CHAPTER 11

Application and stability criteria for rock and artificial units

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1 INTRODUCTION

The design tools given in this chapter on rubble mound structure armour layers are based on tests of schematised structures. Structures in prototype may differ substantially from the test-sections. Results, based on design tools, can therefore only be used in a conceptual design. The confidence bands, which are given for most formulae, support the fact that reality may differ from the mean curve. It is advised to perform physical model investigations for detailed design of all important rubble mound structures.

The armour layer of a rubble mound structure is one of the most important parts to design. Under-design of this layer may consequently lead to large damage or even failure of the structure. Three types of armour layers for seawalls and revetments will be described:

- rock layers
- concrete armour units
- berm type protection

The chapter will start with a description of some basic parameters used in design of armour layers and will finish with stepped and composite slopes.

2 BASIC PARAMETERS

Wave conditions are given principally by the incident wave height at the toe of the structure, H, usually as the significant wave height, H_s (average of the highest 1/3 of the waves) or H_{m0} (based on the spectrum); the mean or peak wave periods, T_m or T_p (based on statistical or spectral analysis); the angle of wave attack, ß, and the water depth, h.

The wave height distribution at deep water can be described by the Rayleigh distribution and in that case one characteristic value, for instance the significant wave height, describes the whole distribution. In shallow and depth limited water the highest waves break and in most cases the wave height distributions cannot longer be described by the Rayleigh distribution. In those situations the actual wave height distribution may be important to consider, or another characteristic value than the significant wave height. Characteristic values that are often used are the two-percent wave height, $H_{2\%}$, and the $H_{1/10}$, being the average of the highest ten percent of the waves. For a Rayleigh distribution the values $H_{2\%} = 1.4\ H_s$ and $H_{1/10} = 1.27\ H_s$ hold.

The influence of the wave period is often described as a wave length and related to the wave height, resulting in a wave steepness. The wave steepness, s, can be defined by using the deep water wave length,

$$s = \frac{2\pi H}{gT^2} \tag{1}$$

If the situation considered is not really in deep water (in most cases) the wave length at deep water and at the structure differ and, therefore, a fictitious wave steepness is obtained. In fact the wave steepness as defined in equation 1 describes a dimensionless wave period. Use of H_s and T_m or T_p in this equation gives a subscript to s, respectively s_{om} and s_{op}.

The most useful parameter describing wave action on a slope, and some of its effects, is the breaker parameter ξ:

$$\xi = \tan\alpha / \sqrt{s} \tag{2}$$

The breaker parameter has often been used to describe the form of wave breaking on a beach or structure, see chapters 2 and 8. It should be noted that different versions of this parameter are defined, reflecting the approaches of different researchers. In this chapter ξ_m and ξ_p are used when s is described by s_{om} or s_{op}.

The most important parameter which gives a relationship between the structure and the wave conditions is the stability number $H_s/\Delta D_{n50}$. The relative buoyant density in here is described by:

$$\Delta = \frac{\rho_r}{\rho_w} - 1 \tag{3}$$

where:

ρ_r = mass density of the rock or concrete (saturated surface dry mass density),

ρ_w = mass density of water.

The diameter used is related to the average mass of the rock and is called the nominal diameter:

$$D_{n50} = \left(\frac{M_{50}}{\rho_r}\right)^{1/3} \tag{4}$$

where:

D_{n50} = nominal diameter,

M_{50} = median mass of rock grading given by 50% on the mass distribution curve.

With the use of concrete units there is only one mass and no grading. This means that the nominal diameter and the stability number are described without the subscript 50, giving D_n and $H_s/\Delta D_n$, respectively.

3 ROCK ARMOUR LAYERS

3.1 Hudson formula

Many methods for the prediction of (rock) size of armour units designed for wave attack have been proposed in the last 50 years. Those treated in more detail here are the Hudson formula as used in SPM (1984) and the formulae derived by Van der Meer (1988a).

The Hudson formula written in its most simple form is:

$$H_s/\Delta D_{n50} = (K_D \cot\alpha)^{1/3} \tag{5}$$

K_D is a stability coefficient taking into account all other variables. K_D-values suggested for design correspond to a "no damage" condition where up to 5% of the armour units may be displaced. In the 1973 edition of the Shore Protection Manual the values given for K_D for rough, angular stone in two layers on a breakwater trunk were:

K_D = 3.5 for breaking waves,
K_D = 4.0 for non-breaking waves.

The definition of breaking and non-breaking waves is different from plunging and surging waves, which were described in chapters 2 and 8. A breaking wave in equation 5 means that the wave breaks due to the foreshore in front of the structure directly on the armour layer. It does not describe the type of breaking due to the slope of the structure itself.

No tests with random waves had been conducted and it was suggested to use H_s in equation 5. By 1984 the advice given was more cautious. The SPM now recommends $H = H_{1/10}$ being the average of the highest 10 percent of all waves. For the case considered above the value of K_D for breaking waves was revised downward from 3.5 to 2.0 (for non-breaking waves it remained 4.0). The effect of these two changes is equivalent to an increase in the unit stone mass required by a factor of about 3.5!

The main advantages of the Hudson formula are its simplicity, and the wide range of armour units and configurations for which values of K_D have been derived. The Hudson formula also has many limitations. Briefly they include:

- the use of regular waves only,
- no account taken in the formula of wave period or storm duration,
- no description of the damage level,
- the use of non-overtopped and permeable core structures only.

The use of $K_D \cot\alpha$ does not always best describe the effect of the slope angle. It may therefore be convenient to define a single stability number $H_s/\Delta D_{n50}$ without this $K_D \cot\alpha$.

3.2 Van der Meer formulae - deep water conditions

Based on earlier work of Thompson and Shuttler (1975) an extensive series of model tests was conducted at Delft Hydraulics (Van der Meer (1988a), Van der Meer (1987), Van der Meer (1988b)). These include structures with a wide range of core/underlayer permeabilities and a wider range of wave conditions. Two formulae were derived for plunging and surging waves, respectively, which are now known as the Van der Meer formulae:

for plunging waves:

$$\frac{H_s}{\Delta D_{n50}} = 6.2\, P^{0.18} \left(\frac{S}{\sqrt{N}}\right)^{0.2} \xi_m^{-0.5} \tag{6}$$

and for surging waves:

$$\frac{H_s}{\Delta D_{n50}} = 1.0\, P^{-0.13} \left(\frac{S}{\sqrt{N}}\right)^{0.2} \sqrt{\cot\alpha}\, \xi_m^{P} \tag{7}$$

where:

P = notional permeability factor
S = damage level
N = number of waves (storm duration)

The transition from plunging to surging waves can be calculated using a critical value of ξ_{mc}:

$$\xi_{mc} = \left[6.2\, P^{0.31} \sqrt{\tan\alpha}\right]^{\frac{1}{P+0.5}} \tag{8}$$

Equation 6 applies for $\xi_m < \xi_{mc}$ and equation 7 for larger values. For $\cot\alpha > 3$ the transition from plunging to surging waves only occurs for very low wave steepnesses (long waves). In that case the transition does not take place for ξ_{mc}, as described in equation 8, but for the same critical value as for $\cot\alpha = 3$. This gives for those gentle slopes the following critical value:

$$\xi_{mc} = \left[3.58\, P^{0.31}\right]^{\frac{1}{P+0.5}} \tag{9}$$

In those situations, where $\cot\alpha > 3$ and $\xi_m > \xi_{mc}$, one should use $\cot\alpha = 3$ in equation 7, also for calculation of ξ_m.

The maximum number of waves N which should be used in equations 6 and 7 is 7500. After this number of waves the structure more or less has reached an equilibrium. This means that damage for more than 7500 waves is found by using N = 7500 in the equations. For small numbers of waves (less than 1000) the equations give a slight over-prediction of damage, specially if N < 500. Van der Meer (1988a) gives a more accurate description for such a situation.

The wave steepness should lie between $0.005 < s_{om} < 0.06$ (almost the complete possible range). The mass density varied in the tests between 2000 kg/m^3 and 3100 kg/m^3, which is also the possible range of application.

Two parameters have not yet been described: the damage level S and the notional permeability factor P.

