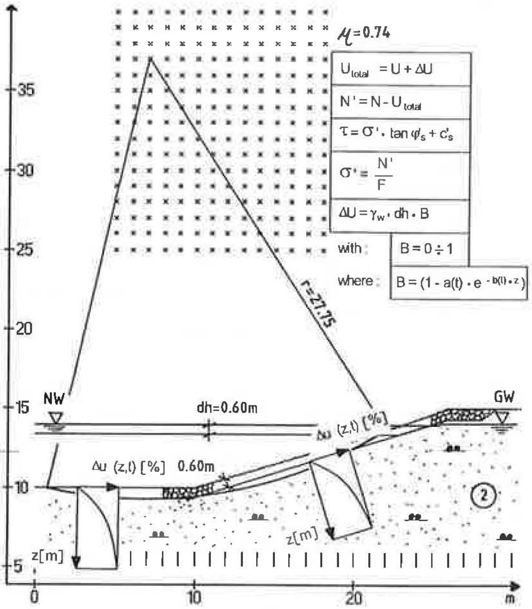

Fig. 11: Example of slip circle calculation, see also Chapter 5.

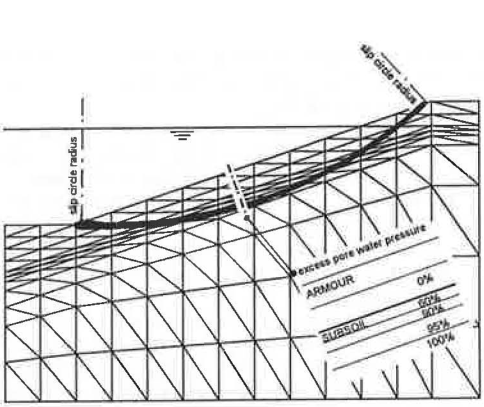

Fig. 12: Bank protection with slip circle and finite elements.

In FEM the soil is considered a continuum, represented by an assembly of elements. Soil parameters are constant per element. Excess pore water pressure increasing with depth as it has been described in section 4.1, can be introduced by defining soil layers parallel to the slope surface with the same soil parameters but different pore water pressures (Fig. 12). In this example the stability of the subsoil of a riprap revetment is calculated taken into account the expected pore pressures in the subsoil.

The de-stabilization due to excess pore water pressure in the subsoil can be demonstrated using Mohr's stress circle (Fig. 13): The neutral stress (u) resulting from (excess) pore water pressure remains nearly unchanged while the total stress σ_r, defined as weight per volume of soil and water, is reduced due to the draw-down. The circle of effective stresses ($\sigma = \sigma_r - u$) touches or passes the Mohr-Coulomb failure surface: the stability is gone. Since this situation cannot be simulated in FEM because of numerical stability reasons (the effective stress must not be below zero), it is a clear advice that stability is not given at least in certain parts of the system.

Slip circle analysis or similar calculation methods are not able to detect areas with effective stress equal or to zero. So stability of the whole system is pretended where local failure will occur: If there is an armour layer of high strength and the toe is reaching sufficiently deep below the bed (a comparable situation as is shown in Fig. 17), sufficient stability will be calculated, even if there is no friction in the subsoil (because of $\sigma' = 0$), but only in the armour layer. In such a case, numerical stability

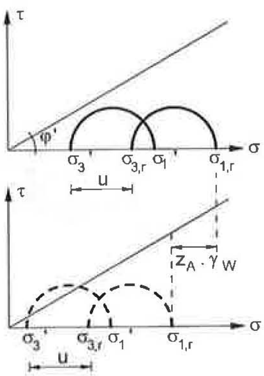

Fig.13: Mohr's stress circles for draw-down.

will not be reached in finite element calculation unless σ' > 0, which can be attained by sufficient weight of the armour layer.
Sand boiling (or fluidization) in the unprotected top layer of the bed in front of the revetment (see section 4.5) will not harm the stability of the bank as long as the revetment toe is reaching down to the stable soil. The fluidized layer has to be represented as a material with nearly no shear strength ($\varphi'=1°$), thus taken into consideration the effect, but without numerical difficulties.

One might assume that fluidization of a layer below the revetment is without problems, since it is confined by the filter and the revetment above and the stabile subsoil below. Indeed, the fluidized soil can't be eroded, but the soil grains will be able to rearrange, preferably in downslope direction. Due to the stress relieve during a draw-down, the revetment will be lifted a few millimetres, thus allowing for movements of single grains. There will be no great effect per load cycle, but a cumulation with time, since any rearrangement after one cycle is the starting situation for the next. So the well known S-shape of a bank will develop, even if the revetment is resistant against erosion and/or wave impact, but too light to attain a sufficient state of stress in the subsoil.

