TOE STRUCTURE STABILITY OF RUBBLE MOUND BREAKWATERS

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SYNOPSIS
Besides armour sizes on sea side, crest and inner side of a rubble mound breakwater, the toe structure has to be designed too. This paper focusses entirely on the stability of this toe structure, as good design rules are lacking, and is based on small scale model tests. The influence of the governing parameters on stability has been described. It appeared that the wave steepness and toe width had no or only minor influence on this stability. A new design formula has beed suggested, with the range of application, and fills in the gap in existing knowledge.

INTRODUCTION
In the classical design of a rubble mound breakwater, both, functional and structural considerations play a role. Functional considerations are mainly wave overtopping, reflection and wave transmission by the breakwater. Important structural elements can be distinguished along the perimeter of the structure:

* armour layer,
* crest and inner slope,
* toe.

Data about the stability of the armour layer are widely available (Van der Meer, 1993), and data about wave overtopping and wave transmission, and the stability of crest and inner slope have recently been presented by the authors on several occasions (Van der Meer and Pilarczyk, 1990, Van der Meer and d'Angremond, 1991, Van der Meer and Daemen, 1994). The function of the toe is mainly to support the armour layer, and to provide a transition to low(er) weight units in the base of the structure (Fig. 1). Data about the stability of the toe are less in number. That is the reason to attach special attention to this part of the structure in this paper.

EXISTING INFORMATION

Influence of toe depth
The stability of the toe was traditionally related to the stability of the armour layer. For values of $h_t/H_s = 1.5$, a minimum weight of the units in the toe was given as W/2. For a clarification of the parameters used, one is referred to Fig. 1 and the table of notations.

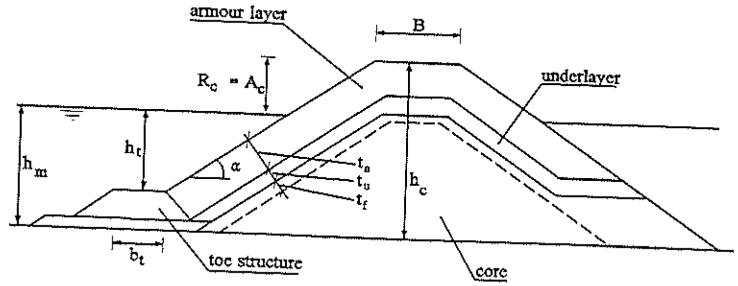

Fig. 1 Cross-section of rubble mound breakwater

For a greater submergence, with values of $h_t/H_s > 2$, the toe unit weight could be reduced to W/10 to W/15 (Shore Protection Manual, 1984). These design rules were attractive because of their simplicity; they were, however, not based on extensive research or comprehensive theoretical considerations.
Though not really intended for use in the toe of a rubble mound breakwater, the Shore Protection Manual refers to experimental work by Brebner and Donnelly (1962). Their research (using regular waves) was aimed at defining the required unit weight for foundations of vertical breakwaters. They indicate a relation between the relative depth of the toe h_t/h_m and the stability number $H/\Delta D_{n50}$, which shows systematically larger values of $H/\Delta D_{n50}$ for larger values of h_t/h_m. This tendency was not confirmed by work done in the context of the CIAD-report (1985). It was attempted in vain to find a relation between $H_s/\Delta D_{n50}$ and h_t/H_s, probably because of the presence of H_s in both dimensionless parameters. Eventually, a stability number is recommended, but the standard deviation proved to be considerable.

Influence of wave period
Gravesen and Sørensen (1977) indicated that waves with a high steepness (short wave period) are causing more damage than waves with a low steepness. This assumption was based on a very small number of tests, and it was not confirmed by the CIAD report.

Re-analysis of existing data
As one of the authors of the CUR/CIRIA Manual, Van der Meer re-analyzed existing test series of Delft Hydraulics and the Danish Hydraulic Institute to establish a more acceptable relationship between wave action, structural design and damage of the toe. He defined damage levels and found 3-10% acceptable, as only flattening out of the toe occurred and the support to the armour layer was maintained.The tests that were available for re-analysis were mainly done in depth limited conditions, with a ratio H_s/h_m close to 0.5.

