STABILITY OF RUBBLE MOUND BREAKWATERS WITH A BERM: THE UPPER SLOPE

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Physical model tests were performed to obtain information on the stability of rock slopes with a horizontal berm. This paper is focussed on the rock slope stability of the slope *above* the berm. By applying a berm the rock size in the upper slope can be significantly smaller than for a straight slope without a berm. The influence of the width of the berm, the level of the berm, and the wave steepness have been investigated. Based on the test results a prediction formula has been derived for 1:2 slopes with various dimensions of the berm.

INTRODUCTION

For rubble mound breakwaters and dikes the presence of a berm in the seaward slope can be very effective to reduce wave overtopping. To assess the influence of a berm on the wave overtopping discharge several methods are available; these can be based on empirical formulae or based on the data-driven Neural Network technique (see *e.g.* Van Gent *et al*, 2007). A berm in the seaward slope of a rubble mound breakwater does not only reduce the wave overtopping discharge, it can also increase the stability of the rock in the armour layer. Although for the effects of a berm on the wave overtopping relatively accurate methods are available, for the influence of berms on the stability only limited information is available (*e.g.* Vermeer, 1986, and the Rock Manual, 2007, for a structure with a slope below the berm of 1:6, and Dijkstra, 2008, with information on the stability of the slope below and at the berm). To fill the knowledge gap for steeper slopes (steeper than 1:6; which is relevant for many practical applications) and also for the slope above the berm, new physical model tests have been performed. Distinction has been made between the stability of the rock material above the berm and below the berm. This paper is focussed on the rock material above the berm and the reduced size of the rock that can be applied to this upper slope.

PHYSICAL MODEL TESTS

Physical model test were performed in a wave flume (length 55m, width 1m, height 1.2m) at Deltares, Delft. The wave board is equipped with second-order wave steering and active reflection compensation.

Wave conditions were measured by arrays of three gauges at deep water and at the location of the structure toe. The analysis was based on the time series of the incident waves at the toe. These signals, without reflected waves, were

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obtained using the method by Mansard and Funke (1980). The spectral significant wave height H_s and the wave period $T_{m-1,0}$ ($T_{m-1,0} = m_{-1}/m_0$ with $m_n = {}_0\int^\infty f^n S(f)\, df$ with n = -1 or 0) were obtained from the measured wave energy spectra. In Van Gent (2001) the wave period $T_{m-1,0}$ was found to appropriately describe the influence of wave energy spectra on wave run-up, while in later studies this wave period was found to be the most appropriate wave period for wave overtopping, wave reflection and the stability of rock slopes. (*e.g.* Van Gent *et al*, 2003). In all tests a Jonswap wave spectrum has been applied.

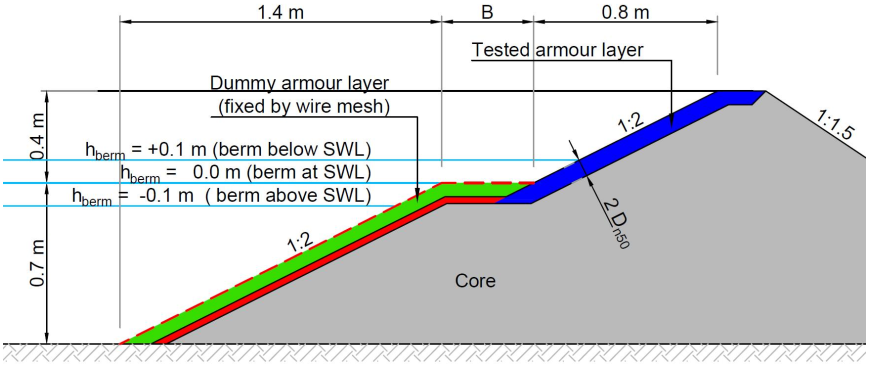

Figure 1. Configuration of tested structures with a berm (for stability of upper slope).

