CONCEPTUAL DESIGN OF RUBBLE MOUND BREAKWATERS

JENTSJE W. VAN DER MEER

Delft Hydraulics, PO Box 152, 8300 AD Emmeloord, the Netherlands

1. Introduction..... 17-2
1.1 Processes involved with rubble mound structures..... 17-2
1.2 Classification of rubble mound structures..... 17-4

2. Governing parameters..... 17-6
2.1 Hydraulic parameters..... 17-6
2.1.1 Wave steepness and surf similarity or breaker parameter..... 17-6
2.1.2 Run-up and run-down..... 17-8
2.1.3 Overtopping..... 17-8
2.1.4 Wave transmission..... 17-9
2.1.5 Wave reflections..... 17-9
2.2 Structural parameters..... 17-9
2.2.1 Structural parameters related to waves..... 17-9
2.2.2 Structural parameters related to rock..... 17-10
2.2.3 Structural parameters related to the cross-section.. 17-11
2.2.4 Structural parameters related to the response of the structure..... 17-13

3. Hydraulic response..... 17-16
3.1 Introduction..... 17-16
3.2 Wave run-up and run-down..... 17-16
3.3 Overtopping..... 17-21
3.4 Transmission..... 17-27
3.5 Reflections..... 17-31

4. Structural response..... 17-33
4.1 Introduction..... 17-33
4.2 Rock armour layers..... 17-34
4.3 Armour layers with concrete units..... 17-42
4.4 Low-crested structures..... 17-45
4.4.1 Reef breakwaters..... 17-47
4.4.2 Statically stable low-crested breakwaters..... 17-48
4.4.3 Submerged breakwaters..... 17-49
4.5 Berm breakwaters..... 17-50
4.6 Underlayers and filters..... 17-54
4.7 Toe protection..... 17-54
4.8 Breakwater head..... 17-56
4.9 Longshore transport at berm breakwaters..... 17-57

REFERENCES

SYMBOLS

1. Introduction

This paper gives first an overall view of physical processes involved with rubble mound structures and a classification of these structures. After the description of governing parameters, the hydraulic response is treated.

This is divided into:

- Wave run-up and run-down,
- Wave overtopping,
- Wave transmission,
- Wave reflection.

The main part of the paper describes the structural response which is divided into:

- Rock armour layers,
- Armour layers with concrete units,
- Low-crested structures,
- Berm breakwaters,
- Underlayers and filters,
- Toe protection,
- Breakwater head,
- Longshore transport at berm breakwaters.

The design tools given in this paper and by Delft Hydraulics' pc-program BREAKWAT are based on tests of schematised structures. Structures in prototype may differ (substantially) from the test-sections. Results, based on these design tools, can therefore only be used in a conceptual design. The confidence bands 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.

1.1 PROCESSES INVOLVED WITH RUBBLE MOUND STRUCTURES

The processes involved with rubble mound structures under wave (possibly combined with current) attack are given in a basic scheme in Fig. 1.

The environmental conditions (wave, current and geotechnical characteristics) lead to a number of parameters which describe *the boundary conditions at or in front of the structure (A)*. These parameters are not influenced by the structure itself, and generally, the designer of a structure has no influence on these parameters. Wave height, wave height distribution, wave breaking, wave period, spectral shape, wave angle, currents, foreshore geometry, water depth, set-up and water levels are the main hydraulic environmental parameters. These environmental parameters are not described in this paper. A specific geotechnical environmental condition is an earthquake.

Governing parameters can be divided into parameters related to *hydraulics (B in Fig. 1)*, related to *geotechnics (C)* and parameters related to the *structure (D)*. *Hydraulic* parameters are related to the description of the wave action on the structure (hydraulic response). These hydraulic parameters are described in Section 2.1. The main hydraulic responses are wave run-up, run-down, wave overtopping, wave transmission and reflection. These are described in Chapter 3. *Geotechnical* parameters are related to, for instance, liquefaction, dynamic gradients and excessive pore pressures. They are not described in this paper.

The structure can be described by a large number of *structural* parameters (D). Some important structural parameters are the slope of the structure, the mass and mass density of the rock, rock or grain shape, surface smoothness, cohesion, porosity, permeability, shear and bulk moduli and the dimensions and cross-section of the structure. The structural parameters related to hydraulic stability are described in Section 2.2.

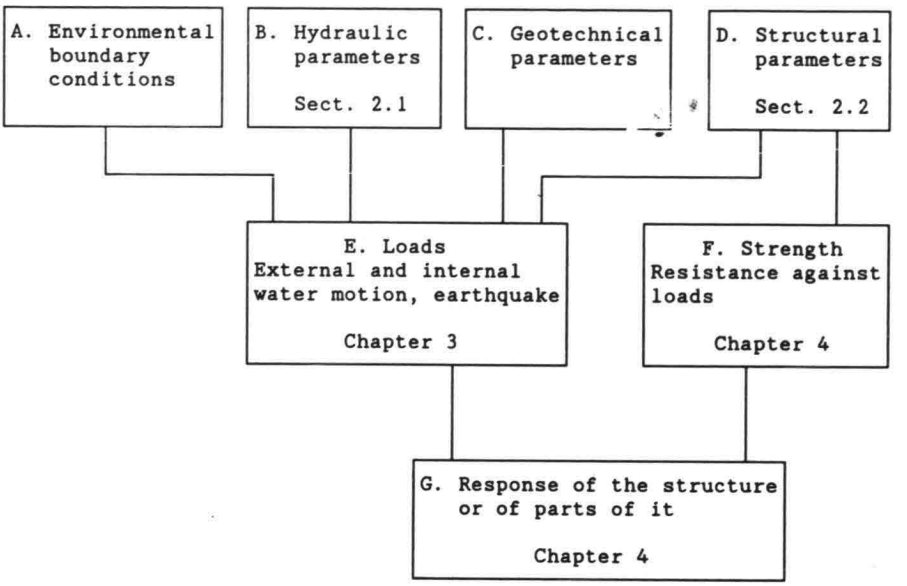


Fig. 1. Basic scheme of assessment of rubble mound structure response

The *loads on the structure* or on structural elements are given by the environmental, hydraulic, geotechnical and structural parameters together (E in Fig. 1). These loads can be divided into loads due to external water motion on the slope, loads generated by internal water motion in the structure and earthquakes. The external water motion is affected by amongst others the deformation of the wave (breaking or not breaking), the run-up and run-down, transmission, overtopping and reflection. These topics are described in Section 2.1. The internal water motion describes the penetration or dissipation of water into the structure, the variation of pore pressures and the variation of the freatic line. These topics are not treated in this paper.

Almost all structural parameters might have some or large influence on the loads. Size, shape and grading of armour stones have influence on the roughness of the slope, and therefore on run-up and run-down. Filter size and grading, together with the above mentioned characteristics of the armour stones, have an influence on the permeability of the structure, and hence on the internal water motion.

The resistance against the loads (waves, earthquakes) can be called the *strength of the structure* (F in Fig. 1). Structural parameters are essential in the formulation of the strength of the structure. Most of them have influence too on the loads, as described above.

Finally the comparison of the strength with the loads leads to a description of the response of the structure or elements of the structure (G in Fig. 1), the description of the so-called failure mechanisms. The failure mechanism may be treated in a deterministic or probabilistic way.

Hydraulic structural responses are stability of armour layers, filter layers, crest and rear, toe berms and stability of crest walls and dynamically stable slopes. These structural responses are described in Chapter 4. Geotechnical responses or interactions are slip failure, settlement, liquefaction, dynamic response, internal erosion and impacts. They are not described in this paper.

Figure 1 can be used too in order to describe the various ways of physical and numerical modelling of the stability of coastal and shoreline structures. A black box method is used if the environmental parameters (A in Fig. 1) and the hydraulic (B) and structural (D) parameters are modelled physically, and the responses (G) are given in graphs or formulae. Description of water motion (E) and strength (F) is not considered.

A grey box method is used if parts of the loads (E) are described by theoretical formulations or numerical models which are related to the strength (F) of the structure by means of a failure criterion or reliability function. The theoretical derivation of a stability formula might be the simplest example of this.

Finally, a white box is used if all relevant loads and failure criteria can be described by theoretical/physical formulations or numerical models, without empirical constants. It is obvious that it will take a long time and a tremendous research effort before coastal and shoreline structures can be designed by means of a white box.

The colours black, grey and white, used for the methods described above do not suggest a preference for one of them. Each method can be useful in a design procedure.

1.2 CLASSIFICATION OF RUBBLE MOUND STRUCTURES

Rubble mound structures can be classified by use of the $H/\Delta D$ parameter, where: H = wave height, Δ = relative mass density and D = characteristic diameter of structure, armour unit (rock or concrete), stone, gravel or sand. Small values of $H/\Delta D$ give structures as caissons or structures with large armour units. Large values imply gravel beaches and sand beaches.

Only two types of structures have to be distinguished if the response of the various structures is concerned. These types can be classified into *statically stable* structures and *dynamically stable* structures.

Statically stable structures are structures where no or minor damage is allowed under design conditions. Damage is defined as displacement of armour units. The mass of individual units must be large enough to withstand the wave forces during design conditions. Caissons and traditionally designed breakwaters belong to the group of statically stable structures. The design is based on an optimum solution between design conditions, allowable damage and costs for construction and maintenance. Static stability is characterised by the design parameter damage, and can roughly be classified by $H/\Delta D = 1\text{-}4$.

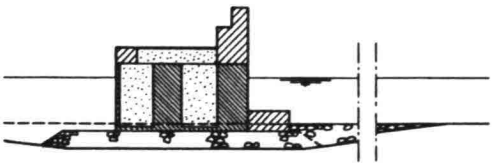
Dynamically stable structures are structures where profile development is concerned. Units (stones, gravel or sand) are displaced by wave action until a profile is reached where the transport capacity along the profile is reduced to a very low level. Material around the still water level is continuously moving during each run-up and rundown of the waves, but when the net transport capacity has become zero the profile has reached an equilibrium. Dynamic stability is characterised by the design parameter profile, and can roughly be classified by $H/\Delta D > 6$.

The structures concerned in this paper are rock armoured breakwaters and slopes and berm type breakwaters. The structures are roughly classified by $H/\Delta D = 1 - 10$.

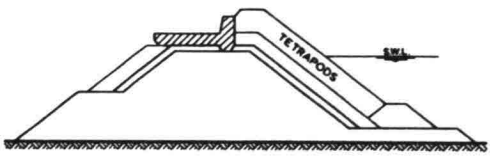
An overview of types of structures with different $H/\Delta D$ values is shown in Figure 2.

Figure 2 gives the following rough classification:

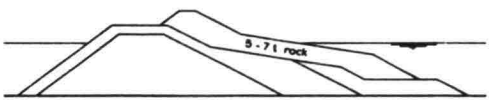
- $H/\Delta D < 1$ **Caissons or seawalls**
  No damage is allowed for these fixed structures. The diameter, D, can be the height or width of the structure.



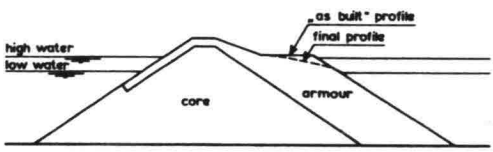
caisson
H/ΔD < 1



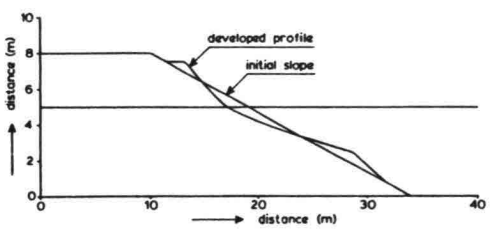
rubble mound breakwater
H/ΔD = 1 - 4



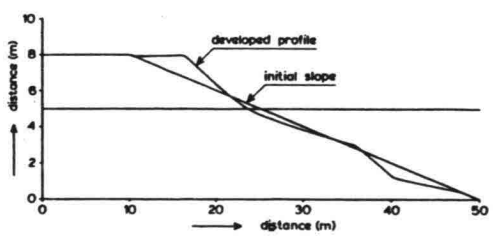
S-shaped breakwater
H/ΔD = 3 - 6



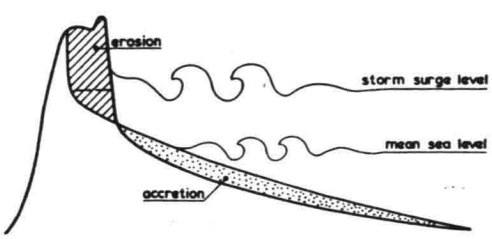
berm breakwater
H/ΔD = 3 - 6



rock beach
H/ΔD = 6 - 20



gravel beach
H/ΔD = 20 - 500



dune erosion (sand beach)
H/ΔD > 500

Fig. 2. Type of structure as a function of H/ΔD

- $H/\Delta D = 1 - 4$ **Stable breakwaters**
  Generally uniform slopes are applied with heavy artificial armour units or natural rock. Only little damage (displacement) is allowed under severe design conditions. The diameter is a characteristic diameter of the unit, such as the nominal diameter.

- $H/\Delta D = 3 - 6$ **S-shaped and berm breakwaters**
  These structures are characterised by more or less steep slopes above and below the still water level with a more gentle slope in between. This gentle part reduces the wave forces on the armour units. Berm breakwaters are designed with a very steep seaward slope and a horizontal berm just above the still water level. The first storms develop a more gentle profile which is stable further on. The profile changes to be expected are important.

- $H/\Delta D = 6 - 20$ **Rock slopes/beaches**
  The diameter of the rock is relatively small and can not withstand severe wave attack without displacement of material. The profile which is being developed under different wave boundary conditions is the design parameter.

- $H/\Delta D = 15 - 500$ **Gravel beaches**
  Grain sizes, roughly between ten centimetres and four millimetres, can be classified as gravel. Gravel beaches will change continuously under varying wave conditions and water levels (tide). Again the development of the profile is one of the design parameters.

- $H/\Delta D > 500$ **Sand beaches (during storm surges)**
  Also material with very small diameters can withstand severe wave attack. The Dutch coast is partly protected by sand dunes. The dune erosion and profile development during storm surges is one of the main design parameters. Extensive basic research has been performed on this topic (Vellinga, 1986).

2. Governing parameters

2.1 HYDRAULIC PARAMETERS

The main hydraulic responses of rubble mound structures are wave run-up and run-down, overtopping, transmission and reflections. The governing parameters related to these hydraulic responses are illustrated in Figure 3, and are discussed in this Section. The hydraulic responses itself are described in Chapter 3.

2.1.1 WAVE STEEPNESS AND SURF SIMILARITY OR BREAKER PARAMETER

Before run-up, run-down, overtopping, transmission and reflection are described, the wave boundary conditions will be defined. Wave conditions are given principally by the incident wave height at the toe of the structure, H_i, usually as the significant wave height, H_s (average of the highest 1/3 of the waves) or H_{m0} ($4\sqrt{m_0}$, based on the spectrum); the mean or peak wave periods, T_m or T_p; the angle of wave attack, β, and the local water depth, h.

The wave period is often written 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, $L = gT^2/2\pi$:

$$s = 2\pi H/gT^2 \qquad (1)$$

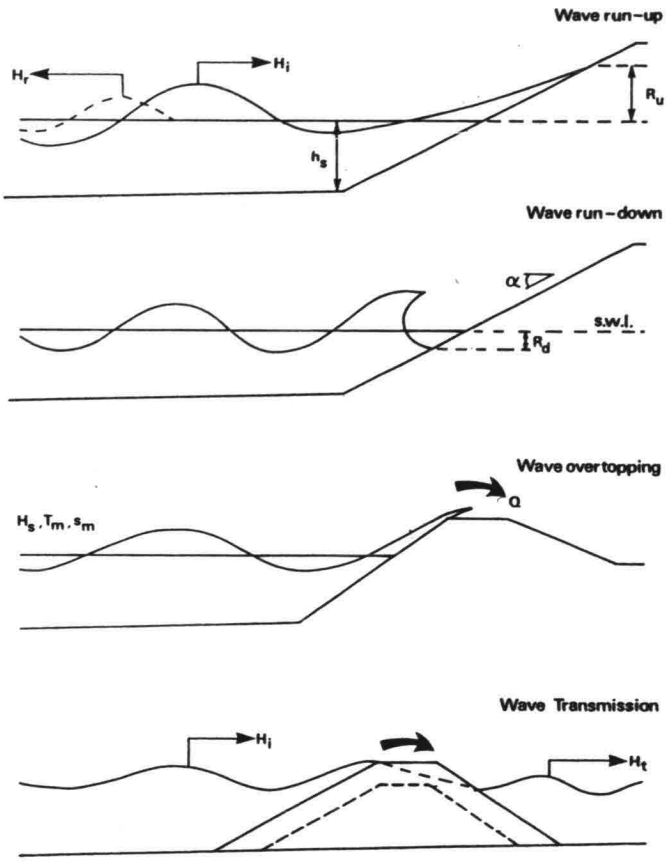


Fig. 3. Governing hydraulic parameters

If the wave height in front of the structure is used in Eq. 1, a fictitious wave steepness is obtained. This steepness is fictitious because H is the wave height in front of the structure and L is the wave length on deep water. Use of H_s and T_m or T_p in Eq. 1 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 surf similarity or breaker parameter, ξ, also termed the Iribarren number Ir:

$$\xi = \tan\alpha/\sqrt{s} \qquad (2)$$

The surf similarity parameter has often been used to describe the form of wave breaking on a beach or structure, Fig. 4. It should be noted that different versions of this parameter are defined within this paper, reflecting the approaches of different researchers. In this Section ξ_m and ξ_p are used when s is described by s_{om} or s_{op}.

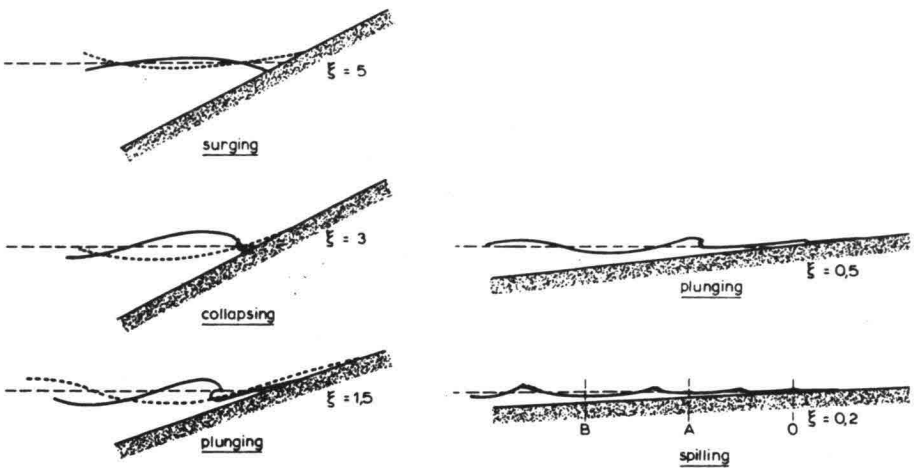


Fig. 4. Breaker types as a function of ξ, Battjes (1974)

2.1.2 RUN-UP AND RUN-DOWN

Wave action on a rubble mound structure will cause the water surface to oscillate over a vertical range generally greater than the incident wave height. The extreme levels reached in each wave, termed run-up and run-down, R_u and R_d respectively, and defined relative to the static water level, constitute important design parameters (see Fig. 3). The design run-up level will be used to determine the level of the structure crest, the upper limit of protection or other structural elements, or as an indicator of possible overtopping or wave transmission. The run-down level is often taken to determine the lower extent of main armour protection, and/or a possible level for a toe berm.

Run-up and run-down are often given in a dimensionless form:
R_{ux}/H_s and R_{dx}/H_s

where the subscript x describes the level considered, for instance 2% or significant, s.

2.1.3 OVERTOPPING

If extreme run-up levels exceed the crest level the structure will overtop. This may occur for relatively few waves under the design event, and a low overtopping rate may often be accepted without severe consequences for the structure or the area protected by it. Sea walls and breakwaters are often designed on the basis that some (small) overtopping discharge is to be expected under extreme wave conditions. The main design problem therefore reduces to one of dimensioning the cross-section geometry such that the mean overtopping discharge, $\bar{Q}$, under design conditions remains below acceptable limits. A dimensionless parameter for the mean overtopping discharge, Q^*, was defined by Owen (1980):

$$Q^* = \frac{\bar{Q}}{\sqrt{gH_s^3}} \sqrt{s/2\pi} \qquad (3)$$

Also here Q_m^* and Q_p^* will be used when s_{om} and s_{op} are used in Eq. 3.

2.1.4 WAVE TRANSMISSION

Breakwaters with relatively low crest levels may be overtopped with sufficient severity to excite wave action behind. Where a breakwater is constructed of relatively permeable construction, long wave periods may lead to transmission of wave energy through the structure. In some cases the two different responses will be combined. The quantification of wave transmission is important in the design of low-crested breakwaters intended to protect beaches or shorelines, and in the design of harbour breakwaters where long wave periods transmitted through the breakwater could cause movement of ships or other floating bodies.

The severity of wave transmission is described by the coefficient of transmission, C_t, defined in terms of the incident and transmitted wave heights, H_i and H_t respectively, or the total incident and transmitted wave energies, E_i and E_t:

$$K_t = H_t/H_i = \sqrt{E_t/E_i} \tag{4}$$

2.1.5 WAVE REFLECTIONS

Wave reflections are of importance on the open coast, and at commercial and small boat harbours. The interaction of incident and reflected waves often lead to a confused sea in front of the structure, with occasional steep and unstable waves of considerable hazard to small boats. Reflected waves can also propagate into areas of a harbour previously sheltered from wave action. They will lead to increased peak orbital velocities, increasing the likelihood of movement of beach material. Under oblique waves, reflection will increase littoral currents and hence local sediment transport. All coastal structures reflect some proportion of the incident wave energy. This is often described by a reflection coefficient, C_r, defined in terms of the incident and reflected wave heights, H_i and H_r respectively, or the total incident and reflected wave energies, E_i and E_r:

$$C_r = H_r/H_i = \sqrt{E_r/E_i} \tag{5}$$

When considering random waves, values of C_r may be defined using the significant incident and reflected wave heights as representative of the incident and reflected energies.

