

STABILITY OF HARDLY RESHAPING BERM BREAKWATERS EXPOSED TO LONG WAVES

Jonas Bjerg Thomsen¹, Mads Sønderstrup Røge¹, Nicole Færch Christensen¹, Thomas Lykke Andersen¹, Jentsje W. van der Meer²

Stability of hardly reshaping berm breakwaters has been investigated in this paper, mainly focusing on exposure to long waves. The study continues previous work by Aalborg University, which suggested that stability of berm breakwaters follow the plunging equation of the Van der Meer stability formula also in the surging domain. Different combinations of berm widths and elevations were tested in order to see if berm breakwaters in general follow the plunging formula or if they start to behave more like conventional breakwaters.

Keywords: breakwater; berm breakwater; stability; damage; hardly reshaping; long waves

INTRODUCTION

The presence of a berm in a breakwater design induces several advantages e.g. use of cheaper construction methods and effective reduction of wave overtopping.

Depending on the reshaping and construction method, berm breakwaters are divided into different categories, defined by several authors. Sigurdarson and Van der Meer (2013) introduced a classification based on the structural behaviour, such as hardly reshaping, partly reshaping and fully reshaping. The classification is by means of the stability number $H_0 = H_s/\Delta D_{n,50}$, the damage $S_d = \frac{A_e}{D_{n,50}^2}$ and the recession $Rec/D_{n,50}$. Mass armoured berm breakwaters are classified as partly reshaping when $S_d > 10$ and approximately $H_0 > 2$ and become fully reshaping when $S_d > 20$, $H_0 = 2.5-3$. For this classification the 100-year wave height is used, or in case of scientific tests, the one but last test condition.

Traditionally stability was dealt with by berm recession, which is the relevant parameter for partly and fully reshaping structures, where the structure, when exposed to wave attack, reshapes into a S-shaped profile (cf. Fig. 1 (right)). Several authors like Van der Meer (1992), Tørum and Krogh (2000), Lykke Andersen and Burcharth (2010), Moghim et al. (2011), Shekari and Shafieifar (2013) and Sigurdarson and Van der Meer (2013) dealt with this. For hardly reshaping structures Lykke Andersen et al. (2012), Burcharth (2013) and Sigurdarson and Van der Meer (2012) suggested that the damage parameter S_d or the eroded area A_e , gives a more accurate description of the reshaping than the recession (cf. Fig. 1).

Lykke Andersen et al. (2012) suggested that for hardly reshaping berm breakwaters different types of damage progression occurs depending on the berm elevation and front slope. If the berm elevation is high and/or the front slope less steep, the damage might be similar to a straight slope, hence the damage starts as local erosion, cf. Fig. 1 (Left), and the formulae by Van der Meer (1988) is expected to provide good results.

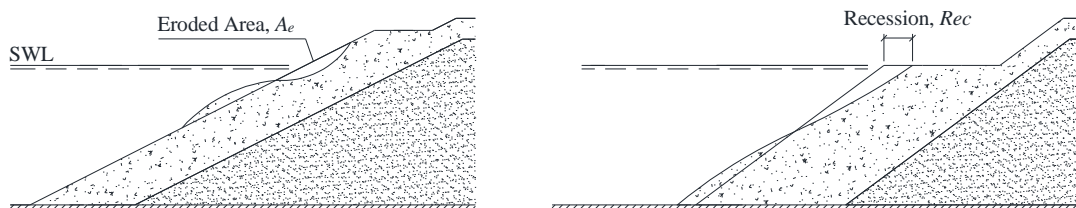


Figure 1: Types of damage progression on a berm breakwater with high berm and gentle front slope (left), and low berm and steep front slope (right). After Lykke Andersen et al. (2012).

For berm breakwaters with low berm elevation and/or a steep front slope, the damage progression is significantly different from that of a non-overtopped straight slope. Here the damage develops from the berm and progresses downwards, cf. Fig. 1 (right). Lykke Andersen et al. (2012) stated five reasons that might cause difference between the stability of hardly reshaping berm breakwaters compared to straight slopes:

¹ Department of Civil Engineering, Aalborg University, Sofiendalsvej 9-11, DK-9200 Aalborg SV, Denmark, jbt@civil.aau.dk, msr@civil.aau.dk, nicolefaerch@hotmail.com, tla@civil.aau.dk

² Van der Meer Consulting BV, P.O. Box 11, 8490 AA Akkrum, The Netherlands, and UNESCO-IHE, P.O. Box 3015, 2601 DA Delft, The Netherlands, jm@vandermeerconsulting.nl

1. A large part of the energy passes over the berm causing less damage than on a straight non-overtopped slope. In contrast to low-crested structures a large part of the water though return from the berm and upper slope.
2. The waves feel a flatter slope due to the berm. The berm causes a different breaking type than for a non-overtopped slope.
3. The stones on the top of the berm move more easily due to lack of interlocking from units above.
4. If the berm is low, the damage cannot progress as high above SWL as it would on a non-overtopped slope.
5. The front slope might be very steep, much steeper than for conventional rock structures and this might influence the effect of wave period on the stability of the steep slope.

From model tests Lykke Andersen et al. (2012) found that the Van der Meer (1988) formulae could be used to predict the damage for steep and hardly reshaping berm breakwaters (1:1.25), but saw that the stability always followed the plunging formula even in the surging regime (low wave steepness). From this, it was concluded that the berm changes the type of wave breaking (point 2).