The basic configuration consists of a non-overtopped rubble mound slope with a berm, with a 1:2 slope above the berm and a 1:2 slope below the berm. A horizontal foreshore on which no wave breaking occurred has been applied. The size of the rock in the armour layer was D_{n50} =20mm for the upper slope and D_{n50} =40mm for the lower slope and the berm (model scale). The berm was horizontal. The width and position of the berm with respect to the water level was varied. Structures with a berm width of 0m, 0.2m and 0.4m were tested. For the stability of the lower slope also a width of 0.8m was applied. The still water levels (SWL) were 0.1m below, at, and 0.1m above the berm.
The damage to the rock armour layer was measured with a surface profiler consisting of 9 gauges. With the profiles measured by each of the 9 gauges the average erosion profile and eroded area A_e were computed. The measured damage levels $S = A_e / D_{n50}^2$ were mainly between S=0 and S=12.
Each configuration was tested with a series of test runs with an increasing wave height and a constant wave steepness. Each test run consisted of 1000 waves. The significant wave height was increased in steps of 0.02m until the amount of damage reached a level of S=12 or a maximum significant wave height of H_s=0.24m. Damage has been repaired after each series of test runs, but not after each individual test run. This means that the measured damage could be higher than if the slope would have been repaired after each individual test run. The latter would have been a better alternative but much more time consuming. The

initial movements of stones in the first performed test run of each series of wave heights (*i.e.* test run with the lowest waves) were not considered as damage. Each configuration was tested with 2 wave steepnesses (s_p=0.015 and s_p=0.04 which corresponds to $s_{m-1,0}$ =0.018 and $s_{m-1,0}$ =0.048 respectively).
Tests were performed to analyse the stability of the slope above the berm, while the material at the lower slope was fixed, and tests were performed to analyse the stability of the slope below the berm, while the material at the upper slope was fixed. In the tests for the lower slope 4 berm widths were tested; in the tests for the upper slope 3 berm widths were tested. For the stability tests of the upper slope (1:2) between 3 and 7 test runs were performed in each test series. Table 1 provides an overview of the values of the most important parameters.

Table 1. Parameters of the data set.

Parameter	Symbol	Value
Slope angle	cot α	2
Relative density	Δ	1.7
Rock size upper slope (m)	D_{n50}	0.020
Rock size lower slope and berm (m)	D_{n50}	0.040
Grading armour material	D_{n85}/D_{n15}	1.5
Core material	$D_{n50\text{-}core} / D_{n50}$	0.5
Berm width (m)	B	0.0 – 0.8
Berm level w.r.t. SWL (m)	h_b	-0.1 – 0.1
Berm level ratio (positive: submerged)	h_b / H_s	-1.2 – 1.4
Wave steepness ($T_{m-1,0}$)	$s_{m-1,0}$	0.018 - 0.048
Wave steepness (T_p)	s_p	0.015 - 0.040
Surf-similarity parameter ($T_{m-1,0}$; cot α)	$\xi_{m-1,0}$	2.3 - 3.7
Wave height ratio	$H_{2\%} / H_s$	1.4
Deep-water wave height over depth	H_s / h_{toe}	< 0.4
Number of waves	N	1000
Stability parameter	$H_s / \Delta D_{n50}$	2 – 7
Damage level	S	< 20

PREDICTION METHOD FOR UPPER SLOPE

Here, a method with empirical formulae will be described to provide guidance in the conceptual design stage to determine the required stone size above the berm. Such a reduction in stone size facilitates important cost savings for rubble mound breakwaters with a berm. The increased stability for a structure with a berm compared to the stability of a straight slope can be quantified based on the described data-set.
The proposed approach to assess the damage to the upper slope is as follows:

1. Determine the damage S for a straight slope without berm using an existing stability formula.
2. Determine the damage $S_{z\text{-}berm}$ for a slope without berm, but only for the part of the slope that is above the vertical position of the berm: $S_{z\text{-}berm} = \gamma_{pos} S$
3. Determine the reduction to the damage due to the presence of the berm: $S_{berm} = \gamma_{berm} S_{z\text{-}berm} = \gamma_{berm} \gamma_{pos} S$