Damage level S

The damage to the armour layer can be given as a percentage of displaced rocks related to a certain area (the whole or a part of the layer). In this case, however, it is difficult to compare various structures as the damage figures are related to different totals for each structure. Another possibility is to describe the damage by the erosion area around still-water level. When this erosion area is related to the size of the rocks, a dimensionless damage level is presented which is independent of the size (slope angle and height) of the structure. This damage level is defined by:

$$S = \frac{A_e}{D_{n50}^2} \tag{10}$$

where:

S = damage level
A_e = erosion area around still-water level

A plot of a structure with damage is shown in Figure 1, the damage level taking both settlement and displacement into account. A physical description of the damage, S, is the number of squares with a side D_{n50} that fit into the erosion area. Another description of S is the number of cubic stones with a side of D_{n50} eroded within a D_{n50}-wide strip of the structure. The actual number of stones eroded within this strip can be more or less than S, depending on the porosity, the grading of the armour rocks and the shape of the rocks. Generally the actual number of rocks eroded in a D_{n50}-wide strip is equal to 0.7 to 1 times the damage S.

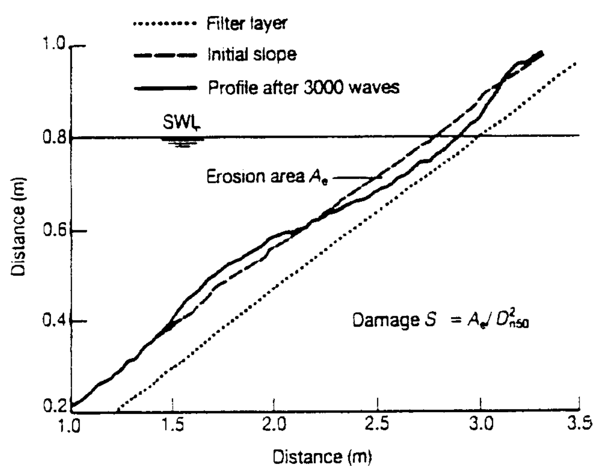

Figure 1 Damage S based on erosion area A_e

The limits of S depend mainly on the slope angle of the structure. For a two-diameter thick armour layer the values in Table 1 can be used. The initial damage of S = 2-3 is according to the criterion of the Hudson formula which gives 0-5% damage. Failure is defined as exposure of the filter layer. For S-values higher than 15-20 the deformation of the structure results in an S-shaped profile.

slope	**initial damage**	**intermediate damage**	**failure (under layer visible)**
1:1.5	2	3-5	8
1:2	2	4-6	8
1:3	2	6-9	12
1:4	3	8-12	17
1:6	3	8-12	17

Table 1 Design values of S for a two-diameter thick armour layer

Notional permeability factor P

The permeability of the structure has influence on the stability of the armour layer. This depends on the size of filter layers and core. The more permeable the structure underneath the armour layer, the more water can penetrate into the structure during wave run-up and the smaller the forces on armour units will be, both in the wave run-up and run-down phase. Larger permeability will give a more stable structure. The permeability of a structure with respect to stability can be given by a notional permeability factor P. Examples of P are shown in Figure 2, based on the work of Van der Meer (1988a).

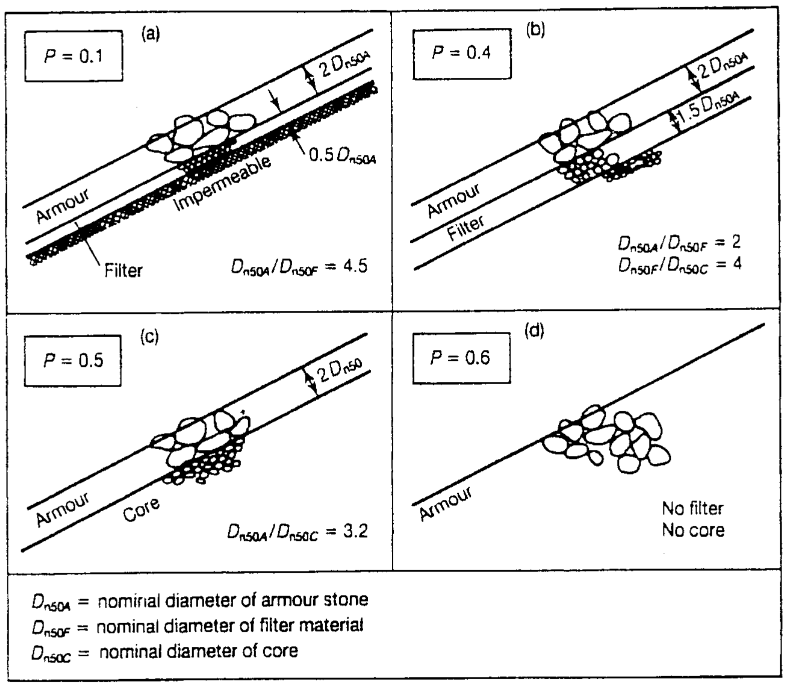

Figure 2 Notional permeability factor P for various structures

The lower limit of P is an armour layer with a thickness of two diameters on an impermeable core (sand or clay) and with only a thin filter layer. These situations often occur for seawalls and revetments. This lower boundary is given by P = 0.1. The upper limit of P is given by a homogeneous structure, which consists only of armour rocks. In that case P = 0.6. Two other values are shown in Figure 2 and each particular structure should be compared with the given structures in order to make an estimation of the P-factor. It should be noted that P is not a measure of porosity!

The estimation of P from Figure 2 for a particular structure must, more or less, be based on engineering judgement. Although the exact value may not precisely be determined, a variation of P around the estimated value may well give an idea about the importance of the permeability.

The permeability factor P can also be determined by using a numerical (pc)-model that can give the volume of water that penetrates through the armour layer during run-up. Calculations should be done for the structures with P = 0.5 and 0.6 (see Figure 2) and for the actual structure. As P = 0.1 gives no penetration at all, a graph can be made with P versus penetrated volume of water. The calculated volume for the actual structure gives then in the graph the P-value. The procedure has been described in more detail in Van der Meer (1988a).

With numerical models developed by Kobayashi and Wurjanto (1990) and Van Gent (1995)) the penetration of water during run-up can be calculated fairly easy.

Reliability of formulae

The reliability of the formulae depends on the differences due to random behaviour of rock slopes, accuracy of measuring damage and curve fitting of the test results. The reliability of equations 6 and 7 can be expressed by giving the coefficients 6.2 and 1.0 in the equations a normal distribution with a certain standard deviation. The coefficient 6.2 can be described by a standard deviation of 0.4 (variation coefficient 6.5%) and the coefficient 1.0 by a standard deviation of 0.08 (8%). These values are significantly lower than that for the Hudson formula at 18% for K_D (with mean K_D of 4.5). With these standard deviations it is simple to include 90% or other confidence bands.

Equations 6 – 9 are more complex than the Hudson formula. They include also the effect of the wave period, the storm duration, the permeability of the structure and a clearly defined damage level. This may cause differences with the Hudson formula, but they are more precise. The complexity of the formulae may easily be overcome by programming the formulae into a spreadsheet, or by using Delft Hydraulics' users friendly pc-program BREAKWAT.

Application of formulae

For a good design it is required to perform a sensitivity analysis for all parameters in the equations. The deterministic procedure is to make design graphs where one parameter is evaluated. Three examples are shown in Figures 3 - 5. Two give a wave height versus breaker parameter plot, which shows the influence of both wave height and wave steepness (the wave climate). The other shows a wave height versus damage plot, which is comparable with the conventional way of presenting results of model tests on stability. The same kind of plots can be derived from equations 6 - 9 for other parameters, see Van der Meer (1988b), and by the use of the earlier mentioned pc-program.

The parameter which influence is shown in Figure 3 is the damage level S. Four damage levels are shown: S = 2 (start of damage), S = 5 and 8 (intermediate damage) and S = 12 (filter layer visible). The structure itself is described by: D_{n50} = 1.0 m (M_{50} = 2.6 t), Δ = 1.6, $\cot\alpha$ = 3.0, P = 0.5 and N = 3000.