It appears that a similar tendency is found as in the Shore Protection Manual and in the original work of Brebner and Donnelly: a high location of the toe leads to a considerable reduction of stability. When the actual results of Brebner and Donnelly were compared with the analysis of CUR/CIRIA, they appeared to be too low as long as H was taken equal to H_s. Introduction of a value $H = H_{1/10}$, as recommended in the Shore Protection Manual for the Hudson formula, brought the tests with regular waves close to those with random waves. The design curve recommended by CUR/CIRIA (Fig. 2) is not completely satisfactory as the evaluation is based on experiments under depth limited conditions only and the possible influence of wave period or wave steepness has not been investigated. The curve can be expressed by the following equation:

$$\frac{H_s}{\Delta D_{n50}} = 8.7 \left(\frac{h_t}{h_m}\right)^{1.4} \tag{1}$$

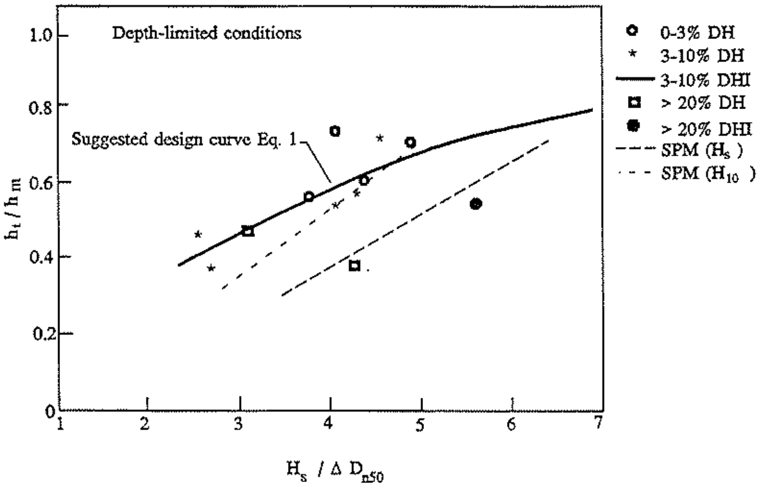

Fig. 2 Survey test results on toe stability; CUR/CIRIA (1991)

PRESENT INVESTIGATIONS

Because of the unsatisfactory situation from a designers point of view, it was decided by Delft University and Delft Hydraulics to carry out additional investigations. The investigations were carried out by Gerding as fulfilment of his masters thesis (Gerding, 1993).

Test Conditions

Model tests were carried out in the "Scheldt" flume of Delft Hydraulics. The length of this flume is 50 m, the width 1.0 m, and the depth 1.2 m. A short and relatively steep foreshore was present with a slope of 1:20, the structure was placed 18 m from the wave generator. (Fig. 3).

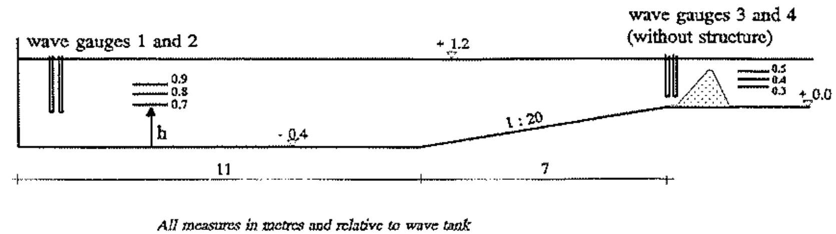

Fig. 3 Longitudinal section of the "Scheldt" flume

Before running the actual tests, the wave characteristics were determined in the absence of the breakwater. In this way, the relation between wave characteristics at the location of the breakwater could be compared with the "deep water" wave characteristics. Tests were carried out for "deep" water depths of 0.7, 0.8 and 0.9 m. It is evident that these wave characteristics changed due to shoaling and breaking on the foreshore. The test programme comprised of a variation of wave height (0.15, 0.20 and 0.25 m) and wave steepness (0.02 and 0.04), all values given for the deep section of the flume, close to the wave generator.