2.2 STRUCTURAL PARAMETERS

Structural parameters can be divided into four categories which will be treated in this Section:

- Structural parameters related to waves.
- Structural parameters related to rock.
- Structural parameters related to the cross-section.
- Structural parameters related to the response of the structure.

2.2.1 STRUCTURAL PARAMETERS RELATED TO WAVES

The most important parameter which gives a relationship between the structure and the wave conditions has been used in Section 1.2. In general the $H/\Delta D$ gives a good classification. For the design of rubble mound structures this parameter should be defined in more detail.

The wave height is usually the significant wave height H_s, either defined by the average of the highest one third of the waves or by $4\sqrt{m_0}$. For deep water both definitions give more or less the same wave height. For shallow water conditions substantial differences may be present.

The relative buoyant density is described by:

$$\Delta = (\rho_r/\rho_w) - 1 \quad (6)$$

where:
ρ_r = mass density of the rock (saturated surface dry relative 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:

$$D_{n50} = (M_{50}/\rho_r)^{1/3} \quad (7)$$

where:
D_{n50} = nominal diameter,
M_{50} = median mass of unit given by 50% on mass distribution curve.

The parameter $H/\Delta D$ changes to $H_s/\Delta D_{n50}$.

Another important structural parameter is the surf similarity parameter, which relates the slope angle to the wave steepness, and which gives a classification of breaker types. The surf similarity parameter ξ (ξ_m, ξ_p with T_m, T_p) is defined in Section 2.1.1.

For dynamically stable structures with profile development a surf similarity parameter can not be defined as the slope is not straight. Furthermore, dynamically stable structures are described by a large range of $H_s/\Delta D_{n50}$ values. In that case it is possible to relate also the wave period to the nominal diameter and to make a combined wave height - period parameter. This parameter is defined by:

$$H_oT_o = H_s/\Delta D_{n50} * T_m\sqrt{g/D_{n50}} \quad (8)$$

The relationship between $H_s/\Delta D_{n50}$ and H_oT_o is listed below.

Structure	$H_s/\Delta D_{n50}$	H_oT_o
Statically stable breakwaters	1 - 4	< 100
Rock slopes and beaches	6 - 20	200 - 1500
Gravel beaches	15 - 500	1000 - 200,000
Sand beaches	> 500	> 200,000

Another parameter which relates both wave height and period (or wave steepness) to the nominal diameter was introduced by Ahrens (1987). In the Shore Protection paper $H_s/\Delta D_{n50}$ is often called N_s. Ahrens included the wave steepness in a modified stability number N_s^*, defined by:

$$N_s^* = N_s\, s_p^{-1/3} = H_s/\Delta D_{n50}\, s_p^{-1/3} \quad (9)$$

In this equation s_p is the local wave steepness and not the deep water wave steepness. The local wave steepness is calculated using the local wave length from the Airy theory, where the deep water wave steepness is calculated by Eq. 1. This modified N_s^* number has a close relationship with H_oT_o defined by Eq. 8.

2.2.2 STRUCTURAL PARAMETERS RELATED TO ROCK

The most important parameter which is related to the rock is the nominal diameter defined by Eq. 7. Related to this is of course M_{50}, the 50% value on the mass distribution curve. The grading of the rock can be given by the D_{85}/D_{15}, where D_{85} and D_{15} are the 85% and 15% values of the sieve curves,

respectively. These are the most important parameters as far as stability of armour layers is concerned. Examples of gradings are shown in box 2 showing the relationship between class of stone (here simply taken as W_{85}/W_{15}) and D_{85}/D_{15}.

Box 1 **Wave height-period parameters**

$$H_s/\Delta D_{n50} = N_s$$

$$H_s/\Delta D_{n50}\, s_p^{-1/3} = N_s^*$$

$$H_s/\Delta D_{n50}\, T_m\sqrt{g/D_{n50}} = H_o T_o$$

$$H_s/\Delta D_{n50}\, s_{om}^{-0.5}\sqrt{2\pi H_s/D_{n50}} = H_o T_o$$

$$\xi_m = \tan\alpha/\sqrt{s} = \tan\alpha \,/\, \sqrt{2\pi H_s/gT_m^2}$$

Box 2 **Examples of gradings**

narrow grading $D_{85}/D_{15} < 1.5$		wide grading $1.5 < D_{85}/D_{15} < 2.5$		Very wide grading $D_{85}/D_{15} > 2.5$	
Class	D_{85}/D_{15}	Class	D_{85}/D_{15}	Class	D_{85}/D_{15}
15-20 t	1.10	1-9 t	2.08	50-1000 kg	2.71
10-15 t	1.14	1-6 t	1.82	20-1000 kg	3.68
5-10 t	1.26	100-1000 kg	2.15	10-1000 kg	4.64
3-7 t	1.33	100-500 kg	1.71	10-500 kg	3.68
1-3 t	1.44	10-80 kg	2.00	10-300 kg	3.10
300-1000 kg	1.49	10-60 kg	1.82	20-300 kg	2.46

2.2.3 STRUCTURAL PARAMETERS RELATED TO THE CROSS-SECTION

There are a lot of parameters related to the cross-section and most of them are obvious. Figure 5 gives an overview. The parameters are:

crest freeboard, relative to swl	R_c
armour crest freeboard relative to swl	A_c
difference between crown wall and armour crest	F_c
armour crest level relative to the seabed	h_c
structure width	B
width of armour berm at crest	G_c
thickness of armour, underlayer, filter	t_a, t_u, t_f
area porosity	n_a
angle of structure slope	α
depth of the toe below swl	h_t

The permeability of the structure has influence on the stability of the armour layer. The permeability depends on the size of filter layers and core and can be given by a notional permeability factor, P. Examples of P are shown in Fig. 6, based on the work of Van der Meer (1988-1). 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. 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 stones. In that case P = 0.6. Two other values are shown in Fig. 6 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!

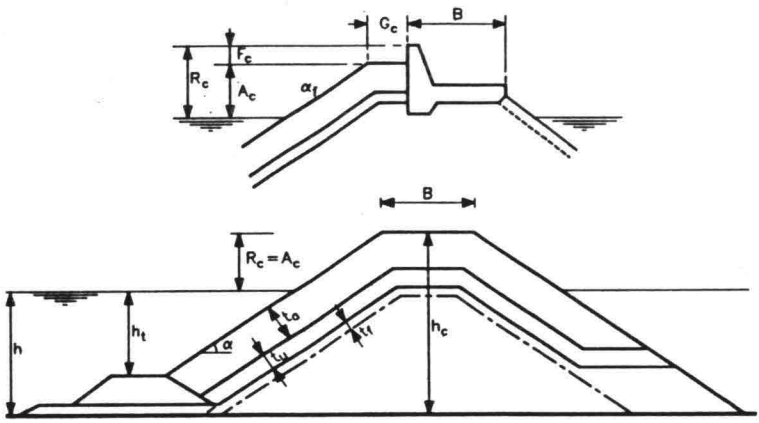


Fig. 5. Governing parameters related to the cross-section

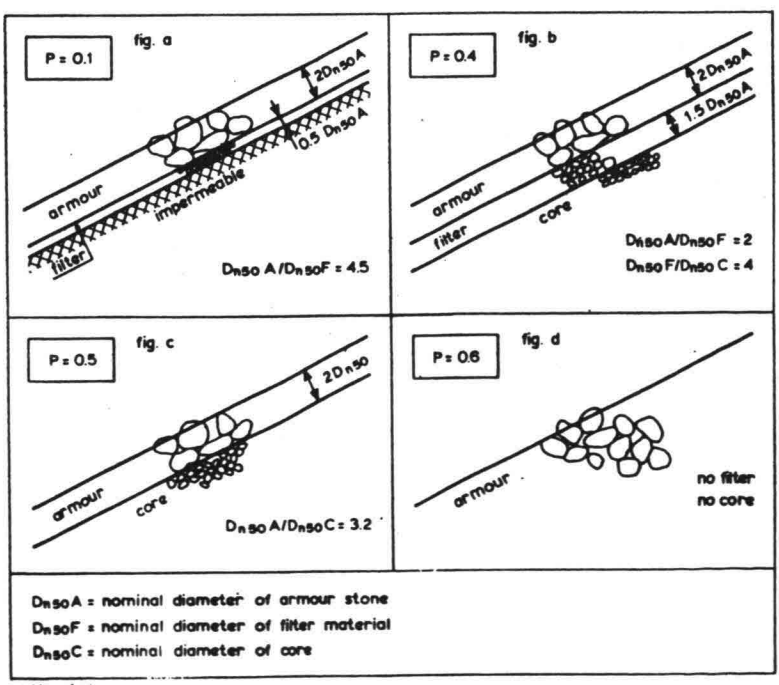


Fig. 6. Notional permeability factor P for various structures

2.2.4 STRUCTURAL PARAMETERS RELATED TO THE RESPONSE OF THE STRUCTURE

The behaviour of the structure can be described by a few parameters. Statically stable structures are described by the development of damage. This can be the amount of rock that is displaced or the displaced distance of a crown wall. Dynamically stable structures are described by a developed profile.

The damage to the armour layer can be given as a percentage of displaced stones 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 swl. When this erosion area is related to the size of the stones, 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:

$$S = A_e / D_{n50}^2 \qquad (10)$$

where:
S = damage level
A_e = erosion area around swl

A plot of a structure with damage is shown in Fig. 7. The damage level takes into account settlement and displacement. A physical description of the damage, S, is the number of squares with a side D_{n50} which 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 stones and the shape of the stones. Generally the actual number of stones eroded in a D_{n50} wide strip is equal to 0.7 to 1 times the damage S.

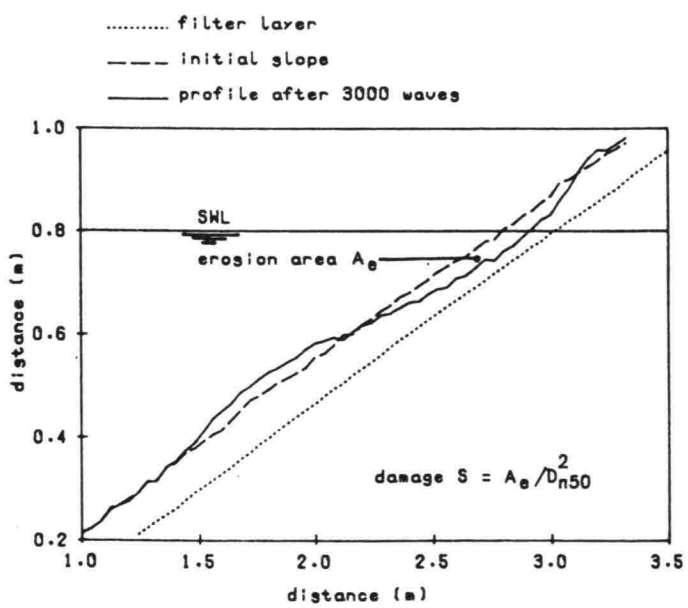


Fig. 7. Damage S based on erosion area A_e

The acceptable 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 and should be called dynamically stable.

Slope	Initial damage	Intermediate damage	Failure
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

Another definition is 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 and for accropode $D_n = 0.7D$.

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 has own damage description, but the actual number is related to a width of one D_n. The following damage descriptions will be used in this paper:

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 two times N_{od}.

Dynamically stable structures are structures where profile development is accepted. Units (stones, gravel or sand) are displaced by wave action until a profile is reached where the transport capacity along the profile is reduced to a minimum. Dynamic stability is characterised by the design parameter *profile*.

An example of a schematised profile is shown in Figure 8. The initial slope was 1:5 which is relatively gentle and one should notice that Fig. 8 is shown on a distorted scale. The profile consists of a beach crest (the highest point of the profile), a curved slope around swl (above swl steep, below swl gentle) and a steeper part relatively deep below swl. For gentle slopes (shingle slope > 1:4) a step is found at this deep part. The profile is characterised by a number of lengths, heights and angles and these were related to the wave boundary conditions and structural parameters (Van der Meer (1988-1)).

Other typical profiles, but for different initial slopes, are shown in Fig. 9. The main part of the profiles is always the same. The initial slope (gentle or steep) determines whether material is transported upwards to a beach crest or downwards, creating erosion around swl.

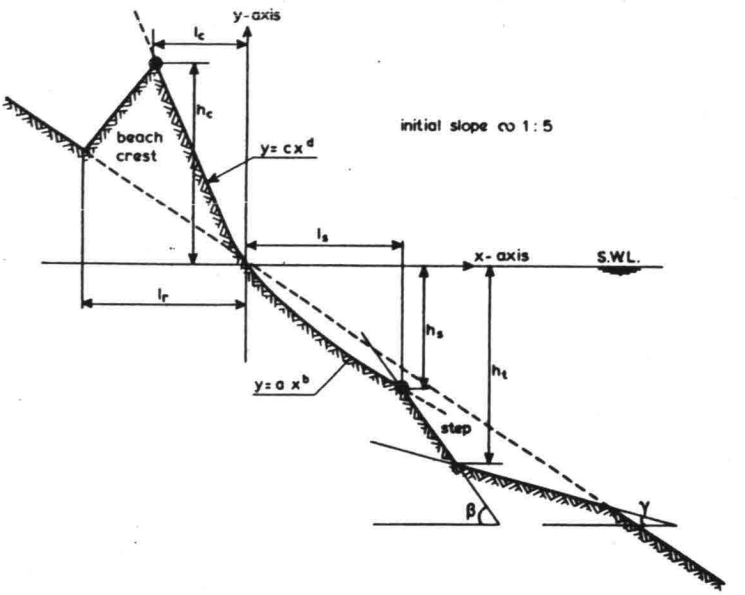


Fig. 8. Schematised profile on a 1:5 initial slope

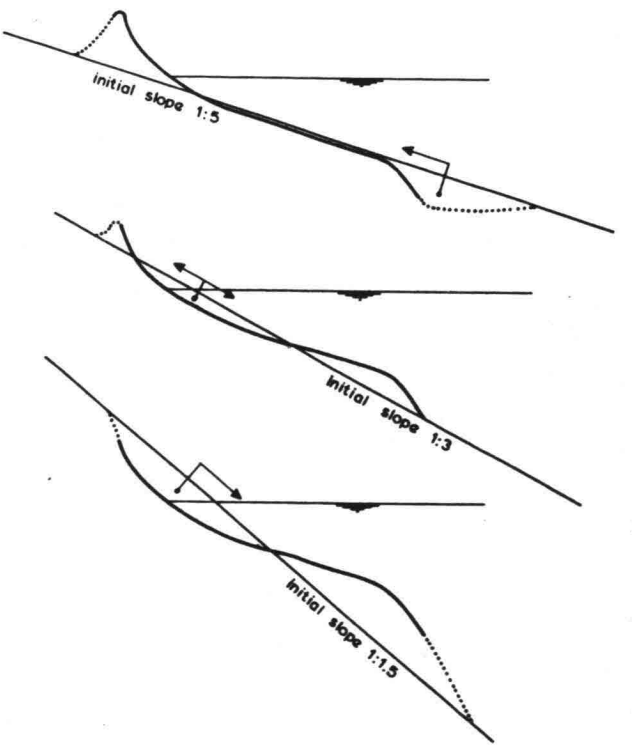


Fig. 9. Examples of profiles for different initial slopes

3. Hydraulic response

3.1 INTRODUCTION

This section presents methods that may be used for the calculation of the hydraulic response parameters which were also given in Figure 3:
run-up and run-down levels,
overtopping discharges,
wave transmission,
wave reflections.

Where possible, the prediction methods are identified with the limits of their application. The reader is advised that prediction methods are generally available to describe the hydraulic response for only a few simplified cases. Often tests have been conducted for a limited range of wave conditions. Similarly the structure geometry tested often represents a simplification in relation to many practical structures. It is therefore necessary to estimate the performance from predictions for related, but non-similar, structure configurations. Where this is not possible, or the predictions are less reliable than are needed, physical model tests should be conducted.

3.2 WAVE RUN-UP AND RUN-DOWN

Prediction of R_u and R_d may be based on simple empirical equations, supported by model test results, or upon numerical models of wave/structure interaction. A few simple numerical models of wave run-up have been developed recently, but have only been tested for a few cases and will not be discussed here.

All calculation methods require parameters to be defined precisely. Run-up and run-down levels are defined relative to still water level (swl), see Fig. 3. On some bermed and shallow slopes run-down levels may not fall below still water. All run-down levels in this paper are given as positive if below swl, and all run-up levels will also be given as positive if above swl.

The upward excursion is generally greater than the downward, and the mean water level on the slope is often above swl. Again this may be most marked on bermed and shallow slopes. These effects often complicate the definition, calculation, or measurement of run-down parameters.

Much of the field data available on wave run-up and run-down applies to gentle and smooth slopes. Some laboratory measurements have been made on steeper smooth slopes, and on porous armoured slopes. Prediction methods for smooth slopes may be used directly for armoured slopes that are filled or fully grouted with concrete or bitumen. These methods can also be used for rough non-porous slopes with an appropriate reduction factor.

The behaviour of waves on rough porous (rubble mound) slopes is very different from that on non-porous slopes, and the run-up performance is not well predicted by adapting equations for smooth slopes. Different data must be used. This difference is illustrated in Fig. 10, where 2% relative run-up, $R_{u2\%}/H_s$, is plotted for both smooth and rock slopes. The greatest divergence between the performance of the different slope types is seen for $1 < \xi_p < 5$. For ξ_p above about 6 or 7 the run-up performance of smooth and porous slopes tends to very similar values. In that case the wave motion is surging up and down the slope without breaking and the roughness and porosity is then less important.

Run-up and run-down will be treated for armoured rubble slopes only. Smooth slopes are used for comparison. Measurements of wave run-up on smooth slopes have been analysed by Ahrens (1981), Delft Hydraulics (M1983, 1989), and by Allsop et al (SR2, 1985). In each instance the test results are scattered, Figs. 10 and 11, but simple prediction lines have been fitted to the data.

Figure 10 shows the data of Ahrens (1981) for slopes between 1:1 and 1:4, of Van Oorschot and d'Angremond (1968) for slopes 1:4 and 1:6 and Allsop et al (1985) for slopes between 1:1.33 and 1:2. All mentioned data points are for smooth slopes. The other points in Fig. 10 are for rock slopes (Delft Hydraulics, M1983, 1989). The scatter in Ahrens' data is large. He measured only 100-200 waves and the 2% value is not very reliable in that case.

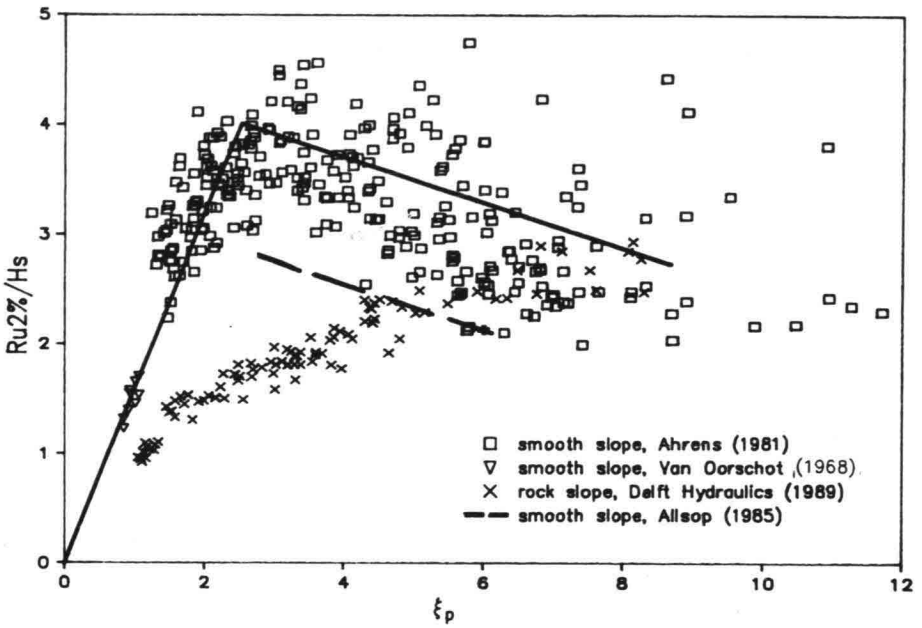


Fig. 10. Comparison of relative 2% run-up for smooth and rubble slopes

Figure 11 shows the same data, but now for the significant levels. The scatter around Ahrens data is much less now. In both Figs. the data of Allsop et al is about 20-30% lower than the data of Ahrens. Reasons for the differences are hard to give, but possibly different definitions in run-up level and different test methods have caused it. Based on these Figs. the data of Ahrens give probably a conservative estimate.

A rubble mound slope will dissipate significantly more wave energy than the equivalent smooth or non-porous slope in most cases. Run-up levels will therefore generally be reduced. This reduction is influenced by the permeability of the armour, filter and under-layers, and by the steepness and period of the waves.

Run-up levels on rubble slopes armoured with rock armour or rip-rap have been measured in laboratory tests. In many instances the rubble core has been reproduced as fairly permeable, except for those particular cases where an impermeable core has been used. Test results often therefore span a range within which the designer must interpolate.

Analysis of test data from measurements by van der Meer (1988-1) has given prediction formulae for rock slopes with an impermeable core, described by a notional permeability factor P = 0.1, and porous mounds of relatively high permeability given by P = 0.4 - 0.6 (Delft Hydraulics M1983 pt3, 1988). The notional permeability factor P was described in Section 2.2.4, Fig. 6.

