CHAPTER 103

STABILITY OF LOW-CRESTED AND REEF BREAKWATERS

Jentsje W. van der Meer[1] and Krystian W. Pilarczyk[2]

Abstract

Low-crested structures are designed for some or even severe overtopping. The stability of these structures is sometimes higher than the non-overtopped structures, due to the fact that wave energy can pass over the crest, giving lower wave forces on the armour layer of the seaward slope.

Low-crested structures can be classified into three categories: dynamically stable reef breakwaters, statically stable low-crested structures with the crest above swl and statically stable submerged structures. Well described investigations at various institutes (in total about 275 tests) were re-analysed and this has led to practical design formulas and graphs for each of the three classes mentioned above.

Introduction

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 it. 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

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1) Delft Hydraulics, PO Box 152, 8300 AD Emmeloord, The Netherlands
2) Rijkswaterstaat, PO box 5044, 2600 GA Delft, The Netherlands

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.

Classification of low-crested structures

Low-crested rock structures can be divided into three categories, see also Figs. 1-3.

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. 1). The equilibrium crest height, with corresponding transmission, are the main design parameters. The transmission will not be treated in this paper (one is referred to Van der Meer (1990b)).

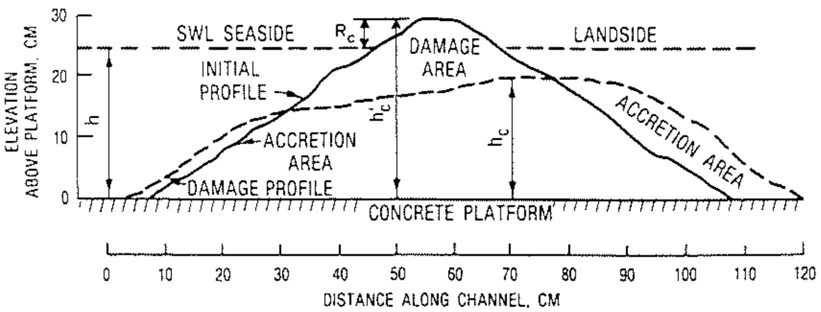

Figure 1 Example of reef type breakwater (Ahrens (1987))

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. 2).

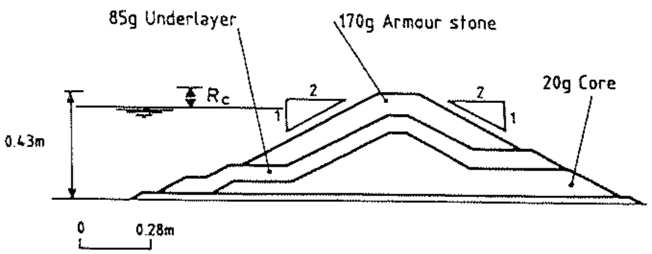

Figure 2 Example of low-crested breakwater (Powell and Allsop (1985))

Statically stable submerged breakwaters ($R_c < 0$)

All waves overtop these structures and the stability increases remarkably if the crest height decreases (Fig. 3).

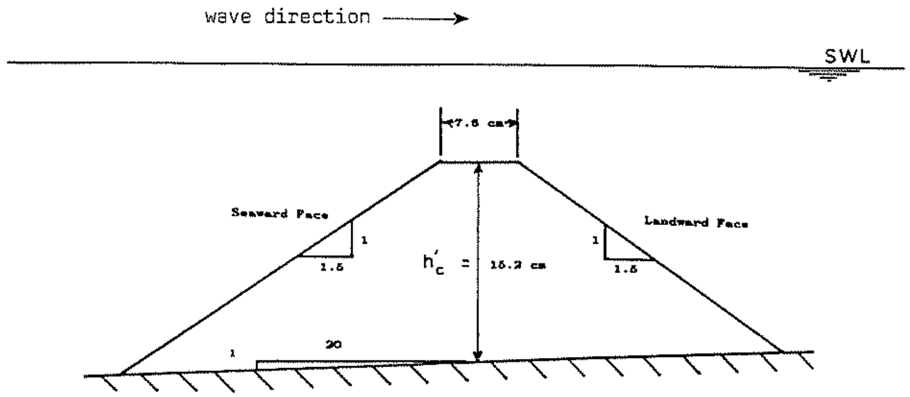

Figure 3 Example of submerged breakwater Givler and Sørensen (1986))

Description of data sets

Ahrens (1987) described the stability of reef type breakwaters. This type of breakwater is little more than a homogeneous pile of stones with individual stone weights similar to those ordinarily used in the armour and first underlayer of conventional breakwaters. The initial crest height is just above the water level. Under severe wave conditions it is allowed that the crest height decreases to a certain equilibrium crest height. Ahrens performed about 15 tests on stability of these structures and gave a formula for the equilibrium crest height.