The stone diameter D_{n50} used for the toe in the model was 0.017 m, 0.025 m, 0.035m, and 0.040 m, with a ratio D_{85}/D_{15} = 1.15 to 1.30. Three stone sizes were tested at the same time by sub-dividing the width of the model in three sections. A range of toe heights and toe widths was investigated for each combination of wave steepness and wave height (Fig. 4). A few tests were carried out with an intermediate wave steepness of 0.03. The waves were generated according to a preset Jonswap spectrum. During the tests, H_s, H_{m0}, $H_{2\%}$ were measured, as well as the average period and the peak period of the spectrum.
For the larger water depths and lower wave heights no breaking occurred, which means that at the location of the structure the ratio between H_s and $H_{2\%}$ was about 1.4. For the smaller water depths and larger wave heights a clear shallow water situation developed, in which the wave heights are not any longer Rayleigh-distributed.

The duration of each test was such that it contained about 1000 waves. The damage was determined after each test by counting and weighing the total number of stones removed from the toe structure. The damage was expressed in the damage number N_{od}, obtained by dividing the number of stones removed from the section by the number of D_{n50} wide strips in that section. Actually, N_{od} is the actual number of stones removed, related to a width (along the longitudinal axis of the structure) of one nominal diameter D_{n50}.

Test Results

All data for a particular configuration (wave steepness s_{op}, deep water depth h, toe height h_m - h_t and toe width b_t) were plotted in graphs of N_{od} versus H_s, and distinguishing between the various stone sizes. The values of H_s used in these damage curves are the values measured at the location of the structure. Examples of such damage curves are given in Fig. 5.

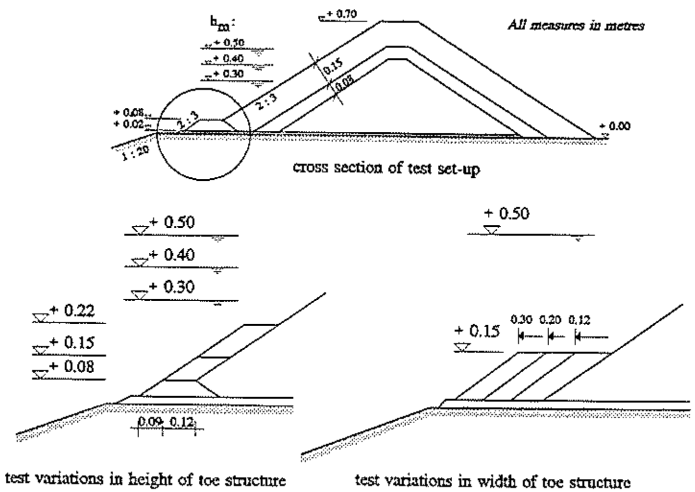

Fig. 4 Breakwater cross-section of test set-up

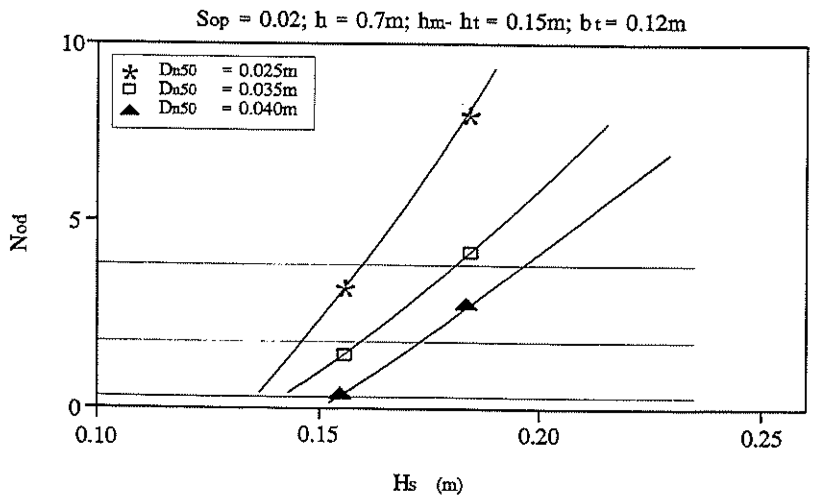

Fig. 5 Damage level N_{od} as a function of significant wave height H_s for various stone diameters D_{n50}

A relation was established between the damage number N_{od} as defined above and the earlier work by Van der Meer in the CUR/CIRIA Manual (1991). Van der Meer distinguished three damage levels:

0-3%	no (or only few) movement of stones in the toe;
3-10%	toe flattened out but the supporting function with respect to the armour layer is still present;
>20-30%	failure; the toe has lost its function.