For the first step an existing stability formula can be applied. In Van Gent *et al* (2003) the stability equations by Van der Meer (1988) were modified to extend its field of application. These modified equations are for plunging waves:

$$\frac{S}{\sqrt{N}} = \left(\frac{1}{c_{plunging}} \frac{H_s}{\Delta D_{n50}} \xi_{m-1,0}{}^{0.5} P^{-0.18} \frac{H_{2\%}}{H_s} \right)^5 \tag{1}$$

and for surging waves:

$$\frac{S}{\sqrt{N}} = \left(\frac{1}{c_{surging}} \frac{H_s}{\Delta D_{n50}} \xi_{m-1,0}{}^{-P} P^{0.13} \tan\alpha^{0.5} \frac{H_{2\%}}{H_s} \right)^5 \tag{2}$$

with the transition at $\xi_c = (c_{plunging} / c_{surging} \; P^{0.31} \sqrt{\tan \alpha})^{1/(P+0.5)}$. For slope angles more gentle than cot α = 4 the equation for plunging waves needs to be used, irrespective of whether $\xi_{m-1,0}$ is larger than ξ_c or not. The optimal values for the coefficients are $c_{plunging}$ = 8.4 and $c_{surging}$ = 1.3. For the tested structures P=0.5 is used.

Since the effect of berms on the expected damage or required rock diameter in the upper slope can be large, using Eqs.1 and 2 as a reference for straight slopes without a berm leads for many applications to very high damage levels for straight slopes. For instance, if the rock diameter in the upper part of the slope can be reduced with a factor 2 to 3, the calculated S values for straight slopes may be 30 to 250 times larger. This means that Eqs.1 and 2 are often applied in the calculation method outside their range of validity.

The present data is both in the range of “plunging waves” and “surging waves”. However, the data on “surging waves” is close to the transition from “plunging waves” to “surging waves”. Since structures with a berm can also be seen as structures that are somewhat more gentle than without a berm, such that the waves would actually be more plunging than surging, here all data is treated as “plunging waves” (Eq.1 is applied).

The effect of cumulative damage (sequential storms) can be taken into account by using a method such as proposed by Van der Meer (1988), see also Fig.5.44 in the Rock Manual (2007).

The second step is to account for the vertical position of the berm to facilitate a comparison for configurations with and without a berm. Use is made of the data without a berm, in which the part below the level of the berm was fixed such that only damage could occur above the berm (z_{berm} is positive if the level of the berm is above SWL; $z_{berm} = -h_b$). Although for this step limited data is available a first estimate is made to obtain an empirical formula for $\gamma_{pos} = S_{z\text{-}berm} / S$. For this purpose data with $s_{m-1,0}$ = 0.018 and $S_{z\text{-}berm}$ < 12 was used. The empirical formula used here is:

$$\gamma_{pos} = \left(\frac{2H_s + h_b}{4H_s} \right)^5 \qquad \text{with } 0 \le \gamma_{pos} \le 1 \tag{3}$$

This means that if the level of the berm would be at $2H_s$ above SWL (emerged berm) this would lead to no damage to the upper slope; for levels equal or higher than h_b =-$2H_s$ (submerged berm) there is no reduction: γ_{pos}=0. If the berm is equal or lower than h_b =$2H_s$ there is no reduction for the upper slope: γ_{pos}=1. For the situation that the berm is at the level of the SWL, $\gamma_{pos}=(0.5)^5$ which means that a diameter that is a factor 2 smaller, would lead to the same value for *S*; thus, if for instance *S*=4 would be acceptable for a straight slope, and *S*=4 would also be acceptable for the upper slope, the diameter can be a factor 2 smaller in the upper slope. However, it should be noted that if a certain damage level is considered acceptable for a straight slope, it is likely that for the upper slope a somewhat smaller damage level should be applied since part of the damage is likely to occur to the lower part of the slope. If half of the acceptable damage would be attributed to the upper slope (*e.g.* *S*=2), in this example the size of the diameter in the upper slope could be reduced with a factor 1.75 (with *S*=2) compared to the required diameter for the entire straight slope (*S*=4). In this paper no analysis is made of what is an acceptable damage level for the part of the slope above the berm.

Fig.2 shows the measured damage to the part above the level of the berm (with *B*=0) compared to the calculated damage using Eq.1 and Eq.3. For the present purpose the match between the measured and calculated damage is considered sufficiently good. If more data becomes available, for instance for other slope angles and/or for a wider range of berm levels, Eq.3 may have to be adjusted or extended.