Allsop (1983) and Powell and Allsop (1985) described about 45 tests on the stability of breakwaters with the crest above swl and which were conventional breakwaters. Only small damage (displacement of stones) was allowed during design conditions.

Givler and Sørensen (1986) described about 45 tests on the stability of submerged breakwaters. The tests were performed with regular waves and included a large range of wave heights, but also wave periods. The damage at the crest was measured and the design criteria are similar to the conventional breakwaters (no or only small damage allowed).

Finally Van der Meer (1988) performed about 45 tests during his very extensive model investigation (in total about 500 tests) on structures with a low crest. These tests cover all three structure types described above (reef type, low-crested above swl and submerged) and were used as a connection between the various structures and data sets.

A more extensive description of the above described data sets together with the complete re-analysis of the data can be found in Van der Meer (1990a).

Reef breakwaters

The analyses of stability by Ahrens (1987, 1989) and Van der Meer (1990a) was concentrated on the change in crest height due to wave attack, see Fig. 1. 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, h_c, to the height at the beginning of the test, h_c'. The natural limiting values of h_c/h_c' are 1.0 and 0.0 respectively.

The wave height can be characterised by $H_s/\Delta D_{n50}$ (Van der Meer (1988)) or N_s (stability number: Ahrens (1987, 1989)).

$$H_s/\Delta D_{n50} = N_s \tag{1}$$

where:
H_s = significant wave height, H_s or H_{m0} (H_{m0} was used in this study)
Δ = relative mass density; $\Delta = \rho_a/\rho_w - 1$
ρ_a = mass density of armour rock
ρ_w = mass density of water
D_{n50} = nominal diameter of rock; $D_{n50} = (M_{50}/\rho_a)^{1/3}$
M_{50} = average mass (50% value on mass distribution curve)

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:

$$N_s^* = H_s^{2/3} L_p^{1/3}/\Delta D_{n50} \tag{2}$$

where: L_p = the Airy wave length calculated using T_p and the water depth at the toe of the structure h. In fact a local wave steepness is introduced in Eq. 2 and the relationship between the stability number N_s and the spectral stability number N_s^* can simply be given by:

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

where: s_p = the local wave steepness; $s_p = H_s/L_p$

That a longer wave period gives more damage than a shorter period is not always true. Ahrens concluded that it was true for reef breakwaters where the crest height lowered substantially. It is however not true for non-overtopped breakwaters (Van der Meer (1987 or 1988)). The influence of the wave period in that case is much more complex than suggested by Eq. 3.

The crest height (reduction) of a reef type breakwater can be described by:

$$h_c = \sqrt{A_t/\exp(aN_s^*)} \tag{4}$$

where "a" = a coefficient and A_t = area of structure cross-section. Ahrens gave various equations for the coefficient a. The most recent and refined one is given by Ahrens (1989):

$$\text{"a"} = 0.046(h_c'-h_c)/h + 0.2083(h_c/h)^{1.5} - 0.144(h_c/h)^2 + 0.4317/\sqrt{B_n} \tag{5}$$

where:
h = water depth at structure toe and
$B_n = A_t/D^2_{n50}$ (bulk number)

The structures of Van der Meer (1988) had other crest heights, water depths, bulk numbers and slope angles than Ahrens' structures. A first fit of Eqs. 4 and 5 with these data is shown in Fig. 4. The breakwater response slope is shown on the vertical axis and is defined by:

$$C = A_t/h_c^2 \tag{6}$$

The graph shows the data sets with different bulk numbers B_n and response slopes C. Where Ahrens' data were nicely located around the curve (Eqs. 4 and 5), the data of Van der Meer did not. It is clear from Fig. 4 that the average slope "as built", C', has also influence on the crest height, besides the parameters h_c'/h and B_n.