Using FEM calculations to evaluate the limit state can only be done with sufficient accuracy if, what is called, higher order elements can be used with sufficient integration point for each element. Most FEM program packages can deal with different elements. Calculation results evaluating the limit state are more reliable if results obtained with different elements and meshes are in agreement.

Finite element calculations are at the moment not a standard tool in the design of revetments. However, in this case an interesting result was obtained. In Fig. 12 a slip circle is drawn through the mesh, because this is supposed to be the sliding plane. Looking at the deformation pattern that is the result of the calculation (Fig. 14) it appeared that a sliding plane parallel to the revetment is in better agreement with the FEM calculations than such a slip circle.

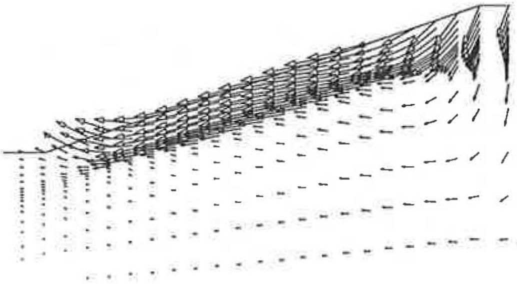

Fig. 14: Deformation pattern in limit state.

Consequences geotextile
The calculation methods dealt with in this chapter all assume that the permeability of the geotextiles larger than the permeability underneath. If this is not the case, for example because of blocking or clogging, then there will be a pressure building underneath the geotextile. As a result the potential sliding plane will be just underneath the geotextile instead of in the subsoil. At this location the stabilizing weight of the revetment and subsoil will be less, increasing the risk of failure.

5 DESIGN EXAMPLE

Sliding plane parallel to slope
In order to compare the results out of revetment calculations by using the plane sliding and slip circle method, the results were compared in calculations where the necessary thickness of a rip-rap layer is investigated.

The ruling subsoil parameters of a silty sand were chosen:
- internal angle of friction $\phi'_S = 32.5°$
- submerged unit weight of the soil of $\gamma' = 9.5$ kN/m³ , (i.e. $\gamma_r/\gamma = 18{,}0/19{,}5$ kN/m³)
- soil water permeability $k = 1{,}5 - 2{,}5 \; 10^{-5}$ m/s
- pore water pressure parameter b(t) = 8 (i.e. a(t) = 1), using Fig. 7 (for example 1).
- slope inclination 1:3
- factor of safety: f = 1.1 (32.5°/1.1= 30° --> reduced internal friction angle of the subsoil)

The revetment is constructed by stone rip-rap placed on a geotextile filter. The revetment has to be designed for an inland navigational canal to withstand rapid draw down effects and ship induced waves. In Fig. 7 the range of the ruling pore water pressure parameter b(t) [1/m] is been plotted against the soil water permeability k [m/s], the parameter a(t) is kept constant and has been chosen to 1. For the design example 1 the parameter b(t) can be taken to b(t) = 8 (1/m). With this value b(t) the standard loading of a draw down value of about 0.60 m WH and draw down velocity between 12 cm/s and 18 cm/s has been used as design load. This will include all occurring water loadings due to navigation (ship induced draw down and waves), which have to expected by passing vessels. The range of the ruling b-values differ with the modulus soil compressibility, but can be neglected, if the recommended b-value (black and white dotted thick line) has been chosen. With this simplification, the decrease in modulus of soil compressibility of more silty soils, which lead to more deformation has been taken into consideration and has been proved by numerical calculation methods and laboratory tests.

Adopting the formulas for parallel embankment sliding, it clearly can be demonstrated, that a thickness of stone cover layer ($\gamma'_{dc} = 9.1$ kN/m³ , i.e. $\gamma_r/\gamma = 14{,}6/19{,}1$ kN/m³) with an internal angle of friction $\phi'_{dc} = 55°$ of at least $d_c = 0.60$ m is requested to ensure safety against slope sliding, but only for the case, a toe supporting force of more than 14.4 kN/m (per unit length of canal) is be supplied. If less toe support is available, a thicker stone cover layer will be needed, to prevent slope sliding.

For the case, an elongated revetment toe of 0.60 m thickness should be designed, the allowable toe support will drop down to more or less 1 kN/m, taking into consideration, the unprotected sea bed in front of the embankment toe will be fluidized during wave and draw down loading. No effective stress can been taken over by the fluidized soil layer thickness up to the depth of the waterlevel draw-down $z_a = 0.6$ m and therefore no earth pressure can been taken into account.