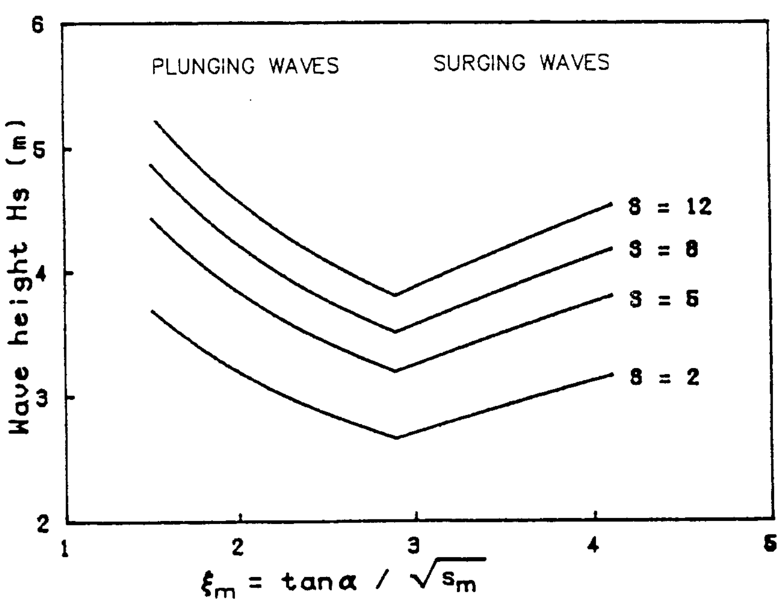

Figure 3 Wave height versus breaker parameter; influence of damage level

The influence of the notional permeability factor P is shown in Figure 4. Four values are shown: P = 0.1 (impermeable core), P = 0.3 (some permeable core), P = 0.5 (permeable core) and P = 0.6 (homogeneous structure). The structure itself is described by: D_{n50} = 1.0 m (M_{50} = 2.6 t), Δ = 1.6, cotα = 3.0 and N = 3000.

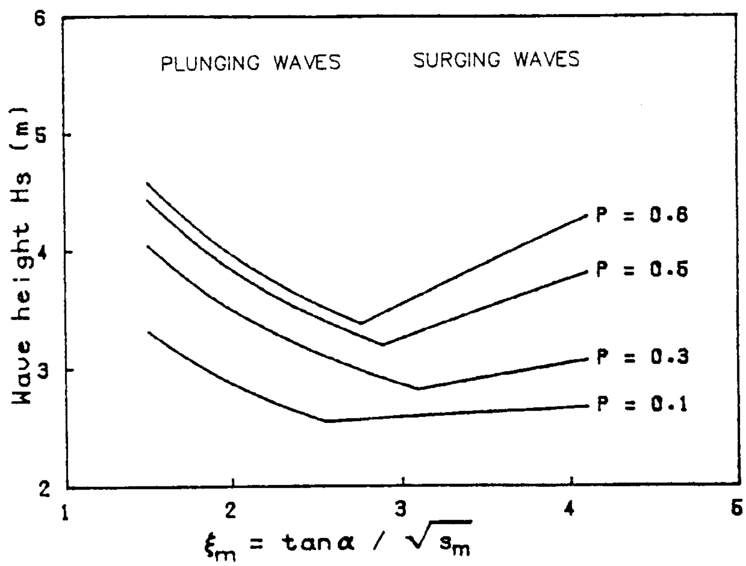

Figure 4 Wave height versus breaker parameter; influence of permeability

Damage curves are shown in Figure 5. Two curves are given, one for a slope angle with cotα = 2.0 and a wave steepness of s_{om}= 0.02 and one for a slope angle with cotα = 3.0 and a wave steepness of 0.05. If the extreme wave climate is known, graphs as Figure 5 are very useful to determine the stability of the armour layer of the structure. Low values of S could be accepted for storms with small return periods and larger values for more extreme storms with (very) large return periods. Figure 5 shows also the 90% confidence levels which give a good idea about the possible variation in stability. This variation should be taken into account by the designer of a structure.

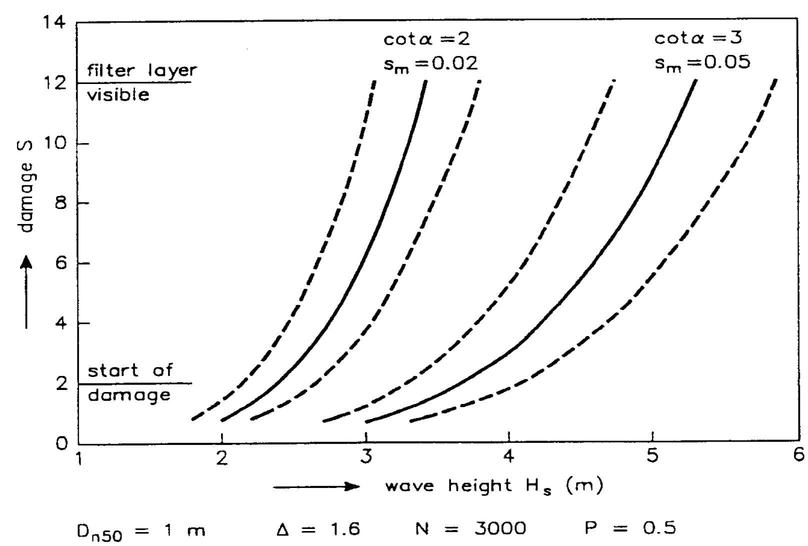

Figure 5 Wave height versus damage

An estimation of the damage profile of a straight rock slope can be made by use of equations 6 and 7 and some additional relationships for the profile. The profile can be schematised to an erosion area around still-water level, an accretion area below still-water level and for gentle slopes a berm or crest above the erosion area. The transitions from erosion to accretion, etc. can be described by heights measured from still-water level, see Figure 6, which gives a measured profile from a test. The heights are respectively h_r, h_d, h_m and h_b.

The relationships for the height parameters were based on the tests described by Van der Meer (1988a) and will not be given here. The assumption for the profile is a spline through the points given by the heights and with an erosion (and accretion) area according to the stability equations. The method is only applicable for straight slopes and is a part of Delft Hydraulics' program BREAKWAT. Figure 7 gives an example of a calculated damaged profile.