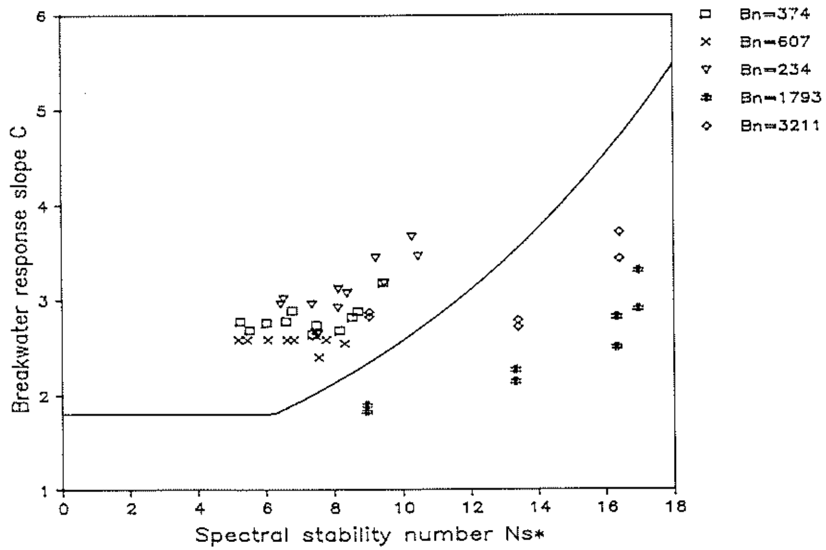

Figure 4 Data of Van der Meer (1988) with Eqs. 4 and 5

Therefore all the data of Ahrens (1987) were re-analysed together with the data of Van der Meer (1988). The complete analysis is given by Van der Meer (1990a), the basis of this paper. It will not be given here.

The final equation that was derived from the analysis is given by:

$$h_c = \sqrt{A_t/\exp(aN_s^*)} \tag{4}$$

$$\text{with "a"} = -0.028 + 0.045C' + 0.034h_c'/h - 6.10^{-9}\ B_n^{\ 2} \tag{7}$$

and $h_c = h_c'$ if h_c in Eq. 4 > h_c'.

The same data as shown in Fig. 4 are given in Fig. 5, but now with the new equations 4 and 7, and with on the vertical axis the relative crest height h_c'/h in stead of the response slope C. The agreement is good. Eq. 7 gives almost the same results for Ahrens´ test range as Eq. 5. Eq. 7 is, therefore, valid for a wider range of conditions.

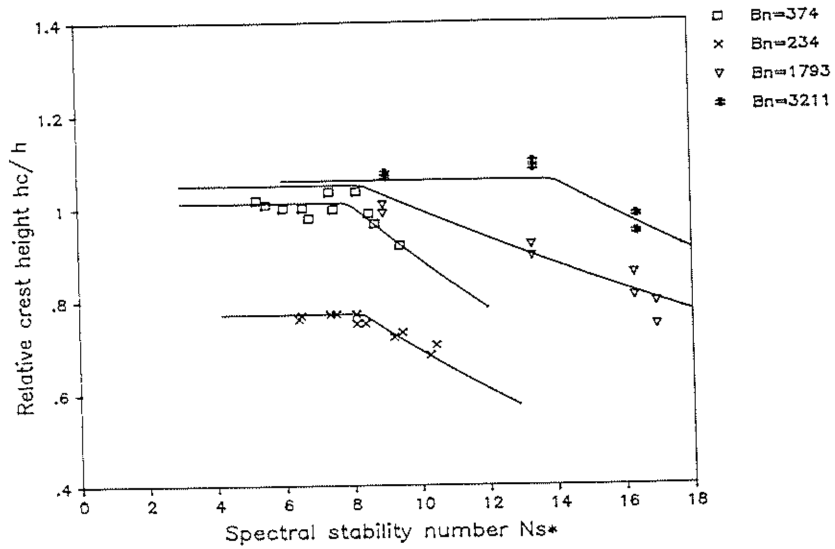

Figure 5 Data of Van der Meer (1988) with Eqs. 4 and 7

The lowering of the crest height of reef type structures as shown in Fig. 1, can be calculated with Eqs. 4 and 7. 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 H_s (produced by Delft Hydraulics´ program BREAKWAT) is shown in Fig. 6. The reliability of Eq. 4 can be described by giving 90% confidence bands. The 90% confidence bands are given by h_c ± 10%.