To stabilise the embankment against slope sliding, an embankment toe structure or an deeply into the sea bed embedded revetment toe is requested. The same effort can be reached by a sheet pile wall, which will supply enough toe support, in order to reduce the requested thickness of the stone cover layer. As long as the outer failure mechanism is dominating the design concept, the thickness of the cover layer is a function of b.

Using the parameter mentioned before the necessary revetment thickness is calculated for different toe supporting forces. The result is plotted in Fig. 15. In this figure the requested cover layer thickness is plotted against soil depth z[m], indicating the critical depth of sliding plane will be at $d_{s\ crit} = 0.31$ m, where the maximum thickness of cover layer is been requested to prevent embankment sliding. It can be seen, that for the plotted curves 1 to 7 the requested cover layer thickness ranges between 1.19 m (curve 1) and 0.78 m (curve 7). The plotted curves represent toe supporting forces between

1: F= 0 kN/m; 2: F= 1 kN/m; 3: F= 2 kN/m; 4: F= 4 kN/m, 5: F= 6 kN/m; 6: F= 8 kN/m;
7: F= 10 kN/m; 8: F=15 kN/m; 9: F=20 kN/m and 10: F= 30 kN/m.

For the curves from 1 to 7 the outer failure mechanism is dominating, for the curves 8 to 10 the inner failure mechanism is acting. As it can be seen, embankment sliding will not occur with a revetment structure of a thickness of 0.6 m stone cover layer, but cannot be reduced any further, although a reasonable large toe support above 15 kN/m could be provided. On the other hand a quite large thickness $d_C = 1.19$ m of stone cover layer would be requested, when the anticipated design structure of an elongated revetment toe (standard width of about 2 m) would be recommended. Therefore a more reasonable revetment construction could be built only by a better toe structure. This can been done by embedding the revetment toe into the sea bed up to the depth where scouring and sea bed fluidization effects can be excluded.

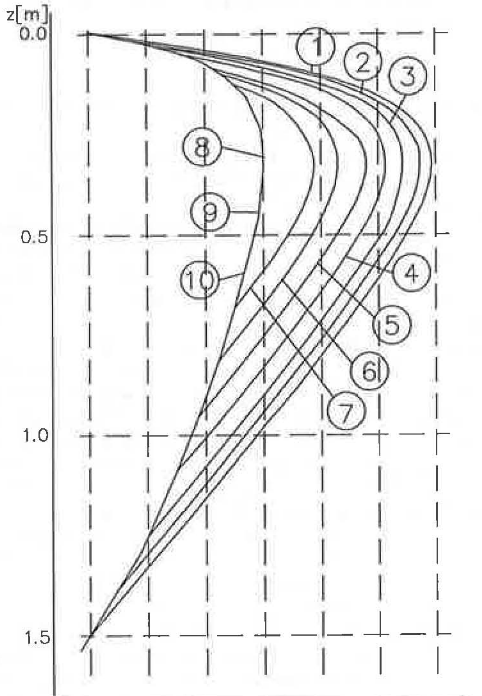

Fig. 15: Required cover layer thickness (d_s) to stabilize the subsoil at various depths as a function of the toe support. Numbers see text.

In case the geotextile filter should be replaced by a 0.40 m thick granular filter, the requested stone cover layer thickness would drop down to about 0.19 m thickness as long as a large toe support above 15 kN/m could be provided (inner failure mechanism). For a 0.58 m thick stone cover layer a toe support of F= 4 kN/m would be needed to prevent embankment sliding (outer failure mechanism). The curves 8 to 10 represent the inner failure mechanism, the curves 1 to 7 describe the available toe supporting forces dominated by the outer failure mechanism.

Calculating the requested cover thickness using a geotextile filter for the above mentioned design parameters (d_h=0.60 m) as a function of b(t), or as a function of ϕ'_S (with b=8), or a function of draw down value d_h, the different requested thicknesses of the cover layer are plotted in Fig. 16. The left hand figure shows the necessary thickness as a function of b(t), the middle figure the as a function ϕ'_S and the right hand figure as a function of d_h.

Slip circle

The results from plane sliding failure mechanisms has been compared with the conventional method of slip circle calculations, the excess pore water pressure Δu (z,t) has to be calculated in different soil layers parallel to the embankment surface with a thickness of about 10 to 20 cm, in order to introduce different excess pore water proportion, as it is shown in Fig. 11.