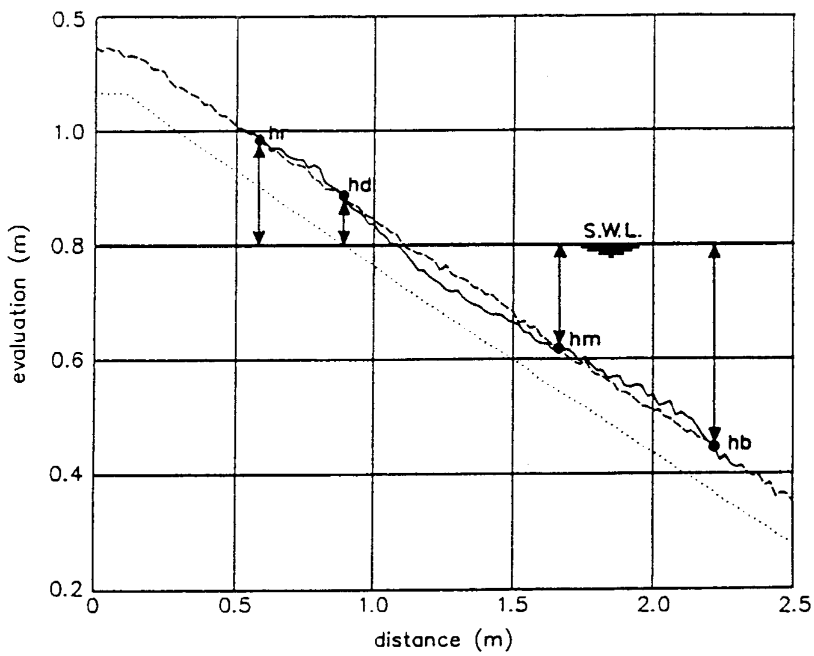

Figure 6 Damage profile (as measured) of a statically stable rock slope

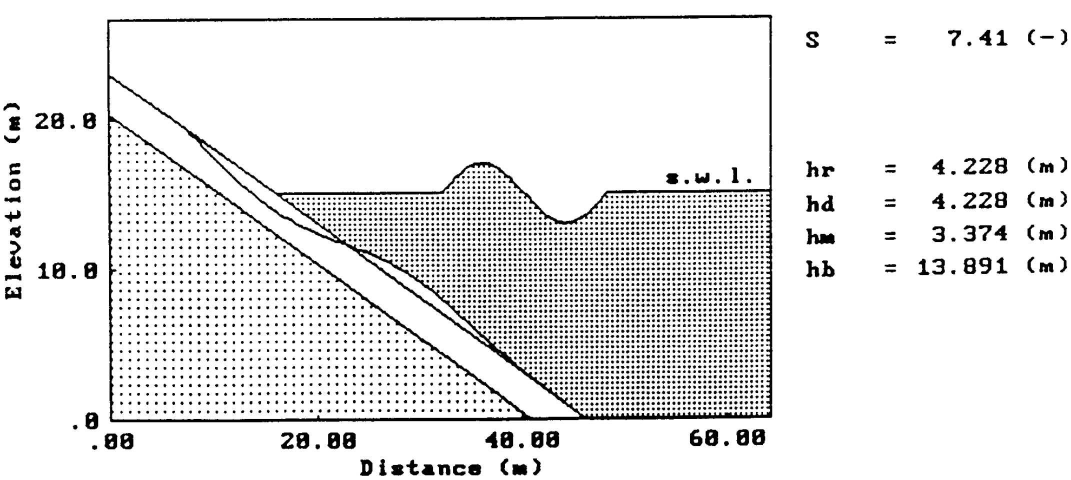

Figure 7 Calculated damage profile of a statically stable rock slope

Probabilistic approach

A deterministic design procedure is followed if the stability equations are used to produce design graphs as H_s versus ξ_m and H_s versus damage (see Figs. 3 - 5) and if a sensitivity analysis is performed. Another design procedure is the probabilistic approach. Equations 6 and 7 can be rewritten to so-called reliability functions and all the parameters can be assumed to be stochastic with an assumed distribution. Here, one example of the approach will be given. A more detailed description can be found in Van der Meer (1988b).

The structure parameters with the mean value, distribution type and standard deviation are given in Table 2. These values were used in a level II first-order second-moment (FOSM) with approximate full distribution approach (AFDA) method. With this method the probability that a certain damage level would be exceeded in one year was calculated. These probabilities were used to estimate the probability that a certain damage level would be exceeded in a certain lifetime of the structure.

parameter	distribution	average	standard deviation
D_n	normal	1.0	0.03
Δ	normal	1.6	0.05
$\cot\alpha$	normal	3.0	0.15
P	normal	0.5	0.05
N	normal	3000	1500
H_s	Weibull	B=0.3	C=2.5
FH_s	normal	0	0.25
s_{om}	normal	0.04	0.01
a (Eq. 6)	normal	6.2	0.4
b (Eq. 7)	normal	1.0	0.08

Table 2 Parameters used in Level II probabilistic computations

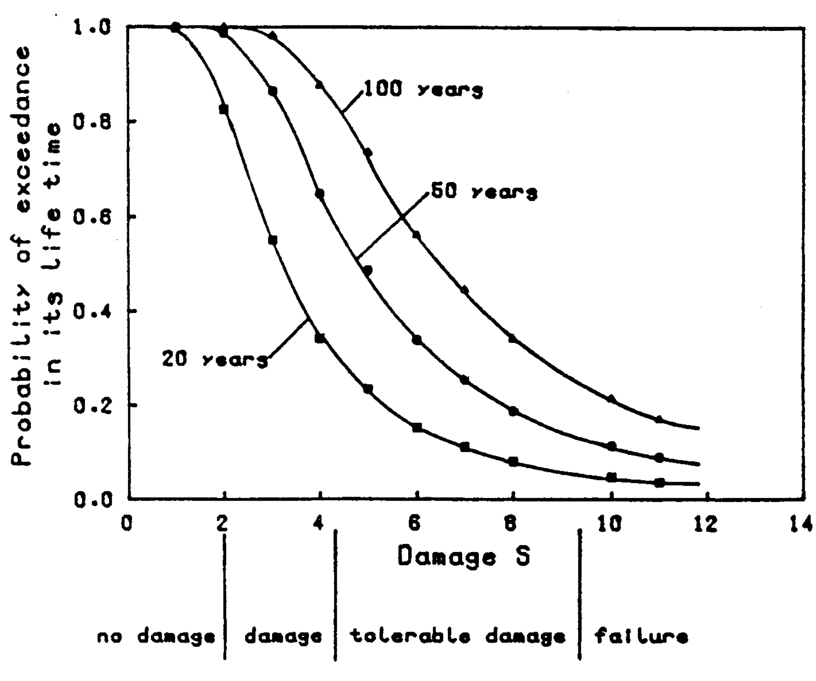

Figure 8 Probability of exceedance of the damage level S in the lifetime of the structure

The parameter FH_s represents the uncertainty of the wave height at a certain return period. The wave height itself is described by a two-parameter Weibull distribution. The coefficients a and b take into account the reliability of the formulae, including the random behaviour of rock slopes.

The final results are shown in Figure 8 where the damage S is plotted versus the probability of exceedance in the lifetime of the structure. From this graph it follows that start of damage (S = 2) will certainly occur in a lifetime of 50 years. Tolerable damage (S = 5-8) in the same lifetime will occur with a probability of 0.2 - 0.5. The probability that the filter layer will become visible (failure) is less than 0.1. Probability curves as shown in Figure 8 can be used to make a cost optimisation for the structure during its lifetime, including maintenance and repair at certain damage levels, see for instance Vrijling et al. (1998).

Probabilistic level II calculations as described above can easily be performed if the required computer programs are available. As this is not the case for many designers of breakwaters, it may be easier to use a level I approach. This means that one applies design formulae with partial safety factors to account for the uncertainty of the relevant parameters. Most guidelines and building codes are based on a level I approach. The determination of the partial safety factors is again based on level II calculations, but this work has to be done by the composers of the guideline and not by the user.

The safety of rubble mound breakwaters has been described with a level I approach (partial safety factors) in PIANC (1993). Two safety factors appeared to be enough to describe the uncertainty: one for the wave height and one for all other parameters together. Partial safety factors for most formulae in this chapter are given in PIANC (1993).

3.3 Shallow water conditions

Up to now the significant wave height H_s has been used in the stability equations. In shallow water conditions the distribution of the wave heights deviate from the Rayleigh distribution (truncation of the curve due to wave breaking). Further tests on a 1:30 sloping and depth limited foreshore by Van der Meer (1988a) showed that $H_{2\%}$ was a better value for design on depth limited foreshores than the significant wave height H_s, i.e. that the stability of the armour layer in depth limited situations is better described by a higher characteristic value of the wave height distribution $H_{2\%}$ than by H_s.

Equations 6 and 7 can be re-arranged with the known ratio of $H_{2\%}/H_s$ for a Rayleigh distribution. The equations become:

for plunging waves:

$$\frac{H_{2\%}}{\Delta D_{n50}} = 8.7\, P^{0.18} \left(\frac{S}{\sqrt{N}}\right)^{0.2} \xi_m^{-0.5} \qquad (11)$$

and for surging waves:

$$\frac{H_{2\%}}{\Delta D_{n50}} = 1.4\, P^{-0.13} \left(\frac{S}{\sqrt{N}}\right)^{0.2} \sqrt{\cot\alpha}\; \xi_m^{P} \qquad (12)$$

Equations 11 and 12 take into account the effect of depth limited situations. A safe approach, however, is to use equations 6 and 7 with H_s. In that case the truncation of the wave height exceedance curve due to wave breaking is not taken into account which can be assumed as a safe approach. If the wave heights are Rayleigh distributed, equations 11 and 12 give the same results as equations 6 and 7, as this is caused by the known ratio of $H_{2\%}/H_s = 1.4$. For depth limited conditions the ratio of $H_{2\%}/H_s$ will be smaller, and one should obtain information on the actual value of this ratio. On the other hand, information about the actual wave heights in depth limited situations is small. Very often the wave heights that are calculated are based on energy, i.e. H_s is based on m_0. In those situations H_s (actually the $H_{1/3}$) differs from H_{m0}, which means that it is very difficult to find a good estimation of $H_{2\%}$. Only if $H_{2\%}$ is really known equations 11 and 12 can be used; otherwise equations 6 and 7 give a safer approach.