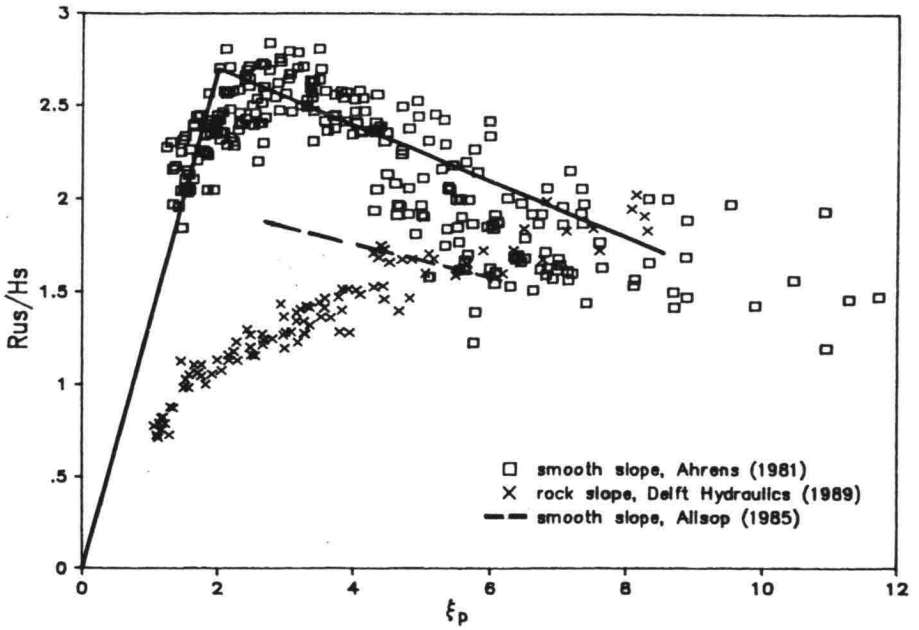


Fig. 11. Comparison of relative significant run-up for smooth and rubble mound slopes

Two sets of empirically derived formulae can be given for run-up on rock slopes. The first set gives the run-up as a function of the surf similarity or breaker parameter. Coefficients for various run-up levels were derived. Secondly the run-up was described as a Weibull distribution, including all possible run-up levels.

The formulae for run-up versus surf similarity parameter are:

$$R_{ux}/H_s = a\xi_m \quad \text{for } \xi_m < 1.5 \tag{11}$$

$$R_{ux}/H_s = b\xi_m^{\,c} \quad \text{for } \xi_m > 1.5 \tag{12}$$

The run-up for permeable structures (P > 0.4) is limited to a maximum:

$$R_{ux}/H_s = d \tag{13}$$

Values for the coefficients a, b, c and d have been determined for exceedence levels of i= 0.1%, 1%, 2%, 5%, 10%, significant, and mean run-up levels and are shown in the table below.

level (%)	a	b	c	d
0.1	1.12	1.34	0.55	2.58
1	1.01	1.24	0.48	2.15
2	0.96	1.17	0.46	1.97
5	0.86	1.05	0.44	1.68
10	0.77	0.94	0.42	1.45
sign.	0.72	0.88	0.41	1.35
mean	0.47	0.60	0.34	0.82

Results of the tests and the equations are shown for example values of i = 2%, and significant, for each of P = 0.1 and P > 0.4, in Figs. 12 and 13.

The reliability of Eqs. 11 - 13 can be described by assuming coefficients a, b and d as stochastic variables with a normal distribution. The variation coefficients for these coefficients are 7 % for P < 0.4 and 12 % for P ≥ 0.4. Confidence bands can be calculated based on these variation coefficients.

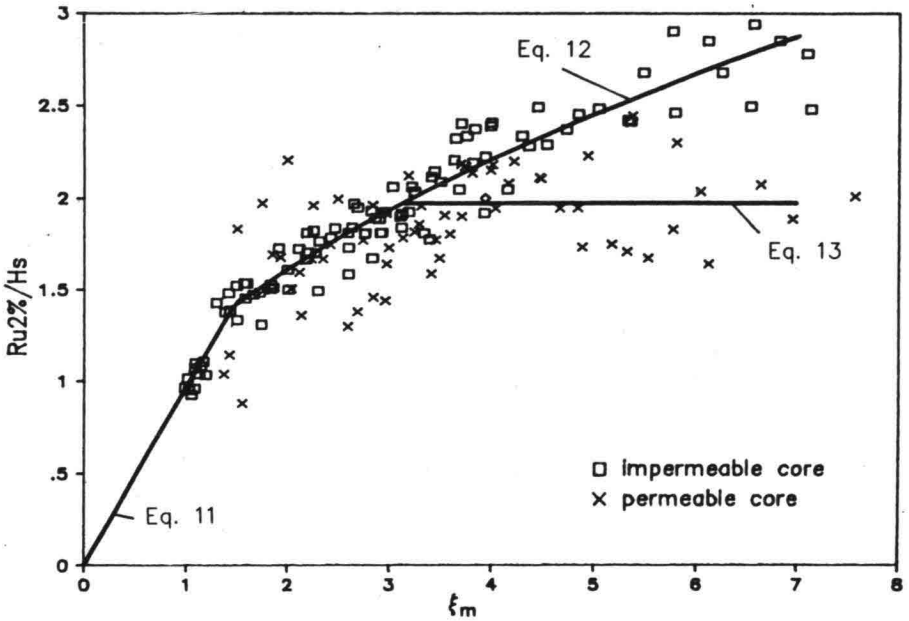


Fig. 12. Relative 2% run-up on rock slopes

The second method is to describe the run-up as a Weibull distribution:

$$p = Pr\{R_u > R_{up}\} = \exp\{-(R_{up}/b)^c\} \quad (14)$$

$$\text{or: } R_{up} = b(-\ln p)^{1/c} \quad (15)$$

where:
p = probability (between 0 and 1),
R_{up} = run-up level exceeded by p * 100% of the run-ups,
b = scale parameter,
c = shape parameter.

The shape parameter defines the shape of the curve. For c=2 a Rayleigh distribution is obtained. The scale parameter can be described by:

$$b/H_s = 0.4\, s_m^{-0.25} \cot\alpha^{-0.2} \quad (16)$$

The shape parameter is described by:

for plunging waves:

$$c = 3.0\, \xi_m^{-0.75} \quad (17)$$

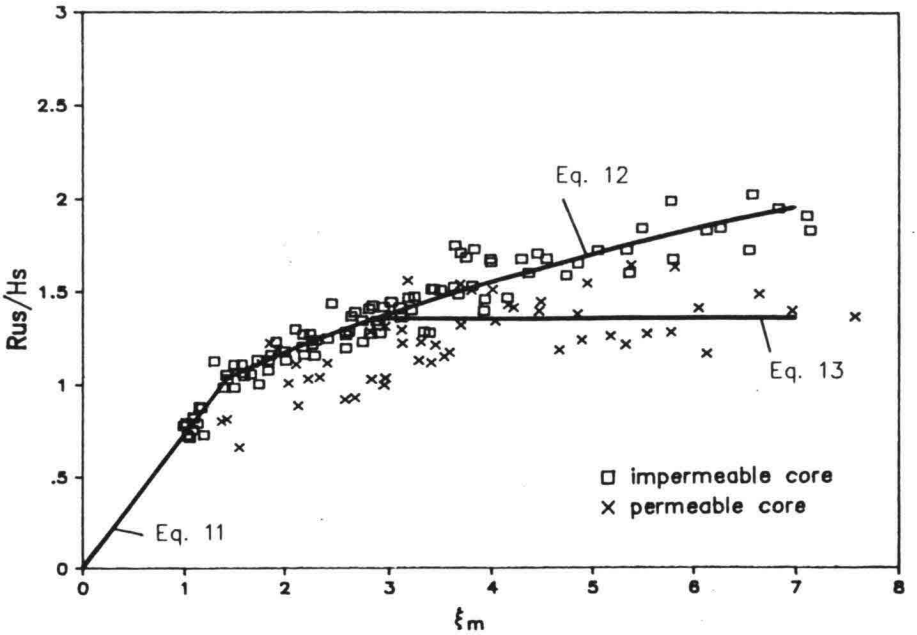


Fig. 13. Relative significant run-up on rock slopes

for surging waves:

$$c = 0.52\ P^{-0.3}\ \xi_m^P \sqrt{\cot\alpha} \qquad (18)$$

The transition between Eqs. 17 and 18 is described by a critical value for the surf similarity parameter, ξ_{mc}:

$$\xi_{mc} = [5.77\ P^{0.3} \sqrt{\tan\alpha}]^{\ 1/(P+0.75)} \qquad (19)$$

For $\xi_m < \xi_{mc}$, Eq. 17 should be used and for $\xi_m > \xi_{mc}$, Eq. 18. The formulae are only applicable *for slopes with cota* ≤ 2. For steeper slopes the distributions on a 1:2 slope may give a first estimation.

Examples of run-up distributions are shown in Fig. 14. The reliability of Eqs. 15 - 18 can be described by assuming b as a stochastic variable with a normal distribution. The variation coefficient of b is 6% for $P < 0.4$ and 9% for $P \geq 0.4$. Confidence bands can be calculated by means of these variation coefficients.

Run-down levels on porous rubble slopes are also influenced by the permeability of the structure, and by the surf similarity parameter. Analysis of the 2% run-down level on the sections tested by Van der Meer (1988-1) has given an equation which includes the effects of structure permeability, and wave steepness:

$$R_{d2\%}/H_s = 2.1\sqrt{\tan\alpha} - 1.2P^{0.15} + 1.5\exp(-60\ s_m) \qquad (20)$$

Test results are shown in Fig. 15 for an impermeable and a permeable core. The presentation with ξ_m only gives a large scatter. Including the slope angle and the wave steepness separately and including also the permeability as in Eq. 20, reduces the scatter considerably.

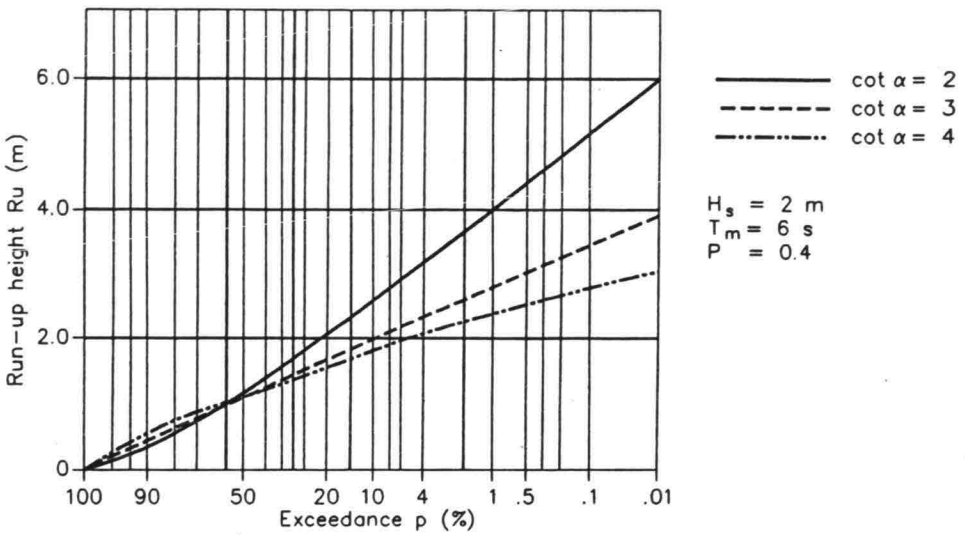


Fig. 14. Run-up distributions on a rock slope

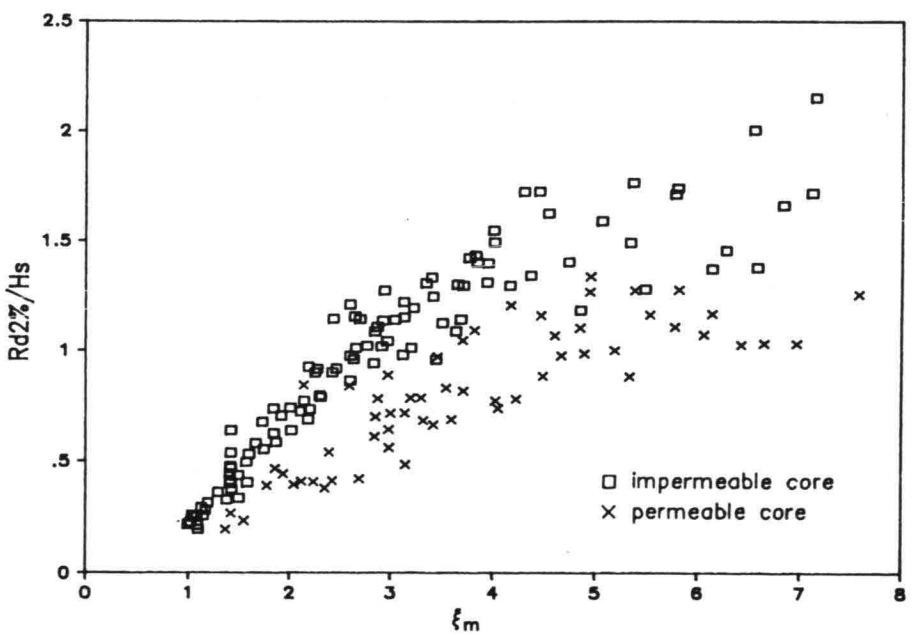


Fig. 15. Run-down $R_{d2\%}/H_s$ on impermeable and permeable rock slopes

3.3 OVERTOPPING

In the design of many sea walls and breakwaters, the controlling hydraulic response is often the wave overtopping discharge. Under random waves this varies greatly from wave to wave. There is very little data available

to quantify this variation. For many cases it is sufficient to use the mean discharge, $\bar{Q}$, usually expressed as a discharge per metre run ($m^3/s.m$). The dimensionless discharge, Q^*_m or Q^*_p, was already given in Section 2.1.3 and Eq. 3.

The calculation of overtopping discharge for a particular structure geometry, water level, and wave condition is based on empirical equations fitted to hydraulic model test results. The data available on overtopping performance is restricted to a few structural geometries. A well-known and wide data set applies to plain and bermed smooth slopes without crown walls, Owen (1980). More restricted studies have been reported by Bradbury et al (1988), and Aminti & Franco (1988). Recently Delft Hydraulics finished two extensive studies on wave runup and overtopping, De Waal and Van der Meer (1992).

Each of these studies have developed dimensionless parameters of the crest freeboard for use in prediction formulae. Different dimensionless groups have been used by each author, and no direct comparisons have yet been made. The simplest such parameter is the relative freeboard, R_c/H_s. This simple parameter however omits the important effects of wave period, and other dimensionless parameters have been required to include the wave length or steepness.

For *plain and bermed smooth slopes* Owen (1980) relates a dimensionless discharge parameter, Q^*_m, to a dimensionless freeboard parameter, R^*_m, by an exponential equation of the form:

$$Q^*_m = a \exp(-bR^*_m/r) \tag{21}$$

where Q^*_m is defined in Eq. 3 and the dimensionless freeboard is defined:

$$R^*_m = R_c/H_s * \sqrt{s_m/2\pi} \tag{22}$$

and values for the coefficients a and b were derived from the test results, and are given in Table 2.

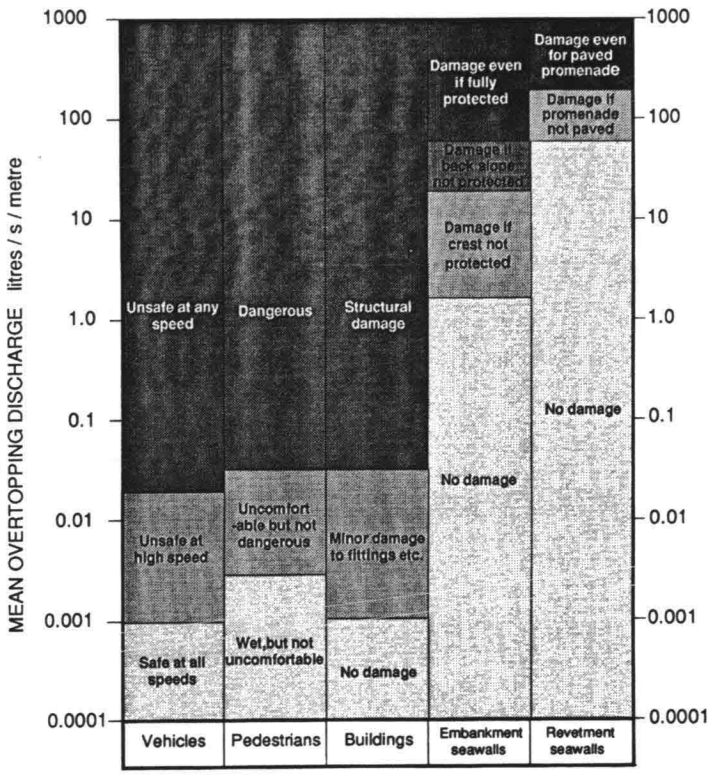
slope	a	b
1:1	0.00794	20.12
1:1.5	0.0102	20.12
1:2	0.0125	22.06
1:3	0.0163	31.9
1:4	0.0192	46.96
1:5	0.025	65.2

Table 2. Values of the coefficients a and b in Eq. 21 for straight smooth slopes

Delft Hydraulics has recently performed various applied fundamental research studies in physical scale models on wave runup and overtopping on various structures, De Waal and Van der Meer (1992). Run-up has extensively been measured on rock slopes. The influence on runup and overtopping of berms, roughness on the slope and shallow water, has been measured for smooth slopes. Finally, the influence of short-crested waves and oblique (long- and short-crested) waves has been studied on wave run-up and overtopping. All research was commissioned by the Technical Advisory Committee for Water Defenses (TAW) in The Netherlands. The paper gives an overall view of the final results, such as design formulae and design graphs and will be summarized here.

Box 3 **Overtopping discharges**

Limiting values of $\bar{Q}$ for different design cases have been suggested, and are summarised in the figure below. This incorporates recommended limiting values of the mean discharge for the stability of crest and rear armour to types of sea walls, and or the safety of vehicles and people.



WAVE RUNUP ON SMOOTH SLOPES

A general runup formula can be given for smooth slopes, based on large scale tests in Delft Hydraulics' large Delta flume and on the research mentioned above. The general formula for the 2%-runup $Ru_{2\%}$ is given by:

$$Ru_{2\%}/H_s = 1.5\ \gamma\ \xi_{op} \quad \text{with a maximum of } 3.0\ \gamma \qquad (23)$$

where: H_s = the significant wave height, γ = a total reduction factor for various influences and ξ_{op} = the surf similarity parameter based on the peak period. This general formula is shown in Fig. 16. The influence of berms,

roughness, shallow water and oblique wave attack on wave runup and overtopping can be given as reduction factors γ_b, γ_f, γ_h and γ_β, respectively. They are defined as the ratio of runup on a slope considered to that on a smooth impermeable slope under otherwise identical conditions (TAW, 1974). The total reduction factor becomes than:

$$\gamma = \gamma_b \gamma_f \gamma_h \gamma_\beta \qquad (24)$$

The reduction factors will be described in the next sections.

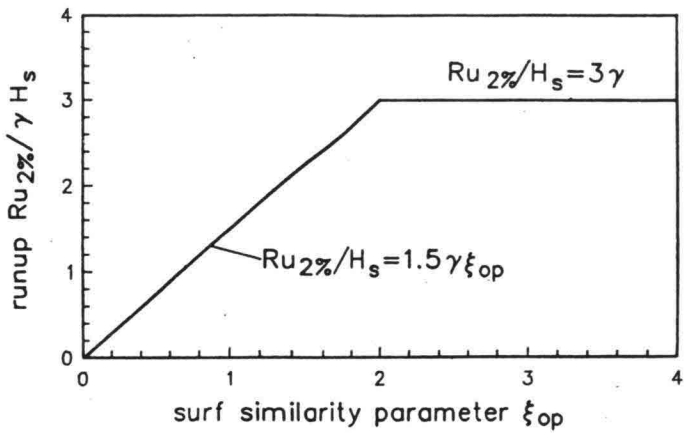


Fig. 16. Wave runup on slopes

BERMS, ROUGHNESS AND SHALLOW WATER

About 150 tests were performed in a wave flume on smooth slopes of 1:3 and 1:4. Berms with various lengths and depths were tested. Various roughness elements were placed on a 1:3 slope, such as cubic blocks, ribs and one layer of rock. Finally the effect of depth limited waves (which do not follow the Rayleigh distribution) on a foreshore was studied.

Covering			Reductionfactor γ_f
Smooth, concrete, asphalt			1,0
Impermeable smooth block revetment			1,0
Grass			0,90 - 1,0
1 layer of rock			0,55 - 0,60
2 layers of rock			0,50 - 0,55
Ribs. k/H_s = 0,12 - 0,19 en and $1/k$ = 7 (optimum)			0,60 - 0,70
Blocks on smooth slope. Height f_h, width f_b			
f_h/f_b	f_b/H_s	surface covered	
0,88	0,12 - 0,24	1/25	0,75 - 0,85
0,88	0,12 - 0,19	1/9	0,70 - 0,75
0,44	0,12 - 0,24	1/25	0,85 - 0,95
0,88	0,12 - 0,18	1/25	0,85 - 0,95
0,18	0,55 - 1,10	1/4	0,75 - 0,85

Table 3. Reduction factors γ_f for runup on slopes including roughness

The reduction factor for berms γ_b can easiest be described by using an equivalent slope. This slope is simply a straight line between points on the slope $1.5H_s$ below and above the slope. The tests on roughness resulted in a table with reduction factors γ_f for various rough slopes and can be seen as an update of Table II.5.5 in TAW (1974) or the similar Table 7-2 in the Shore Protection Manual (CERC, 1984). Table 3 shows this update (now with random waves). The influence of depth limited waves on runup can be described by $\gamma_h = H_{2\%}/1.4H_s$. For a Rayleigh distribution of the wave heights γ_h becomes 1.