The present study follows up on the investigations by Lykke Andersen et al. (2012) in order to clarify to what point the stability of the berm breakwater follows the tendency of the plunging formula and when a surging domain exists. The study is based on new two-dimensional model tests for hardly reshaping berm breakwaters with varying berm widths and berm elevations for a range of surf similarities ξ , mainly focussing on low steepness (surging regime). It should be noted that the cross-section of the berm breakwater is a kind of academic one, as hardly reshaping berm breakwaters in practical design have various rock gradings and only the largest grading is present on top of the berm and at the front side. The present set-up uses one large berm with only one rock grading and it also does not consider a toe structure. Despite that, the mentioned effects can be studied with this set-up of testing.

MODEL TEST SET-UP

For the present study, 24 new tests (denoted 2014 tests) were conducted at Aalborg University, and used together with 14 tests previously carried out as part of a master thesis (denoted 2013 tests).

The tests were carried out in a wave flume with dimensions of 25.0 x 1.5 x 1.0 m (L x W x H), cf. Fig. 2. The flume had a bottom slope of 1:100 leading to 0.13 m deeper water at the wave maker than at the structure, making it possible to generate depth-limited waves without wave breaking at the paddle.

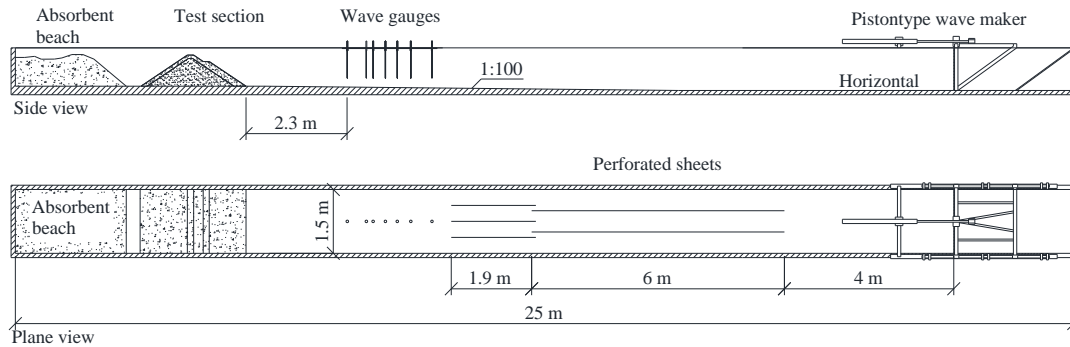


Figure 2: Two-dimensional wave flume at Aalborg University.

The cross-sections used in the present tests, are illustrated in Fig. 3 and Fig. 4. Different combinations of berm widths and elevations were covered, though only emerged berms were tested, with the lowest berm at SWL. One of the berm elevations was chosen to $0.65 H_{m0}$ (where H_{m0} is the significant wave height for the design event), according to preferred design dimensions by Sigurdarson and Van der Meer (2013). The design wave height corresponded to the second highest in a series with a following overload condition. Also higher berm elevations were tested (up to $1.9 H_{m0}$). The narrow berm width corresponded to an armour thickness of two layers of rocks and the wide berm corresponded to four layers of rock. The 2013 tests were conducted with front slopes $\cot \alpha = 1.25$ and 1.5 while the 2014 tests only used $\cot \alpha = 1.5$.