Statically stable low-crested breakwaters above swl

The stability of a low-crested conventional breakwater can be related to the stability of a non-overtopped structure. Stability formulas as the Hudson formula or more advanced formulas (Van der Meer (1987, 1988) can be used for example. The required stone diameter for an overtopped breakwater can then be determined by a reduction factor for the mass of the armour, compared to the mass for a non-overtopped structure.

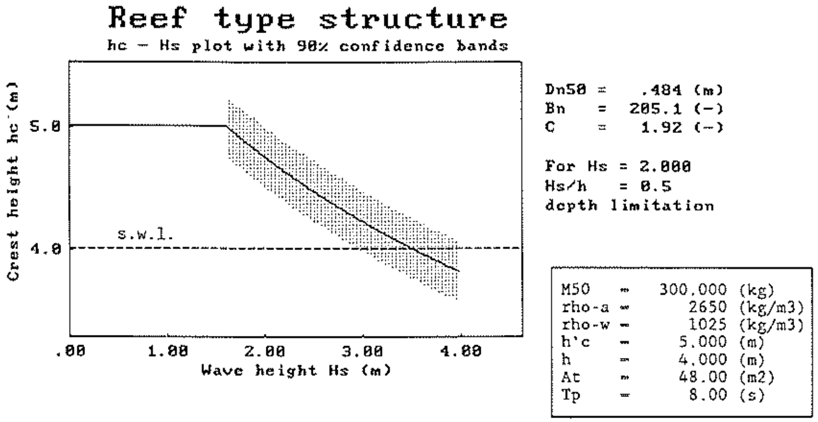

Figure 6 Design graph of reef type breakwater

Data sets that could be used for analysis were a part of Ahrens data (with small damage to the crest), Powell and Allsop (1985) and Van der Meer (1988). Fig. 7 gives the damage curves of a part of Van der Meer´s tests with four crest heights, R_c, and for a constant wave period of 1.7 s. From this figure it is obvious that a decrease in structure crest height results in an increase in stability.

Furthermore, from the tests it could be concluded that a longer period of 2.2 s gave an increase in stability for $R_c/H_s \lesssim 1.3$ and the shorter period of 1.7 s for a lower value of $R_c/H_s \lesssim 0.8$. This can also be explained in a physical way. A long period gives higher run-up on a slope than a short period. Therefore more energy is lost by overtopping for a long period at the same crest level as for a short period.

The transition height where the increase in stability starts (given as a R_c/H_s value) should in fact be a function of the wave period (or wave steepness) too. From the mentioned data sets the following transition heights R_c/H_s were derived and the corresponding (average) wave steepness $s_{op} = 2\pi H_s/gT_p^2$ was taken from the original data.

Author	transition	
	R_c/H_s	s_{op}
Van der Meer (T_p = 2.0 s)	0.8	0.025
Van der Meer (T_p = 2.6 s)	1.3	0.015
Ahrens	1.3	0.010
Allsop (long wave)	2.0	0.006
Allsop (short wave)	0.7	0.027

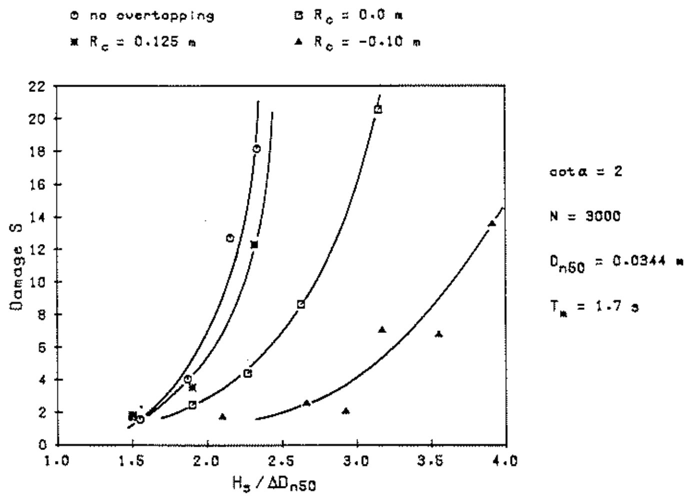

Figure 7 Influence of crest height on damage curves (from Van der Meer (1988))

