Design of Berm Breakwaters: Recession, Overtopping and Reflection

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Summary

The front slope stability of the berm breakwater has often been assessed through the recession parameter, Rec. A large number of stability tests on berm breakwaters has been analysed and a main conclusion is that the effect of the wave height is far more important than the wave period. Therefore it is proposed to consider only the stability number and a prediction formula has been given. It is a simple design formula, but adaptations should be made, depending on some other design parameters. These are the seaward slope angle, the level of the berm and the depth of the toe below the still water level. These adaptations have been given as a guideline for design.

The paper presents also the development of a new overtopping formula for berm breakwaters. Overtopping data from hydraulic model tests of berm breakwaters have been gathered and reanalysed in line with the procedure in the EurOtop Manual (2007). The data shows a clear dependency on wave period or wave steepness, which is in contrast to the main conclusion of the CLASH project and the EurOtop Manual for conventional rubble mound breakwaters. Overtopping also depends on whether the berm breakwater has reached fully reshaped condition or only partly or hardly reshaped condition.

Similar to overtopping also tests on wave reflection from berm breakwaters have been gathered and analysed. Again a division has to be made between fully reshaping berm breakwaters and hardly or partly reshaping berm breakwaters.

Introduction

The Icelandic-type berm breakwater concept has been in development over the past 30 years and nearly 40 such structures have been constructed worldwide over a wide range of wave climates, water depths and tidal conditions. The paper focuses on two main aspects in the design of berm breakwaters, the front slope stability and overtopping, as well as reflection from berm breakwaters. A new classification of berm breakwaters is introduced that distinguishes between mass armoured berm breakwaters and the Icelandic-type berm breakwater.

New Classification of Berm Breakwaters

A new classification of berm breakwaters was presented in Sigurdarson and van der Meer (2012) which is a modification and update of the PIANC (2003) classification. Firstly the classification distinguishes between different types of berm breakwaters, mass armoured berm breakwaters, MA, with a homogeneous berm of mainly one stone class, and Icelandic-type berm breakwaters, IC, constructed with more rock classes, Figure 1.

The behaviour of both types will be very different if relatively small rock is used for the mass armoured berm breakwater and very large rock for the Icelandic-type berm breakwater. The first one may fully reshape, where the second one may show static stability without significant reshaping. But it is also possible that similar rock classes are used and where both types may show partly reshaping. The type of breakwater does not always give similar behaviour and therefore this behaviour, the recession of the berm, is a part of the classification.

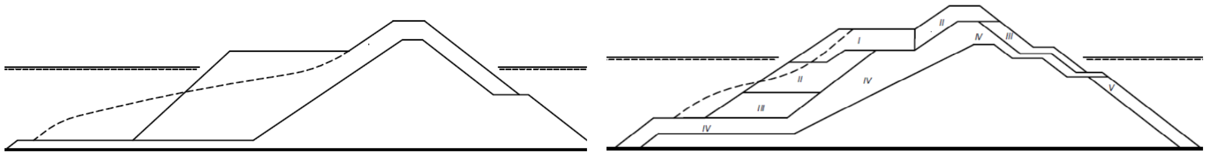

Figure 1. Mass armoured berm breakwater to the left, MA, and Icelandic-type berm breakwater to the right; IC. On both cross sections the expected reshaping is shown with a dotted line.

Secondly the classification takes into account the structural behaviour of berm breakwaters. Three degrees of reshaping are defined:

- Hardly reshaping HR
- Partly reshaping PR
- Fully reshaping FR

Both the two types of berm breakwaters and the different structural behaviour lead to a classification with four typical types of berm breakwaters:

- Hardly reshaping Icelandic-type berm breakwater HR-IC
- Partly reshaping Icelandic-type berm breakwater PR-IC
- Partly reshaping mass armoured berm breakwater PR-MA
- Fully reshaping berm breakwater (mass armoured) FR-MA

Table 1 shows the new classification for berm breakwaters, including indicative values for the stability number, $H_s/\Delta D_{n50}$, the damage and the recession. These values are given for a 100-years wave condition. For wave conditions with smaller return periods the values will be smaller and consequently, for more severe wave conditions, like overload tests, the values may be larger.

Table 1. Classification of berm breakwaters based on 100-years wave condition.

	Abbreviation	$H_s/\Delta D_{n50}$	S_d	Rec/D_{n50}
Hardly reshaping Icelandic-type berm breakwater	HR-IC	1.7 - 2.0	2 - 8	0.5 - 2
Partly reshaping Icelandic-type berm breakwater	PR-IC	2.0 - 2.5	10 - 20	1 - 5
Partly reshaping mass armoured berm breakwater	PR-MA	2.0 - 2.5	10 - 20	1 - 5
Reshaping mass armoured berm breakwater	FR-MA	2.5 - 3.0	--	3 - 10

Front Slope Stability of Berm Breakwaters

PIANC (2003) presented recession data of many research projects, mainly with traditional reshaping berm breakwaters as well as partly Icelandic-type berm breakwaters. Most of the data represent a recession larger than $5*D_{n50}$ and a stability parameter H_oT_{om} larger than 70. A large scatter is present due to various influences. Some of them would be the definition of wave height (at the toe or more at deep water), placement of rock (dumped or carefully placed), way of measuring recession, seaward slope angle, etc.