In Fig. 11 a 0.6 m thick revetment of stone cover layer, placed on a geotextile filter, looses its stability

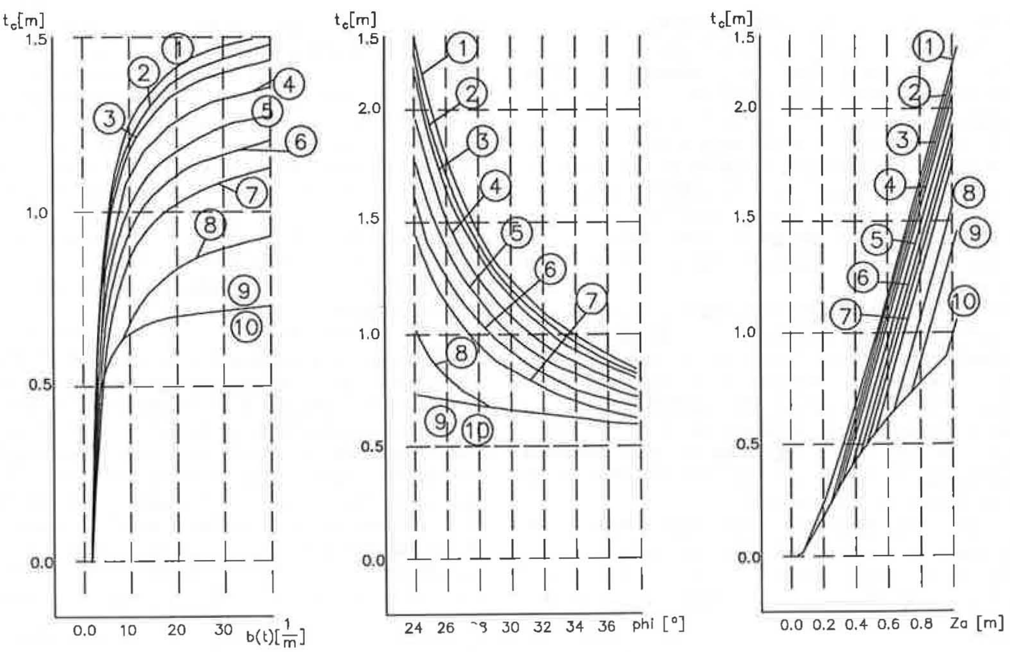

Fig. 16: Required revetment thickness as a function of b(t), the friction angle of the subsoil ϕ and z_A. For various values of toe support. Numbers see text.

due to rapid draw-down values (z_A=0.6 m). This draw-down is initiating a fluidization of the sea bed in front of the elongated revetment toe up to the a depth level of 0.6 m. The toe supporting force is therefore temporally reduced below F_{limit} (the outer failure mechanism), which will, which the reduced value of about 1 kN/m, be insufficient to prevent embankment sliding in the critical soil depth layer $d_{c\ crit}$ below the revetment. The factor of safety is calculated with the temporally acting excess pore $\Delta u(z,t) = \rho g\ z_a.B$ where the proportion factor B has been given by the exponential function (24) plotted against soil depth level z [m] perpendicular to the slope. The result coincides well with the results obtained the using the plane sliding mechanism. In Figure 17 the resulting safety factor is $\mu = 0.97$ for a toe structure, which is embedded up to 2 m below sea bed. The fluidization effect (supposed to be equal $z_A = 0.60$ m) reduces the toe supporting force below F_{limit}, causing embankment sliding similar to the case described in fig. 11, although much less endangered.

Additional comparing calculations for the cases fluidization will not occur and for the case of an increased cover layer thickness off $d_c = 0.90$ m result in safety factors, which are given in Table 3.

The safety factor of $\mu = 1.0$ for the 60 cm thick revetment cover layer, 2 m deep embedded toe into the sea bed, where fluidization is not occurring, coincides exactly with the result from plane mechanism causing sliding through the toe of the cover layer directly at sea bed level with F_{limit}, given by the toe supporting formula (25).

It has to be mentioned, that the above described calculation methods are governed by limit state conditions, which can be described by Coulombs statement to the mobilised friction on consideration shear stress condition. But deformation of the soil structure is not bound to limit state conditions, when large deformations occur before failure happens. The effect may cause different failure situations, such as heaving and settling effects of the endangered soil areas below the revetment. Fig. 14 shows the size of the endangered soil area at draw down time, where great hydraulic gradients may be initiated, without reaching failure state. Destabilisation leading to unacceptable deformations of the embankment structure, which destroys the requested service level of the revetment structure.