These data points are shown in Figure 8. Although the points are only a rough estimate, they show a consistent trend: a decreasing transition crest height with increasing wave steepness. The possible maximum steepness in nature will be in the order of s_{op} = 0.04 - 0.045, which in fact was not tested by one of the authors. Although not the best fit, the following equation (also shown in Figure 8) gives a good fit with the data:

$$\text{transition crest height: } R_c/H_s = 0.13\ s_{op}^{-0.5} \qquad (8)$$

In Powell and Allsop (1985) a dimensionless crest height R_p^* was introduced which was used to describe overtopping and which included the wave steepness. The definition is given by:

$$R_p^* = R_c/H_s \sqrt{s_{op}/2\pi} \qquad (9)$$

Comparison and rewriting of Eqs. 8 and 9 shows that the transition crest height can simply be described by:

$$R_p^* = 0.052 \qquad (10)$$

The average increase in stability ($H_s/\Delta D_{n50}$ or N_s^*) for a structure with the crest at the water level, in comparison with a non-overtopped structure, is in the order of 20-30%. If the increase in stability is set at 25%, independent of wave steepness, and if a linear increase in stability is assumed between R_p^* = 0.052 and R_p^* = 0, the increase in stability can be described as a function of R_p^* only. Furthermore, if not the increase in stability is taken as a measure, but the reduction in required nominal diameter D_{n50}, the final equation becomes:

$$\text{Reduction factor for } D_{n50} = 1/(1.25 - 4.8\ R_p^*) \qquad (11)$$

for $0 < R_p^* < 0.052$

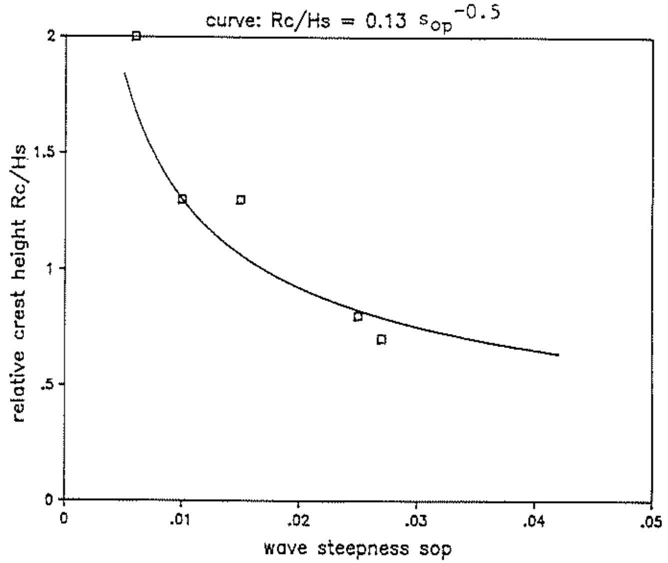

Figure 8 Transition crest height where the influence of a low crest starts, as a function of wave steepness

This final equation 11 describes the stability of a statically stable low-crested breakwater with the crest above swl in comparison with a non-overtopped structure. Eq. 11 is shown in Fig. 9, for various wave steepnesses, and can be used as a design graph. The reduction factor for the required nominal diameter can be read from this graph (or calculated by Eq. 11) in comparison with a non-overtopped structure.

An average reduction of 0.8 in diameter is obtained for a structure with the crest height 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.