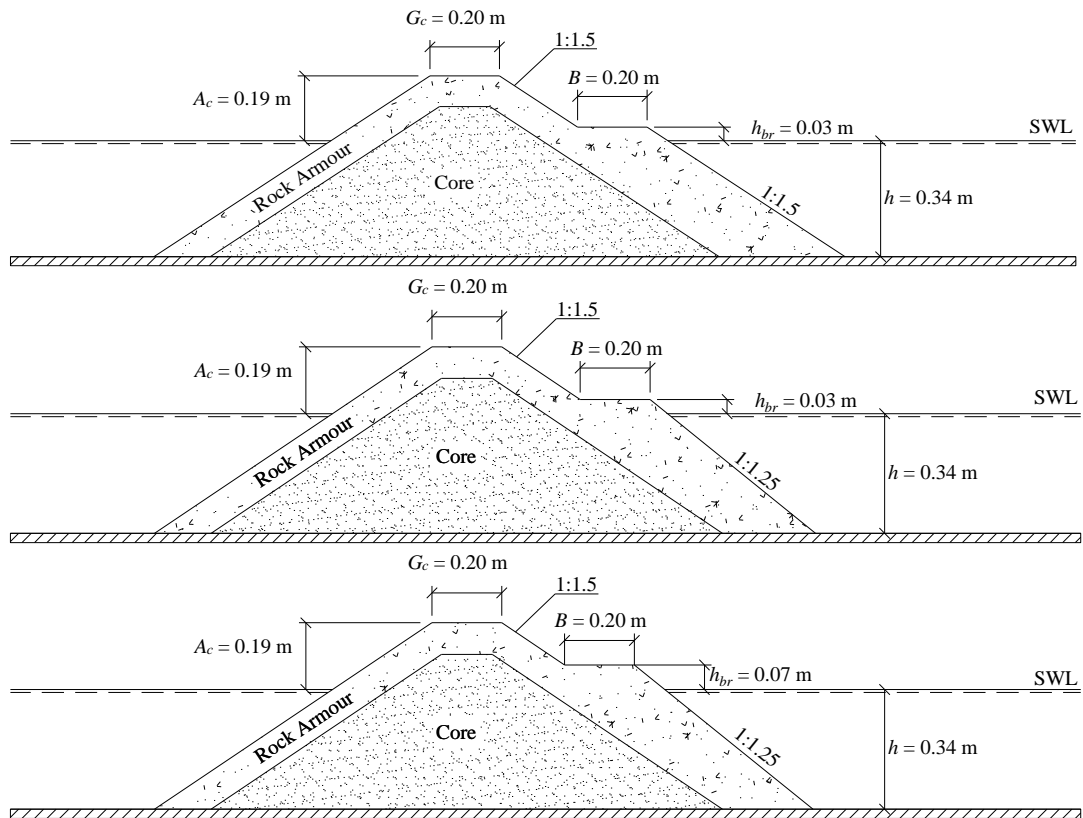


Figure 3: Cross-sections used for the 2013 tests.

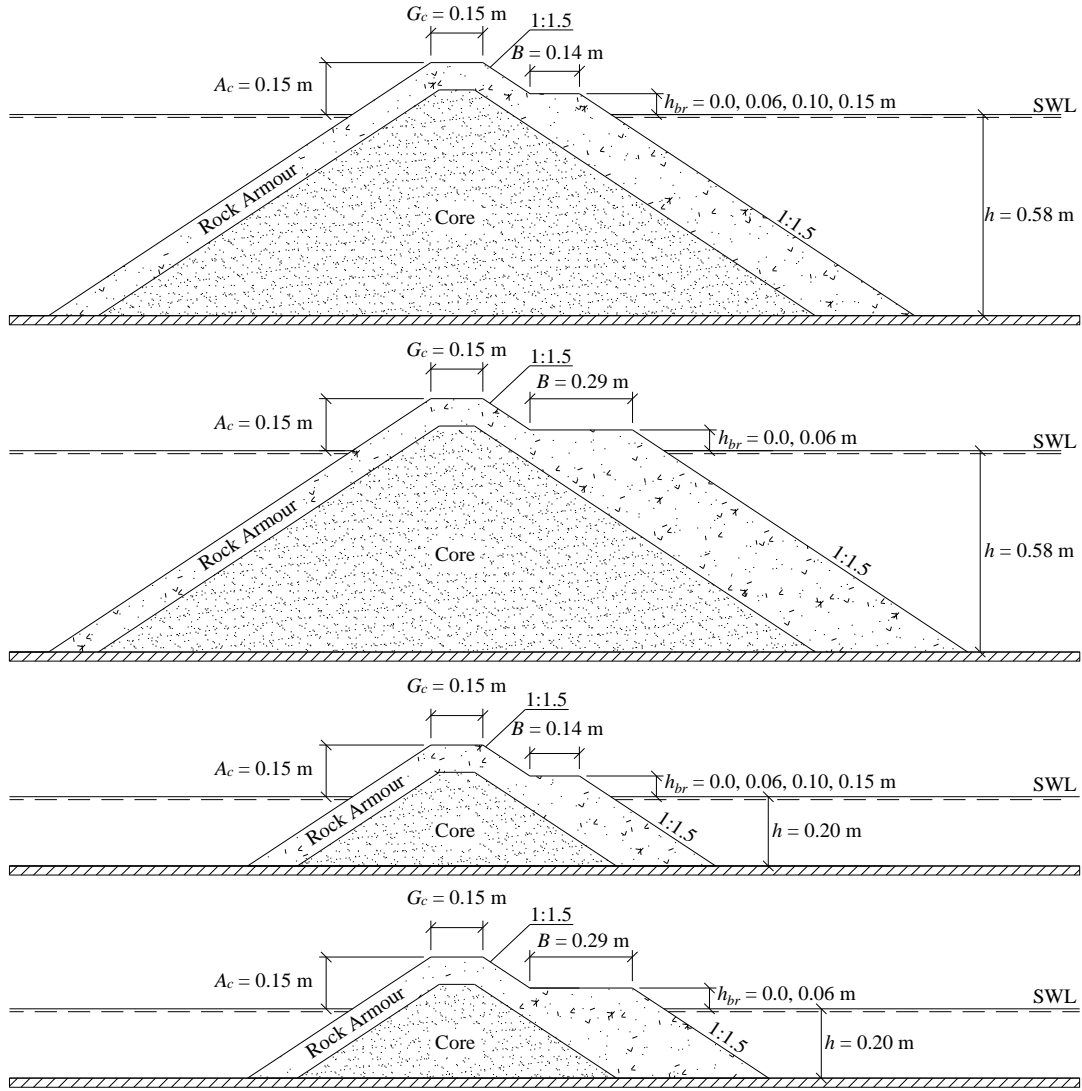


Figure 4: Cross-sections used for 2014 tests.

The core material used in the tests was coarse and the berm homogeneous, leading to a notional permeability $P \approx 0.6$, similar to a homogeneous structure according to Van der Meer (1988). Material properties used in the tests are stated in Table 1.

Table 1: Material properties used in the 2013 and the present 2014 tests.

	2013 tests		2014 tests	
	Rock armour	Core	Rock armour	Core
Median weight, W_{50} [kg]	0.087	0.007	0.094	0.006
Mass density, ρ_s [kg/m ³]	2,743	2,700	2,654	2,713
Nominal diameter, $D_{n,50}$ [m]	0.032	0.014	0.033	0.013
Gradation, $f_g = D_{n,85}/D_{n,15}$ [-]	1.16	1.39	1.43	1.38

TEST PROGRAMME AND TEST PROCEDURE

The viscous scale effects are often determined by use of the Reynolds number, Re , given in Eq. (1).

$$Re = \frac{\sqrt{g H_{m0}} D_{n,50}}{\nu} \quad (1)$$

Where ν is the kinematic viscosity, $D_{n,50}$ the characteristic stone diameter and $\sqrt{g H_{m0}}$ is a characteristic velocity.