The above described method to predict excess pore water pressure and the evaluation of the stability is not limited to only waterlevel draw-down situations. It is successfully used in flood crest and high tide situations as well as for draw-down loadings, subjected i.e. to reservoir embankments. Fig. 7 shows

Table 3: Examples of slip circle calculations. Comparison between elongated (2 m) and embedded toe (to 2 m depth). d_c= thickness cover.

toe structure	d_c (m)	safety factor μ	
		not fluid.	fluidized
elongated toe	0.9 0.6	1.33 0.83	0.9 0.74
embed. toe	0.9 0.6	1.33 1.00	1.17 0.97

that the numbers of b(t) can change considerably and this means that also the time scale involved can change considerably.

6 CONCLUSIONS

Calculation methods for rip-rap and placed block revetments have been presented. Theory and example calculations showed the large influence of the subsoil reaction on the stability of the revetment. Stability of revetments can only be evaluated adequately if the subsoil reaction is incorporated in the calculation method. The calculation method for the stability of a rip-rap revetment can in principle also be used to evaluate the stability of the subsoil for a placed block revetment. However, for this last type there will be more loading on the cover layer and less on the subsoil. The examples show that, in case of water level draw-down, using the exponential equation (24) to calculate the pore pressure in the subsoil, the stability of the subsoil can be analyzed with sufficient accuracy using slip circle analysis. Although it is also shown that a sliding plane parallel to the slope is more likely to occur.

The design methods showed that the permeability of the geotextile is very important. This permeability is also determined by the filter stability. At the moment the only permeability test harmonised in CEN is the index test on permittivity. This is probably sufficient for a product standard, but it is not when the aim is to base a design at the results of tests.

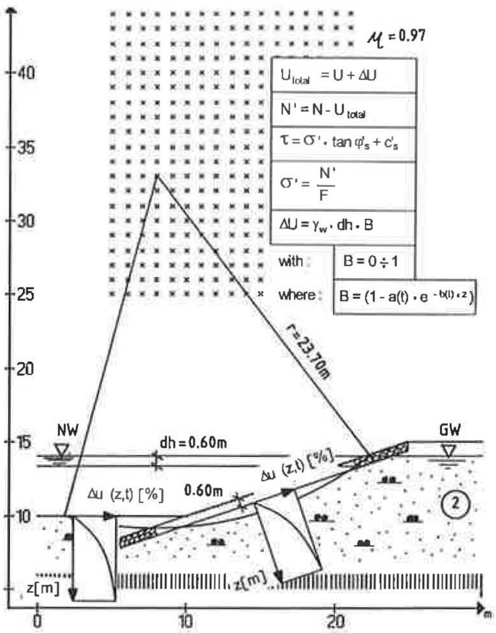

Fig. 17: Example slip circle calculation, showing the influence of an embedded toe.

References

Bezuijen, A., Klein Breteler, M. Pilarczyk, W. (1986) Large scale model tests on a block revetment placed on sand with a geotextile as separation layer. Proc. 3rd Int. Conf. on Geotextiles, Vienna.

Bezuijen, A. Klein Breteler. M. and Bakker, K.J. 1987. Design criteria for placed block revetments and granular filters. Proc. 2nd. COPEDEC, Beijing.

Bezuijen, A., Klein Breteler M., Burger, A.M., 1990. Placed block revetments. chapter in Coastal Protection, Balkema Rotterdam ISBN 90 6191 1273.

Bezuijen, A. Klein Breteler M., 1996. Design formulas for block revetments, To be published in J. of Waterway, Port, Coastal and Ocean Eng.

Coastal Engineering Reseach Center, 1984. Shore Protection Manual. U.S. Army Corps of Engineers.

CUR/TAW (1995) Design manual on pitched slope protection. Balkema ISBN 90 5410 606 9

Hjortnæs-Pedersen, A.G.I., Bezuijen, A., Best, H. 1987. Non-stationary flow under revetments using the Finite Element Method. Proc. 9th. Euro. conf. Soil Mechanics. and Found. Eng., Dublin. Aug/Sept.

Köhler, H.-J.; Bezuijen, A. Permeability influence of filter layers on the stability of rip-rap revetments under wave attack, Proc. of the 5th Int. Conf. of geotextiles, geomembranes and related products, Singapore, 5th - 9th September 1994, (SEAC-IGS), Vol. 2, ISBN 981-00-5822-5, p. 767-770.

Köhler, H.-J. Größe und Einflus von Porenwasserspannungen in Kanalböschungen, Aussprachete "Kanalausleidungen" in der Bundesanstalt für Wasserbau, Außenstelle Berlin, 10.-11. Mai 1995

Köhler, H.-J. Porenwasserdruckausbreitung im Boden, Messverfahren un Berechnungsansätze, Beitrag zum Seminar Messen in der Geotechnik '96, vom 22-23.02.1996, Universität Braunschweig.