3.4 Effects of armour shape and wide gradings

The effects of armour shape on stability has been described by Latham et al. (1988) and Van der Meer et al. (1996). Latham et al. (1988) tested five classes of rock with different shape classifications such as fresh, equant, semi-round, very round and tabular. The damage to the test sections using each of the armour shapes tested was compared with damage calculated using equations 6 and 7. As expected, very round rock suffered more damage than any of the other shapes. The performances of the fresh and equant rock were broadly similar. Surprisingly, the tabular rock exhibited higher stability than other armour shapes.

The coefficient 6.2 in equation 6 and 1.0 in equation 7 were used to describe the shape effects. These coefficients are summarised in Table 3.

rock shape class	plunging waves; alternative for coefficient 6.2	surging waves; alternative for coefficient 1.0
fresh	6.32	0.81
equant	6.24	1.09
semi-round	5.96	0.99
very round	5.88	0.81
tabular	6.72	1.30

Table 3 Suggested coefficients for "non-standard" armour shapes in equations 6 and 7

The stability of rock armour of (very) wide grading has been investigated by Allsop (1990). Model tests on a 1:2 slope with an impermeable core were conducted to identify whether the use of rock armour of grading wider than $D_{85}/D_{15} = 2.25$ will lead to armour performance substantially different from that predicted by equations 6 and 7. The test results confirmed the validity of these equations for rock of narrow to wide gradings ($D_{85}/D_{15} < 2.25$). Very wide gradings, such as $D_{85}/D_{15} = 4.0$, may in general suffer slightly more damage than predicted for narrower gradings. On any particular structure, there will be greater local variations in the sizes of the individual rocks in the armour layer than for narrow gradings. This will increase spatial variations of the damage, giving a higher probability of severe local damage. Considerable difficulties will be encountered in measurement and checking such wide gradings. An other problem might be with these wide gradings that the very big stones do not anymore fit into a two-layer system based on D_{n50}. If very wide gradings are applied one should consider a layer thickness of 3 D_{n50}. More information can be found in above mentioned references and in Allsop and Jones (1993).

Van der Meer et al. (1996) have tested the influence of rock shape and grading on stability of a low crested structure. The crest was two diameters above the water level. Six rock shapes and gradings were carefully selected and tested. The rock shape was also determined by the ratio

L/D. Here L is the largest dimension and D the smallest. L/D=1 is a sphere or cube and L/D=3 is a fairly long and/or flat stone. In many cases values of L/D>2 are not permitted in construction specifications, which in reality is difficult to obtain. Furthermore, such requirements are hardly based on testing.

Figure 9 gives a part of the results of the testing. Data points of three rock classes are given, all with a nominal diameter D_{n50} = 0.035 m and a grading of D_{85}/D_{15} = 1.75. The gradings had only different quantities of long and flat rock. Rock type 5 in Figure 9 had 80% of the rock with a dimension L/D>2 and even 40% with L/D>3. Figure 9 shows clearly that the length-width ratio L/D shows no influence on stability.

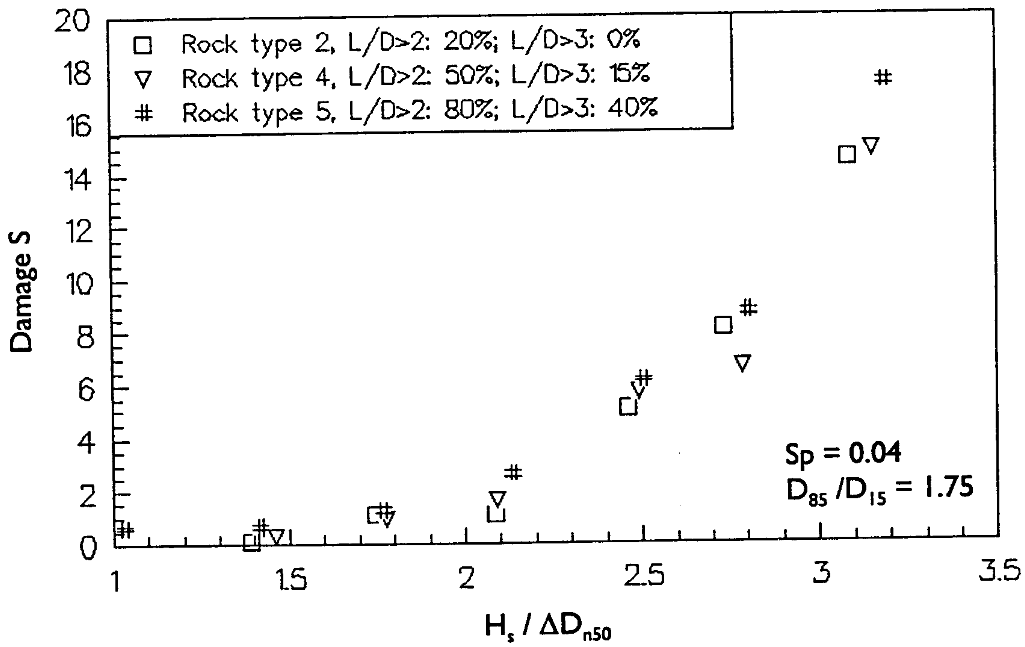

Figure 9 Influence of rock shape L/D on stability

The grading itself has only minor influence. It has no influence if the grading D_{85}/D_{15} is smaller than about 2. The material factors L/D and grading gave hardly cause for the rejection of amounts of rock during construction. Hence, in future it is recommended to be less strict as to the requirements for constructing (low) breakwaters than was customary up to now. In particular, for the differences in length/width ratios of the rock this will yield gains in time and material to be used. This will benefit both principal and contractor.

4 CONCRETE ARMOUR LAYERS

The Hudson formula (equation 5) was given in section 3.1 with K_D-values for rock. The SPM (1984) gives a table with values for a large number of concrete armour units. The most important ones are: K_D= 6.5 and 7.5 (breaking/non-breaking waves) for cubes, K_D = 7.0 and 8.0 for tetrapods and K_D = 15.8 and 31.8 for Dolosse. For other units one is referred to SPM (1984).

Extended research by Van der Meer (1988c) on breakwaters with concrete armour units was based on the governing variables found for rock stability. The research was limited to only one cross-section (i.e. one slope angle and permeability) for each armour unit.

Therefore the slope angle, cotα, and consequently the breaker parameter, ξ_m, is not present in the stability formulae developed on the results of the research. The same holds for the notional permeability factor, P. This factor was P = 0.4.

Breakwaters with armour layers of interlocking units are generally built with steep slopes in the order of 1:1.5. Therefore this slope angle was chosen for tests on cubes and tetrapods. Accropode are generally built on a slope of 1:1.33, and this slope was used for tests on accropode. Cubes were chosen as these elements are bulky units which have good resistance against impact forces. Tetrapods are widely used all over the world and have a fair degree of interlocking. Accropode were chosen as these units can be regarded as the latest development, showing high interlocking, strong elements and a one-layer system. A uniform 1:30 foreshore was applied for all tests. Only for the highest wave heights which were generated, some waves broke due to depth limited conditions.

Damage to rock armour was measured by considering the eroded area around the water level. It is not usual to measure profiles for concrete armour layers. Very often damage is based on an actual number of units. Therefore, another definition has been suggested for damage to concrete armour units. Damage there can be defined as the relative damage, N_o, which is the actual number of units (displaced, rocking, etc.) related to a width (along the longitudinal axis of the structure) of one nominal diameter D_n. For cubes D_n is the side of the cube, for tetrapods D_n = 0.65 D, where D is the height of the unit, for accropode D_n = 0.7D and for Dolosse D_n = 0.54 D (with a waist ratio of 0.32).

An extension of the subscript in N_o can give the distinction between units displaced out of the layer, units rocking within the layer (only once or more times), etc. In fact the designer can define his own damage description, but the actual number is related to a width of one D_n. The following damage descriptions are used in this chapter:

- N_{od} = units displaced out of the armour layer (hydraulic damage);
- N_{or} = rocking units;
- N_{omov} = moving units, $N_{omov} = N_{od} + N_{or}$.

The definition of N_{od} is comparable with the definition of S, although S includes displacement and settlement, but does not take into account the porosity of the armour layer. Generally S is about twice N_{od}. Further, N_{od} can be easily related to a percentage of damage. If the number of units in a cross-section is known with a length of 1 D_n, the percentage of damage to a structure is simply the ratio of N_{od} and this number. Suppose a damage of N_{od}=0.3 and a total of 25 units in a cross-section. The damage percentage becomes then 0.3/25*100% = 1.2%.