Dai and Kamel (1969) stated that for conventional rubble mound breakwaters conservative results are obtained when using Reynold numbers lower than the critical value, $Re_{crit} = 3 \cdot 10^4$. Typically the Reynold number is lower for small scale tests with reshaping berm breakwaters as the armour stones are relatively small. For the 2014 tests the Reynold numbers are in the range of $2.5 \cdot 10^4 < Re < 2.7 \cdot 10^4$, and for the 2013 tests $1.8 \cdot 10^4 < Re < 2.7 \cdot 10^4$. As these values are smaller than the recommended value by Dai and Kamel (1969) it cannot be ruled out that small viscous scale effects have influenced the results to some extent.

The conditions covered by the tests are stated in Table 2. Each test series was performed by measuring cumulative damage, meaning that the wave height H_{m0} was stepwise increased while the wave steepness was kept constant. Each test consisted of approximately 1,000 waves and after each test series the berm was rebuilt.

Table 2. Range of parameters used in test series.		
	2013 tests	2014 tests
Number of tests	14	24
Front slope, $\cot \alpha$	1.25, 1.5	1.5
Mean wave steepness, S_{0m}	0.015-0.049	0.010-0.016
Relative wave height, H_{m0}/h	0.18-0.37	0.16-0.42
Relative freeboard, A_d/H_{m0}	1.51-3.59	1.23-1.94
Relative berm width, B/H_{m0}	1.59-3.20	1.3-3.76
Relative berm elevation, h_{br}/H_{m0}	0.17-0.75	0.0-1.92
Stability number, $H_0 = H_{m0}/\Delta D_{n,50}$	1.0-2.3	1.4-2.3

WAVE GENERATION AND WAVE ANALYSIS

Generation of waves was based on a JONSWAP spectrum, defined by H_{m0}, f_p ($f_p = 1/T_p$) and the peak enhancement factor γ ($\gamma = 3.3$ for all tests). The generation was done by use of the software package AwaSys 6 (2014), using linear generation by the filtered white noise method. Linear generation was used as active absorption was needed in the tests, which is based on linear theory. All tests were performed with active absorption of reflected waves based on gauges placed at the paddle face and tuned to be effective also for long waves (operation area with low paddle reflection $0.2 \text{ Hz} \leq f \leq 1.2 \text{ Hz}$) (cf. Lykke Andersen et al. (2014)). The system was also effective for very long waves so that seiches was not building up in the flume.

Measurement and separation of incident and reflected waves was done by use of seven resistant type wave gauges placed in front of the structure, cf. Fig. 2.

The method by Zelt and Skjelbreia (1992), based on linear theory, was used to separate incident and reflected waves. Analysis of waves was done by use of the software package WaveLab 3 (2014). To validate the use of the linear separation algorithm a few tests were replicated without the structure in place. For the frequency domain analysis a cut-off was used at $1/3 f_p$ and $3 f_p$. For the time domain analysis no filtering was performed due to the very long waves.

DAMAGE MEASUREMENT

The initial and reshaped profile was measured in a grid spacing of $10 \times 5 \text{ mm}$, by use of a computer controlled non-contact laser profiler, using the software EPro (2014). The profiles were averaged over the width disregarding 20 cm in each side to limit the influence of wall effects. From the measurements the eroded area A_e and damage S_d was determined.

The set-up of the profiler is shown in Fig. 5.



Figure 5: Set-up of erosion profiler.

The present paper only investigates damage on the berm and the lower front slope. In the tests damage up to $S_d \approx 9$ was measured.

STABILITY OF HARDLY RESHAPING BERM BREAKWATERS

The most common method of stability assessment of berm breakwaters has previously been to describe it by the recession, Rec . It has however been stated that for hardly and partly reshaping berm breakwaters using the method for conventional rock slopes might be an additional method. A commonly used method for determining stability of conventional non-overtopped rubble mound breakwaters was presented by Van der Meer (1988). In the present work the significant wave height $H_{1/3}$ has been substituted with $H_{2\%}/1.4$, where $H_{2\%}$ is the wave height exceeded by 2% of the waves. This is done as many of the new tests were conducted with non-Rayleigh distributed waves.

The stability formulae are defines as:

Plunging waves ($\xi_{0m} < \xi_{0m,cr}$):

$$\frac{H_{2\%}}{\Delta D_{n,50}} = 8.7 P^{0.18} \xi_{0m}^{0.5} \left(\frac{S_d}{\sqrt{N}} \right)^{0.2} \quad (2)$$

Surging waves ($\xi_{0m} > \xi_{0m,cr}$):

$$\frac{H_{2\%}}{\Delta D_{n,50}} = 1.4 P^{-0.13} \sqrt{\cot(\alpha)} \xi_{0m}^P \left(\frac{S_d}{\sqrt{N}} \right)^{0.2} \quad (3)$$

Transition point:

$$\xi_{0m,cr} = \left(6.2 P^{0.31} \sqrt{\tan(\alpha)} \right)^{\frac{1}{P+0.5}} \quad (4)$$

Where $\Delta = \rho_{armour} / \rho_{water} - 1$ is the relative mass density, N is the number of waves and ξ_{0m} is the deep water surf similarity parameter.