OBLIQUE AND SHORT CRESTED WAVES

About 160 tests were performed in a multi-directional wave basin on wave runup and overtopping. The structure was 15 m long and was divided in 3 sections with different crest levels. Overtopping was measured at two sections and runup at the other. Smooth 1:2.5 and 1:4 slopes were tested and a 1:4 slope with a berm at the still water level.

Short crested perpendicular wave attack gave similar results on both wave runup and overtopping than long crested perpendicular wave attack. The results were different when the wave attack on the structure was oblique, see Fig. 17. Long crested waves gave a reduction factor γ_β of 0.6 when the angle of wave attack was larger than 60°.

Short crested oblique waves, more similar to nature, give a different picture. From 0° to 90° the runup reduction factor reduced linearly to 0.8. The reduction in runup is much less than for long crested waves.

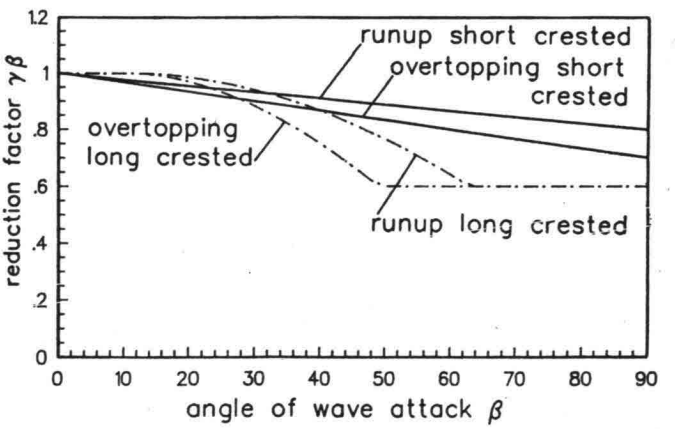


Fig. 17. Influence of oblique long and short crested waves

Wave overtopping is given per meter structure width. With oblique wave attack less wave energy will reach this meter structure width and therefore reduction factors for oblique wave attack are smaller for overtopping than for runup. The reduction factors are given in Fig. 17.

WAVE OVERTOPPING

The most simple approach for determining wave overtopping (given as a mean overtopping discharge $\bar{Q}$ in m^3/s per m width) is followed when the crest freeboard R_c is related to an expected runup level on a non-overtopped slope, say the $Ru_{2\%}$. This "shortage in runup height" can than be described by $(Ru_{2\%}-R_c)/H_s$. The approach followed by others (Owen 1980) with R_c only in stead of $(Ru_{2\%}-R_c)$ leads to different formulas and different dimensionless

parameters for plunging (breaking) and surging (non-breaking) waves. Eq. 23 and 24 can be used to determine $Ru_{2\%}$, including all influences of berms, etc.

The most simple dimensionless description of overtopping is $\bar{Q}/\sqrt{gH_s^3}$. Fig. 18 shows the final results on overtopping and gives all available data, including data of Owen (1980), Führböter et al (1989) and various tests at Delft Hydraulics. The horizontal axis gives the "shortage in runup height" $(Ru_{2\%}-R_c)/H_s$. For the zero value the crest height is equal to the 2% runup height. For negative values the crest height is even higher and overtopping will be (very) small. For a value of 1.5 the crest level is 1.5 H_s lower than the 2% runup height and overtopping will obviously be large. The vertical axis gives the logaritmic of the mean overtopping discharge $\bar{Q}/\sqrt{gH_s^3}$.

Fig. 18 gives about 500 data points. The formula that describes more or less the average of the data is given by an exponential function (according to Owen 1980):

$$\mu(\bar{Q}) = 8.10^{-5} \sqrt{gH_s^3} \exp[3.1(Ru_{2\%}-R_c)/H_s] \qquad (25)$$

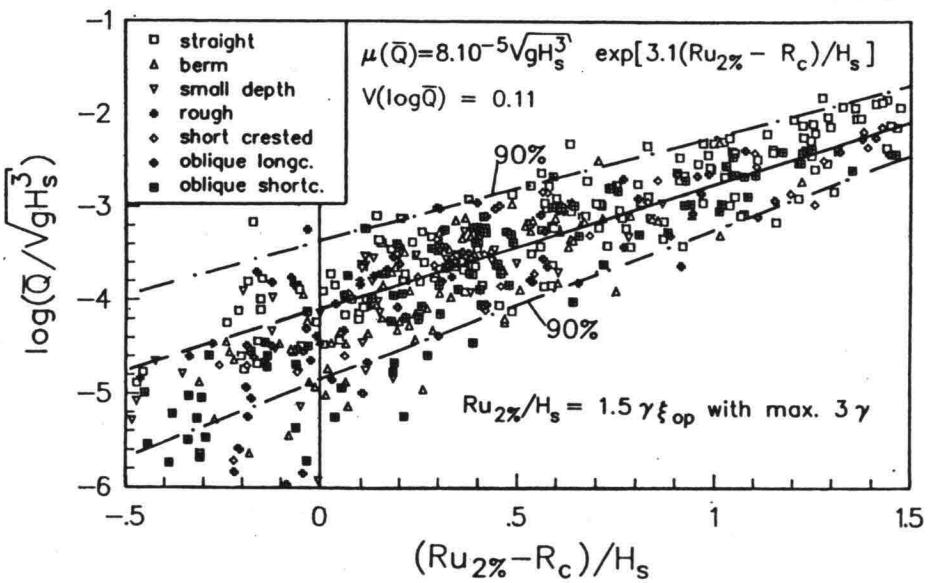


Fig. 18. Final results on wave overtopping of slopes

The reliability of Eq. 25 can be given by assuming that log $\bar{Q}$ (and not $\bar{Q}$) has a normal distribution with a variation coefficient $V = \sigma/\mu = 0.11$. Reliability bands can than be calculated for various practical values of mean overtopping discharges. The 90% reliability bands for some overtopping discharges are:

mean discharge	90% reliability bands
0.1 l/s per m	0.02 to 0.5 l/s per m
1.0 l/s per m	0.3 to 3.5 l/s per m
10 l/s per m	4.4 to 23 l/s per m

Surprisingly there is very little data available describing the overtopping performance of rock armoured sea walls without crown walls. However the results from two tests by Bradbury et al (1988) may be used to give estimates of the influence of wave conditions and relative freeboard. Again the test results have been used to give values of coefficients in an empirical equation. To give the best fit to the prediction equation, Bradbury et al have revised Owen's parameter R_m^* to give F^*:

$$F^* = R_c/H_s * R_m^* = [R_c/H_s]^2 \sqrt{s_m/2\pi} \quad (26)$$

Predictions of overtopping discharge can then be made using

$$Q_m^* = a(F^*)^{-b} \quad (27)$$

Values of a and b have been calculated from the results of tests with a rock armoured slope at 1:2 with the crest details shown in Figure 19. For section A, $a = 3.7*10^{-10}$ and $b = 2.92$. For section B, $a = 1.3*10^{-9}$ and $b = 3.82$.

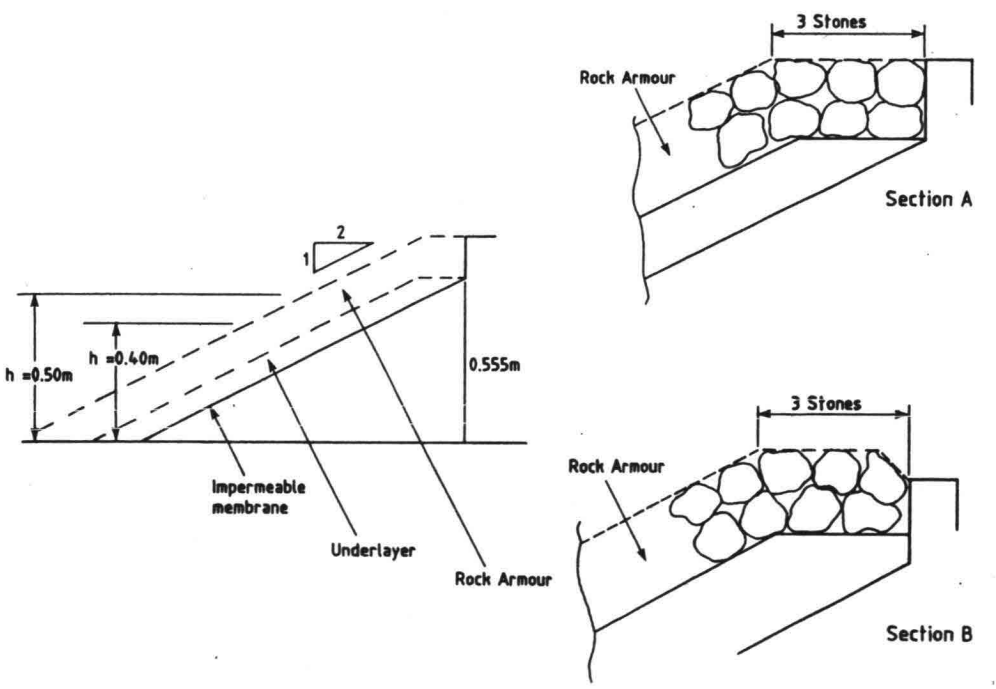


Fig. 19. Overtopped rock structures with low crown wall

3.4 TRANSMISSION

Structures such as breakwaters constructed with low crest levels will transmit wave energy into the area behind the breakwater. The transmission performance of low-crested breakwaters is dependent upon the structure geometry, principally the crest freeboard, crest width and water depth, but also the permeability; and on the wave conditions, principally the wave height and period.

Hydraulic model test results measured by Seelig (1980), Allsop & Powell (1985), Daemrich & Kahle (1985), Ahrens (1987) and van der Meer (1988-1) have been re-analysed by Van der Meer (1990-2) to give a single prediction method. This relates K_t to the relative crest freeboard, R_c/H_s. The data used is plotted in Fig. 20. The prediction equations describing the data may be summarised:

Range of validity	Equation	
$-2.00 < R_c/H_s < -1.13$	$K_t = 0.80$	(28)
$-1.13 < R_c/H_s < 1.2$	$K_t = 0.46 - 0.3R_c/H_s$	(29)
$1.2 < R_c/H_s < 2.0$	$K_t = 0.10$	(30)

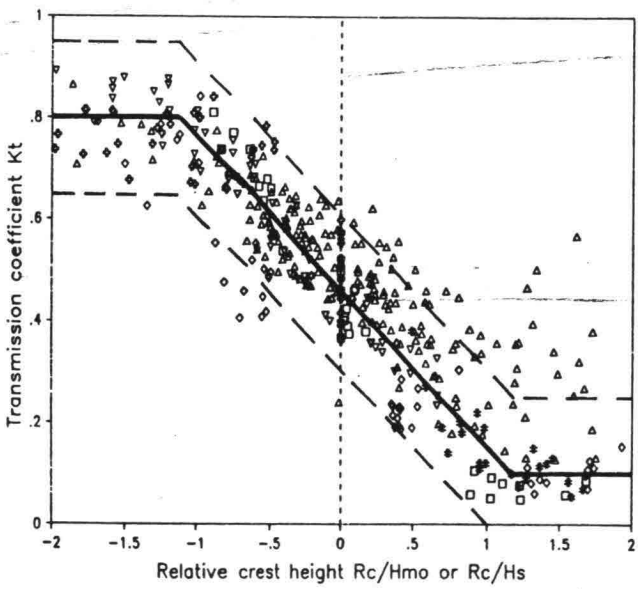


Fig. 20. Wave transmission over and through low-crested structures

These equations give a very simplistic description of the data available, but will often be sufficient for a preliminary estimate of performance. The upper and lower bounds of the data considered is given by lines 0.15 higher, or lower, than the mean lines given above. This corresponds with the 90% confidence bands (the standard deviation was 0.09).

A second analysis on the data was performed by Daemen (1991) and he performed also more tests on wave transmission. A summary has been described by Van der Meer and d'Angremond (1991) and is given here.

Until now wave transmission has been described in the conventional way as a function of R_c/H_i. It is not clear, however, that the use of this combination of crest freeboard and wave height produces similar results with on the one hand constant R_c and variable H_i and on the other variable R_c and constant H_i. Moreover, when R_c becomes zero, all influence of the wave height is lost which leads to a large spreading in the figure at $R_c = 0$.

Therefore, it was decided to separate R_c and H_i in the second analysis. The mass or nominal diameter of the armour layer of a rubble mound structure is determined by the extreme wave attack that can be expected during the

life time of the structure. There is a direct relationship between the design wave height and the size of armour stone, which is often given as the stability factor $H_s/\Delta D_{n50}$, where Δ is the relative buoyant density. It can be concluded that the nominal diameter of the armour layer characterises the rubble mound structure. It is, therefore, also a good parameter to characterise the wave height and crest height in a dimensionless way.

The relative wave height can then be given as H_i/D_{n50}, in accordance with the stability factor, and the relative crest height as R_c/D_{n50}, the number of rocks that the crest level is above or below SWL. Finally, D_{n50} can be used to describe other breakwater properties as the crest width B. This yields the parameter B/D_{n50}.

The primary parameters for wave transmission can now be given as:

Relative crest height : R_c/D_{n50}

Relative wave height : H_i/D_{n50}

Fictitious wave steepness : s_{op}

Secondary parameters are relative crest width, B/D_{n50}, permeability factor, P, and slope angle cotα. Furthermore, it should be noted that reef type breakwaters differ considerably from the conventional rubble mound structure.

The outcome of the second analysis on wave transmission, including the data of Daemen (1991), was a linear relationship between the wave transmission coefficient K_t and the relative crest height R_c/D_{n50}, which is valid between minimum and maximum values of K_t. In Fig. 21 the basic graph is shown. The linearly increasing curves are presented by:

$$K_t = a\, R_c/D_{n50} + b \tag{31}$$

with: $$a = 0.031\, H_i/D_{n50} - 0.24 \tag{32}$$

Eq. 32 is applicable for conventional and reef type breakwaters. The coefficient "b" for conventional breakwaters is described by:

$$b = -5.42\, s_{op} + 0.0323\, H_i/D_{n50} - 0.0017\, (B/D_{n50})^{1.84} + 0.51 \tag{33}$$

and for reef type breakwaters by:

$$b = -2.6\, s_{op} - 0.05\, H_i/D_{n50} + 0.85 \tag{34}$$

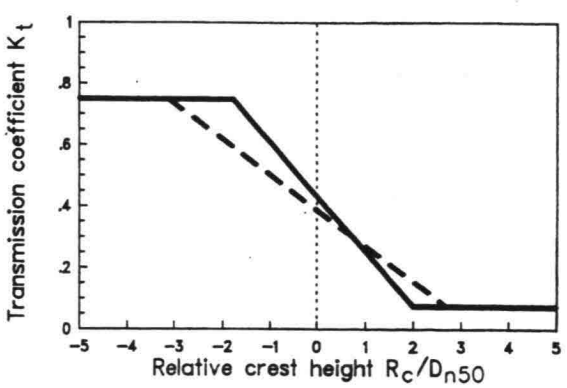


Fig. 21. Basic graph for wave transmission

The following minimum and maximum values were derived:

Conventional breakwaters:
Minimum: $K_t = 0.075$; maximum: $K_t = 0.75$ (35)

Reef-type breakwaters:
Minimum: $K_t = 0.15$; maximum: $K_t = 0.60$ (36)

The analysis was based on various groups with constant wave steepness and a constant relative wave height. The validity of the wave transmission formula (Eq. 31) corresponds, of course, with the ranges of these groups that were used. The formula is valid for:

$$1 < H_i/D_{n50} < 6 \text{ and } 0.01 < s_{op} < 0.05$$

Both upper boundaries can be regarded as physically bound. Values of $H_i/D_{n50} > 6$ will cause instability of the structure and values of $s_{op} > 0.05$ will cause waves breaking on steepness. In fact, boundaries are only given for too low wave heights relative to the rock diameter and for very low wave steepnesses.

The formula is applicable outside the range given above, but the reliability is low. Fig. 22 shows the measured wave transmission coefficient versus the calculated one from Eq. 31, for various data sets of conventional breakwaters. The reliability of the formula can be described by assuming a normal distribution around the line in Fig. 22. With the restriction of the range of application given above, the standard deviation amounted to $\sigma(K_t) = 0.05$, which means that the 90 per cent confidence levels can be given by $K_t \pm 0.08$. This is a remarkable increase in reliability compared to the simple formula given by Eqs. 28 - 30 and Fig. 20, where a standard deviation of $\sigma(K_t) = 0.09$ was given.

The reliability of the formula for reef-type breakwaters is more difficult to describe. If only tests are taken where the crest height had been lowered less than 10 per cent of the initial height h_c, and the test conditions lie within the range of application, the standard deviation amounts to $\sigma(K_t) = 0.031$. If the restriction on the crest height is not taken into account the standard deviation amounts to $\sigma(K_t) = 0.054$.

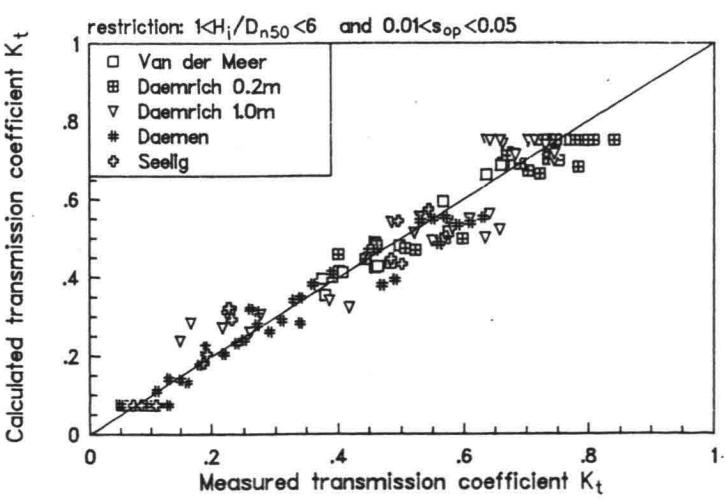


Fig. 22. Calculated versus measured wave transmission for conventional breakwaters

3.5 REFLECTIONS

Waves will reflect from nearly all coastal or shoreline structures. For structures with non-porous and steep faces, approximately 100% of the wave energy incident upon the structure will reflect. Rubble slopes are often used in harbour and coastal engineering to absorb wave action. Such slopes will generally reflect significantly less wave energy than the equivalent non-porous or smooth slope. Although some of the flow processes are different, it has been found convenient to calculate the reflection performance given by C_r using an equation of the same form as for non-porous slopes, but with different values of the empirical coefficients to match the alternative construction. Data for random waves is available for smooth and armoured slopes at angles between 1:1.5 and 1:2.5 (smooth) and 1:1.5 and 1:6 (rock).

Data of Allsop and Channell (1988) will be given here and data of Van der Meer (1988-1), analysed by Postma (1989). Formulae of other references will be used for comparison.

Battjes (1974) gives for smooth impermeable slopes:

$$C_r = 0.1\xi^2 \tag{37}$$

Seelig and Ahrens (1981) give:

$$C_r = a\,\xi_p^2/(b + \xi_p^2) \tag{38}$$

with:
a = 1.0, b = 5.5 for smooth slopes
a = 0.6, b = 6.6 for a conservative estimate of rough permeable slopes

Eqs. 37 and 38 are shown in Fig. 23 together with the reflection data of Van der Meer (1988-1) for rock slopes. The two curves for smooth slopes are close. The curve of Seelig and Ahrens for permeable slopes is not a conservative estimate, but even underestimates the reflection for large ξ_p values.

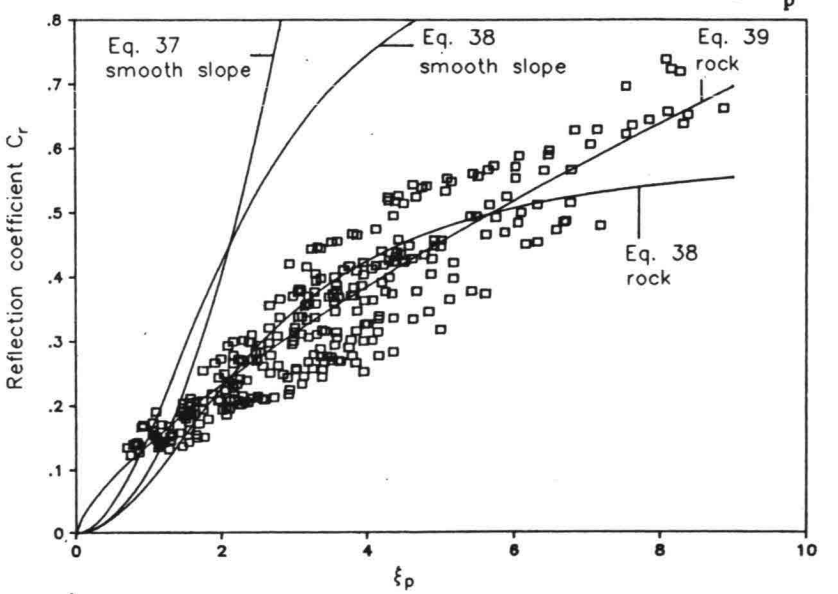


Fig. 23. Comparison of data on rock slopes of Van der Meer (1988-1) with other formulae

The best fit curve through all the data points in Fig. 23 is given by Postma (1989) and is also given in Fig. 23:

$$C_r = 0.14\ \xi_p^{0.73} \qquad \text{with } \sigma(C_r) = 0.055 \tag{39}$$

The surf similarity parameter did not describe the combined slope angle-wave steepness influence in a sufficient way. Therefore, both the slope angle and wave steepness were treated separately and Postma derived the following relationship:

$$C_r = 0.071\ P^{-0.082}\ \cot\alpha^{-0.62}\ s_{op}^{-0.46} \tag{40}$$

with:
$\sigma(C_r) = 0.036$
P = notional permeability factor described in Sect. 2.2.4

The standard deviation of 0.055 in Eq. 39 reduced to 0.036 in Eq. 40 which is a considerable increase in reliability.