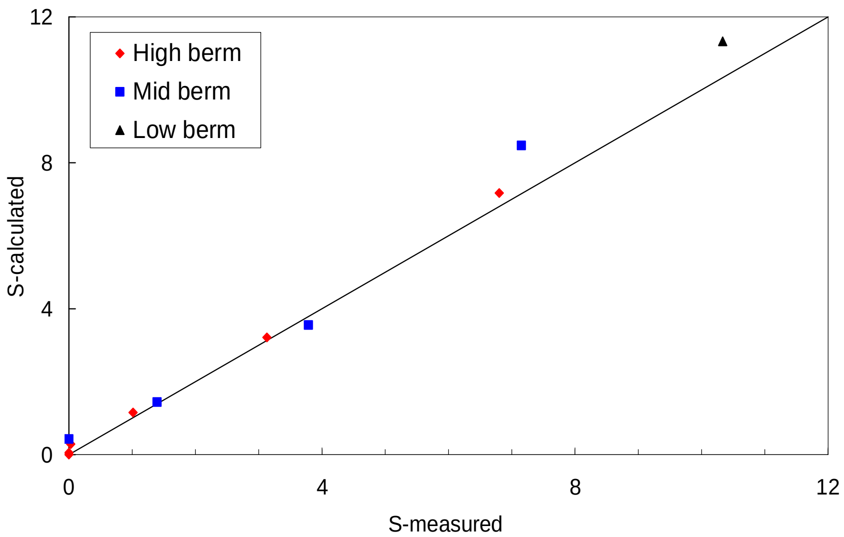

Figure 2. Calculated and measured damage on the upper part of the slope for tests without a berm (Eqs. 1-3).

The third step is to account for the influence of the berm on the stability. The two parameters that are accounted for here are the width of the berm and the wave steepness. The following empirical formula is calibrated:

$$\gamma_{berm} = 1 - c_1 \, s_{m-1,0}^{\;c_2} \left(\frac{B}{H_s} \right)^{c_3} \qquad \text{with } \gamma_{berm} \geq 0 \tag{4}$$

The calibration based on the data with the applied 1:2 slope provided: c_1=0.075, c_2=-0.5 and c_3=0.3. This simple formula may have to be adjusted or extended if more data becomes available, for instance to include the influence of other slope angles and/or for a wider range of berm levels.
The resulting stability formula for plunging waves can be written as follows:

$$S = \gamma_{berm} \gamma_{pos} \left(\frac{1}{c_{plunging}} \frac{H_s}{\Delta D_{n50}} \xi_{m-1,0}^{\;0.5} P^{-0.18} N^{0.1} \frac{H_{2\%}}{H_s} \right)^5 \tag{5}$$

and for surging waves:

$$S = \gamma_{berm} \gamma_{pos} \left(\frac{1}{c_{surging}} \frac{H_s}{\Delta D_{n50}} \xi_{m-1,0}^{\;-P} P^{0.13} \tan\alpha^{0.5} \; N^{0.1} \frac{H_{2\%}}{H_s} \right)^5 \tag{6}$$

Note that based on the tested configurations and conditions only Eq.5 is applied. and for the slope cot α = 2 is used, irrespective of the presence of a berm or not.
Figs.3-6 show the measured and calculated damage, each for the 3 levels of the berm. The results of the measurements in these figures show:

- In all tests a higher wave steepness leads to less damage.
- In all tests a wider berm leads to less damage.
- In all tests a higher berm leads to less damage; for low berms the damage increases relatively quickly with increasing wave heights, while for a high berm the damage progresses more gradually.

The calculated damage levels in these figures show similar trends for the wave steepness, for the width of the berm, and for the level of the berm. The comparison between measured and calculated results show:

- The agreement between measured and calculated damage is somewhat better for the low wave steepness than for the high wave steepness; the agreement for the high wave steepness in combination with a low-wide or mid-wide berm is relatively weak.
- The agreement between measured and calculated damage is rather good; the relatively low damage levels are predicted more accurate than those for high damage levels.