ANALYSIS OF RESULTS

Lykke Andersen et al. (2012) stated that geometrical parameters might influence the damage progression, and this can be seen by investigating the measured profiles of the tests. When having a low berm close to or at SWL the damage tends to progress from above and downward, cf. Fig. 6.

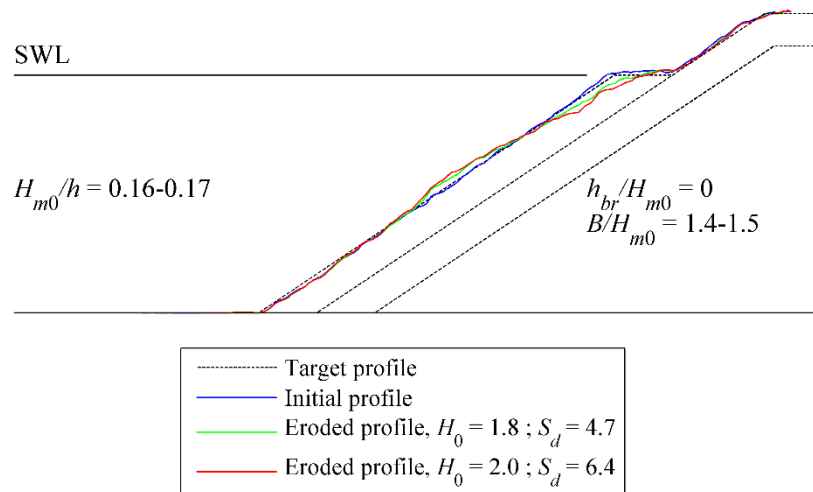


Figure 6: Damage progression with berm at SWL with wave steepness $s_{0m} \approx 0.009$. The stability index is defined as $H_0 = H_{m0}/\Delta D_{n,50}$.

When increasing the berm elevation the damage has a tendency to progress more like a conventional breakwater, hence more into the armour layer instead of downward. The berm was though not high enough to prevent that damage was progressing to the berm level.

The influence of a high berm ($h_{br}/H_{m0} > 0.6$) was tested, showing that the damage progresses into the armour layer as for a conventional rubble mound structure, cf. Fig. 7.

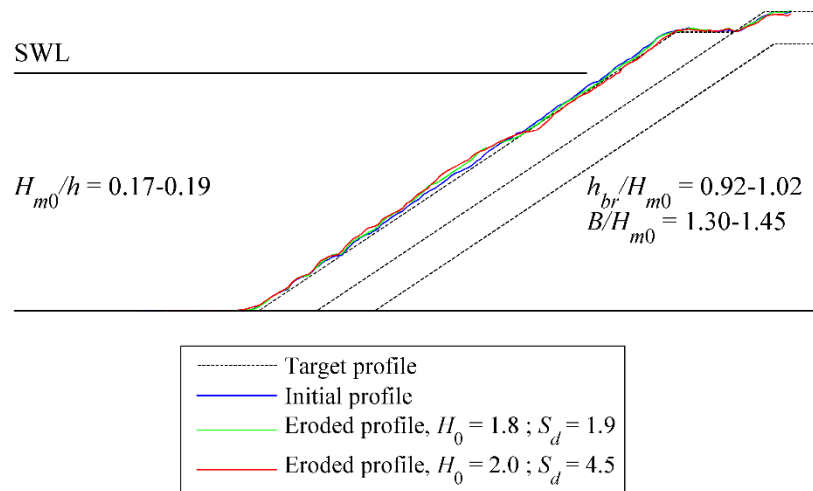


Figure 7: Damage progression for emerged berm with wave steepness $s_{0m} \approx 0.009$. The stability index is defined as $H_0 = H_{m0}/\Delta D_{n,50}$.

The tests clearly showed that by varying the geometrical parameters (berm width/elevation) the stability was influenced. Instead of always following the plunging formula, the stability increased by increasing berm elevation and width. This is illustrated in Fig. 8, where results are plotted by the usual Van der Meer method. For subsequent tests in a series, the cumulative damage was calculated using the Van der Meer (1985) procedure. Here the number of waves corresponding to the calculated damage from the previous test is calculated and then added to the number of waves in the present test.

Fig. 8 shows that increasing the berm elevation leads to an increase in stability. Here both the berm width and berm elevation is of importance. It is observed that when reaching a higher berm (approximately $h_{br}/H_{m0} = 0.85$) the stability has increased significantly.