The results of random wave tests by Allsop & Channell (1989), analysed to give values for the coefficients a and b in equation 38 (but with ξ_m instead of ξ_p) is presented below. The rock armoured slopes used rock in 2 or 1 layer, placed on an impermeable slope covered by underlayer stone, equivalent to P = 0.1. The range of wave conditions for which these results may be used is given by:

$0.004 < s_m < 0.052$, and $0.6 < H_s/\Delta D_{n50} < 1.9$.

Slope type	a	b
Smooth	0.96	4.80
Rock, 2 layer	0.64	8.85
Rock, 1 layer	0.64	7.22

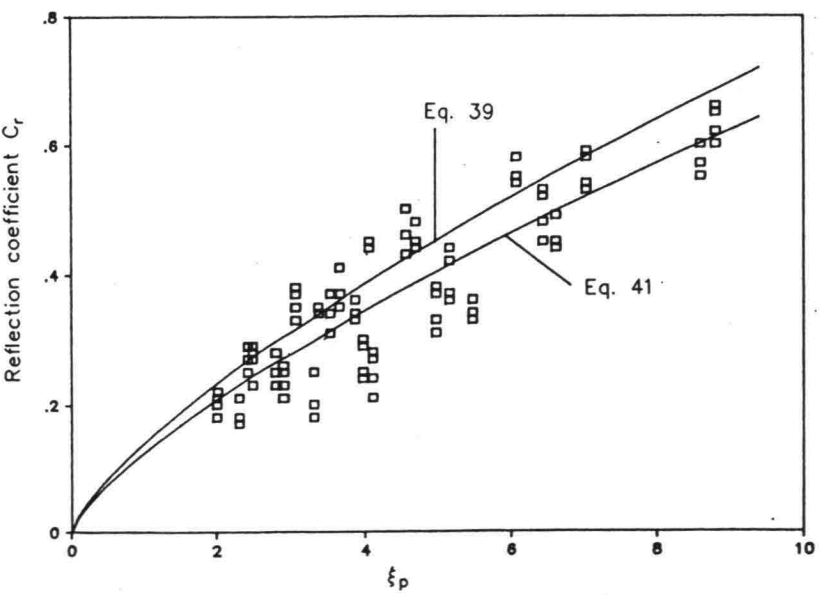


Fig. 24. Data of Allsop and Channel (1989)

Postma (1989) also re-analysed the data of Allsop and Channell which were described above. Fig. 24 gives the data of Allsop and Channel together with Eq. 39. The curve is a little higher than the average of the data. The best fit curve is described by:

$$C_r = 0.125\ \xi_p^{0.73} \qquad \text{with } \sigma(C_r) = 0.060 \tag{41}$$

There are no reliable general data available on the reflection performance of rough, non-porous, slopes. In general a small reduction in the level of reflections might be expected, much as for wave run-up. Reduction factors have not, however, been derived from tests. It is not therefore recommended that values of C_r lower than for the equivalent smooth slope be used, unless test data is available.

4. Structural response

4.1 INTRODUCTION

The hydraulic and structural parameters are described in Chapter 2 and the hydraulic responses in Chapter 3. Figure 3 gives an overview of the definitions of the hydraulic parameters and responses as wave run-up, run-down, overtopping, transmission and reflection. Figure 5 gives an overview of the structural parameters which are related to the cross-section. The response of the structure under hydraulic loads will be described in this Chapter and design tools will be given.

The design tools given in this Chapter will be able to design a lot of structure types. Nevertheless it should be remembered that each design rule has its limitations. For each structure which is important and expensive to built, it is advised to perform physical model studies.

Figure 25 gives the same cross-section as in Fig. 5, but it shows now the various parts of the structure which will be described in the next Sections. Some general points and design rules for the geometrical design of the cross-section will be given here. These are:

- The minimum crest width.
- The thickness of (armour layers).
- The number of units or rocks per surface area.
- The bottom elevation of the armour layer.

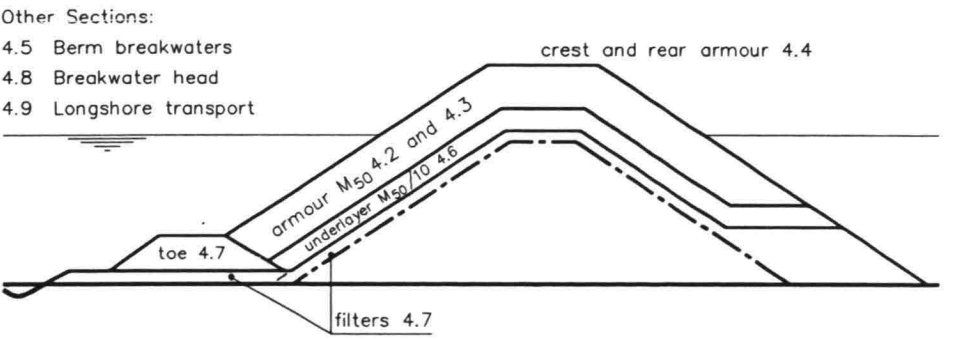


Fig. 25. Various parts of a structure

The crest width is often determined by construction methods used (access on the core by trucks or crane) or by functional requirements (road/crown wall on the top). In case the width of the crest can be small a required minimum width should be taken. According to the SPM (1984) this minimum width is:

$$B_{min} = (3 - 4)\, D_{n50} \qquad (42)$$

The thickness of layers and the numbers of units per m² are given in Box 4. The number of units in a rock layer depends on the grading of the rock. The values of k_t that are given in the Box describe a rather narrow grading (uniform stones). For riprap and even wider graded material the number of stones can not easily be estimated. In that case the volume of the rock on the structure can be used.

Box 4 Thickness of layers and number of units

The thickness of layers is given by:

$$t_a = t_u = t_f = n\, k_t\, D_{n50} \qquad (43)$$

The number of units per m² is given by:

$$N_a = n\, k_t\, (1-n_v) D_{n50}^{-2} \qquad (44)$$

Where: t_a, t_u, t_f = thickness of armour, underlayer or filter
n = number of layers
k_t = layer thickness coefficients
n_v = volumetric porosity

Values of k_t and n_v are taken from the SPM (1984)

	k_t	n_v
smooth rock, n = 2	1.02	0.38
rough rock, n = 2	1.00	0.37
rough rock, n > 3	1.00	0.40
graded rock	-	0.37
cubes,	1.10	0.47
tetrapods,	1.04	0.50
dolosse,	0.94	0.56

The bottom elevation of the armour layer should be extended downslope to an elevation below minimum SWL of at least one (significant) wave height, if the wave height is not limited by the water depth. Under depth limited conditions the armour layer should be extended to the bottom as shown in Fig. 25 and supported by a toe.

4.2 ROCK ARMOUR LAYERS

Many methods for the prediction of rock size of armour units designed for wave attack have been proposed in the last half century. Those treated in more detail here are the Hudson formula as used in the Shore Protection paper (1984) and the formulae derived by Van der Meer (1988-1).

The original Hudson formula is written by:

$$M_{50} = \frac{\rho_r H^3}{K_D \Delta^3 \cot\alpha} \qquad (45)$$

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 paper 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 Section 2.1.1. A breaking wave in formula 45 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, it was suggested to use H_s in Eq. 45. By 1984 the advice given was more cautious. The SPM now recommends $H = H_{10}$, 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:
- Potential scale effects due to the small scales at which most of the tests were conducted,
- 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 without this $K_D\cot\alpha$. Further, it may often be more helpful to work in terms of a linear armour size, such as a typical or nominal diameter. The Hudson formula can be re-arranged to:

$$H_s/\Delta D_{n50} = N_s = (K_D \cot\alpha)^{1/3} \tag{46}$$

Eq. 46 shows that the Hudson formula can be written in terms of the structural parameter $H_s/\Delta D_{n50}$ which was discussed in Section 2.2.1.

Based on earlier work of Thompson and Shuttler (1975) an extensive series of model tests was conducted at Delft Hydraulics (Van der Meer (1988-1), Van der Meer (1987), Van der Meer (1988-2)). The tests included 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. These formulae may be written as:

for plunging waves:

$$H_s/\Delta D_{n50} = 6.2\ P^{0.18}\ (S/\sqrt{N})^{0.2}\ \xi_m^{-0.5} \tag{47}$$

and for surging waves:

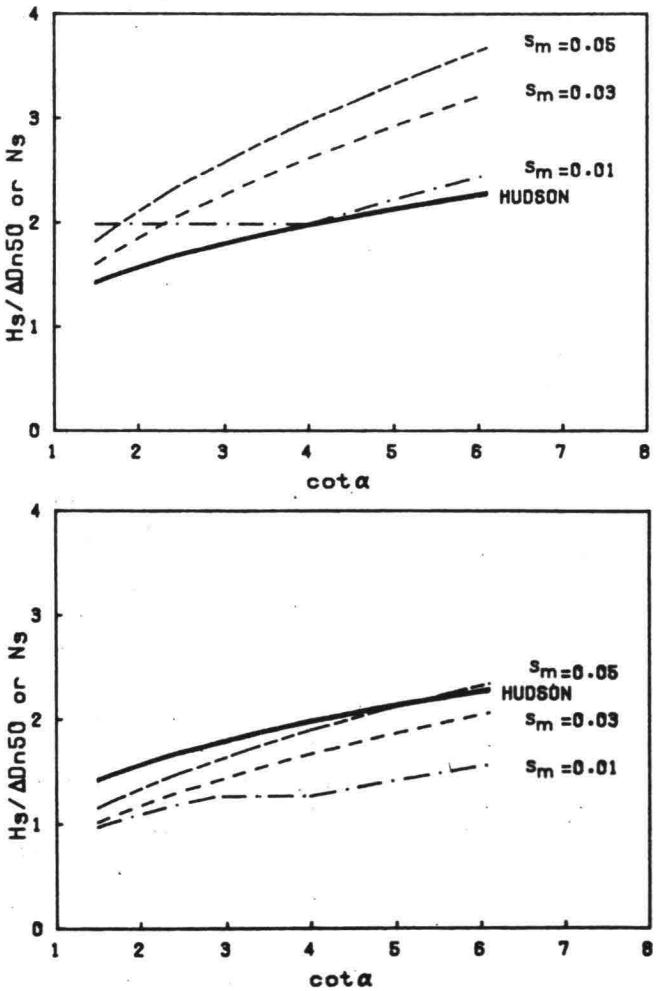
$$H_s/\Delta D_{n50} = 1.0\ P^{-0.13}\ (S/\sqrt{N})^{0.2}\ \sqrt{\cot\alpha}\ \xi_m^{P} \tag{48}$$

The transition from plunging to surging waves can be calculated using a critical value of ξ_m:

$$\xi_{mc} = [6.2\ P^{0.31}\ \sqrt{\tan\alpha}]^{1/(P+0.5)} \tag{49}$$

Box 5 **Comparison of Hudson and new formulae**

The $H_s/\Delta D_{n50}$ in the Hudson formula is only related to the slope angle cotα. Therefore a plot of $H_s/\Delta D_{n50}$ or N_s versus cotα as shows one curve for the Hudson formula. Formulae 47 - 49 take into account the wave period (or steepness), the permeability of the structure and the storm duration. The effect of these parameters are shown here.



The upper graph shows the curves for a permeable structure after a storm duration of 1000 waves (a little more than the number used by Hudson). The lower graph gives the stability of an impermeable revetment after wave attack of 5000 waves (equivalent to 5 - 10 hours in nature. Curves are shown for various wave steepnesses.

For $\cot\alpha \geq 4.0$ the transition from plunging to surging does not exists and for these slope angles only Eq. 47 should be used. All parameters used in Eqs. 47-49 are described in Chapter 2. The notional permeability factor P is shown in Fig. 6. The factor P should lie between 0.1 and 0.6.

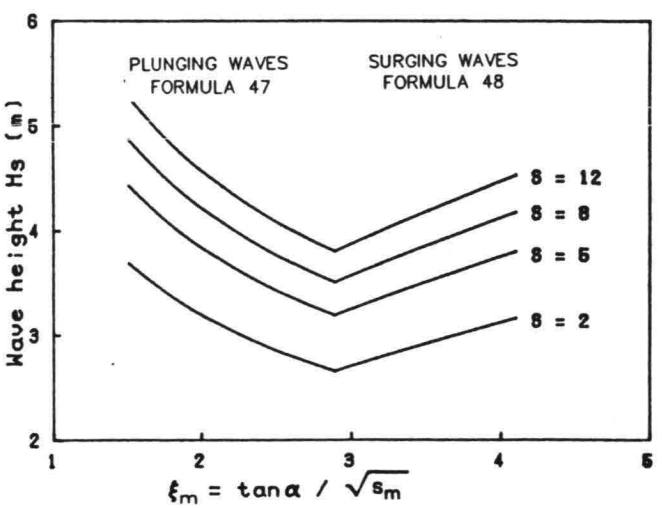
Design values for the damage level S are shown in Table 1. The level "start" of damage, S = 2 - 3, is equal to the definition of "no damage" in the Hudson formula, Eq. 46. The maximum number of waves N which should be used in Eqs. 47 and 50 is 7500. After this number of waves the structure more or less has reached an equilibrium.

The wave steepness should lie between $0.005 < s_m < 0.06$ (almost the complete possible range). The relative mass density varied in the tests between 2000 kg/m³ and 3100 kg/m³, which is also the possible range of application.

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 the formulae 47 and 49 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 0.8 (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 $K_D^{1/3}$ (with mean K_D of 4.5). With these standard deviations it is simple to include 90% or other confidence bands.

Equations 47 - 49 are more complex than the Hudson formula 46. 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 between the Hudson formula and Eqs. 47 - 49. Box 5 gives a comparison between the formulae.

Box 6 H_s **versus** ξ_m **graph (influence damage levels)**



The parameter which influence is shown 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, cotα = 3.0, P = 0.5 and N = 3000.

Nevertheless, it is more difficult to work with Eqs. 47 - 51. 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 boxes 6 - 8. Two for a wave height versus surf similarity plot, which shows the influence of both wave height and wave steepness (the wave climate). The other for 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 Eqs. 47-49 for other parameters, see Van der Meer (1988-2).

An estimation of the damage profile of a straight rock slope can be made by use of Eqs. 47 and 48 and some additional relationships for the profile. The profile can be schematised to an erosion area around swl, an accretion area below swl, 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 swl, see Fig. 26. 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 (1988-1) 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 Eqs. 47 and 48. The method is only applicable for straight slopes.

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 Boxes 6 - 8) and if a sensitivity analysis is performed. Another design procedure is the probabilistic approach. Eqs. 47 and 49 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 (1988-2).

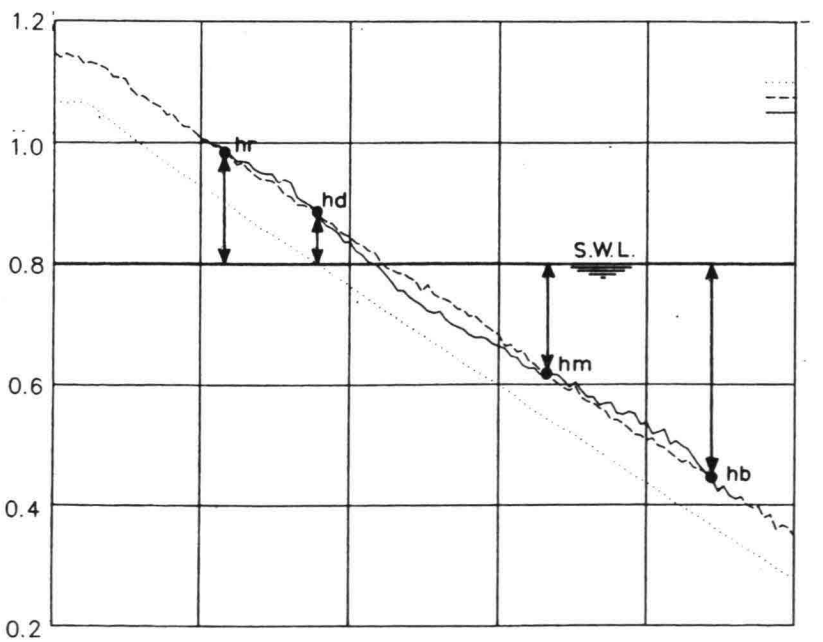


Fig. 26. Damage profile of a statically stable rock slope

The structure parameters with the mean value, distribution type and standard deviation are given in Table 4. 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 calculate the probability that a certain damage level would be exceeded in a certain life time of the structure.

Parameter	Distribution	Average	Standard deviation
D_{n50}	Normal	1.0	0.03
Δ	Normal	1.6	0.05
cotα	Normal	3.0	0.15
P	Normal	0.5	0.05
N	Normal	3000	1,500
H_s	Weibull	B=0.3	C=2.5
FH_s	Normal	0	0.25
s_m	Normal	0.04	0.01
a (Eq. 49)	Normal	6.2	0.4
b (Eq. 50)	Normal	1.0	0.08

Table 4. Parameters used in Level II probabilistic computations

The parameter FH_s describes 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.

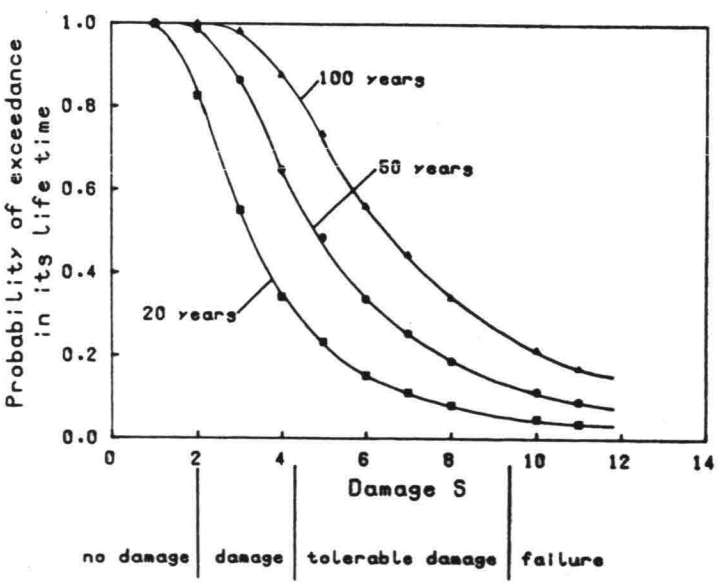


Fig. 27. Probability of exceedance of the damage level S in the life time of the structure

The final results are shown in Fig. 27 where the damage S is plotted versus the probability of exceedance in the life time of the structure. From this Figure follows that start of damage (S = 2) will certainly occur in a life time 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 Fig. 27 can be used to make a cost optimization for the structure during its lifetime, including maintenance and repair at certain damage levels.

Up to now the significant wave height H_s was 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 (1988-1) showed that $H_{2\%}$ was the best value for the design. This means that the stability of the armour layer in depth limited situations is better described by $H_{2\%}$ than by H_s. Eqs. 47 - 49 can be re-arranged with the known ratio of $H_{2\%}/H_s$. The equations become:

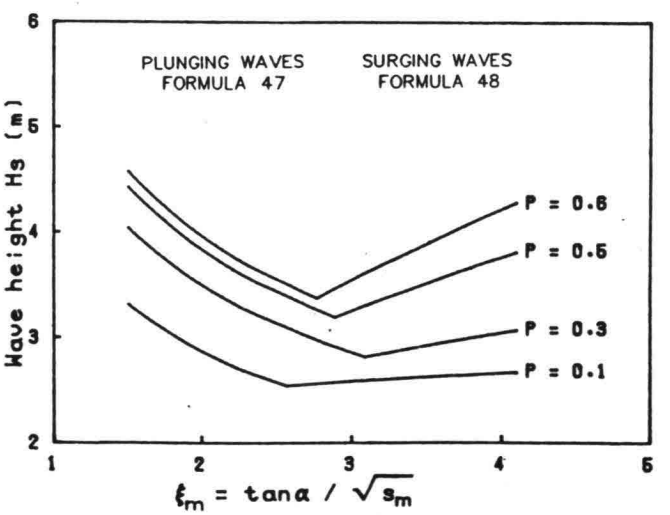
For plunging waves:

$$H_{2\%}/\Delta D_{n50} = 8.7\ P^{0.18}\ (S/\sqrt{N})^{0.2}\ \xi_m^{-0.5} \qquad (50)$$

and for surging waves:

$$H_{2\%}/\Delta D_{n50} = 1.4\ P^{-0.13}\ (S/\sqrt{N})^{0.2}\ \sqrt{\cot\alpha}\ \xi_m^{P} \qquad (51)$$

Box 7 H_s versus ξ_m graph (influence permeability)

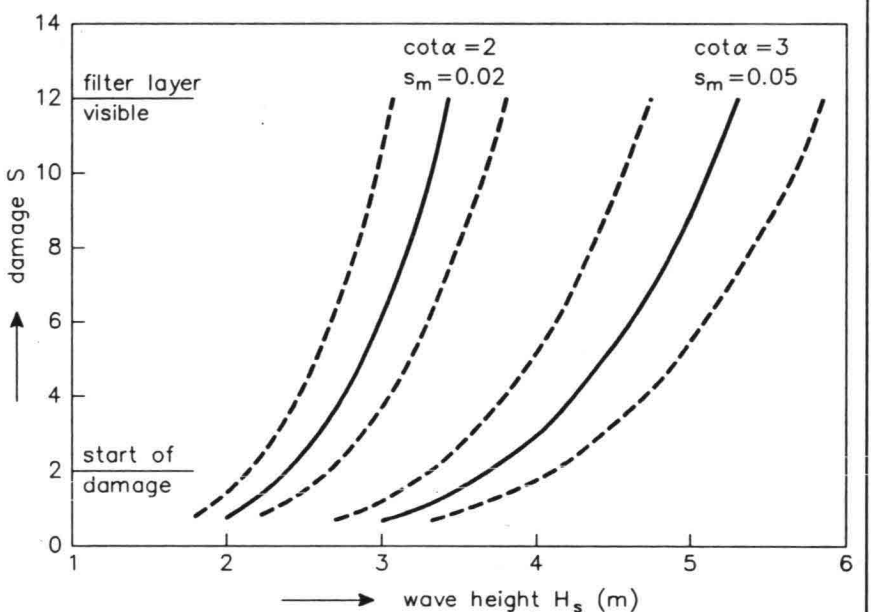


The parameter which influence is shown is the notional permeability factor P. Four values are shown: P = 0.1 (impermeable core), P = 0.3 (some permeable core), P = 0.5 (permeable core) and P = 0.6 (homogeneous structure). The structure itself is described by: D_{n50} = 1.0 m (M_{50} = 2.6 t), Δ = 1.6,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

Eqs. 50 - 51 take into account the effect of depth limited situations. A boundary condition is that one should know the ratio of $H_{2\%}/H_s$ for the depth limited situation. A value of 1.4 can be taken as a maximum (Rayleigh distribution) and a value of 1.1 - 1.2 for very severe wave breaking on a gentle foreshore.