Sigurdarson et al (2008) defined requirements for reliable data representing the Icelandic-type berm breakwater and indentified three data sets which fulfilled these requirements and is really focussed around small recession. They found that the best fit for the recession data was obtained using the parameter H_oT_{op}, which includes the peak period, T_p, instead of the mean period, T_m. But the difference with using only the stability number $H_o = H_s/\Delta D_{n50}$, so not considering the wave period, was not large.

A statically stable design of an Icelandic-type berm breakwater has been tested in a wave flume at HR Wallingford, where the behaviour from the start of moving of the first stones, up to a few times an overload condition, was measured very precisely, Sigurdarson and van der Meer (2011). At the start of damage the recession of the berm profile varied considerably along the profile. But when the damage became larger and the berm really reshaped into the well-know S-profile, the recession becomes more. In that case it is sufficient to measure only a few profiles, average them and measure the recession, the horizontal retreat of the berm, given in nominal diameter of the armour stone protecting the berm, D_{n50}. This has often been done in berm breakwater research, also for the less reshaping Icelandic-type berm breakwater. In the research at HR Wallingford a very accurate laser profiler has been used and according to the method in Van der Meer (1988) ten profiles have been averaged to give a good description of the behaviour of the structure.

Figure 2 shows the average recession versus the stability number $H_s/\Delta D_{n50}$ for the new dataset.

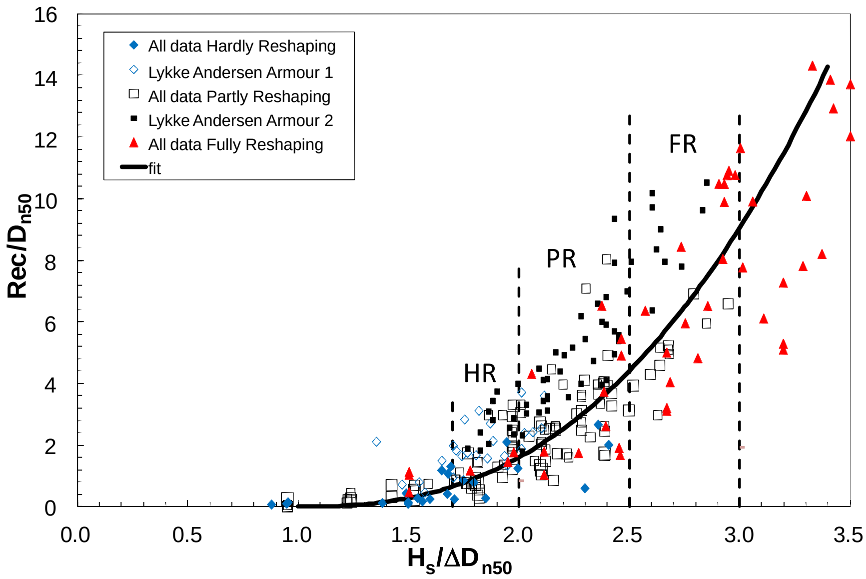

Figure 2. Average recession Rec_{av} versus $H_s/\Delta D_{n50}$, with the new prediction formula.

Based on this new data set for berm breakwaters a new prediction formula for berm recession is presented, which uses the stability number $H_s/\Delta D_{n50}$ and not the dimensionless wave height-wave period parameter H_oT_{op}:

$$Rec_{av}/D_{n50} = 1.6\,(H_s/\Delta D_{n50} - 1.0)^{2.5}$$

with: $$Rec_{av}/D_{n50} = 0 \text{ for } H_s/\Delta D_{n50} < 1.0 \qquad (1)$$

The graph shows also the classification of hardly reshaping (HR), partly reshaping (PR) and fully reshaping (FR). The formula shows that for a statically stable Icelandic-type berm breakwater with a design value of $H_s/\Delta D_{n50}$ = 1.5 the expected recession is not more than about half a stone diameter. For $H_s/\Delta D_{n50}$ = 2.0 this may increase to 1.5 to 3 stone diameters, depending on how accurate the rock above swl has been placed. A fully reshaping mass armoured breakwater may have more than 10 stone diameters reshaping.

Figure 2 shows a large scatter and this has been analysed in depth. The data base and detailed analysis are described in a - for the time being - confidential report, but the intention of the authors is to publish it in the near future. Although the stability number is the most significant parameter to describe the recession of berm breakwaters, other parameters influence the berm recession (which gives the scatter in Figure 2). Here three geometrical parameters are identified that may have substantial influence on berm recession.