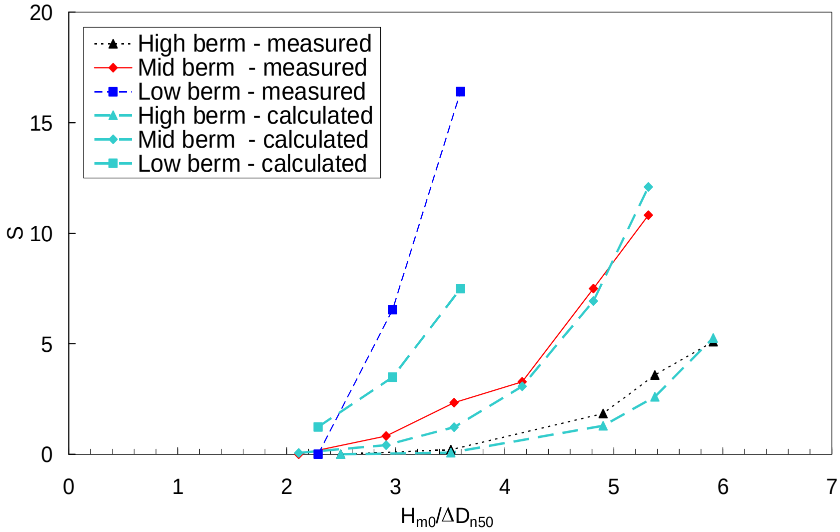

Figure 3. Measured and calculated damage for narrow berm and low wave steepness.

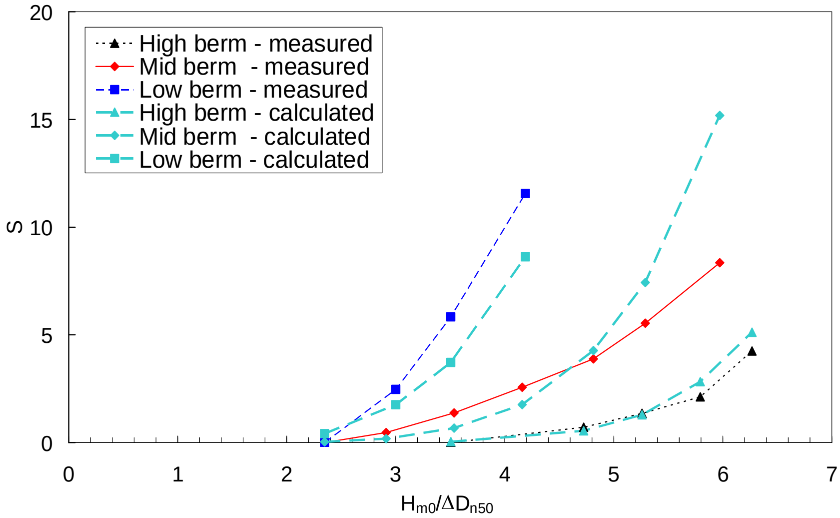

Figure 4. Measured and calculated damage for wide berm and low wave steepness.

- The standard deviation of the differences between measured and predicted values of $S/\sqrt{N}$ is 0.11 for all data with S smaller than 20 and 0.08 for all data with S smaller than 12. Note that the standard deviation for data with straight slopes as described in Van Gent *et al* (2003) and Van Gent (2004) was 0.10 for rock slopes with a permeable core.

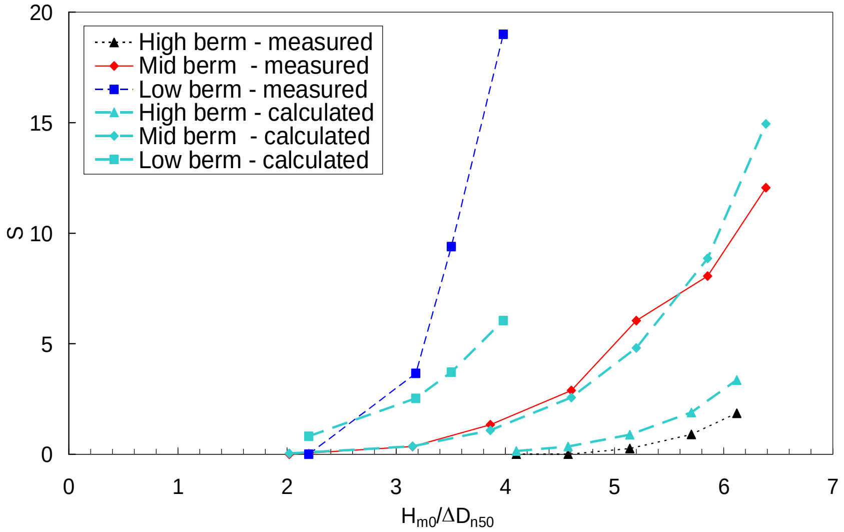

Figure 5. Measured and calculated damage for narrow berm and high wave steepness.