These damage levels were represented by N_{od} values of <0.5, 0.5-2.0, and >4.0 respectively. Eventually, the damage curves were used to determine the wave heights causing the three defined damage levels N_{od} = 0.5, 2.0 and 4.0. In this way a data base was created that contained the values of all relevant parameters.

INFLUENCE OF PARAMETERS

The database thus developed was used to analyze the influence of each parameter. This influence will be briefly discussed for each parameter, and where appropriate, an example is given in the form of a graph.

Influence of wave steepness s_{op}

The influence of the wave steepness on the stability was checked by construing a graph of H_s versus s_{op} for each configuration and indicating the damage level. It appears that the damage level is a clear function of the wave height, but there is no significant trend indicating an influence of the steepness (Fig. 6).

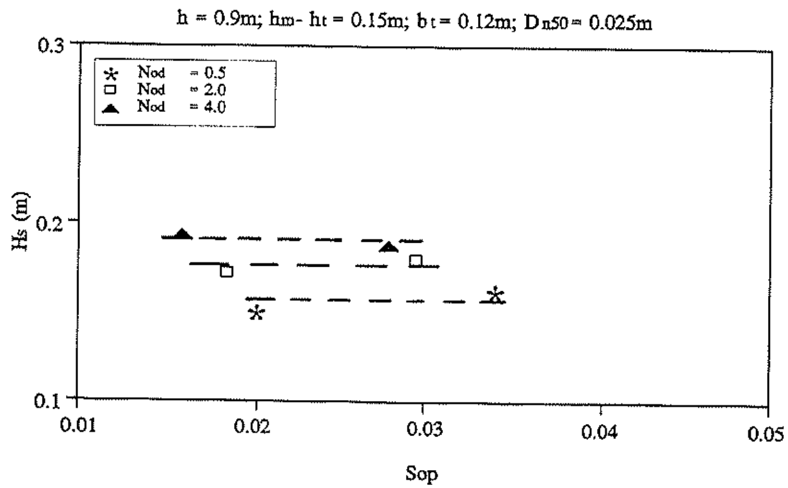

Fig. 6 Influence of wave steepness s_{op} on required wave height H_s to cause fixed damage levels N_{od}

Following this conclusion, the database could be condensed, ignoring the value of the wave steepness. Since this conclusion is based on a limited number of observations, it was also attempted to find a confirmation through the additional tests with wave steepness of 0.03. Figure 7 shows the damage curves for a configuration with a (deep) water depth of 0.7 m, a toe height $h_m' - h_t = 0.15$ m, a toe width of b_t of 0.12 m, and a stone diameter D_{n50} of 0.04 m. It is also clear from this figure that there is just one damage curve, which is not affected by a different wave steepness.

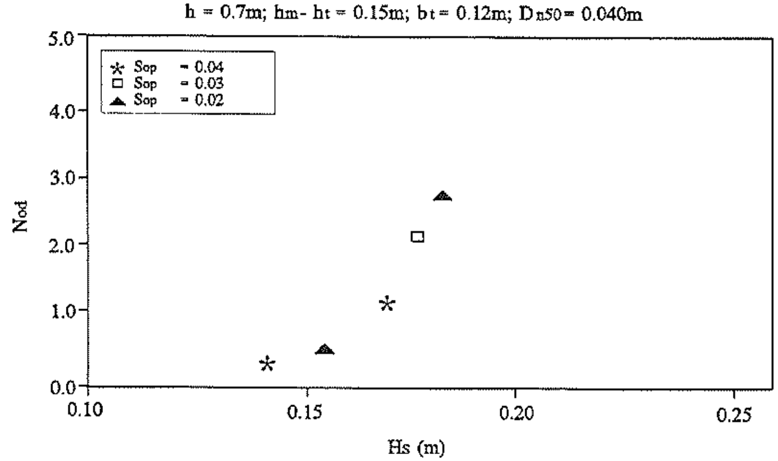

Fig. 7 Influence of wave steepness s_{op} on damage curve

Influence of toe width b_t

To assess the influence of the toe width, all data sets with identical water depth, toe height and stone diameter are combined. Plotting the toe width versus the wave height, again the damage appears to be a function of the wave height only (Fig. 8). This permits us to ignore the width of the toe as a parameter in the remainder of the analysis. For design purposes, one should realize, however, that a larger toe permits acceptance of more damage.