As only one slope angle was investigated, the influence of the wave period should not be given in formulae including ξ_m, as this parameter includes both wave period (steepness) and slope angle. The influence of wave period, therefore, will be given by the wave steepness s_{om}.

Formulae for cubes and tetrapods

Final formulae for stability of concrete units include the relative damage level N_{od}, the number of waves N, and the wave steepness, s_{om}. The formula for cubes is given by:

$$\frac{H_s}{\Delta D_n} = \left(6.7 \frac{N_{od}^{0.4}}{N^{0.3}} + 1.0 \right) s_{om}^{-0.1} \qquad (13)$$

For tetrapods:

$$\frac{H_s}{\Delta D_n} = \left(3.75 \frac{N_{od}^{0.5}}{N^{0.25}} + 0.85 \right) s_{om}^{-0.2} \qquad (14)$$

For the no-damage criterion $N_{od} = 0$, equations 13 and 14 reduce to:

$$\frac{H_s}{\Delta D_n} = 1.0\, s_{om}^{-0.1} \tag{15}$$

$$\frac{H_s}{\Delta D_n} = 0.85\, s_{om}^{-0.2} \tag{16}$$

No damage at all is a very strict criterion and armour layers designed on this criterion will get large concrete units. For rock layers some settlement and small displacement is included in the "start of damage" definition S=2-3. For N_{od}=0.5 a similar situation is found and this is a more economical criterion than no damage at all.

Equations 13 and 14 give decreasing stability with increasing wave steepness. This is similar to the plunging area for rock layers. Due to the steep slopes used, no transition was found to plunging waves. De Jong (1996), however, analysed more data on tetrapods from tests performed at Delft Hydraulics and he found a similar transition. His formula for plunging waves should be considered together with equation 14, which now acts for surging waves only, and becomes:

$$\frac{H_s}{\Delta D_n} = \left(8.6 \left(\frac{N_{od}}{\sqrt{N}} \right)^{0.5} + 3.94 \right) s_{om}^{0.2} \tag{17}$$

Both equations 14 and 17 are shown in Figure 10 for three different damage levels of N_{od}. It is possible, and it might even be expected, that a similar transition can be found for cubes. No data are available, however, on that aspect.

De Jong (1996) also investigated the influence of crest height and packing density on stability of tetrapods. Equation 17 regards to an almost non-overtopped structure. Stability increases if the crest height decreases. With the crest freeboard defined by R_c, he found that the stability number in equation 17 could be increased by a factor with respect to a lower crest height. This factor is given by:

$$1 + 0.17 e^{-0.61 \frac{R_c}{D_n}} \tag{18}$$

It might be possible that this factor could also be applied to stability numbers calculated with equations 13 and 14, but more research is required to prove that. The packing density has been described in chapter 9 and k_t is given there as the layer thickness coefficient. The number of units per square meter gives the actual packing density and relies on k_t. The normal packing density used in the tests amounted to $k_t = 1.02$. Lower packing densities of $k_t = 0.95$ and 0.88 were used to investigate the influence of k_t. It indeed appeared that a lower packing density leads to lower stability. In the equation this could be incorporated by making the coefficient 3.94 in equation 17 dependent on k_t. The total stability formula (for plunging waves) for tetrapods, including crest height and packing density becomes:

$$\frac{H_s}{\Delta D_n} = \left(8.6 \left(\frac{N_{od}}{\sqrt{N}} \right)^{0.5} + 2.64 k_t + 1.25 \right) * s_{om}^{0.2} * \left(1 + 0.17 e^{-0.61 \frac{R_c}{D_n}} \right) \quad (19)$$

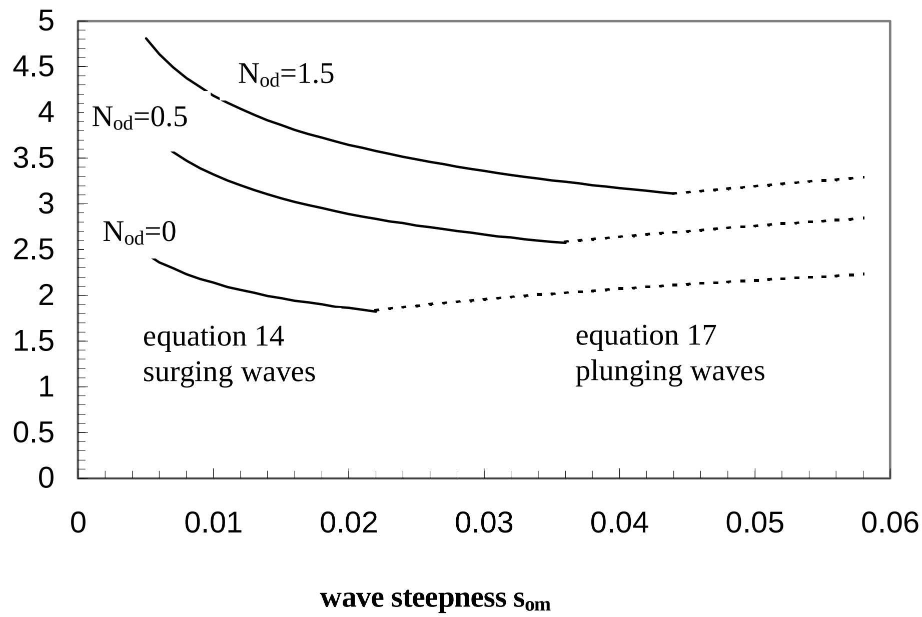

Figure 10 Stability number as a function of wave steepness for tetrapods with N=1000

Formulae for accropode

The storm duration and wave period showed no influence on the stability of accropode and the "no damage" and "failure" criteria were very close. The stability, therefore, can be described by two simple formulae:

start of damage, $N_{od} = 0$:

$$\frac{H_s}{\Delta D_n} = 3.7 \quad (20)$$

failure, $N_{od} > 0.5$:

$$\frac{H_s}{\Delta D_n} = 4.1 \quad (21)$$

Comparison of equations 120and 21 shows that start of damage and failure for accropode are very close, although at very high $H_s/\Delta D_n$-numbers. It means that up to a high wave height accropode are completely stable, but after the initiation of damage at this high wave height, the structure will fail progressively. Therefore, it is recommended that a safety coefficient for design should be used of about 1.5 on the $H_s/\Delta D_n$-value. This means that for the design of accropode one should use the following formula, which is close to design values of cubes and tetrapods:

$$\frac{H_s}{\Delta D_n} = 2.5 \qquad (22)$$

This is also a value that is used by Sogreah to design accropode layers. Although accropode may fail in a progressive way for high wave heights, use of a safety coefficient changes it to a safe structure with has the following advantage with respect to other units. If the design wave height for cubes or tetrapods is under-estimated, a higher wave height than expected, may lead ot increased and undesirable damage. If the wave height for an accropode layer is under-estimated up to 50%, in fact nothing happens. No damage is expected as the stability number is still lower than the one for start of damage.

Reliability of formulae

The reliability of equations 13-21 can be described with a similar procedure as for rock. The coefficients 3.7 and 4.1 in equations 20 and 21 for accropode can be considered as stochastic variables with a standard deviation of 0.2. The procedure for equations 13-17 is more complicated. Assume a relationship:

$$\frac{H_s}{\Delta D_n} = a\, f\, (N_{od}, N, s_{om}) \qquad (23)$$

The function $f(N_{od}, N, s_{om})$ is given in equations 13-17. The coefficient, a, can be regarded as a stochastic variable with an average value of 1.0 and a standard deviation. From analysis it followed that this standard deviation is $\sigma = 0.10$ for both formulae on cubes and tetrapods.

Equations 6-8 and 13-22 describe the stability of rock, cubes, tetrapods and accropode. A comparison of stability is made in Figure 11 where for all units curves are shown for two damage levels: "start of damage" (S = 2 for rock and $N_{od} = 0$ for concrete units) and "failure" (S = 8 for rock, $N_{od} = 2$ for Cubes, $N_{od} = 1.5$ for tetrapods and $N_{od} > 0.5$ for accropode). The curves are drawn for N = 3000 and are given as the stability number $H_s/\Delta D_n$ versus the wave steepness, s_{om}.