Some of them have also been described in the extensive formula of Lykke Andersen (2006). These are, with their dimensionless form:

- Down slope $\cot\alpha_d$
- Berm level d_b/H_s
- Toe depth h_t/H_s

Down Slope

Conventional two layer rock armour structures are normally not designed for slope angles steeper than 1:1.5. Steeper slope angles simply become too unstable and large rock is needed to create a stable steep conventional structure. Same is true for berm breakwaters. Therefore hardly or partly reshaping berm breakwater should not be designed with a steeper down slope than 1.1.5. But in the case of fully reshaping berm breakwater, where large reshaping is expected, steeper slopes of 1:1.25 or even 1:1.1, which are close to the natural angle of response of the rock, can be considered. Figure 3 shows steep slope to the left with relative large recession and more gentle slope to the right with less recession, but the reshaped profile is the same.

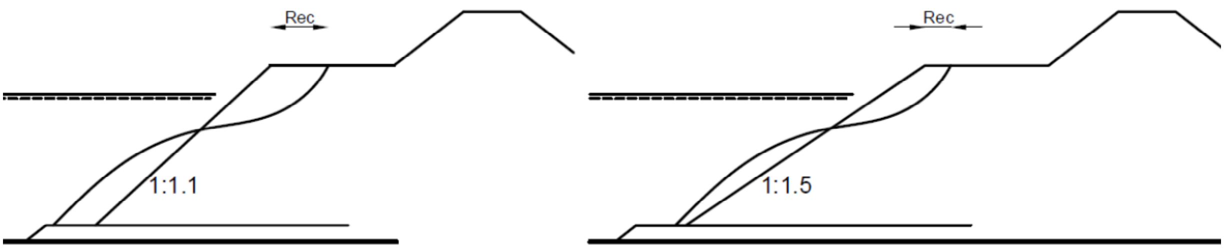

Figure 3. The influence slope angle on recession for reshaping berm breakwaters.

Berm Level

A low berm level may give larger reshaping. To reach a stable S-shape profile a certain amount of rock is needed. Providing more rock volume above the final reshaped profile results in less recession and vice versa. A low berm is sometimes considered for fully reshaping structures and will then show a larger recession. For hardly and partly reshaping berm breakwaters generally the berm is well above the water level, as the berm acts as an area where up-rushing waves can dissipate their energy and this works better with higher berm. As long as the berm is high enough above the water level the energy dissipation of up-rushing waves will work. But an even higher berm will not have extra influence. This would mean that if the berm is higher than a certain threshold, it will work quite well, if it is under this threshold the recession will increase a little.

Toe Depth

The toe depth, which is defined as the water depth above the lowest part in the as built profile where the eroded rock during the reshaping process can settle, Figure 4, may have direct influence on the reshaped profile. A structure with a high and wide toe needs less recession to build up a stable S-shaped profile than a structure with a low and/or a narrow toe berm.

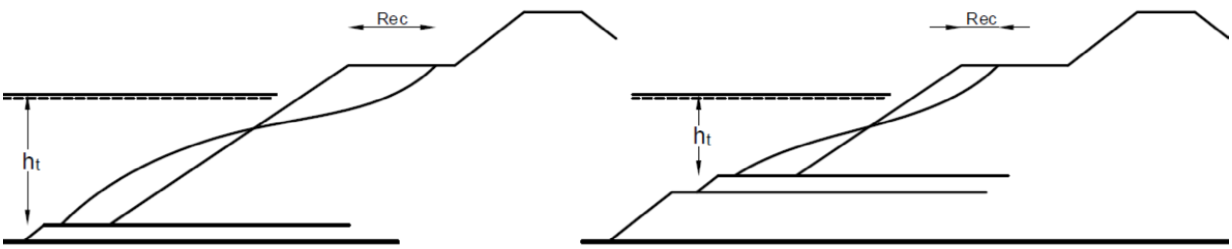

Figure 4. The influence toe depth on recession for reshaping berm breakwaters.

Based on the geometrical data of the structures in the data set, the location of the data points in the recession graph compared to the recession formula and the classification of the berm breakwater, a kind of “average” geometry has been assumed. Deviation from this geometry may then lead to a negative score, if assumed less stable, or a positive score, if assumed more stable. This has resulted in the scoring system of Table 2.

Table 2. Berm breakwaters geometrical dimension scoring system.