A save approach, however, is to use Eqs. 47 and 48 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 save approach. If the wave heights are Rayleigh distributed Eqs. 50 and 51 give the same results as Eqs. 47 and 48, as this is caused by the known ratio of $H_{2\%}/H_s = 1.4$. As said above, 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.

Box 8 Wave height - damage graph



D_{n50} = 1 m Δ = 1.6 N = 3000 P = 0.5

Two curves are shown, one for a slope angle with cotα = 2.0 and a wave steepness of s_m = 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, plots as shown in this box are very useful to determine the stability of the armour layer of the structure. The graph 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.

Box 9 **Very wide gradings**

Normal wide gradings have $D_{85}D_{15} < 2.5$. Allsop (1990) undertook a model study on the stability of very wide gradings with $D_{85}/D_{15} = 4.0$. A limited number of tests were performed on a 1:2 structure with an impermeable core (revetment). The tests showed first displacement of small rock and then larger rock. Two tests showed large damage (S = 10-13) and a repetition of these tests with exactly the same conditions showed no damage at all (S = 2). This large scatter may be an effect of the very wide grading.

Furthermore, it is difficult to obtain a good gradation all along the structure and the M_{50} or D_{n50}, may change considerably along the structure.

Based on the model tests and the difficulties in construction of a homogeneous armour layer it is advised not to use very wide gradings ($D_{85}/D_{15} > 2.5$) for a revetment. It might be possible to use very wide gradings for reef type structures which consist only of a homogeneous mass of stone. Model tests are required in that case. Box 2 gives examples of narrow, wide and very wide gradings.

4.3 ARMOUR LAYERS WITH CONCRETE UNITS

The Hudson formula 46 was given in Section 4.2 with K_D values for rock. The Shore Protection paper 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 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 the Shore Protection Manual (1984).

Extended research by Van der Meer (1988-3) 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 (slope angle and permeability) for each armour unit. Therefore the slope angle, $\cot\alpha$, and consequently the surf similarity parameter, ξ_m, is not present in the stability formula developed on the results of the research. The same yields 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 concrete units can be described by the damage number N_o, described in Section 2.2.4. N_{od} is the actual number of displaced units related to a width (along the longitudinal axis of the breakwater) of one nominal diameter, D_n. N_{or} and N_{omov} are respectively the number of rocking units and the number of moving units (displaced + rocking).

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

$$H_s/\Delta D_n = (6.7\ N_{od}^{0.4}/N^{0.3} + 1.0)\ s_m^{-0.1} \qquad (52)$$

For tetrapods:

$$H_s/\Delta D_n = (3.75\ N_{od}^{0.5}/N^{0.25} + 0.85)\ s_m^{-0.2} \qquad (53)$$

For the no-damage criterion $N_{od} = 0$, Eqs. 54 and 55 reduce to:

$$H_s/\Delta D_n = s_m^{-0.1} \qquad (N_{od} = 0, \text{ cubes}) \qquad (54)$$

$$H_s/\Delta D_n = 0.85\ s_m^{-0.2} \qquad (N_{od} = 0, \text{ tetrapods}) \qquad (55)$$

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, $N_{od} = 0$: $H_s/\Delta D_n = 3.7$ (56)

Failure, $N_{od} > 0.5$: $H_s/\Delta D_n = 4.1$ (57)

The reliability of Eqs. 52 - 57 can be described with a similar procedure as for rock. The coefficients 3.7 and 4.1 in Eqs. 55 and 56 for accropode can be considered as stochastic variables with a standard deviation of 0.2. The procedure for Eqs. 51 - 54 is more complicated. Assume a relationship:

$$H_s/\Delta D_n = a\ f(N_{od}, N, s_m) \qquad (58)$$

The function $f(N_{od}, N, s_m)$ is given in Eqs. 52 and 53. 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.

Eqs. 47 and 48 and 52 - 56 describe the stability of rock, cubes, tetrapods and accropode. A comparison of stability is made in Fig. 28 were 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 $H_s/\Delta D_n$ versus the wave steepness, s_m.

From Fig. 28 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. As start of damage and failure are very close for accropode, a safety coefficient should be used for design (for example a factor 1.5 on the $H_s/\Delta D_n$ value).
- 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.

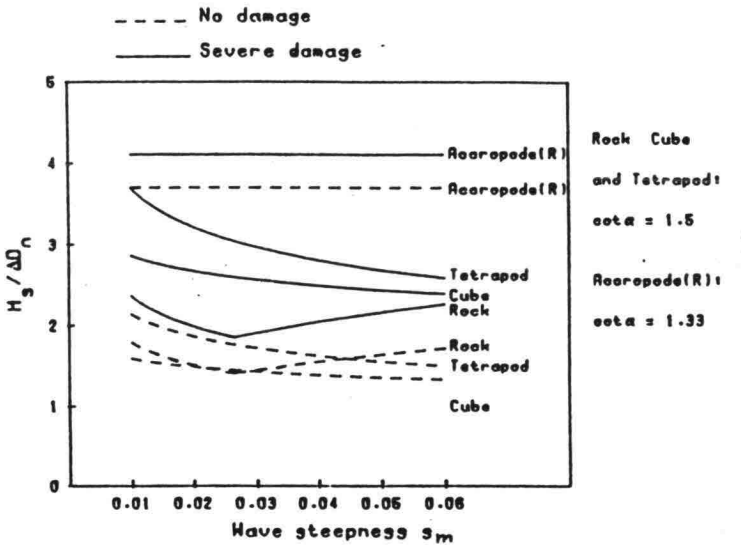


Fig. 28. Comparison of stability of rock, cubes, tetrapods and accropode

Another useful plot tnat directly can be derived from the stability formulae 52 and 53 is a wave height - damage graph. Fig. 29 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 limits 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.

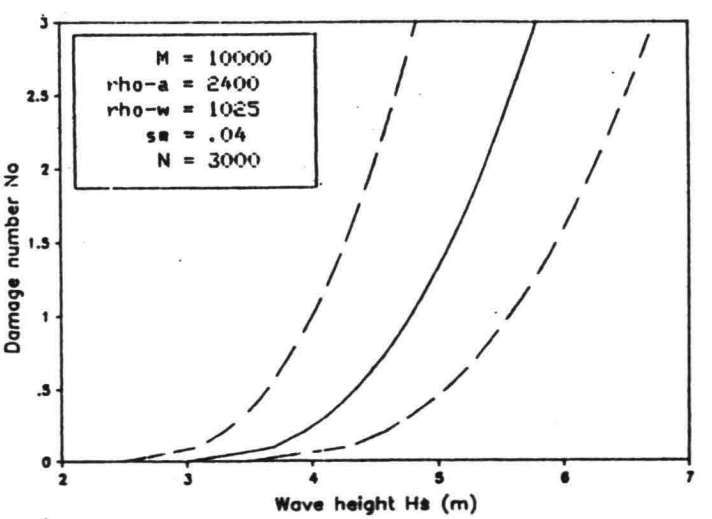


Fig. 29. Wave height - damage curve for cubes with 90% confidence levels

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 when one assumes that each moving unit results in a broken unit. The lower limits (only displaced units) for cubes and tetrapods are given by Eq. 52 and 53. The upper limits (number of moving units) were derived by Van der Meer and Heydra (1990). The Eqs. for the number of moving units are:

$$H_s/\Delta D_n = (6.7\ N_{omov}^{0.4}/N^{0.3} + 1.0)\ s_m^{-0.1} - 0.5 \qquad (59)$$

For tetrapods:

$$H_s/\Delta D_n = (3.75\ N_{omov}^{0.5}/N^{0.25} + 0.85)\ s_m^{-0.2} - 0.5 \qquad (60)$$

The Eqs. are very similar to Eqs. 52 and 53, 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. For armour layers with large concrete units the actual number of broken units will probably lie between the curve of N_{od} (Eqs. 52 and 53) and N_{omov} (Eqs. 59 and 60).

4.4 LOW-CRESTED STRUCTURES

As long as structures are high enough to prevent overtopping, the armour on the crest and rear can be (much) smaller than on the front face. The dimensions of the rock in that case will be determined by practical matters as available rock, etc.

Most structures, however, are designed to have some or even severe overtopping under design conditions. Other structures are so low that also under daily conditions the structure is overtopped. Structures with the crest level around swl and sometimes far below swl will always have overtopping and transmission.

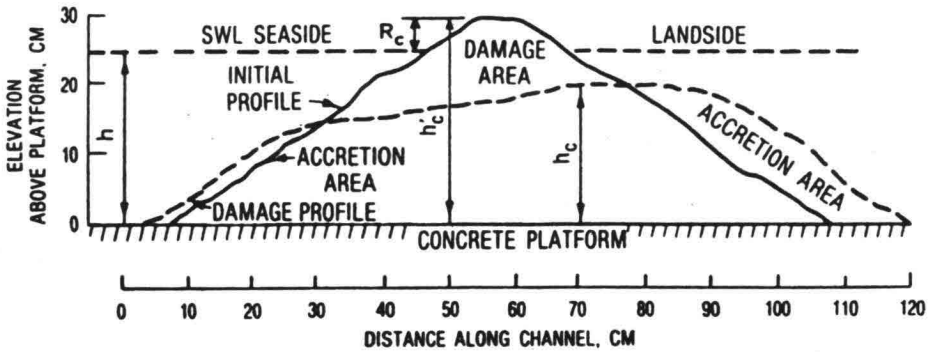
It is obvious that when the crest level of a structure is low, wave energy can pass over the structure. This has two effects. First the armour on the front side can be smaller than on a non-overtopped structure, due to the fact that energy is lost on the front side.

The second effect is that the crest and rear should be armoured with rock which can withstand the attack by overtopping waves. For rock structures the same armour on front face, crest and rear is often applied. The methods to establish the armour size for these structures will be given here. They may not yield for structures with an armour layer of concrete units. For those structures physical model investigations may give an acceptable solution.

Low-crested rock structures can be divided into three categories, also shown in Figs. 30-32.

Dynamically stable reef breakwaters.

A reef breakwater is a low-crested homogeneous pile of stones without a filter layer or core and is allowed to be reshaped by wave attack (Fig. 30). The equilibrium crest height, with corresponding transmission, are the main design parameters. The transmission was already described in Sect. 3.4.



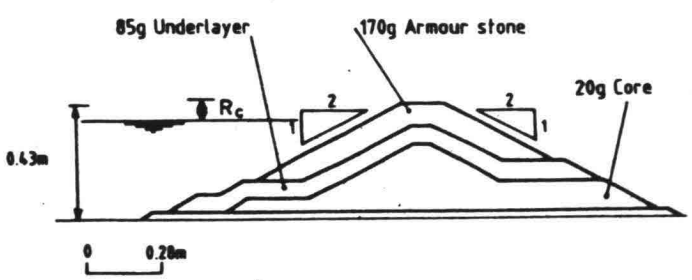
Cross-sectional view of initial and typical damaged reef profiles (swl denotes still-water level)

Ahrens (1987)

Fig. 30. Dynamically stable reef breakwater

Statically stable low-crested breakwaters ($R_c > 0$).

These structures are close to non-overtopped structures, but are more stable due to the fact that a (large) part of the wave energy can pass over the breakwater (Fig. 31).



Powell and Allsop (1985) overtopped breakwater

Fig. 31. Statically stable low-crested breakwater

Statically stable submerged breakwaters (R_c<0).
All waves overtop these structures and the stability increases remarkably if the crest height decreases (Fig. 32).

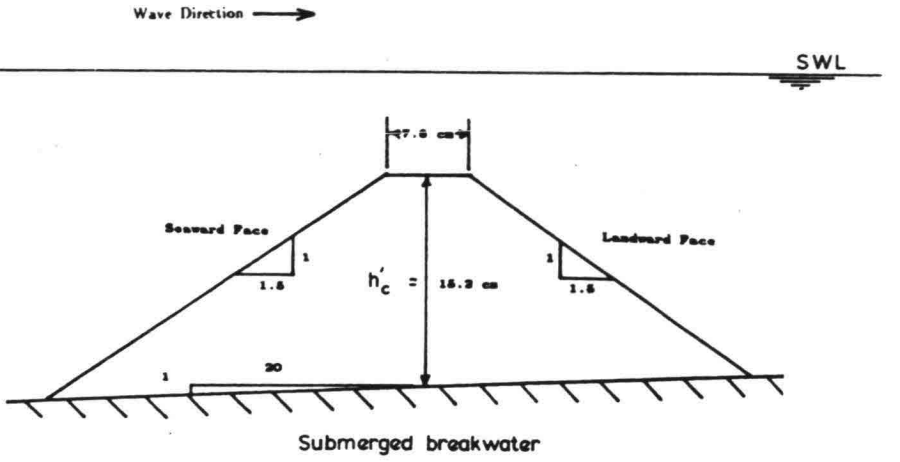


Fig. 32. Submerged breakwater

4.4.1 REEF BREAKWATERS

The analyses of stability by Ahrens (1987) and Van der Meer (1990-1) was concentrated on the change in crest height due to wave attack, see Fig. 30. Ahrens defined a number of dimensionless parameters which described the behaviour of the structure. The main one is the relative crest height reduction factor h_c/h_c'. The crest height reduction factor h_c/h_c' is the ratio of the crest height at the completion of a test to the height at the beginning of the test. The natural limiting values of h_c/h_c' are 1.0 and 0.0 respectively.
Ahrens found for the reef breakwater that a longer wave period gave more displacement of material than a shorter period. Therefore he introduced the spectral (or modified) stability number, N_s^*, defined by Eq. 9.
The relative crest height, according to Van der Meer (1990-1) or Van der Meer and Pilarczyk (1990) can be described by:

$$h_c = \sqrt{A_t/\exp(aN_s^*)} \qquad (61)$$

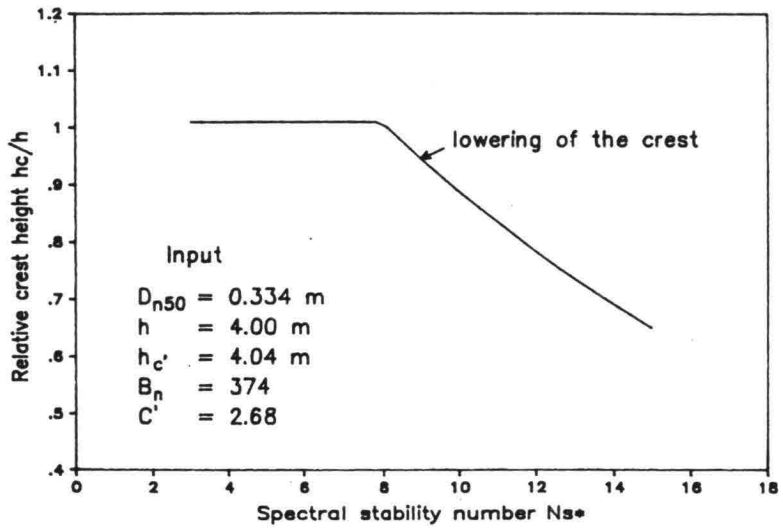
$$\text{with "a"} = -0.028 + 0.045C' + 0.034h_c'/h - 6.10^{-9} B_n^2 \qquad (62)$$

and $h_c = h_c'$ if h_c in Eq. 61 > h_c'.

Eq. 61 was derived by Van der Meer (1990-1), including all Ahrens' (1987) tests. The parameters are given by:
A_t = area of structure cross section,
$C' = A_t/h_c'$ (response slope),
h = water depth at structure toe,
$B_n = A_t/D_{n50}^2$ (bulk number).

The lowering of the crest height of reef type structures as shown in Fig. 30, can be calculated with Eqs. 61 and 62. It is possible to draw design curves from these equations which give the crest height as a function of N_s^* or even H_s. An example of h_c versus N_s^* is shown in Box 10. The reliability of Eq. 61 can be described by giving 90% confidence bands. The 90% confidence bands are given by $h_c \pm 10\%$.

Box 10 **Stability of reef type breakwater**



4.4.2 STATICALLY STABLE LOW-CRESTED BREAKWATER

The stability of a low-crested breakwater (overtopped, $R_c > 0$) can be related to the stability of a non-overtopped structure. Stability formulae as 47 and 48 can be used for example. The required stone diameter for an overtopping breakwater can then be determined by a reduction factor for the mass of the armour, compared to the mass for a non-overtopped structure. The derived equations are based on Van der Meer (1990-1).

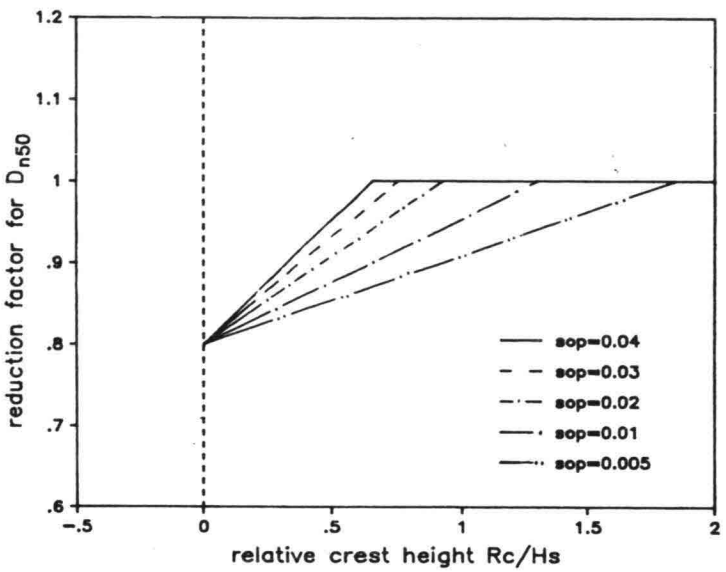
$$\text{Reduction factor for } D_{n50} = 1/(1.25 - 4.8\ R_p^*) \qquad (63)$$

$$\text{for } 0 < R_p^* < 0.052$$

$$\text{where } R_p^* = R_c/H_s \sqrt{s_{op}/2\pi} \qquad (64)$$

The R_p^* parameter is a combination of relative crest height, R_c/H_s and wave steepness s_{op}. Design curves are shown in Box 11.

Box 11 **Design curves for low-crested breakwaters (R_c>0)**



An average stability increase of 20 % is obtained for a structure with the crest level at the water level. The required mass in that case is a factor $(1/1.25)^3 = 0.51$ of that required for a non-overtopped structure.

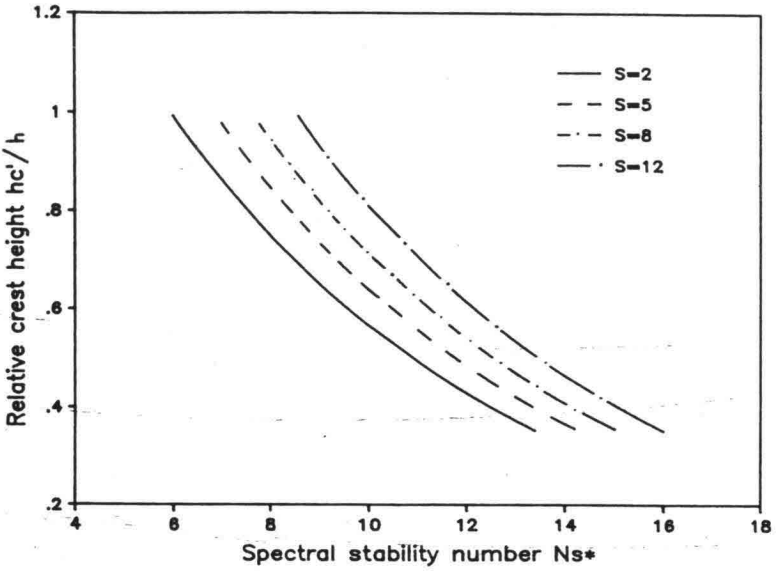
4.4.3 SUBMERGED BREAKWATERS

The stability of submerged breakwaters depends on the relative crest height, the damage level and the spectral stability number. The given formulae are based on a re-analysis of the tests of Givler and Sørensen (1986) by van der Meer (1990-1). The stability is described by:

$$h_c'/h = (2.1 + 0.1\ S)\ \exp(-0.14\ N_s^*) \qquad (65)$$

For fixed crest height, water level, damage level, and wave height and period, the required ΔD_{n50} can be calculated, giving finally the required stone weight. Also wave height versus damage curves can be derived from Eq. 65.

Box 12 **Design curves for sub-merged breakwaters (R_c<0)**



Eq. 65 is shown in the graph for three damage levels and can be used as a design graph. Here again S = 2 is start of damage, S = 5-8 is moderate damage and S = 12 is "failure" (lowering of the crest by more than one D_{n50}

4.5 BERM BREAKWATERS

Statically stable structures can be described by the damage parameter S, see Section 2.2.4. Dynamically stable structures can be described by a profile, see Figs. 8 and 9. Based on extensive model tests (Van der Meer (1988-1)) relationships were established between the characteristic profile parameters as shown in Fig. 8 and the hydraulic and structural parameters. These relationships were used to make the computational model Profiles in the program 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 swl.
- Computation of an (established or assumed) sequence of storms (or tides) by using the previous 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 done on the same initial profile or on the computed profile.