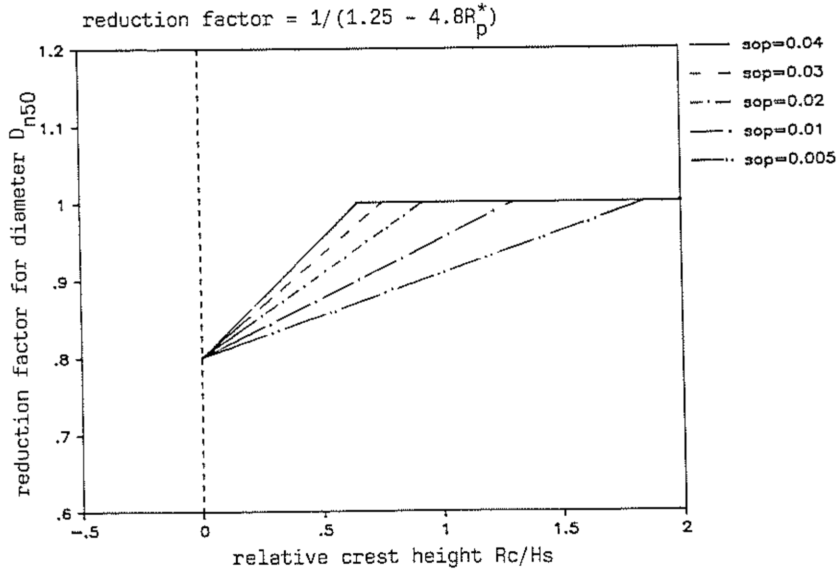

Figure 9 Design graph with the reduction factor for the stone diameter of a low-crested structure ($R_c < 0$) as a function of relative crest height and wave steepness

<u>Submerged breakwaters</u>

The tests of Ahrens (1987, 1989) and Allsop (1983) and Powell and Allsop (1985) had always an initial crest level at or above the water level. Only Van der Meer (1988) and Givler and Sørensen (1986) had initial crest heights below the water level. The total amount of data is limited, however. Van der Meer (1988) tested only a slope angle of 1:2 and Givler and Sørensen (1986) tested only a slope of 1:1.5. The seaward slope angle might have some influence on the stability of the submerged structure. Therefore the analysis of submerged structures here will be only valid for rather steep slopes, say about 1:1.5 to 1:2.5.

The slope angle has large influence on non-overtopped structures. In the case of submerged structures the wave attack is concentrated on the crest and less on the seaward slope. Therefore it might be allowed to exclude the slope angle of submerged structures as being a governing parameter for stability.

The relationship between relative crest height h_c'/h and spectral or modified stability number N_s^* (Eq. 2) for a fixed damage level of S = 5, where S is defined by Van der Meer (1988), is shown in Fig. 10. It is noted again that the tests of Givler and Sørensen were

performed with monochromatic waves and that therefore, differences in results between Van der Meer´s tests and Givler and Sørensen´s tests might be due to this effect.

The N_s^* values of Givler and Sørensen for $h_c'/h = 0$ are a little smaller than those of Van der Meer. This might be caused by the above described difference in wave testing, but also by a less accurate measuring technique for the damage.

The N_s^* values for $h_c'/h = 0.75$, however, are lower in Van der Meer´s tests, see Figure 10. The data points of Van der Meer for this crest height are very close which means that for statically stable submerged structures stability might better be described by N_s^* in stead of $H_s/\Delta D_{n50}$. That this is not the case for low-crested structures with the crest above swl, can also be concluded from Figure 10, where for $h_c'/h > 0$ substantial difference is found for the two wave periods tested by Van der Meer.

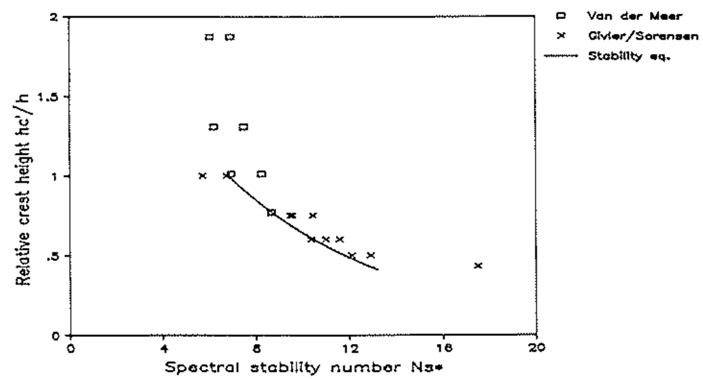

Figure 10 Spectral stability number as a function of relative crest height for submerged structures and for a fixed damage level of S = 5

For structures with the crest above swl it was concluded that a reduction of 0.8 in nominal diameter (or an increase of 25% in N_s^*) value can be expected for breakwaters with the crest at the water level, in comparison with non-overtopped structures. This factor is also present in Figure 10. For submerged structures, however, the increase in stability is much larger. The difference in N_s^* between $h_c'/h = 1.0$ and 0.5 is about a factor 2.