	Preferred dimension	Alternative and score	
Hardly reshaping berm breakwaters HR			
Down slope	$\cot\alpha_d = 1.5$	$\cot\alpha_d < 1.5$	÷
Berm level	$d_b/H_s > 0.6$	$d_b/H_s \leq 0.6$	÷
Toe depth	h_t/H_s: no influence (hardly any reshaping)		
Partly reshaping berm breakwaters PR			
Down slope	$\cot\alpha_d = 1.5$	$\cot\alpha_d < 1.5$	÷
Berm level	$d_b/H_s > 0.6$	$d_b/H_s \leq 0.6$	÷
Toe depth	$2.0 < h_t/H_s \leq 2.5$	$h_t/H_s > 2.5$	÷
		$1.6 < h_t/H_s \leq 2.0$	+
		$h_t/H_s \leq 1.6$	++
Fully reshaping berm breakwaters FR			
Down slope	$\cot\alpha_d = 1.25/1.33$	$\cot\alpha_d < 1.25$	÷
		$\cot\alpha_d > 1.33$	+
Berm level	$d_b/H_s > 0.6$	$d_b/H_s \leq 0.6$	÷
Toe depth	$2.0 < h_t/H_s \leq 2.5$	$h_t/H_s > 2.5$	÷
		$1.6 < h_t/H_s \leq 2.0$	+
		$h_t/H_s \leq 1.6$	++

Above points are of course very useful when designing a berm breakwater. Equation 1 gives the average expected recession, but this may be changed to a more stable breakwater by adapting the design parameters of Table 2 in a positive way.

Wave Overtopping at Berm Breakwaters

The EurOtop Manual (2007) presents an overtopping formula for rubble mound structures, which is independent on wave period or wave steepness. For berm breakwaters the same manual presents the overtopping formula of Lykke Andersen, 2006, which is rather complicated to use. The formula on the other hand shows that wave period has significant influence on wave overtopping.

The EurOtop Manual (2007) gives overtopping formulae for smooth slopes, like dikes, and for rubble mound structures with a straight slope. Both are considered here first as it will give the basis for describing wave overtopping for berm breakwaters. The wave overtopping of "non-breaking" or surging waves on steep slopes can be described by:

$$\frac{q}{\sqrt{g \cdot H_{m0}^3}} = 0.2 \cdot \exp\left(-2.6 \frac{R_C}{H_{m0} \cdot \gamma_f \cdot \gamma_\beta}\right) \qquad (2)$$

where:

q	= mean overtopping discharge per meter structure width	[m^3/s/m]
g	= acceleration due to gravity	[m/s²]
H_{m0}	= estimate of significant wave height from spectral analysis = $4\sqrt{m_0}$	[m]
R_c	= crest freeboard of structure	[m]
γ_f	= influence factor for the permeability and roughness of or on the slope	[-]
γ_β	= influence factor for oblique wave attack	[-]

According to Eq. (2) the dimensionless wave overtopping discharge $q/(gH_{m0}^3)^{0.5}$ is given as an exponential function of the relative crest freeboard R_c/H_{m0} and two influence factors, one for oblique wave attack and one for the influence of permeability or roughness of the slope. Exponential functions show a straight line on a log-linear graph.

A berm breakwater is also a rubble mound breakwater with large roughness and permeability. But it has not a straight slope, but a steep seaward slope with a berm and often a partly or fully reshaped berm. Nevertheless, it may be expected that overtopping data for berm breakwaters will give a similar graph as for structures with straight slope. The influence factor may then be a function of geometry of the berm breakwater or wave conditions. If this influence factor can be found and described in a sophisticated way, Eq. 2 can be used replacing the γ_f factor with γ_{BB}, which is the influence factor for a berm breakwater.

Available Data Sets

Data on overtopping at berm breakwaters have been gathered, partly from research and partly from projects, and reanalysed in line with the procedure in the EurOtop Manual, Sigurdarson and van der Meer (2011). The data has a large variation in wave period or wave steepness and shows a clear dependency on those parameters. The data was grouped according to the new classification of berm breakwaters in Table 1 as these may respond differently with regard to wave overtopping.

Overtopping in a laboratory can be measured very accurately, but the meaning of very small overtopping is not always realistic. Overtopping rates lower than 1 l/s per m are affected by scale effects. Wave overtopping graphs are given in relative form, using $q/(gH_s^3)^{0.5}$ as dimensionless overtopping rate. A value of $q/(gH_s^3)^{0.5} < 10^{-5}$ will mostly give an overtopping rate less than 0.5 l/s per m. This is already a threshold were scale effects play a role. For analysis of overtopping data the focus will therefore be on $q/(gH_s^3)^{0.5} > 10^{-5}$. For smaller values the scatter also increases. In the overtopping graphs the area below the given threshold will be shaded in a way that small overtopping data are still visible, but the focus is on larger overtopping rates rather than the very small ones.

Development of Influence Factor γ_{BB}

Each data set was separately plotted in a graph with relative wave overtopping rate versus relative crest freeboard. The data were compared with Equation (2) for steep slopes, with various values for the influence factor γ_f = 1.0 (smooth slope); 0.6; 0.5; 0.4 and 0.3. For berm breakwaters one could also read $\gamma_f = \gamma_{BB}$, that is replacing the influence factor γ_f with a berm breakwater influence factor γ_{BB}.