Influence of local water depth h_m

For given dimensions of the toe (note that this refers now to the height of the toe only), the influence of the water depth directly in front of the structure represents the submergence of the toe. In view of the limited data that were found in literature, it is no surprise that a larger local water depth demands higher waves to create the same damage (Fig.9).

Influence of toe height h_m - h_t

The influence of the toe height shows a similar tendency as the influence of the local water depth.

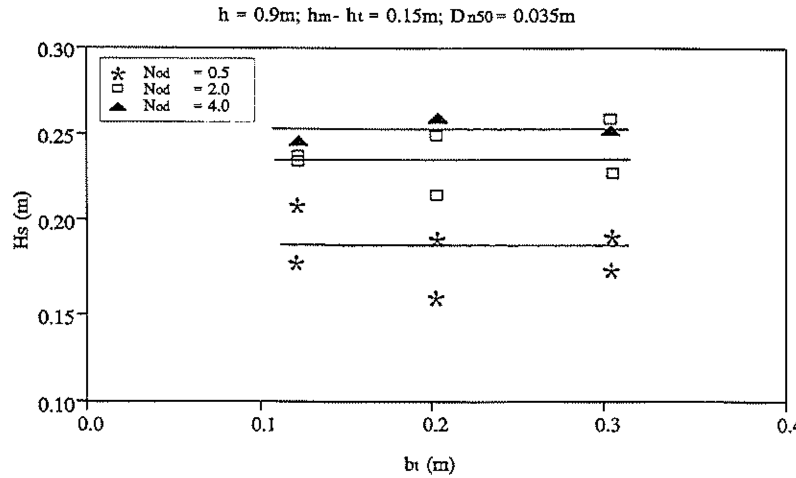

Fig. 8 Influence of toe width b_t on required wave height H_s to cause fixed damage levels N_{od}

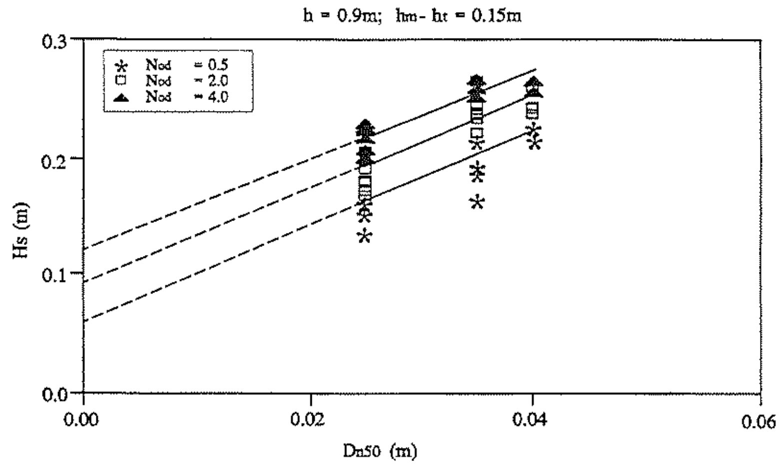

Fig. 9 Relation between wave height H_s and stone diameter D_{n50} for fixed damage levels. Note that the lines do not cross the origin.

Influence of the stone diameter D_{n50}

When wave height and stone diameter are plotted against each other, it is again not surprising that using larger stones leads to a more stable toe. The same is found in the stability analysis of the armour layer. When analyzing available literature on the stability of the armour layer, all authors use a stability number $H_s/\Delta D_{n50}$. This implies that the damage curves in a graphical representation cross the origin. This appears, however, not to be the case for the present tests on toe stability (Fig. 10). Physically, this is understandable because the armour layer is situated in the zone of direct wave attack, whereas the toe is protected by its submerged position. This leads apparently to a threshold value and an offset in the damage curves.