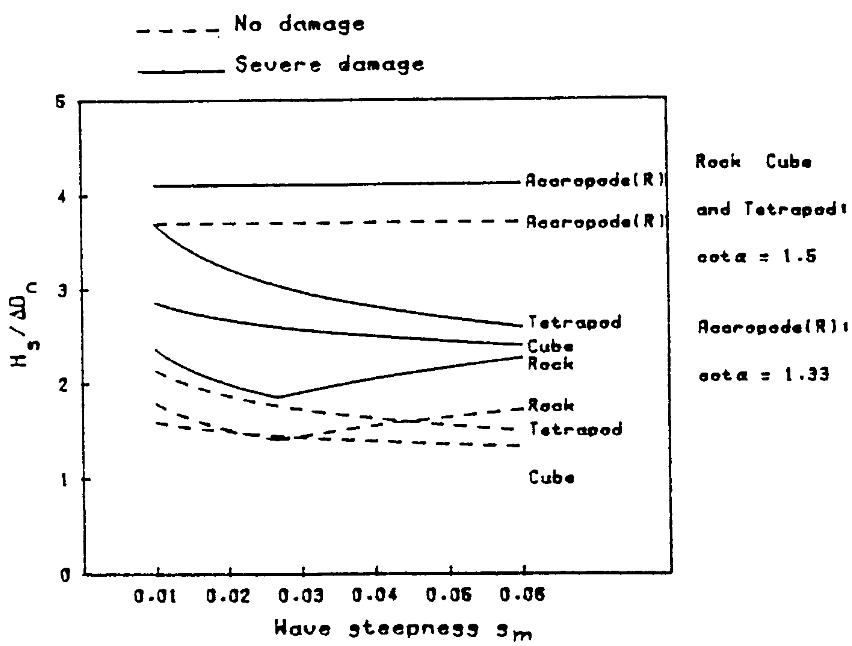

Figure 11 Comparison of stability of rock, cubes, tetrapods and accropode as a function of wave steepness

From Figure 11 the following conclusions can be drawn:

- start of damage for rock and cubes is almost the same. This is partly due to a more stringent definition of "no damage" for Cubes ($N_{od} = 0$). The damage level S = 2 for rock means that a little displacement is allowed (according to Hudson's criterion of "no damage", however).
- the initial stability of tetrapods is higher than for rock and cubes and the initial stability of accropode is much higher.
- failure of the slope is reached first for rock, then cubes, tetrapods and accropode. The stability at failure (in terms of $H_s/\Delta D_n$-values) is closer for tetrapods and accropode than at the initial damage stage.

A similar graph, but now for a fixed wave steepness of s_{om}=0.04 and a fixed mass of 10 ton is given in Figure 12. It is clear that cubes are a little less stable than tetrapods and that accropode are much more stable, but with progressive failure, however. The left vertical line gives the design value for accropode and is close to design values for tetrapods with N_{od} around 0.5.

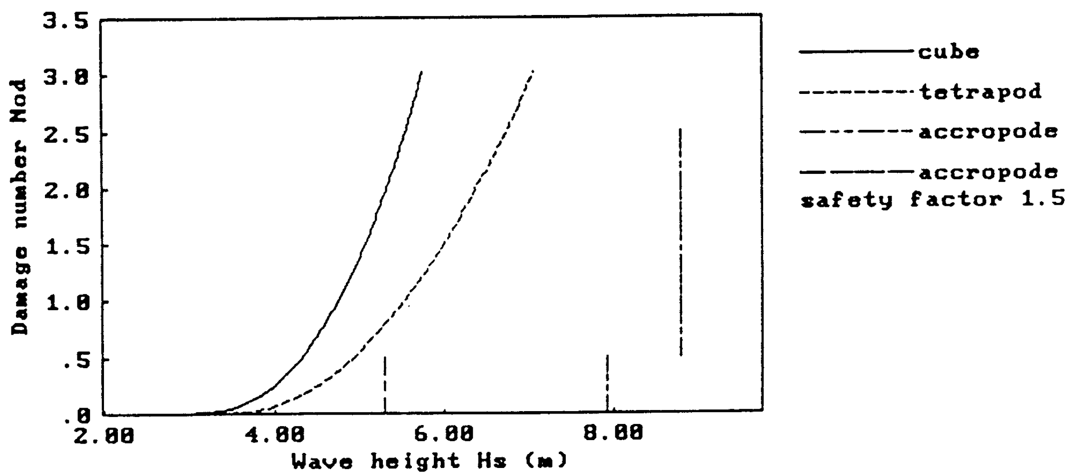

Figure 12 Comparison of 10 ton concrete units with s_{om}=0.04 and N=3000

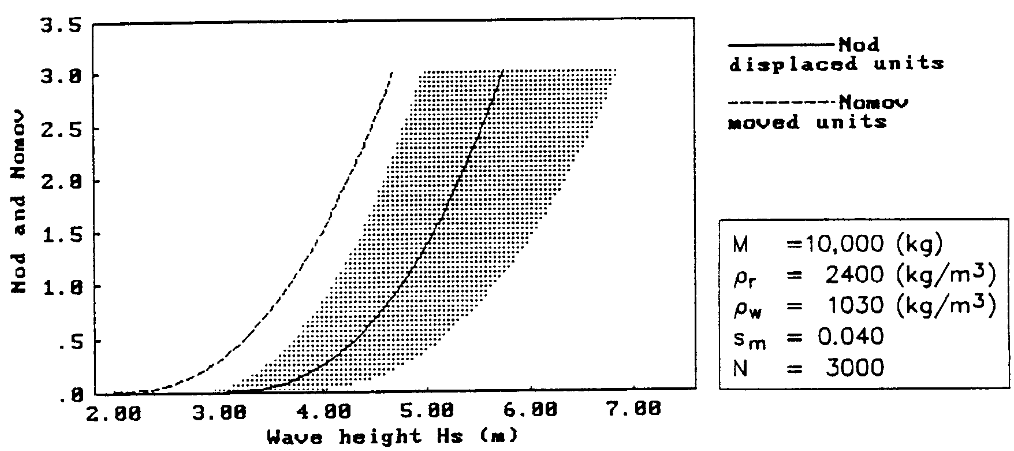

Figure 13 Wave height – damage curve for cubes with 90% confidence bands

Another useful graph that directly can be derived from stability formulae 13 and 14 is the wave height - damage graph for one type. Figure 13 gives an example for cubes and gives the 90% confidence bands too, using the standard deviations described before.

Up to now damage to a concrete armour layer was defined as units displaced out of the layer (N_{od}). Large concrete units, however, can break due to limitations in structural strength. After the failures of the large breakwaters in Sines, San Ciprian, Arzew and Tripoli, a lot of research all over the world was directed to the strength of concrete armour units. The results of that research will not be described here.

In case where the structural strength may play a role, however, it is interesting to know more than only the number of displaced units. The number of rocking units, N_{or}, or the total number of moving units, N_{omov}, may give an indication of the possible number of broken units. A (very) conservative approach is followed if one assumes that each moving unit results in a broken unit. The lower limits (only displaced units) for cubes and tetrapods are given by equations 13 and 14. The upper limits (number of moving units) were derived by Van der Meer and Heydra (1991). The equations for the number of moving units are:

For cubes:

$$\frac{H_s}{\Delta D_n} = \left(6.7 \frac{N_{omov}^{0.4}}{N^{0.3}} + 1.0\right) s_{om}^{-0.1} - 0.5 \qquad (24)$$

For tetrapods:

$$\frac{H_s}{\Delta D_n} = \left(3.75 \frac{N_{omov}^{0.5}}{N^{0.25}} + 0.85\right) s_{om}^{-0.2} - 0.5 \qquad (25)$$

The equations are very similar to equations 13 and 14, except for the coefficient -0.5. In a wave height - damage graph the result is a curve parallel to the one for N_{od}, but shifted to the left, see Figure 13. For armour layers with large concrete units the actual number of broken units will probably lie between the curve of N_{od} (equations 13 and 14) and N_{omov} (equations 24 and 25).

Dolosse

Holtzhausen and Zwamborn (1992) investigated the stability of dolosse in a basic way, similar to the research on cubes, tetrapods and accropode, described above. Damage was defined as units displaced more than one diameter and rocking or movements were not taken into account. The aspect of rocking (and breakage) should be considered for heavy dolosse, say heavier than 10-15 t.