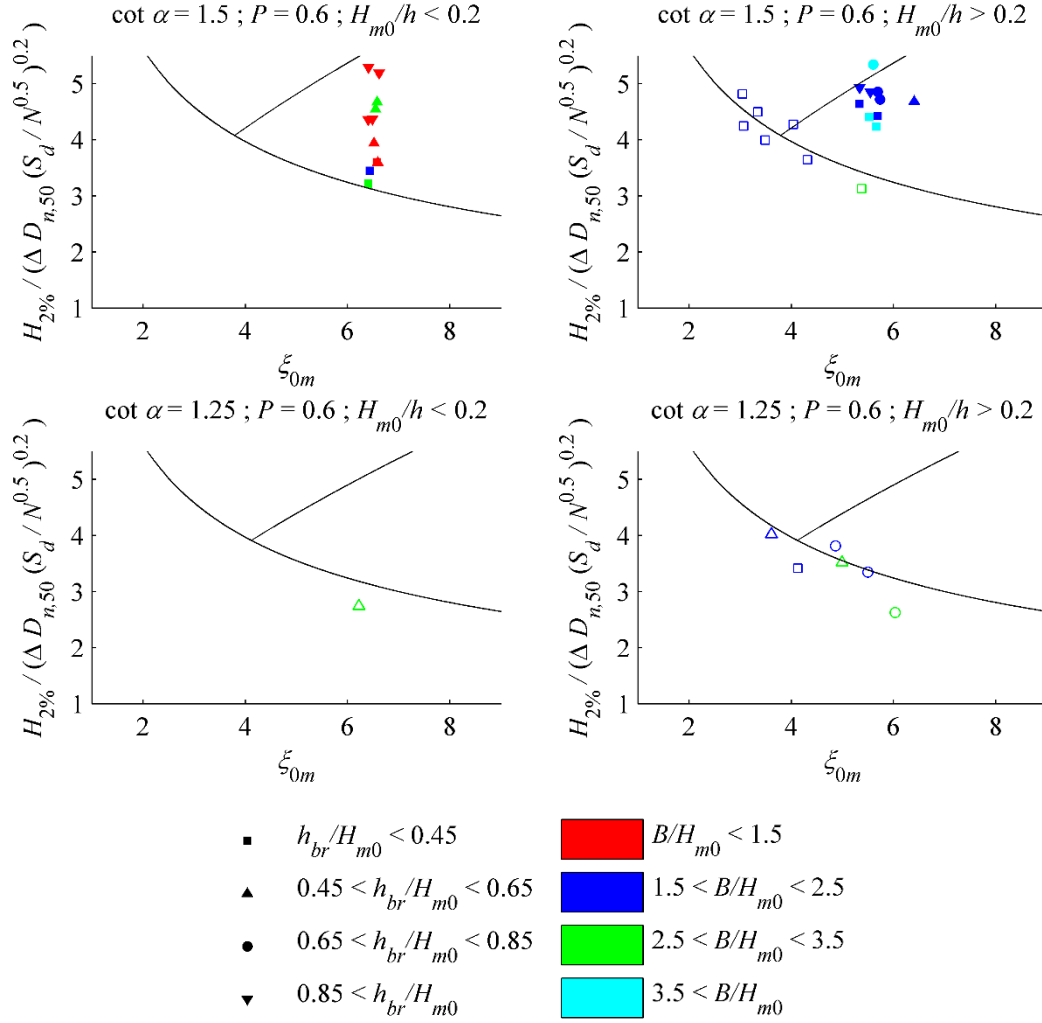


Figure 8: Comparison between the Van der Meer (1988) formula and test results. Filled markers correspond to 2014 tests and open markers to 2013 tests.

STABILITY OF BREAKWATERS WITH A BERM BY VAN GENT (2013)

Van Gent (2013) investigated the rock stability of a two layer conventional rubble mound breakwater with a berm, focussing on both the upper and lower slope. His tests covered slope angles of $\cot \alpha = 2$ and 4, but was mainly conducted with plunging waves. The surging waves used in his tests were so close to the transitions point ($\xi_{m-1,0} = 1.2 - 4.2$), that all was treated as plunging waves. Also it was stated that because of the presence of a berm, the structure could be seen as more gentle, hence the waves would be more plunging and not as surging. This statement supports the conclusion made by Lykke Andersen et al. (2012) that the plunging formula could be used. As the present tests are mainly in surging waves and for steep slopes, the two studies can supplement each other.

From the tests Van Gent (2013) concluded that when the berm is located at SWL the stability always follows the plunging formula (here the modified Van der Meer formulae by Van Gent et al. (2003)), and saw that when submerging or emerging the berm, the damage was reduced. He tested a range of relative berm elevations $-1.2 < h_{br}/H_{m0} < 1.4$ and berm widths $0 < B/H_{m0} < 11$. For the emerged berms, he stated that berm elevation, berm width, wave steepness and slope angle was of importance. He gave a reduction factor γ_{berm} (cf. Eq. (5)) to be included in the plunging formula.

$$\gamma_{berm} = 1 - 0.02 \xi_{m-1,0} \frac{B}{H_s} \frac{h_{br}}{H_s} \quad \text{for } h_{br}/H_s \leq 0 \quad (5)$$

The present tests were used to investigate whether the formula of Van Gent (2013) can be applied to berm breakwaters which has much steeper front slopes. However, the present tests are also outside his tested range of surf similarities.

Van Gent (2013) reduction factor, Eq. (5) is applied in the Van der Meer (1988) plunging formula (Eq. (2)) to calculate the damage and plotted by the usual Van der Meer method, cf. Fig. 9.