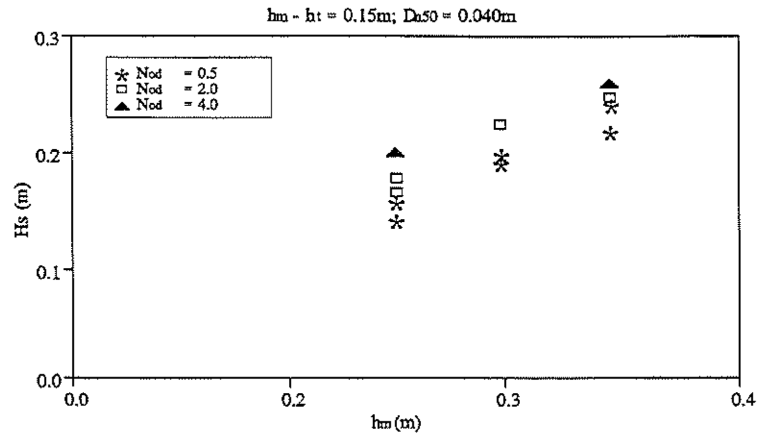

Fig. 10 Influence of local water depth h_m on required wave height H_s to cause fixed damage levels N_{od}

ANALYSIS

Although the data show quite some scatter, a relation could be established between wave height and damage level. The shape of such a damage curve can be described by the relation:

$$H_s = b * N_{od}^{0.15} \tag{2}$$

Introduction of the stability number $H_s/\Delta D_{n50}$ as a dimensionless parameter in the analysis has the advantage that it relates directly to the traditional presentation of breakwater stability. It has the disadvantage that the above mentioned offset is introduced. For the first analysis, it has been decided to stick to the traditional presentation. Under the assumption that the damage is also a (power) function of the relative submergence of the toe h_t/h_m, we can plot all tests as in Fig. 11. The best fit expression for the damage becomes:

$$\frac{H_s}{\Delta D_{n50}} = 6.5 \left(\frac{h_t}{h_m}\right)^{1.2} N_{od}^{0.15} \tag{3}$$

This relation is valid in the range $0.4 < (h_t/h_m) < 0.9$.

After rewriting and transformation of the damage percentage into the damage level N_{od} it was possible to introduce the expression by Van der Meer, as shown in Fig. 2 and Eq. (1), into the plot of the present data in Fig. 11. It becomes clear that the earlier recommendation and the present best fit are quite similar. The remaining difference between the two curves can be explained by the definition of acceptable damage, in percentages in the CUR/CIRIA manual, and in the form of N_{od} in the present study. Judging Eq. (3) on the basis of Fig. 11, it is clear that the scatter of results around the best fit is not very satisfactory.

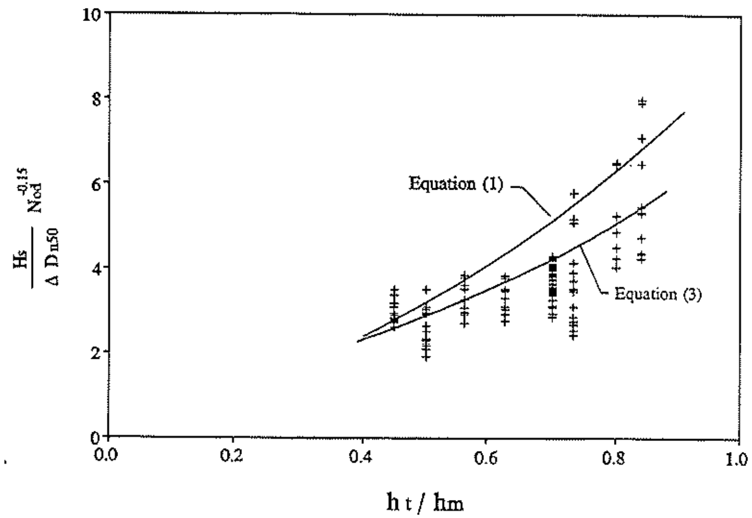

Fig. 11 Stability as a function of h_t/h_m

It is evident that another parameter had to be found to take into account the submergence of the toe. In the Shore Protection Manual, the relation h_t/H_s is used. This implies a second use of one of the parameters that was already included in the stability number $H_s/\Delta D_{n50}$. Instead of using H_s again, one could also opt for D_{n50} to make h_t dimensionless. In the study, both options have been worked out. The results are demonstrated in Fig. 12a and 12b. It is evident that eventually h_t/D_{n50} was chosen as parameter in the ultimate relation. Best fit procedures resulted in the following expression:

$$\frac{H_s}{\Delta D_{n50}} = \left(0.24 \frac{h_t}{D_{n50}} + 1.6\right) N_{od}^{0.15} \tag{4}$$