It is known that the wave period has influence on overtopping at berm breakwaters, this in contrast to steep smooth slopes and also to conventional breakwaters with a straight and steep slope. The reason may be the berm itself, which is very permeable and is most effective for dissipation of energy of short waves. For this reason each test on overtopping was classified into a wave steepness range, given by s_{op} = 0.005 - 0.01; 0.01 - 0.02; 0.02 - 0.03; 0.03 - 0.04; 0.04 - 0.05 and 0.05 - 0.06. Here only two graphs are taken as an example, the data of Keilisnes and Armour 3 of Lykke Andersen (2006).

Figure 5 shows the overtopping data of Keilisnes, a partly reshaping Icelandic-type berm breakwater, Sigurdarson and Viggosson (1994). Although half of the data lies below the threshold, it is very clear that lower wave steepness (larger wave periods) give larger wave overtopping.

With the data of Armour 3 of Lykke Andersen (2006) in Figure 6 there is again a clear influence of the wave steepness. A large scatter is present for steepness s_{op} = 0.03 - 0.04, where a large number of tests were performed with different berm widths, crest heights and crest widths. The structure was a fully reshaping mass armoured berm breakwater FR MA.

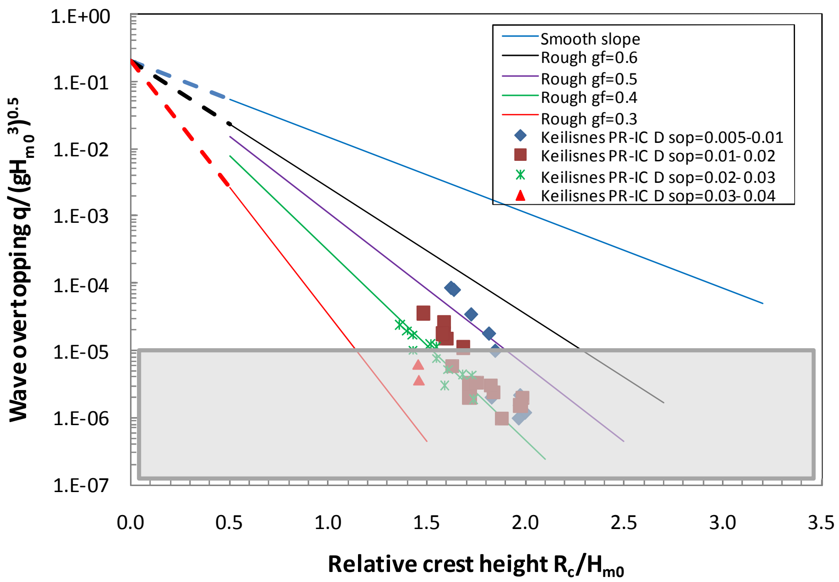

Figure 5. Wave overtopping for Keilisnes, PR IC.

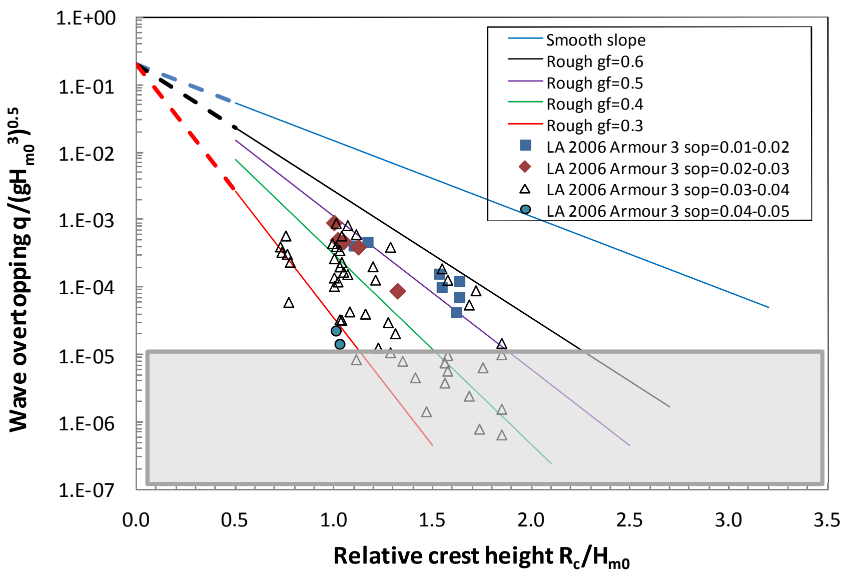

Figure 6. Wave overtopping for Lykke Andersen (2006), Armour 3, FR MA.

Figure 7 and Figure 8 give then the final result with all data combined into these two graphs, one for hardly and partly reshaping berm breakwaters and one for fully reshaping berm breakwaters. The prediction line is given together with the 5% exceedance curves, using a standard deviation of $\sigma = 0.35$, which was the standard deviation for steep smooth and rough slopes on the coefficient 2.6 in overtopping formula Eq. 2. The fast majority of the data lies indeed between these two exceedance curves. Project 5 may be the main outlier with half of the data outside the confidence band.