The results of a computation on a berm breakwater is shown in Fig. 33, together with a listing of the input parameters.

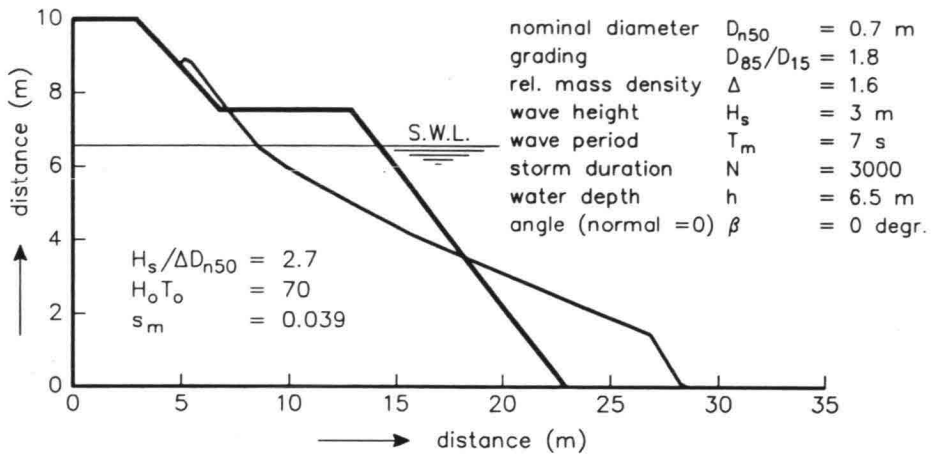


Fig. 33. Example of a computed profile for a berm breakwater

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.

The computational model can be used in the same way as the deterministic design approach of statically stable slopes, described in Section 4.2. There the rather complicated stability formulae 47 and 48 were used to make design graphs such as damage curves and these graphs were used for a sensitivity analysis. By making a lot of computations with the computational model a same kind of sensitivity analysis can be performed for dynamically stable structures. Aspects which were considered for the design of a berm breakwater (Van der Meer and Koster (1988)) were for example:
- Optimum dimensions of the structure (upper and lower slope, length of berm).
- Influence of wave climate, stone class, water depth.
- Stability after first storms.

An example to derive optimum dimensions for a berm breakwater will be described below. The influence of the wave climate on a structure is shown in Box 13. Stability after first storms can possibly be described by use of formulae 47 and 49.

OPTIMUM DIMENSIONS FOR A BERM BREAKWATER (example)

A berm breakwater can be regarded as an unconventional design. Displacement of armour stones in the first stage of its lime time is accepted. After this displacement (profile formation) the structure will be more or less statically stable. The cross-section of a berm breakwater can be described by a lower slope 1:m, a horizontal berm with a length b (just above still water level in this case) and an upper slope 1:n. The lower slope is often steep and close to the natural angle of repose.

The critical design point in the example of Van der Meer and Koster (1988). was that erosion was not allowed at the upper slope above the berm.

The minimum required berm length b was established for this criterion with the computational model. The berm length b was determined for various combinations of m and n. Fig. 34 shows the final results. Each combination of m, n and b from this Fig. gives more or less the same stability (no erosion at the upper slope). It is obvious form Fig. 34 that steep slopes require a longer berm and visa versa.

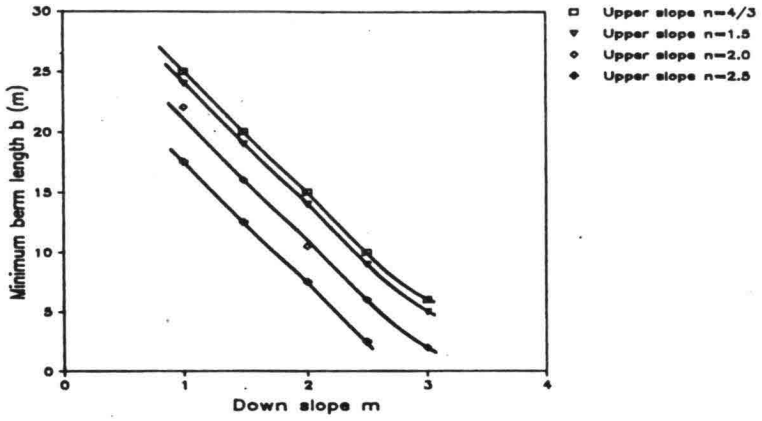


Fig. 34. Minimum berm length as a function of down slope and upper slope for a specific berm breakwater

Fig. 34 gives no information on the optimum values for the slopes. Therefore another criterion was introduced. The amount of stones required for the construction was calculated for each combination of slopes and berm length. This amount of stones (or cross-sectional area), B, was plotted as a function of the upper and down slope and is given in Fig. 35. It shows that the down slope has minor influence on the required amount of stones (almost horizontal lines) and that a steep upper slope reduces this amount considerably. It should be noted that results given in Figs. 34 and 35 were obtained for a specific structure with specific wave boundary conditions and that they are not generally applicable.

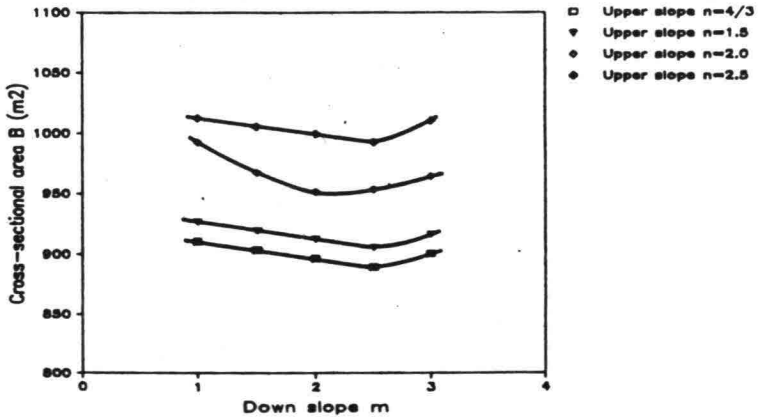
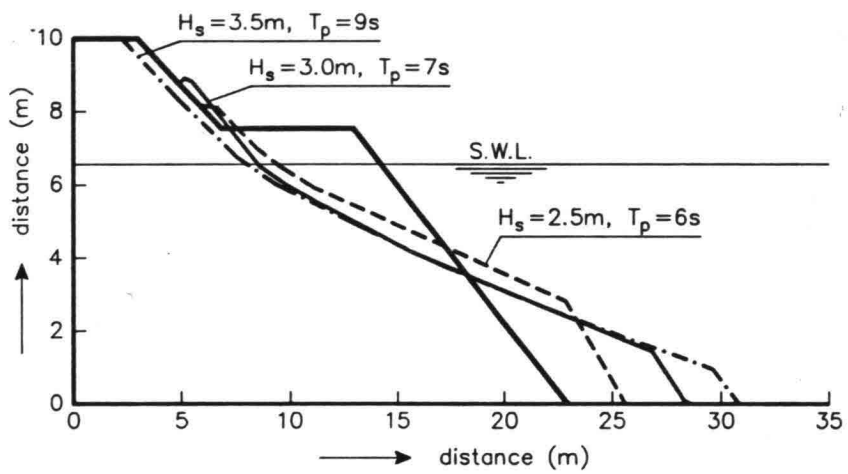


Fig. 35.. Cross-sectional area as a function of down slope and upper slope for a specific berm breakwater

The relationships for the computational model were based on tests in the range of $H_s/\Delta D_{n50}$ = 3-500, see Van der Meer (1988-1). Later on the model was verified specifically for berm breakwaters. This is described by Van der Meer (1990-5). Tests from various institutes all over the world were used for this verification.

The overall conclusion was that the model never showed large unexpected differences with the test results and that in most cases the calculations and measurements were very close. Compaction of material caused by wave attack and damage to the rear of the structure caused by overtopping are not modelled in the program and this was and is a boundary condition for use of the program. The combination of the statically stable formulae or model with the dynamically stable model showed to be a good tool for the prediction of the behaviour of berm breakwaters under all wave conditions.

Box 13 Influence of wave climate on berm breakwater



Design aspects other than the profile development of the seaward side have been investigated in Van der Meer and Veldman (1992). Aspects such as scale effects, rear stability, round head design and longshore transport have been treated there, based on extensive test series on two different berm breakwater designs. A first conclusion is that scale effects were not present in a 1:35 scale model compared with a 1:7 large scale model with wave heights up to 1.7 m.

A first design rule was assessed on the relationship between damage at the rear of a berm breakwater and the crest height, wave height, wave steepness and rock size. The parameter $R_c/H_s * s_{op}^{1/3}$ showed to be a good combination of relative crest height and wave steepness to describe the stability of the rear of a berm breakwater. The following values of $R_c/H_s * s_{op}^{1/3}$ can be given for various damage levels to the rear of a berm breakwater caused by overtopping waves and can be used for design purposes.

$$R_c/H_s * s_{op}^{1/3} = 0.25: \text{ start of damage}$$
$$R_c/H_s * s_{op}^{1/3} = 0.21: \text{ moderate damage} \qquad (66)$$
$$R_c/H_s * s_{op}^{1/3} = 0.17: \text{ severe damage}$$

4.6 UNDERLAYERS AND FILTERS

Rubble mound structures in coastal and shoreline protection are normally constructed with an armour layer and one or more underlayers. Sometimes an underlayer is called a filter. The dimensions of the first underlayer depend on the structure type.

Revetments often have a two diameter thick armour layer, a thin underlayer or filter and than an impermeable structure (clay or sand), with or without a geotextile. The underlayer in this case works as a filter. Small particles beneath the filter should not be washed through the layer and the filter stones itself should not be washed through the armour. In this case the geotechnical filter rules are strongly recommended. Roughly these rules give D_{15}(armour)/D_{85}(filter) < 4 to 5.

Structures as breakwaters have one or two underlayers and than a core of rather fine material (quarry-run). The SPM (1984) recommends for the stone sizes of the underlayer under the armour a range of 1/10 to 1/15 of the armour mass. This criterion is more strict than the geotechnical filter rules and gives D_{n50}(armour)/D_{n50}(underlayer) = 2.2 - 2.3.

A relatively large underlayer has two advantages. First the surface of the underlayer is less smooth with bigger stones and gives more interlocking with the armour. This is specially the case if the armour layer is constructed of concrete armour units. Secondly, a large underlayer gives a more permeable structure and therefore has a large influence on the stability (or required mass) of the armour layer. The influence of the permeability on stability has been described in Section 4.2.

Therefore, it is recommended to use sizes of 1/10 to 1/15 M_{50} of the armour for the mass of the underlayer.

4.7 TOE PROTECTION

In most cases the armour layer on the seaside near the bottom is protected by a toe, see Fig. 36. If the rock in the toe has the same dimensions as the armour, the toe will be stable. In most cases, however, one wants to reduce the size of the stones in the toe. The SPM (1984) shows results of Brebner and Donnelly (1962), who tested toes under monochromatic waves. A relationship is assumed between the ratio h_t/h and the stability number $H/\Delta D_{n50}$ (or N_s), where h_t is the depth of the toe below the water level and h is the water depth (see also Fig. 5). A small ratio of h_t/h = 0.3 -0.5 means that the toe is relatively high above the bottom. In that case the toe structure is more a berm structure. A value of h_t/h = 0.8 means that the toe is near the bottom. $H/\Delta D_{n50}$ values, using regular wave height H (therefore not $H_s/\Delta D_{n50}$!) of 6-7 are recommended if $h_t/h > 0.5$.

Sometimes a relationship between $H_s/\Delta D_{n50}$ and h_t/H_s is assumed where a lower value of h_t/H_s should give more damage. Gravesen and Sørensen (1977) describe that a high wave steepness (short wave period) gives more damage to the toe than a low wave steepness. Above mentioned assumption was based on only a few points. In the CIAD report (1985) this conclusion could not be verified. No relationship was found there between $H_s/\Delta D_{n50}$ and h_t/H_s, probably because H_s is present in both parameters. An average value of $H_s/\Delta D_{n50}$ = 4 was given for no damage and a value of 5 for failure. The standard deviation around these values was 0.8, showing a large seatter.

The results of a more in depth study will be given here. The results presented in the CIAD report were re-analysed and compared with other data. Figure 36 shows the final results. Seven breakwaters (with alternatives) tested at Delft Hydraulics were taken and the behaviour of the toe was examined. The wave boundary conditions for which the criteria "0-3%", 3-10%" and "failure, >20-30%" occurred were established. Here "0-3%" means no movement of stones (or only a few) in the toe. "3-10%" means that the toe flattened out a little, but the function of the toe (supporting the armour

layer) was intact and the damage is acceptable. "Failure" means that the toe has lost its function and this damage level is not acceptable.

In almost all cases the structure was attacked by waves in a more or less depth limited situation, which means that H_s/h was fairly close to 0.5. This is also the reason why it is acceptable that the location of the toe, h_t, is related to the water depth, h. It would not be acceptable for breakwaters in very large water depths (more than 20 - 25 m). The results of the analysis are, therefore, applicable for depth limited situations.

Fig. 36 shows that if the toe is high above the bottom (small h_t/h ratio) the stability is much smaller than for the situation were the toe is close to the bottom. The results of DHI (internal paper) are also shown in the Figure and correspond well with the 3-10% values of Delft Hydraulics. If the curve of Brebner and Donnelly (1962) is added with $H = H_s$, the curve is too low compared with the other results. If one assumes $H = H_{10}$ (as was done in SPM (1984)), the curve corresponds well with the other results.

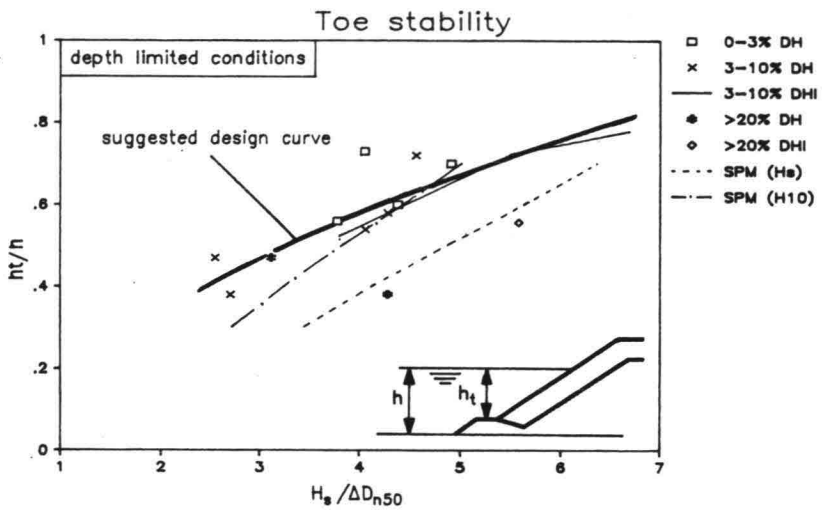


Fig. 36. Toe stability as a function of h_t/h

A suggested line for design purposes is given in the Figure. In general it means that the depth of the toe below the water level is an important parameter. If the toe is close to the bottom the diameter of the stones can be more than twice as small as when the toe is half way the bottom and the water level. Design values for low and acceptable damage (0-10%) and for more or less depth limited situations are:

h_t/h	$H_s/\Delta D_{n50}$
0.5	3.3
0.6	4.2
0.7	5.2
0.8	6.3

A general function between h_t/h and $H_s/\Delta D_{n50}$ is given by:

$$h_t/h = -0.22\ (H_s/\Delta D_{n50})^{0.7} \qquad (67)$$

Based on the limited number of data points it is not easy to give an estimation of the reliability of Eq. 67. Analysis of the variation of the data points gave a standard deviation for the coefficient 0.22 of about 0.02. The 90% confidence bands are reached for $0.22 \pm 1.64\sigma$. The mass for the toe structure is in most cases not related to a maximum available stone size (as it often is for the armour layer). A safe approach for the toe is, therefore, to design on the 90% confidence level in stead of the average (Eq. 67). This leads to:

$$h_t/h = 0.253 \ (H_s/\Delta D_{n50})^{0.7} \qquad (68)$$

Three points are shown in Fig. 36 which indicate failure of the toe. Above given design values are safe for $h_t/h > 0.5$. For lower values of h_t/h one should use the stability formulae for armour stones described in Section 4.2.

4.8 BREAKWATER HEAD

Breakwater heads represent a special physical process. Jensen (1984) described it as follows. "When a wave is forced to break over a roundhead it leads to large velocities and wave forces. For a specific wave direction only a limited area of the head is highly exposed. It is an area around the still water level where the wave orthogonal is tangent to the surface and on the lee side of this point. It is therefore general procedure in design of heads to increase the weight of the armour to obtain the same stability as for the trunk section. Alternatively, the slope of the roundhead can be made less steep, or a combination of both."

An example of the stability of a breakwater head in comparison with the trunk section and showing the location of the damage as described in the previous paragraph is shown in Fig. 37 and was taken from Jensen (1984). The stability coefficient ($H_s/\Delta D_n$ for tetrapods) is related to the stability of the trunk section. Damage is located about 120 - 150° from the wave angle.

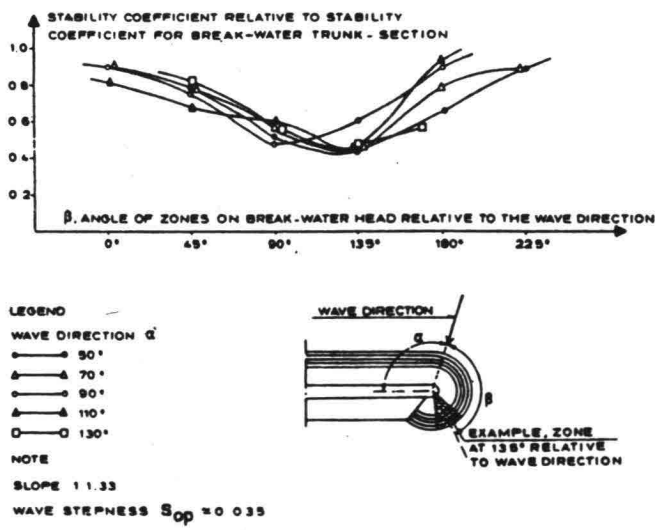


Fig. 37. Stability of a breakwater head armoured with tetrapods (taken from Jensen (1984))

No specific rules are available for the breakwater head. The required increase in weight can be a factor between 1 and 4, depending on the type of armour unit. The factor for rock is more close to 1.

Another aspect of breakwater heads was mentioned by Jensen (1984). The damage curve for a head is often steeper than for a trunk section. A breakwater head may show progressive damage. This means that if both head and trunk were designed on the same (low) damage level, an (unexpected) increase in wave height can cause failure of the head or a part of it, where the trunk still shows acceptable damage. This aspect is less pronounced for heads which are armoured by rock.

Finally the stability of a berm breakwater head should be discussed. Burcharth and Frigaard (1987) have studied longshore transport and stability of berm breakwaters in short basic study. The recession of a breakwater head is shown as an example in Fig. 38, for fairly high wave attack ($H_s/\Delta D_{n50}$ = 5.4). Burcharth and Frigaard (1987) give as a first rule of thumb for the stability of a breakwater head that $H_s/\Delta D_{n50}$ should be smaller than 3.

Tests on a berm breakwater head by van der Meer and Veldman (1992) showed that increasing the height of the berm at this head and therefore creating a larger volume of rock, can be seen as a good measure for enlarging the stability of the round head of a berm breakwater, using the same rock as for the trunk.

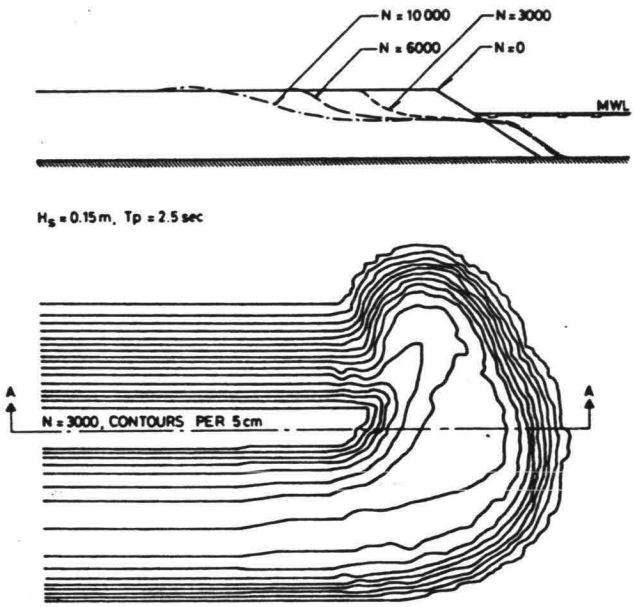


Fig. 38. Example of erosion of a berm breakwater head (taken from Burcharth and Frigaard (1987))

4.9 LONGSHORE TRANSPORT AT BERM BREAKWATERS

Statically stable structures as revetments and breakwaters are only allowed to show damage under very severe wave conditions. Even then the damage can be described by the displacement of only a number of stones from the still water level to (in most cases) a location downwards. Movement of stones in the direction of the longitudal axis is not relevant for these types of structures.

The profiles of dynamically stable structures as gravel/shingle beaches, rock beaches and sand beaches change according to the wave climate. Dynamically stable means that the net cross-shore transport is zero and the profile has reached an equilibrium profile for a certain wave condition. It is possible that during each wave material is moving up and down the slope (shingle beach).

Oblique wave attack gives wave forces parallel to the alignment of the structure. These forces can cause transport of material along the structure. This phenomenon is called longshore transport and is well known for sand beaches. Also shingle beaches change due to longshore transport, although the research on this aspect has always been limited.