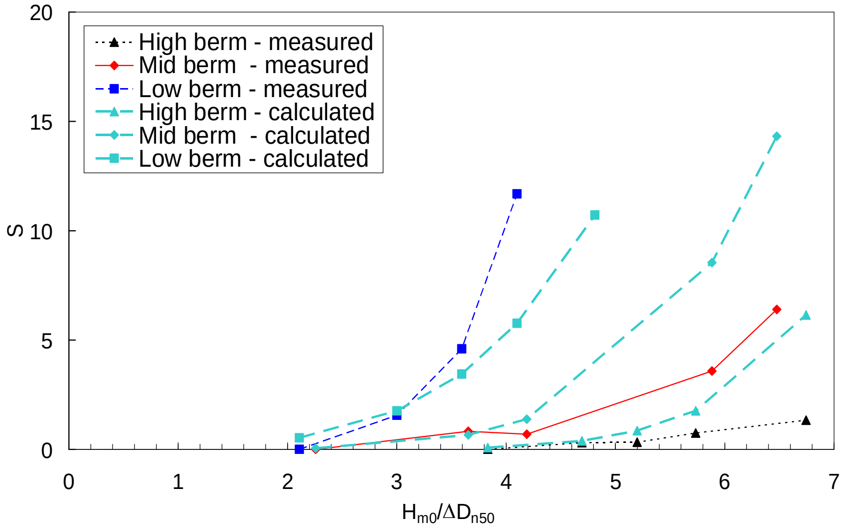

Figure 6. Measured and calculated damage for wide berm and high wave steepness.

Figs.7 and 8 show all data with configurations with berms in comparison with the proposed formulae. Note that for all data Eq.5. ("plunging waves") is applied. Fig.7 shows again that the relatively low damage levels are predicted more accurate than those for high damage levels.

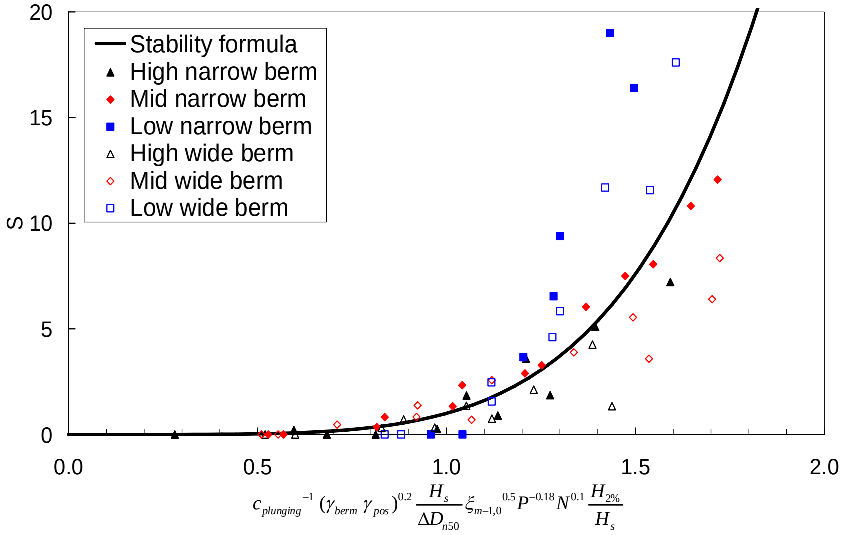

Figure 7. Measured values versus the parameters in the prediction formula (Eq.5).