It means that with the influence factors derived in this section the general overtopping formula for steep smooth and rough slopes can also represent overtopping of berm breakwaters with similar reliability. Summarizing, overtopping for berm breakwaters can be calculated by:

$$\frac{q}{\sqrt{g \cdot H_{m0}^3}} = 0.2 \cdot \exp\left(-2.6 \frac{R_C}{H_{m0} \cdot \gamma_{BB} \cdot \gamma_\beta}\right) \quad (3)$$

with:

$\gamma_{BB} = 0.68 - 4.5s_{op} - 0.05B/H_s$ for HR and PR
$\gamma_{BB} = 0.70 - 9.0s_{op}$ for FR

and B/H_s is given by the design wave height (for a 100-years return period).

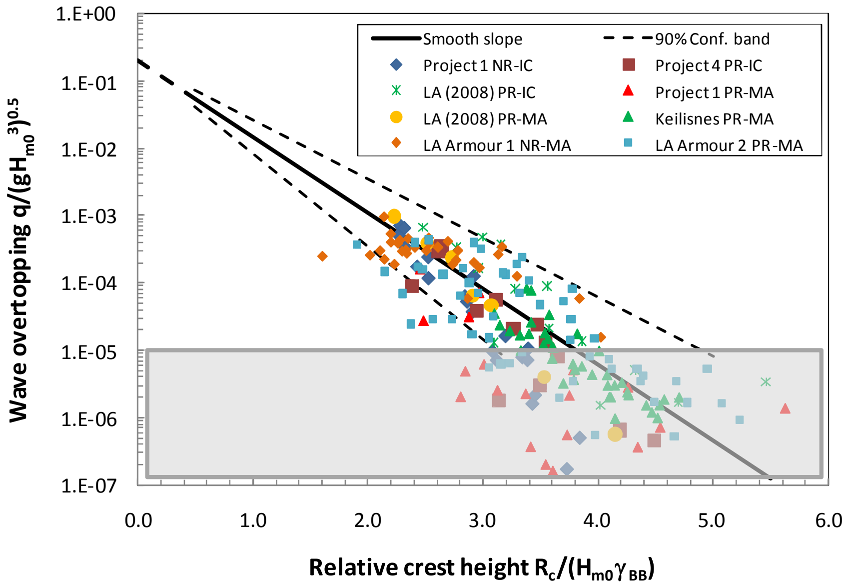

Figure 7. Wave overtopping for hardly and partly reshaping berm breakwaters.

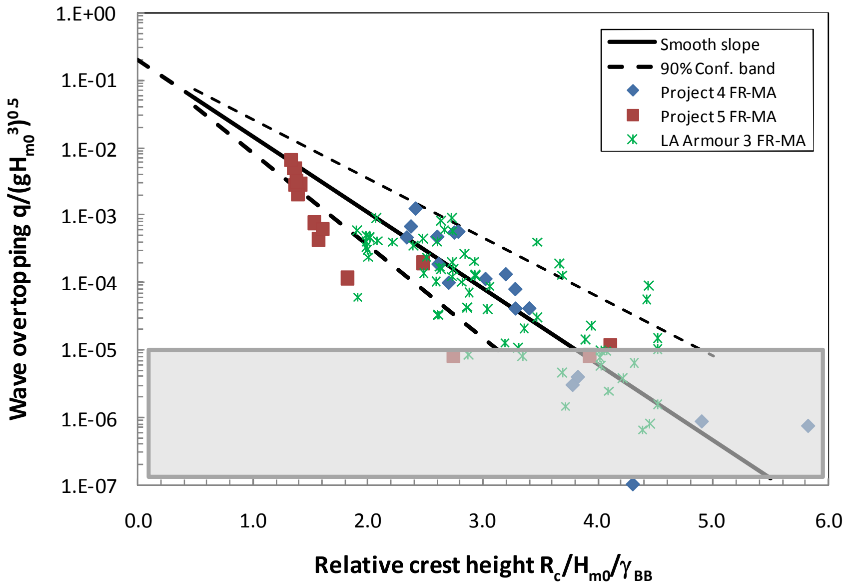

Figure 8. Wave overtopping for fully reshaping berm breakwaters.

For $q/(gH_s^3)^{0.5} < 10^{-5}$ Equation (3) may over-predict the actual wave overtopping. But scale effects may bring the overtopping up again. Mostly a safe design is created if the formulae are also used for $q/(gH_s^3)^{0.5} < 10^{-5}$, but anyhow one should be careful in interpreting model test results for these low values.

Wave Reflection from Berm Breakwaters

Another design aspect is the wave reflection. Large wave reflection may be unwanted as it may hinder ship navigation or increase erosion of adjacent beaches. In general wave reflection from berm breakwaters is fairly low, comparable to or lower than from conventional rock structures.