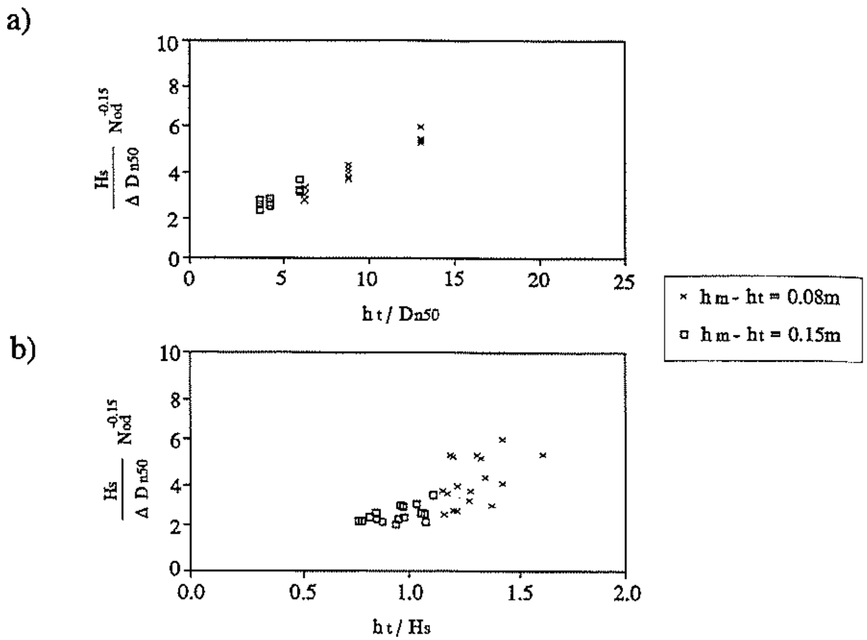

Fig. 12 Stability as a function of dimensionless toe heights h_t/D_{n50} and h_t/H_s

An idea of the overall quality of this result may be obtained from Fig. 13, which shows an acceptable reduction of the scatter as compared with Fig. 11. Eq. (4) can be used in the range:

$$0.4 < h_t/h_m < 0.9$$

$$3 < h_t/D_{n50} < 25$$

Almost no damage means $N_{od} = 0.5$, acceptable damage means $N_{od} = 2$ (some flattening out) and severe damage can be described by $N_{od} = 4$. The latter damage levels relate to a "normal" toe which has a width of a few diameters. For larger toes a higher damage level can be acceptable as more stones are then available in the toe.

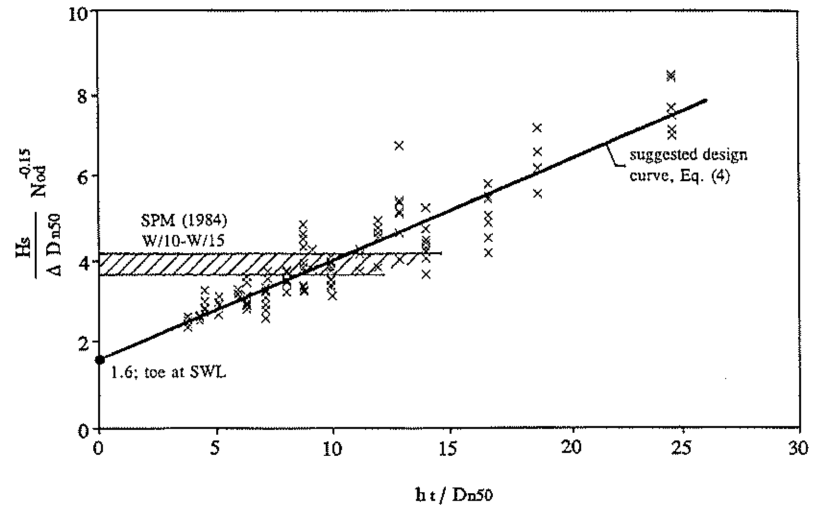

Fig. 13 Overall results with suggested design curve

DISCUSSION

The result presented in Eq. (4) is an acceptable approximation of the test results. For values $h_t \leq 0$, the stability number is 1.6. Physically, this condition represents a situation that the toe extends to a level above MSL. The toe has thus been reconditioned into an armour layer, and one would expect a similar stability behaviour. In comparable conditions, the use of the Van der Meer stability formula for armour rock leads to a stability number of 1.7, indeed very close to the 1.6 found in the present study.
The Shore Protection Manual (1984) gives for toe structures with $h_t/H_s > 2$ the advise to use W/10 to W/15, where W is the weight of the armour layer. Using $H_s/\Delta D_{n50} = 1.7$ for armour layers, as suggested above, this becomes $H_s/\Delta D_{n50} = 3.7$ to 4.2 for the toe. These values are also given in Fig. 13. For fairly deep toe structures this means that the SPM-advice is a conservative one.