Moreover, if $h_c'/h < 0.45$ a remarkable increase in stability is present (the right points in Figure 10). This might be due to a change in phenomenon considered. If the structure becomes too low (and too small) the wave does not "feel" the structure anymore.

Figure 10 can be used to develop a design formula for submerged structures. As the difference between testing with monochromatic or random waves is not known for this type of structure, a design curve should be at the safe side of the data of Givler and Sørensen. According to the description of the crest height for a reef breakwater (see Eq. 4) the following equation can be fitted to the data:

$$h_c'/h = a\ \exp(bN_s^*) \tag{12}$$

where "a" and "b" are coefficients. The coefficient "b" was found to be the same for all three damage levels of S = 2, 5, and 12 which were considered, and amounted to b = -0.14. The coefficients "a" were respectively 2.33, 2.68 and 3.11 for S = 2, 5 and 12. A linear relationship between "a" and S gives the following equation:

$$a = 2.1 + 0.1\ S \tag{13}$$

Eqs. 12 and 13 together give the final stability formula:

$$h_c'/h = (2.1 + 0.1\ S)\ \exp(-0.14\ N_s^*) \tag{14}$$

The stability of submerged breakwaters is only a function of the relative crest height, the damage level and the spectral stability number. Eq. 14 is shown in Figure 10 and gives good agreement with the test results.

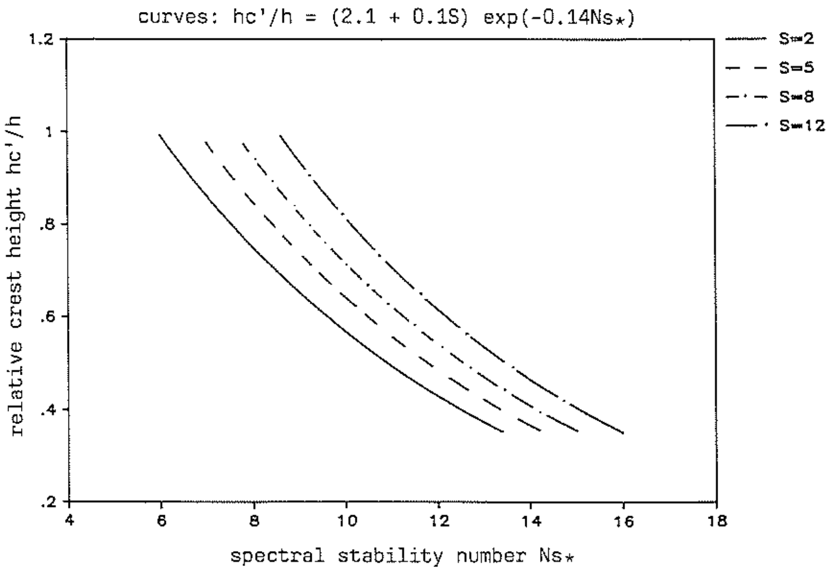

Figure 11 Design curves for a submerged structure

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. 14.

Eq. 14 is shown as a design graph in Fig. 11 for four damage levels. Here S = 2 is start of damage, S = 5-8 is moderate damage and S = 12 severe damage (more than one layer removed from the crest).

Conclusions

Low-crested rubble mound structures can be divided into three categories: dynamically stable reef breakwaters; statically stable low-crested breakwaters ($R_c/H_s > 0$) and statically stable submerged breakwaters. All waves overtop these structures and the stability increases remarkably if the crest height decreases.

The stability of reef breakwaters is described by Eqs. 4 and 7. Design curves can be drawn with the aid of these equations and an example is given in Fig. 6.

The stability of a low-crested breakwater with the crest above swl is first established as being a non-overtopped structure. Stability formulas derived by Van der Meer (1987, 1988) can be used. The required stone diameter for an overtopped breakwater can then be determined by multiplying the derived stone diameter for a non-overtopped structure with a reduction factor, given by Eq. 11. Design curves are shown in Fig. 9.

The stability of submerged breakwaters depends on the relative crest height, the damage level and the spectral stability number. The stability is described by Eq. 14 and a design graph is given in Fig. 11.

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