Data on wave reflection from rubble mound structures in physical model investigation is often available as it is needed in order to establish the incident significant wave height. The reflection coefficient, K_r, is defined as: $K_r = H_{m0,r}/H_{m0,r}$ (with $H_{m0,i}$ = incident significant wave height and $H_{m0,r}$ = reflected significant wave height). Most reflection formulae are given as a function of the breaker parameter $\xi = \tan\alpha/s^{0.5}$, where $\tan\alpha$ = slope angle and s = wave steepness. An overall view of reflection from all kind of rubble mound and smooth structures is given in Zanuttigh and Van der Meer (2008).

The problem with berm breakwaters is that first the bermed structure is in fact a composite slope, but this slope also changes more or less during recession. A fully reshaping berm breakwater finally becomes a nice S-shaped profile, which is different than a structure with straight slopes. This means that it is very difficult to follow the conventional method with the breaker parameter as in Zanuttigh and Van der Meer (2008). An alternative method will be given here.

The conventional way of analysis uses the slope angle as well as the wave steepness in the breaker parameter. Data for other breakwaters show that the wave steepness has a significant influence on wave reflection. Given this fact and the problem in establishing an average slope for a reshaping berm breakwater, a first analysis can be done for the wave steepness only. Figure 9 gives an overall view of all available data for statically stable berm breakwaters with design stability numbers $H_s/\Delta D_{n50} < 3.0$

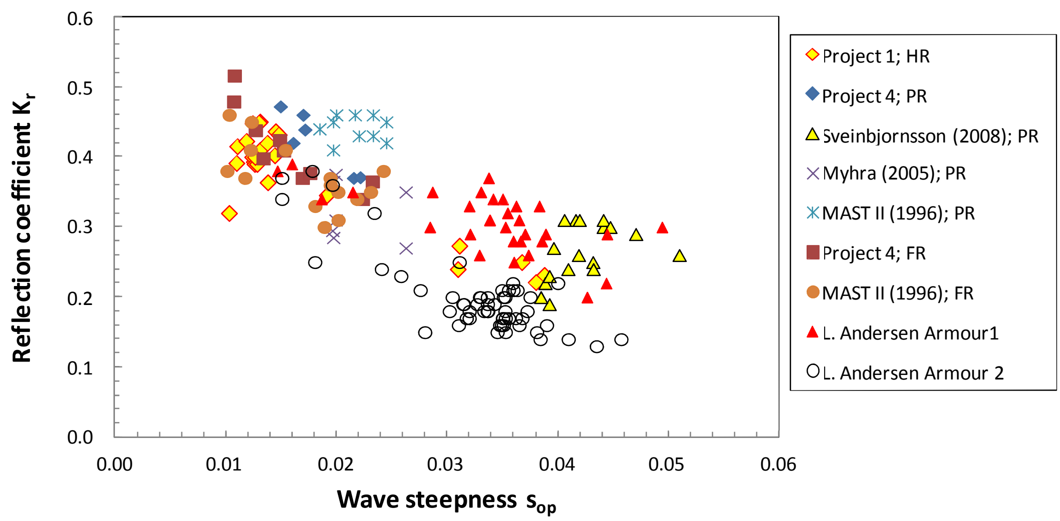

Figure 9. All available data on reflection for statically stable berm breakwaters as function of the wave steepness.

Two conclusions can be drawn from the figure. First that wave reflection decreases with increasing wave steepness, which is according to other breakwaters (see Zanuttigh and Van der Meer 2008)). The second conclusion is that there is quite a lot of scatter if all data are taken together in one graph. Further analysis showed that it was necessary to divide the data in hardly/partly reshaping berm breakwaters and fully reshaping berm breakwaters.

By taking the transition between partly and fully reshaping berm breakwaters around Rec/D_{n50} = 4-5 two graphs have been made, Figure 10 and Figure 11, with the two categories of berm breakwaters.