Attention was also paid to the shallow water effects, and in particular to the deformation of the Rayleigh distribution due to breaking. In such case, one would expect that $H_{2\%}$ would yield better results than the use of H_s. In the present case, such substitution did not decease the existing scatter. It is emphasized that in the present series of tests the density of the stone (and thus Δ) was not varied.

CONCLUSIONS

Besides very rough design rules (Shore Protection Manual, 1984) and a method for depth limited situations only (CUR/CIRIA manual, 1991), good design information on toe structure stability at rubble mound breakwaters is lacking.

Small scale model tests showed that the influence of wave steepness and toe width on stability was neglectable. The stability of the toe can be described by the stability number $H_s/\Delta D_{n50}$, the relative toe depth h_t/D_{n50} and the damage level N_{od}.

A design formula has been suggested (Eq. 4) with the range of application.

NOTATION

b_t	width of the toe structure	(m)
D_{n50}	nominal diameter $(M_{50}/\rho_s)^{1/3}$	(m)
h	deep water depth	(m)
h_m	shallow water depth near structure	(m)
h_t	water depth above the toe structure	(m)
H_{m0}	significant wave height, calculated from energy density spectrum	(m)
H_s	significant wave height, calculated from wave height distribution	(m)
H_{so}	significant wave height near the wave board	(m)
$H_{2\%}$	wave height exceeded by 2% of the waves	(m)
m_0	zero-th moment of wave energy spectrum	(m^2)
M_{50}	average mass of stone class	(kg)
N_{od}	number of stones displaced out of a strip (along the longitudinal axis of the structure) with a width of one D_{n50}	(-)
s_{op}	fictitious wave steepness $(2\pi H_s/gT_p^2)$	(-)
T_m	mean wave period	(s)
T_p	wave period at peak of energy density spectrum	(s)
ρ_s	mass density of stone	(kg/m^3)
ρ_w	mass density of water	(kg/m^3)
Δ	relative density $(\rho_s/\rho_w - 1)$	(-)

REFERENCES

Brebner, A. and Donnelly, P. (1962). Laboratory Study of Rubble Foundations for Vertical Breakwaters. Engineer Report No. 23, Queen's University, Kingston, Ontario.

CIAD (1985). Computer aided evaluation of the reliability of a breakwater design. Report of the CIAD project group on breakwaters. CIAD, Zoetermeer, The Netherlands.

CUR/CIRIA (1991). Manual on the use of rock in coastal and shoreline engineering. CUR Report 154, Gouda, The Netherlands/Ciria special publication 83, London, U.K.

Gerding, E. (1993). Toe structure stability of rubble mound breakwaters. Master's thesis. Delft University of Technology. Delft Hydraulics Report H1874.

Gravesen, H. and Sørensen, T. (1977). Stability of rubble mound breakwaters. Proc. 24th International Navigation Congress, Leningrad, U.S.S.R.

Shore Protection Manual (1984). Coastal Engineering Research Center, Waterways Experiment Station, Vicksburg Miss., U.S.A.

Van der Meer, J.W. and Pilarczyk, K.W. (1990). Stability of low-crested and reef breakwaters, ASCE, Proc. 22nd International Conference on Coastal Engineering, Delft.

Van der Meer, J.W. and d'Angremond, K. (1991). Wave transmission at low-crested structures. In Coastal Structures and Breakwaters, ICCE, London, pp. 25-42.

Van der Meer, J.W. (1993). Conceptual Design of Rubble Mound Breakwaters. Delft Hydraulics Publication number 483.

Van der Meer, J.W., and Daemen, I.F.R. (1994). Stability and Wave Transmission at low-crested Rubble Mound Structures. ASCE, J. of Waterways, Ports, Coastal and Ocean Engineering, Vol. 120, Jan/Feb, pp. 1-19.