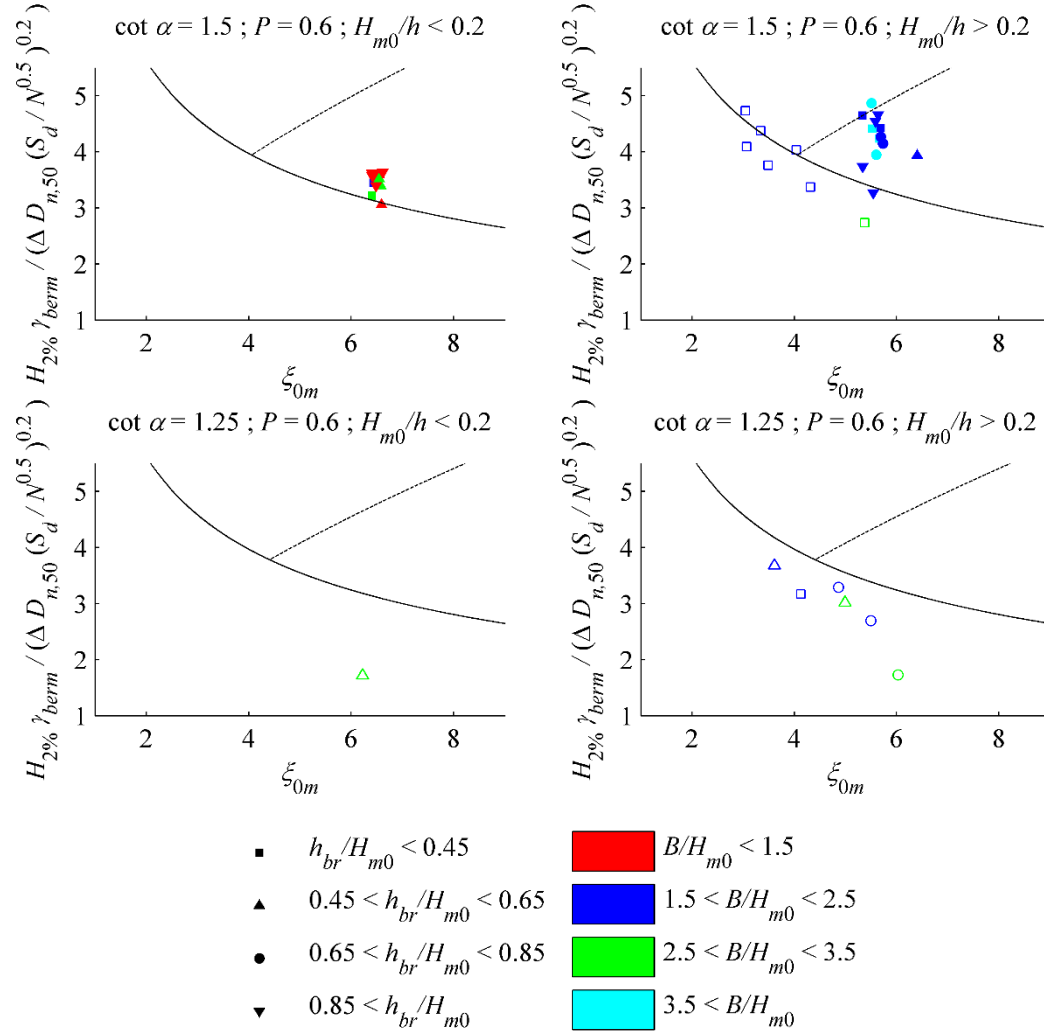


Figure 9: Comparison between Van der Meer (1988) formula together with γ_{berm} (Van Gent (2013)). Filled markers are for 2014 test data and open markers are for 2013 data.

Fig. 9 illustrates that introduction of γ_{berm} brings the results closer to the plunging formula. However, some tests with wide and/or high berm are still close to surging formula. This clearly indicates the influence of berm width and elevation, and proves that at some point, the berm breakwater stability comes closer to behaving as a conventional breakwater. However, tests conducted in shallow water $H_{m0}/h > 0.2$, still results in more scatter than for tests conducted in deep water. This will be investigated further in an ongoing study.

The results for a front slope $\cot \alpha = 1.25$, indicates that the structure is less stable than predicted with γ_{berm} included. For a front slope this steep, the berm might become so unstable that S_d is insufficient to describe the damage, and the recession should be used instead. The present tests are however also outside the validated ranges of Van Gent (2013), and another behaviour might be expected. This subject will be investigated further.

Comparison between measured and calculated damage, (using the Van der Meer (1985) method for accumulated damage) illustrates the influence of γ_{berm} , cf. Fig. 10.

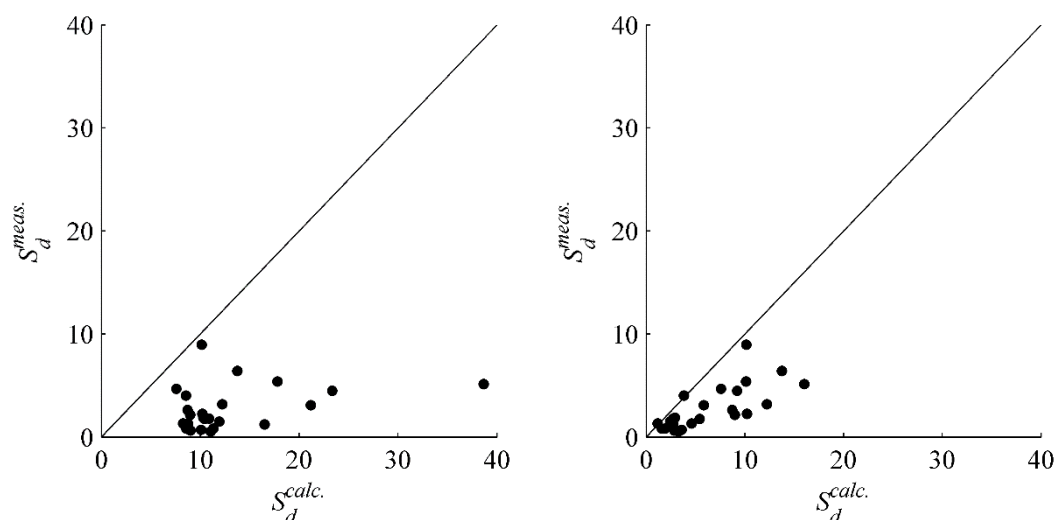


Figure 10: Comparison between measured and calculated damage S_d . Left is by use of Van der Meer (1988) plunging formula. Right is for Van der Meer (1988) plunging formula together with γ_{berm} (Van Gent (2013)). Test data is for $\cot \alpha = 1.5$ only.