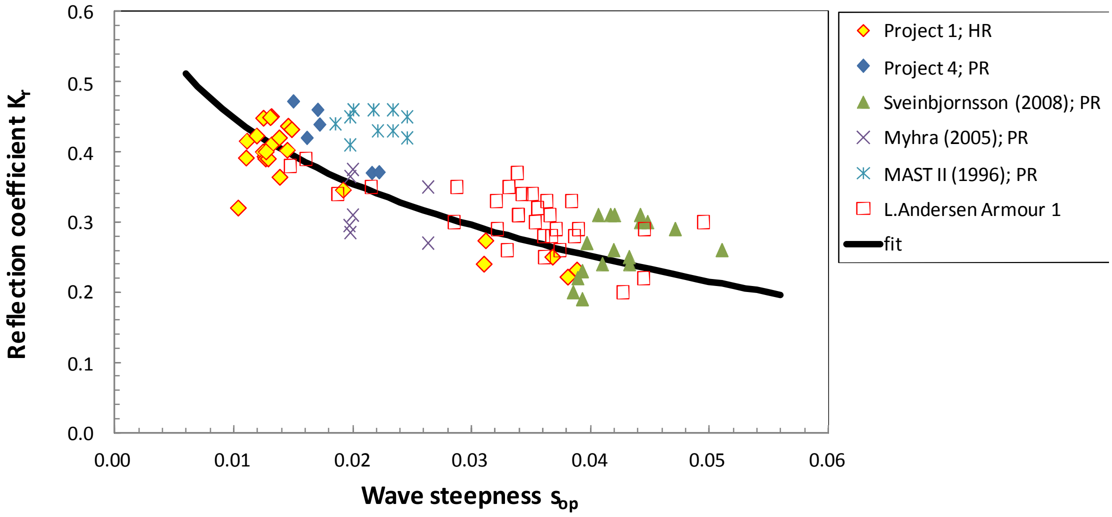

Figure 10. Data on reflection for hardly and partly reshaping berm breakwaters as function of the wave steepness.

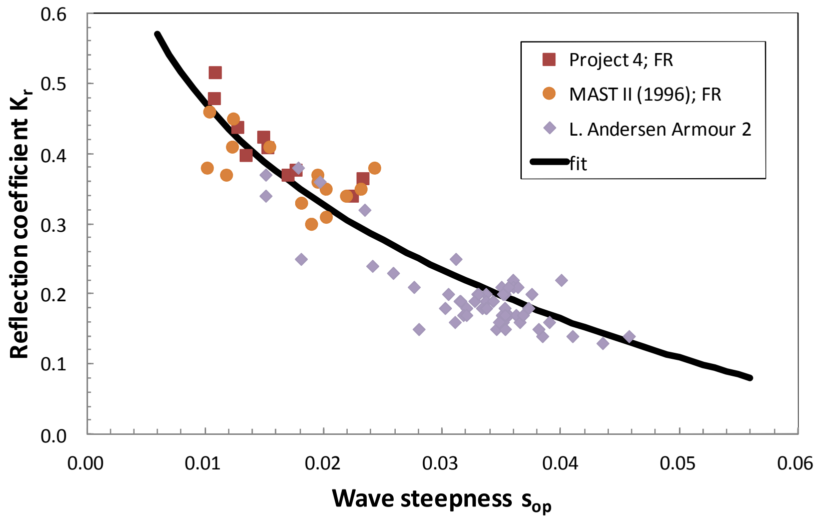

Figure 11. Data on reflection for fully reshaping berm breakwaters as function of the wave steepness.

Both figures show much less scatter than in the overall picture in Figure 9 and a nice trend as function of the wave steepness. Part of the scatter in Figure 9 could probably be explained by the different "as built" down slopes, where a steep slope of 1:1 or 1:1.25 (MAST II and Lykke Andersen data) give slightly more reflection than a slope of 1:1.5 (Project 1). But as a first estimate the trends in Figure 10 and Figure 11 are good enough for a prediction.

Wave reflection for berm breakwaters can now be calculated by the following prediction formulae:

$K_r = 1.3 - 1.7 s_{op}^{0.15}$	HR and PR	($H_s/\Delta D_{n50\ design}$ <2.5 or Rec/D_{n50} < 4-5)	(4)
$K_r = 1.8 - 2.65 s_{op}^{0.15}$	FR	($H_s/\Delta D_{n50\ design}$ >2.5 or Rec/D_{n50} > 4-5)	(5)

Conclusions

A new prediction formula has been developed for recession for the Icelandic-type berm breakwater, which uses the stability number $H_s/\Delta D_{n50}$ and not the dimensionless wave height-wave period parameter H_oT_{op}. Various design measures, like toe depth, berm level and seaward slope angle have influence on the recession of the berm breakwater and design guidance is given.

Data on overtopping at berm breakwaters has been gathered, partly from research and partly from projects, and reanalysed in line with the procedure in the EurOtop Manual. The data has a large variation in wave period or wave steepness and shows a clear dependency on those parameters. A new overtopping formula for berm breakwaters has been developed and roughly validated on prototype performance.

The proposed overtopping formula is based on available results from physical model tests of berm breakwaters. Based on overtopping data with a rather narrow range of wave steepness (only s_{op} = 0.02 and 0.04, not smaller than 0.02) the European CLASH project and the EurOtop Overtopping Manual both concluded that overtopping at conventional rubble mound structures was independent on wave period. Still some dependency was found and with reanalysing of that data it might be able to get better coherence between overtopping of conventional and berm breakwaters, especially if much smaller wave steepness of conventional breakwaters would be considered. The tendency found for berm breakwaters in this paper may also be present, maybe to a lesser extent, for conventional breakwaters with concrete units.

Wave reflection from berm breakwaters is fairly low, comparable to or lower than from conventional rock structures. New reflection formulæ have been developed, one for hardly and partly reshaping berm breakwaters and one for fully reshaping berm breakwaters.

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