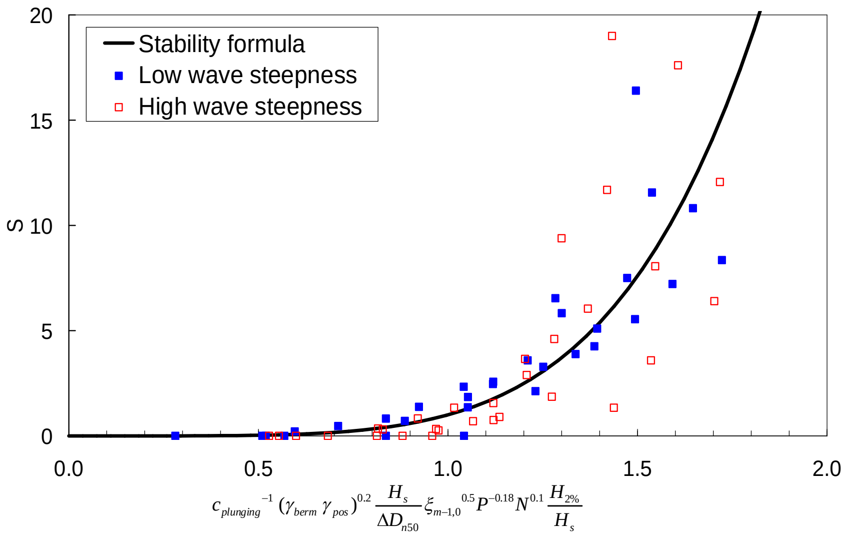

Figure 8. Measured values versus the parameters in the prediction formula (Eq.5).

Fig.8 shows the same data as Fig.7 but now distinction is made between tests with a low wave steepness and a high wave steepness, indicating that the proposed method provides better results for the low wave steepness than for the high wave steepness. The sub-sets of data that provide the largest deviations are the tests with the high wave steepness in combination with the mid-wide berm.

In above described analysis Eq.5 is used (all data treated as “plunging waves”). If Eqs.5 and 6 are used (thus data partly in the range of “plunging waves” and partly just within in the range “surging waves”), the standard deviations are hardly affected: again 0.11 for all data with *S* smaller than 20 and 0.08 for *S* smaller than 12.
Here, in the first step the modified version of the formulae by Van der Meer (1988), as proposed in Van Gent *et al* (2003), has been used. In the latter also a more simple formula was proposed that can be applied, for instance if no information on the wave period or wave energy spectra is available. If no information on the wave period is available also Eq.4 needs to be simplified (c_2=0) and re-calibrated. The latter leads to (c_1=0.4 and c_3=0.3). This leads to the following simplified formula:

$$S = \gamma_{berm}\gamma_{pos}\left(0.57\,\frac{H_s}{\Delta D_{n50}}\,(\tan\alpha)^{0.5}\,(1+D_{n50core}/D_{n50})^{-2/3}\,N^{0.1}\right)^5 \qquad (7)$$

with c_1=0.4, c_2=0 and c_3=0.3 in Eq.4 for γ_{berm}. Using this formula leads to a standard deviation of the differences between measured and predicted values of $S/\sqrt{N}$ of 0.13 for all data with *S* smaller than 20 and 0.10 for all data with *S* smaller than 12. Both are slightly larger than using Eqs.5 and 6.
The measured damage levels were the result of a number of conditions, thus representing cumulative damage. In the analysis each condition has been treated as an individual storm without preceding conditions. As discussed before the applied method also allows for taking the effect of cumulative damage (sequential storms) into account. The influence of this aspect has been analysed by using the mentioned approach with cumulative damage. The influence is rather small. Recalibrating Eq.4 for γ_{berm} would lead to c_1=0.08 instead of c_1=0.075. After this re-calibration the standard deviations of the differences between measured and predicted values of $S/\sqrt{N}$ are again 0.11 for all data with *S* smaller than 20 and 0.08 for all data with *S* smaller than 12. Based on this analysis it is concluded that this aspect is not essential for the performed tests and obtained prediction formulae.
Fig.9 shows a graph where on the vertical axis the increase of stability is shown by using the ratio of required rock size for a straight slope and the required rock size above the berm. This figure shows (for $s_{m-1,0}$ = 0.02) this ratio as a function of the (non-dimensional) water depth above the berm (h_b/H_s) for 3 (non-dimensional) widths of the berm. Note that a berm width of h_b/H_s =0 shows the applied reduction due to focussing on the damage at a level above the berm only. In Fig.9 a ratio of for instance 2.5 means that at the upper slope a rock diameter can be applied that is a factor 2.5 smaller than for a straight slope under the same wave loading. Fig.9 shows that for a low (submerged) berm the reduction in rock size is small, for a berm at SWL the reduction is large, and for a high berm the reduction is very large.