Also rock beaches and berm breakwaters are or can be dynamically stable under severe wave action. This means that oblique wave attack may induce longshore transport which can also cause problems for these types of structures. Longshore transport does not occur for statically stable structures, but it will start for conditions where the diameter is small enough in comparison with the wave height. Then the conditions for start of longshore transport are important.

Start of longshore transport is most interesting for the berm breakwater where profile development under severe wave attack is allowed. The berm breakwater can roughly be described by $H_s/\Delta D_{n50} = 2.5 - 6$. Burcharth and Frigaard (1987) performed model tests to establish the incipient longshore motion for berm breakwaters. Their range of tests corresponded to $3.5 < H_s/\Delta D_{n50} < 7.1$. Longshore transport is not allowed at berm breakwaters and therefore Burcharth and Frigaard (1987) gave the following (somewhat premature) recommendations for the design of berm breakwaters, which in fact give the incipient longshore motion.

For trunks exposed to steep oblique waves	$H_s/\Delta D_{n50} < 4.5$	
For trunks exposed to long oblique waves	$H_s/\Delta D_{n50} < 3.5$	(69)
For roundheads	$H_s/\Delta D_{n50} < 3$	

Van der Meer and Veldman (1992) tested a berm breakwater under angles of wave attack of 25 and 50 degrees. Burcharth and Frigaard (1987 and 1988) tested their structure under angles of 15 and 30 degrees. Longshore transport was measured by the movement of stones from a coloured band. The transport was measured for developed profiles which means that the longshore transport during the development of the profile of the seaward slope was not taken into account. The measured longshore transport, S(x), was defined as the number of stones that was displaced per wave. Multiplication of S(x) with the storm duration (the number of waves) in practical cases would lead to a transport rate of total number of stones displaced per storm. Subsequently, the transport rate can be calculated in m^3/storm or m^3/s.

Figs. 39 and 40 give all the test results on long shore transport, both for the tests of Van der Meer and Veldman (1992) and the tests of Burcharth and Frigaard (1988). Both a higher wave height and a longer wave period result in larger transport. In Van der Meer (1988) the combined wave height-wave period parameter $H_o T_{op}$ was used for dynamically stable structures:

$$H_o T_{op} = H_s/\Delta D_{n50} * T_p\sqrt{g/D_{n50}} \qquad (70)$$

H_o is defined as the stability number $H_s/\Delta D_{n50}$ and T_{op} as the dimensionless wave period related to the nominal diameter: $T_{op} = T_p\sqrt{g/D_{n50}}$. With the parameter $H_o T_{op}$ it is assumed that wave height and wave period have the same influence on longshore transport. Figs. 39 and 40 give the longshore transport S(x) (in number of stones per wave) versus the $H_o T_{op}$. Fig. 39 gives all the data points. The maximum transport is about 3 stones/wave for $H_o T_{op}$ =

350, which is in fact a very high rate for berm breakwaters. The $H_s/\Delta D_{n50}$-value in that case was 7.1, considerable higher than the design value for berm breakwaters. Fig. 39 also shows that quite a lot of tests had a much smaller transport rate than 0.1 - 0.2 stones/wave.

Therefore Fig. 40 was drawn with a maximum transport rate of only 0.1 stones/wave. Now only 4 data points remain of Burcharth and Frigaard (1988), the others are from the tests of Van der Meer and Veldman (1992). Fig. 40 shows that the transport for large wave angles of 50 degr. is much smaller than for the other angles of 15 - 30 degrees. The two lowest points of Burcharth and Frigaard show transport for $H_o T_{op}$ = 100, where the present tests do not give longshore transport up to $H_o T_{op}$ = 117.

Vrijling et al. (1991) use a probabilistic approach to calculate the longshore transport at a berm breakwater over its total life time. In that case the start or onset of longshore transport is extremely important. They use the data of Van der Meer and Veldman (1992) and the data of Burcharth and Frigaard (1987), but not the extended series described in Burcharth and Frigaard (1988). Based on all data points (except for some missing data points this is similar to Fig. 39) they come to a formula for longshore transport:

$$S(x) = 0 \text{ for } H_o T_{op} < 100$$
$$S(x) = 0.000048\ (H_o T_{op} - 100)^2 \qquad (71)$$

Eq. 71 is shown in Figs. 39 and 40 with the dotted line. The equation fits nicely in Fig. 39, but does not fit the average trend for the low $H_o T_{op}$-region, see Fig. 40. The equation overestimates the start of longshore transport a little (except for 2 points of Burcharth and Frigaard). Therefore Eq. 71 was changed a little in order to describe the start of longshore transport better:

$$S(x) = 0 \text{ for } H_o T_{op} < 105$$
$$S(x) = 0.00005\ (H_o T_{op} - 105)^2 \qquad (72)$$

This equation yields for wave angles rougly between 15 and 35 degrees. For smaller or larger wave angles the transport will (substantially) be less. Eq. 71 is shown in Figs. 39 and 40 with the solid line and fits better in the low $H_o T_{op}$-region. The upper limit for Eq. 71 is chosen as $H_s/\Delta D_{n50} < 10$. With Eq. 71 the longshore transport for berm breakwaters has been established.

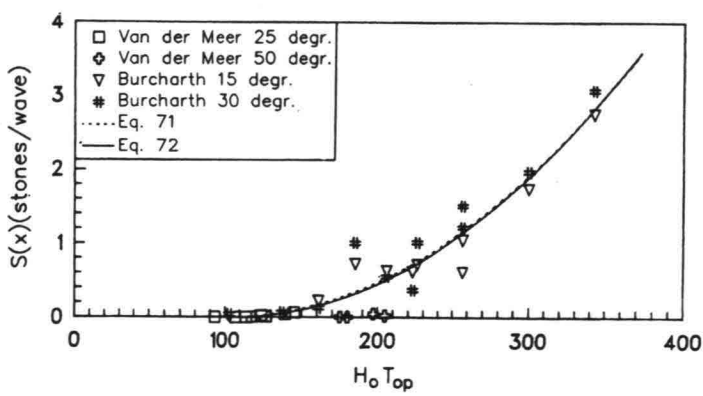


Fig. 39. Longshore transport for berm breakwaters

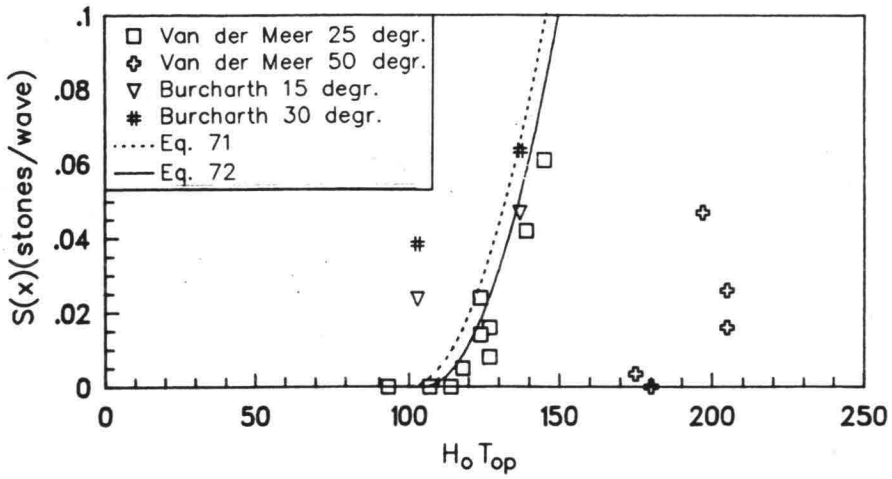


Fig. 40. Onset of longshore transport for berm breakwaters. This figure gives the exploded view of the part in Fig. 39 with S(x) < 0.1

References

Ahrens, J.P., 1981. Irregular wave run-up on smooth slopes. CERC Technical Paper No. 81-17, Fort Belvoir.

Ahrens, J.P., 1987. Characteristics of reef breakwaters. CERC, Vicksburg, Technical Report CERC-87-17.

Allsop, N.W.H. and Channell, A.R., 1989. Wave reflections in harbours: reflection performance of rock armoured slopes in random waves. Report OD 102, Hydraulics Research, Wallingford.

Allsop, N.W.H., Hawkes, P.J., Jackson, F.A. and Franco, L., 1985. Wave run-up on steep slopes - model tests under random waves. Report SR 2, Hydraulics Research, Wallingford.

Allsop, N.W.H., 1990. Rock armouring for coastal and shoreline structures: hydraulic model studies on the effects of armour grading. Hydraulics Research, Wallingford, Report EX 1989.

Aminthi, P. and Franco, L., 1988. Wave overtopping on rubble mound break waters. Proc. 21st ICCE, Malaga.

Battjes, J.A., 1974. Computation of set-up, longshore currents, run-up and overtopping due to wind-generated waves. Comm. on Hydraulics, Dept. of Civil Eng., Delft Univ. of Technology, Report 74-2.

Bradbury, A.P., Allsop, N.W.H. and Stephens, R.V., 1988. Hydraulic performance of breakwater crown wall. Report SR 146, Hydraulics Research, Wallingford.

Brebner, A. and Donnelly, P., 1962. Laboratory study of rubble foundations for vertical breakwater, Engineer Report No. 23. Queen's University Kingston, Ontario, Canada.

Burcharth, H.F. and Frigaard, P., 1987. On the stability of berm breakwater roundheads and trunk erosion in oblique waves. Seminar on Unconventional Rubble-Mound Breakwater, Ottawa.

Burcharth, H.F. and Frigaard, P., 1988. On 3-dimensional stability of reshaping berm breakwaters. ASCE, Proc. 21th ICCE, Malaga, Spain, Ch. 169.

CIAD, Project group breakwaters, 1985. Computer aided evaluation of the reliability of a breakwater design. Zoetermeer, The Netherlands.

Coastal Engineering Research Center, 1984. Shore Protection paper. U.S. Army Corps of Engineers.

Daemen, I.F.R. Wave transmission at low-crested breakwaters. Delft University of Technology, Faculty of Civil Engineering, Delft, 1991, Master Thesis.

Daemrich, K.F. and Kahle, W., 1985. Shutzwirkung von Unterwasserwellen brechern unter dem einfluss unregelmässiger Seegangswellen. Eigenverlag des Franzius-Instituts für Wasserbau und Küsteningenieurswesen, Heft 61.

Delft Hydraulics-M1983, 1987 Taluds van losgestorte materialen. Statische stabiliteit van stortsteen taluds onder golfaanval. Ontwerp formules. Verslag modelonderzoek, deel I. (Slopes of loose materials. Static stability of rubble mound slopes under wave attack. Design formula. Report on model investigation, Part I), in Dutch.

Delft Hydraulics-M1983, 1987 Taluds van losgestorte materialen. Dynamische stabiliteit van grind- en stortsteen taluds onder golfaanval. Model voor profielvorming. Verslag modelonderzoek, deel II. (Slopes of loose materials. Dynamic stability of gravel beaches and rubble mound slopes under wave attack. Model for profile formation. Report on model investigation, Part II), in Dutch.

Delft Hydraulics-M1983, 1989 Taluds van losgestorte materialen. Golfoploop of statisch stabiele stortsteen taluds onder golfaanval. Verslag modelonderzoek, deel III. (Slopes of loose materials. Wave run-up on statically stable rock slopes under wave attack. Report on model investigation, Part III), in Dutch. Author: C.J. Stam.

Führböter, A., Sparboom, U. and Witte, H.H. (1989). Großer Wellenkanal Hannover: Versuchsergebnisse über den Wellenauflauf auf glatten und rauhen Deichböschungen mit de Neigung 1:6. *Die Küßte. Archive for Research and Technology on the North Sea and Baltic Coast.*

Givler, L.D. and Sorensen, R.M., 1986. An investigation of the stability of submerged homogeneous rubble-mound structures under wave attack. Lehigh University, H.R. IMBT Hydraulics, Report #IHL-110-86.

Gravesen, H. and Sorensen, T., 1977. Stability of rubble mound breakwaters. Proc. 24th Int. Navigation Congress.

Jensen, O.J., 1984. A monograph on rubble mound breakwaters. Danish Hydraulic Institute.

Kao, J.S. and Hall, K.R., 1990. Trends in stability of dynamically stable breakwaters. ASCE, Proc. of 22th ICCE, Delft, The Netherlands, Ch. 129.

Komar, P.D., 1969. The longshore transport of sand on beaches. University of Carolina, San Diego.

Owen, M.W., 1980. Design of seawalls allowing for wave overtopping. Report No. EX 924, Hydraulics Research, Wallingford.

Postma, G.M., 1989. Wave reflection from rock slopes under random wave attack. MSc thesis, Delft University of Technology.

Powell, K.A. and Allsop, N.W.H., 1985. Low-crest breakwaters, hydraulic performance and stability. Hydraulics Research, Wallingford. Report SR 57.

Seelig, W.N., 1980. Two-dimensional tests of wave transmission and reflection characteristics of laboratory breakwaters. CERC Technical Report No. 80-1, Vicksburg.

Seelig, W.N., 1983. Wave reflection from coastal structures. Proc. Conf. Coastal Structures '83. ASCE, Arlington.

Technical Advisory Committee on Protection against Inundation (TAW), 1974. Wave run-up and overtopping. Government Publishing Office, The Hague.

Thompson, D.M. and Shuttler, R.M., 1975. Riprap design for wind wave attack. A laboratory study in random waves. HRS, Wallingford, Report EX 707.

Van der Meer, J.W. and Pilarczyk, K.W., 1987. Stability of breakwater armour layers - Deterministic and probabilistic design. Delft Hydraulics Communication No. 378.

Van der Meer, J.W., 1987. Stability of breakwater armour layers - Design formulae. Coastal Eng., 11, p 219 - 239.

Van der Meer, J.W., 1988-1. Rock slopes and gravel beaches under wave attack. Doctoral thesis. Delft University of Technology. Also: Delft Hydraulics Communication No. 396.

Van der Meer, J.W., 1988-2. Deterministic and probabilistic design of breakwater armour layers. Proc. ASCE, Journal of WPC and OE, Vol. 114, No. 1.
Van der Meer, J.W., 1988-3. Stability of Cubes, Tetrapods and Accropode. Proc. Breakwaters '88, Eastbourne. Thomas Telford.
Van der Meer, J.W. and Koster, M.J., 1988. Application of computational model on dynamic stability. Proc. Breakwaters '88, Eastbourne. Thomas Telford.
Van der Meer, J.W. and Heydra, G., 1990. Rocking armour units: number, location and impact velocity. Delft Hydraulics Publication No. 435.
Van der Meer, J.W. and Pilarczyk, K.W., 1990. Stability of low-crested and reef breakwaters. Proc. 22th ICCE, Delft.
Van der Meer, J.W., 1990-1, Low-crested and reef breakwaters, Delft Hydraulics Report H198/Q638.
Van der Meer, J.W., 1990-2. Data on wave transmission due to overtopping. Delft Hydraulics Report, H 986.
Van der Meer, J.W., 1990-3. Extreme shallow water wave conditions. Delft Hydraulics Report, H 198.
Van der Meer, J.W., 1990-4. Taluds van losgestorte materialen. Stabiliteit van lage dammen en overgangskonstrukties bij stortsteen onder golfaanval. (Slopes of loose materials. Stability of low-crested and composite structures of rock under wave attack, in Dutch). Delft Hydraulics Report M 1983 part V.
Van der Meer, J.W., 1990-5. Verification of BREAKWAT for berm breakwaters and low-crested structures. Delft Hydraulics Report, H 986.
Van der Meer, J.W., 1990-6. Static and dynamic stability of loose materials. In Coastal Protection. K.W. Pilarczyk (editor). Balkema, Rotterdam.
Van der Meer, J.W. and d'Angremond, K., 1991. Wave transmission at low-crested structures. In *Coastal structures and breakwaters*. Proc. ICE, London.
Van der Meer, J.W., 1992. Stability of the seaward slope of berm breakwaters. Accepted for publication to the Journal of Coastal Engineering, Elsevier, Amsterdam. September issue.
Van der Meer, J.W. and Stam, C.J.M. (1992). Wave runup on smooth and rock slopes of coastal structures. Paper to be published in ASCE, *Journal of WPC & OE. September issue.*
Van Hijum, E. and Pilarczyk, K.W., 1982. Equilibrium profile and longshore transport of coarse material under regular and irregular wave attack. Delft Hydraulics, Publication No. 274.
Van Oorschot, J.H. and d'Angremond, K., 1968. The effect of wave energy spectra on wave run-up. Proc. 11th ICCE, London, Chapter 68.
Vellinga, P., 1986. Beach and dune erosion during storm surges. Delft University of Technology. Doctoral thesis.
Vrijling, J.K., Smit, E.S.P. and De Swart, P.F., 1991. Berm breakwater design; the longshore transport case: a probabilistic approach. ICE, Proc. Coastal Structures and Breakwaters, London.

Symbols

A_c	Armour crest freeboard, relative to still-water level
A^e	Erosion area on profile
B	Structure width, in horizontal direction normal to face
C_r	Coefficient of wave reflection
D	Particle size, or typical dimension
D_n	Nominal block diameter = $(M/\rho_a)^{1/3}$
D_{n50}	Nominal diameter $(M_{50}/\rho_a)^{1/3}$
D^s	Size of the equivalent volume sphere
D^z	Sieve diameter
D_{50}	Sieve diameter, diameter of stone which exceeds the 50% value of sieve curve
D_{85}	85% value of sieve curve

D_{15}	15% value of sieve curve
D_{85}/D_{15}	Armour grading parameter
d	Thickness or minimum axial breadth
E_i	Incident wave energy
E_r	Reflected wave energy
E_t	Transmitted wave energy
E_d	Energy absorbed or dissipated
F_c	Difference of level between crown wall and armour crest = $R_c - A_c$
f	Frequency of waves = $1/T$
f_p	Peak frequency of waves at which maximum wave energy occurs
G_c	Width of armour berm at crest
g	Gravitational acceleration
H	Wave height, from trough to crest
H_{max}	Maximum wave height in a record
H_{m0}	Significant wave height calculated from the spectrum = $4m_0^{1/2}$
H_o	Offshore wave height, unaffected by shallow-water processes
H_{os}	Offshore significant wave height, unaffected by shallow-water processes
H_s	Significant wave height, average of highest one-third of wave height
$H_{2\%}$	Wave height exceeded by 2% of waves
$H_{1/10}$	Mean height of highest one-tenth of waves
h	Water depth
h_c, h_c'	Armour crest level relative to seabed, after and before exposure to waves
k_t	Layer thickness coefficient
K_t	Coefficient of total transmission, by overtopping or transmission through
L	Wave length, in the direction of propagation
L_o	Deep water or offshore wave length, $gT^2/2\pi$
L_{om}, L_{op}	Offshore wave length of mean, T_m, peak, T_p and periods, respectively
L_s	Wave length in (shallow) water at structure toe
L_{ms}, L_{ps}	Wave length of mean or peak period at structure toe
l	Maximum axial length
M	Mass of an armour unit
M_{50}, M_i	Mass of unit given by 5%, i%, on mass distribution curve
m	Seabed slope
m_0	Zeroth moment of wave spectrum
m_n	nth moment of spectrum
N_n	Number of waves in a storm, record or test, = T_t/T_m
N_a	Total number of armour units in area considered
N_d	Number of armour units displaced in area considered
N_{od}, N_{or}	Number of displaced, or rocking, units per width D_n across armour face
N_r	Number of armour units rocking in area considered
N_s	Stability number = $H_s/(\Delta D_{n50})$
N_s^*	Spectral stability number = $(H_{m0}^2 L_{ps})^{1/3}/(\Delta D_{n50})$
P	Notional permeability factor, defined by van der Meer
P_x	Probability that x will not exceed a certain value; often known as cumulative probability density of x
p_x	Probability density of x
Q	Overtopping discharge, per unit length of seawall
Q^*	Dimensionless overtopping discharge = $Q/(T_m g H_s)$
R	Strength descriptor in probabilistic calculations
R^*	Dimensionless freeboard = $R_c/T_m(gH_s)^{1/2}$
R_c	Crest freeboard, level of crest relative to still-water level
$R_{d2\%}$	Run-down level, below which only 2% pass
R_u	Run-up level, relative to still-water level

R_{us}	Run-up level of significant wave
$R_{u2\%}$	Run-level exceeded by only 2% of run-up crests
S	Loading descriptor in probabilistic design
S	Dimensionless damage, A_e/D^2_{n50}; may be calculated from mean profiles or separately for each profile line, then averaged
s	Wave steepness, H/L_o
s_m	Wave steepness for mean period, $2\pi H_s/gT^2_m$
s_{op}	Offshore wave steepness for peak period, $H_{os}/L_{op} = 2\pi H_{os}/gT^2_p$
T^p	Wave steepness for peak periods $2\pi H_s/gT^2_p$
T_m	Wave period
T^m	Mean wave period
T^p	Spectral peak period, inverse of peak frequency
T^R	Duration of wave record, test or sea state
t	Time, variable
t_a, t_f, t_x	Thickness of armour, underlayer or other layer in direction
W^a	Armour unit weight, = Mg
W_0, W_{15}, W_{50}, W_y	Weight for which a fraction or percentage y is lighter on the cumulative weight distribution curve
Z	Reliability function in probabilistic design; Z = R-S
α	Structure front face angle
β	Angle of wave attack with respect to the structure
γ_f	Roughness value, usually relative to smooth slopes
Δ^f	Relative buoyant density of material considered, e.g. for rock = $(\rho_t/\rho_w)-1$
$\mu(x)$	Mean of x^t
ξ	Surf similarity parameter, or Iribarren number, = $\tan\alpha/s_m^{1/2}$
ξ_p	Modified surf parameter = $\tan\alpha/s^{1/2}_p$
ρ^p	Mass density, usually of fresh water
ρ_t	Mass density, oven-dried density
ρ_{ssd}	Saturated surface dry density
ρ_w	Mass density of sea water
ρ_t, ρ_c, ρ_a	Mass density of rock, concrete, armour
ρ_b	Bulk density of material as laid