Observing Fig. 10 it is seen that better estimates are obtained when applying γ_{berm} but still with a bias. This indicates the importance of geometrical parameters of the berm (berm elevation and width). With the present test, where only a limited number of berm configurations on very few surf similarities was tested, it is not possible to exactly determine when the berm breakwater stability behave as for a conventional breakwater, but the tests clearly indicates that it occurs when adjusting geometrical parameters. Therefore further tests are needed.

CONCLUSION

In the present paper the stability of hardly reshaping berm breakwaters was investigated. Based on new model tests the statement by Lykke Andersen et al. (2012), that the stability follows the plunging formula also for low wave steepness, was proven incorrect when berm elevation and width was increased. The reduction factor γ_{berm} by Van Gent (2013) was tested also outside the ranges of validity (primarily higher surf similarities and steeper front slope) and applied to the Van der Meer (1988) plunging formula. The factor was found to improve the estimation of the damage for some tests, but was insufficient to fully describe the damage. Further tests with a larger range of berm widths, elevation and front slopes are therefore needed.

REFERENCES

- AwaSys 6. 2014. Wave Generation Software, Department of Civil Engineering, Aalborg University, <http://www.hydrosoft.civil.aau.dk/awasys/>.
- Burcharth, H. F. 2012. On Front Slope Stability of Berm Breakwaters. Coastal Engineering, vol 77, no. July, pp. 71-76.
- Dai, Y.B. and Kamel, A.M. 1969. Scale Effect Tests for Rubble Mound Breakwaters. U. S. Army Engineer Waterway Experiment Station, Corps of Engineers, Vicksburg, Mississippi, Research Report H-69-2.
- EPro. 2014. Erosion Profiler Software, Department of Civil Engineering, Aalborg University, <http://www.hydrosoft.civil.aau.dk/epro/>.
- Lykke Andersen, T. and Burcharth, H. F. 2010. A new formula for front slope recession of berm breakwaters. Coastal Engineering, Vol. 57, pp. 359-374.
- Lykke Andersen, T., Van der Meer, J.W., Burcharth, H. F., Sigurdarson, S. 2012. Stability of Hardly Reshaping Berm Breakwaters. ICCE 2012: 33rd International Conference on Coastal Engineering. American Society of Civil Engineers.
- Lykke Andersen, T., Clavero, M., Frigaard, P., Losada, M., Puyol, J.I. 2014. Active Absorption of highly non-linear waves. Submitted to Coastal Engineering for review.

- Moghim, M.N., Shafieefar, M., Tørum, A., Chegini, V. 2011. A new formula for the sea state and structural parameters influencing the stability of homogeneous reshaping berm breakwaters. *Coastal Engineering* (2011), pp. 706-721. Elsevier, Amsterdam.
- Shekari, M.R., and Shafieefar, M. 2013. An experimental study on the reshaping of berm breakwaters under irregular wave attacks, *Applied Ocean Research*, Volume 42, August 2013, Pages 16-23, ISSN 0141-1187.
- Sigurdarson, S. and van der Meer, J., 2012. Wave Overtopping at Berm Breakwaters in line with EurOtop. ASCE, Proceedings of ICCE 2012, Shanghai.
- Sigurdarson, S. and J.W. van der Meer. 2013. Design of berm breakwaters, recession, overtopping and reflection. *Proc. ICE, Coasts, Marine Structures and Breakwaters 2013*, Edinburgh, UK.
- Tørum, A., Krogh, S.R. 2000. Berm breakwaters. Stone quality. SINTEF Report No. STF22 A00207 to the Norwegian Coast Directorate, July 2000.
- Van Gent, M.R.A., Smale, A.J. and Kuiper, C. 2003. Stability of rock slopes with shallow foreshores, ASCE, *Proc. Coastal Structures 2003*, Portland.
- Van Gent, M.R.A. 2013. Rock Stability of Rubble MOUND Breakwaters with a Berm, *Coastal Engineering*, Volume 78, August 2013, Pages 35-45, ISSN 0378-3839
- Van der Meer, J.W. 1985. Stability of revetments and breakwaters under random wave attack. *Breakwaters; Design and Construction*, Thomas Telford, *Proc. Breakwaters '85 Conference*, London.
- Van der Meer, J.W. 1988. Rock Slopes and Gravel Beaches under wave Attack. PhD-thesis Delft University of Technology.
- Van der Meer, J.W. 1992. Stability of the seaward slope of berm breakwaters. *Journal of Coastal Engineering*, 16 (1992), pp. 205-234. Elsevier, Amsterdam.
- WaveLab 3. 2014. Wave Data Acquisition and Analysis Software, Department of Civil Engineering, Aalborg University, <http://www.hydrosoft.civil.aau.dk/wavelab/>
- Zelt, J. A., and Skjelbreia, J. E. 1992. Estimating incident and reflected wave fields using an arbitrary number of wave gauges. *Coastal Engineering Proceedings*, 1(23).