After some rewriting with respect to the damage number N_{od}, which is used in this cahpter, the stability formula for Dolosse becomes according to Holtzhausen and Zwamborn (1992):

$$N_{od} = 6250 \left[\frac{H_s}{\Delta^{0.74} D_n}\right]^{5.26} s_{op}^{3}\, w_r^{20 s_{op}^{0.45}} + E \qquad (26)$$

where:

w_r = the waist ratio of the dolos

E = error term

The waist ratio w_p is a measure to account for possible breakage: a higher waist ratio gives a stronger dolos and should be used for relative severe wave attack. The applicable range for the waist ratio is 0.33 - 0.40.

The error term E describes the reliability of the formula. It is assumed to be normally distributed with a mean of zero and a standard deviation of:

$$\sigma(E) = 0.01936 \left[\frac{H_s}{\Delta^{0.74} D_n} \right]^{3.32} \qquad (27)$$

As equation 26 is a power curve, the no-damage criterion N_{od} = 0 cannot be substituted. According to Holtzhausen and Zwamborn (1992) no damage should be described as N_{od} = 0.1. The test duration was between 2000 and 3000 waves (one hour in the model). The influence of storm duration was not investigated, which means that equation 26 holds for storm durations in the same order as the tests.

Breakage of units has not been treated in this chapter. Some relevant references on this topic are: Burcharth et al. (1991), Burcharth and Liu (1992), Ligteringen et al. (1992), Scott et al. (1990), Van der Meer and Heydra (1991) and Van Mier and Lenos (1991).

5 BERM TYPE PROTECTIONS

Statically stable structures can be described by the damage parameters S or N_{od} as described above. If the deformation becomes too large one can speak of dynamically stable structures which can be described by a profile.

Berm breakwaters can also be used as seawall or revetment and are a special type of structure. A berm breakwater, see Figure 14, is a core which on the seaward side is protected by a large heap of stones, the berm. This berm is constructed in an easy way: the stones are just dumped into the sea by crane or bulldozer. This gives normally a fairly steep seaward slope of 1:1 to 1:1.5. The berm level is located at a safe working level above the highest tide. As the seaward side is steep it will not be stable under storm conditions. The first storms will take the stones of the berm and shape it into an S-shaped profile, which becomes stable as the slope becomes gentler. In fact, a berm breakwater is initially statically unstable, but becomes statically stable during its life time.

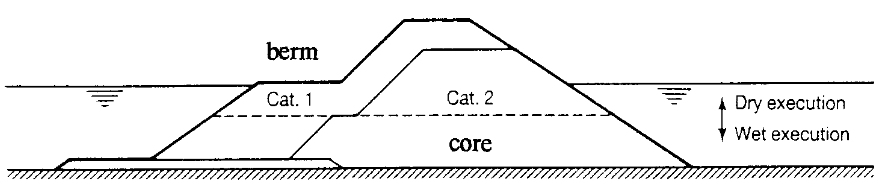

Figure 14 Cross-section of a berm breakwater

Except for the easy construction there is another advantage. Berm breakwaters can be designed with not too large rock (1-10 for example) up to wave heights of about 6 m. If this kind of rock is available a berm breakwater could be much cheaper than a protection with concrete units (which requires weights in the order of 20 ton). In all cases, except for rocky bottoms, a good filter layer should be designed at the bottom, see Figure 14. This layer should cover the whole area where the rock of the reshaped profile may be present.

A first design of a berm breakwater is made with $H_s/\Delta D_{n50}$=3.0. Most of the berm breakwaters built have stability numbers close to 3. A first estimation of the required berm width can be made as follows: draw a straight line from the crest of the structure to the filter layer at the bottom under a slope of 1:4. This gives more or less the total amount of rock that is required at the seaward side. Then draw a steep upper slope (around 1:1.5) until the desired berm level. Redistribution of the required amount of rock will give a first estimation of the berm width.

Further design should be made with a model that can predict reshaping and/or with model testing. Based on extensive model tests (Van der Meer (1988a)) relationships were established between characteristic profile parameters and the hydraulic and structural parameters. These relationships were used to make the computational model BREAKWAT, which simply gives the profile in a plot together with the initial profile. Boundary conditions for this model are:

- $H_s/\Delta D_{n50}$ = 3-500 (berm breakwaters, rock and gravel beaches)
- arbitrary initial slope
- crest above still-water level
- computation of an (established or assumed) sequence of storms (or tides) by using the previously computed profile as the initial profile.

The input parameters for the model are the nominal diameter of the stone, D_{n50}, the grading of the stone, D_{85}/D_{15}, the buoyant mass density, Δ, the significant wave height, H_s, the mean wave period, T_m, the number of waves (storm duration), N, the water depth at the toe, h and the angle of wave incidence, β. The (first) initial profile is given by a number of (x,y) points with straight lines in between. A second computation can be made on the same initial profile or on the computed one.

The result of a computation on a berm breakwater is shown in Figure 15, together with a listing of the input parameters. The model can be applied to:

- design of rock slopes and gravel beaches
- design of berm breakwaters
- behaviour of core and filter layers under construction during yearly storm conditions.

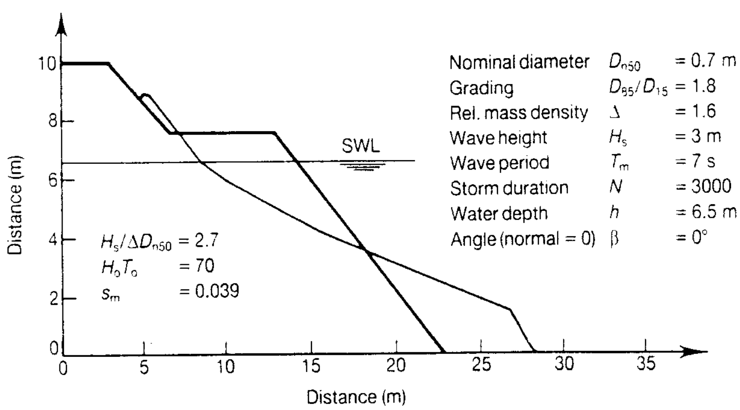

Fig. 15 Example of a computed profile for a berm breakwater

The computation model can be used in the same way as the deterministic design approach of statically stable slopes, described in section 3. There the rather complicated stability equations 6 and 7 were used to make design graphs such as damage curves, and these graphs were used for a sensitivity analysis. By making a large number of computations with the computational model the same kind of sensitivity analysis can be performed for berm breakwater of berm type protections and for dynamically stable structures. The influence of the wave climate on a structure is shown in Figure 16 and shows the difference in behaviour of the structures for various wave climates. Stability after first (less severe) storms can possibly be described by use of equations 6 and 7.

A little more information is given in van der Meer (1993). Most of the experience with construction of berm breakwaters at this moment is available at Iceland. In the past 15 years about 20 berm breakwaters have been built there. The most recent work is described by Sigurdarson et al. (1996, 1998) and Sigurdarson and Viggosson (1994).

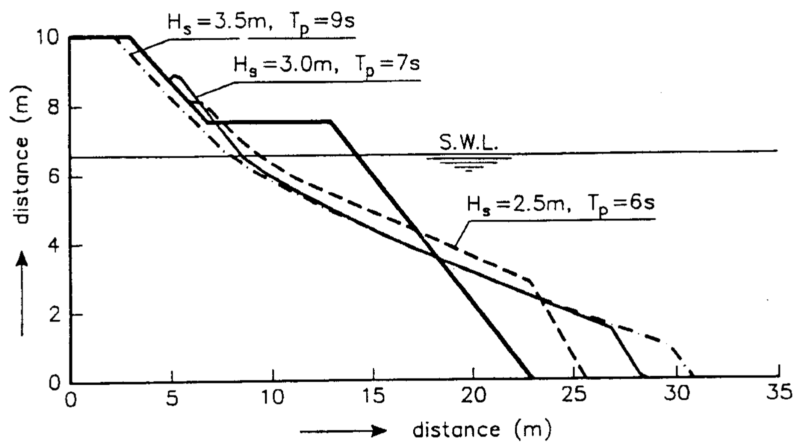

Figure 16 Example of influence of wave climate on a berm breakwater profile

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