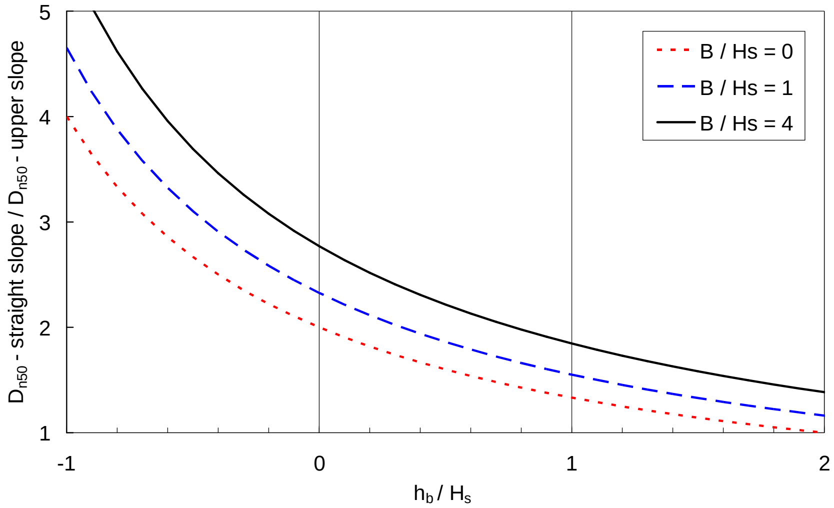

Figure 9. Stability increase in terms of required rock size at the upper slope as function of the level of the berm (a positive h_b is a submerged berm; $s_{m-1,0}$ = 0.02).

In the Rock Manual (2007) Fig.5.69 shows a diagram for an upper slope of 1:3 in combination with a 1:6 lower slope. At h_b/H_s=1 the diagram shows an increase in stability in the range of 1.75-2.5, which is rather close to values shown in Fig.9 for a 1:2 slope below and above the berm (for comparison B/H_s=4 corresponds to about B/L=0.1).

Within the ranges of parameters shown in Table 1, Fig.9 can be used as a guideline for the conceptual design of a structure with a horizontal berm and a 1:2 slope.

CONCLUSIONS

This study on the stability of rubble mound structures with a berm has lead to the following conclusions on the stability of the upper berm:

- The rock size in the upper slope, *i.e.* the slope above the berm, can be significantly smaller than for a straight slope without a berm.
- The increase in stability of the rock in the upper slope depends on the level of the berm, the width of the berm, and the wave steepness.
- A method is proposed to estimate the increase in stability. Within the ranges of parameters shown in Table 1, this method can be used as a guideline for the conceptual design of a structure with a horizontal berm and 1:2 slopes.

It is recommended to study the influence of a berm on the stability of the upper slope also for other slopes, for instance 1:4 slopes. Furthermore, it is recommended to study the effects of a berm on the required rock size at the berm and at the lower slope (*i.e.* an extension of tests by Dijkstra, 2008), and to study the effects of berms in combination with oblique wave attack.

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REFERENCES

Dijkstra , O.P.J., 2008, Armour stability on a bermed slope breakwater, M.Sc. thesis, *Delft University of Technology*.

Mansard, E. and Funke, E., 1980, The measurement of incident and reflected spectra using a least-square method, *Proc. ICCE'80*, ASCE, pp.154-172.

Rock Manual, 2007, *The use of rock in hydraulic engineering* (2^{nd} edition), CIRIA, CUR, CETMEF, Published by C683, CIRIA, London (ISBN 978-0-86017-683-1 and 5).

Van der Meer, J.W., 1988, Rock slopes and gravel beaches under wave attack, Ph.D. thesis, *Delft University of Technology*.

Van Gent, M.R.A., 2001, Wave run-up on dikes with shallow foreshores, *Journal of Waterway, Port, Coastal and Ocean Engineering*, ASCE, Vol.127, No.5, Sept/Oct 2001, pp.254-262.

Van Gent, M.R.A., Van den Boogaard, H.F.P., Pozueta, B. and Medina, J.R., 2007, Neural network modelling of wave overtopping at coastal structures, *Coastal Engineering*, Elsevier, Vol.54, pp. 586-593.

Van Gent, M.R.A., Smale, A. and Kuiper, C., 2003, Stability of rock slopes with shallow foreshores, *Coastal Structures conference 2003*, Portland, USA.

Van Gent, M.R.A., 2004, On the stability of rock slopes, *Balkema, Keynote, Proc. NATO-Workshop on Environmental friendly coastal protection structures*, Varna, Bulgaria.

Vermeer, A.C.M., 1986, Stability of rubble mound berms and toe constructions; report on literature survey and model investigation (in Dutch), *Delft Hydraulics Report M2006*, Delft.

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Physical model tests
Wave flume