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AVERAGE OVERTOPPING DISCHARGE PREDICTION FOR BERM BREAKWATERS

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The present paper deals with overtopping prediction for berm breakwaters in line with the EurOtop methodology. The basis for the paper is the recent advances proposed for EurOtop for conventional breakwaters with respect to the influence of the wave steepness and the crest width. New model tests have been performed to investigate the applicability of these influence factors to berm breakwaters. To cover a white spot in existing data for berm breakwaters, the model tests included wave conditions with very low wave steepness. The results show that the recently developed influence factors for conventional breakwaters also improve predictions for berm breakwaters. Based heron an additional influence factor for the dimensionless berm width is established. The berm width was in previous studies made dimensionless by the wave height, but the present study indicates that the wavelength is more appropriate.

Keywords: overtopping; berm breakwaters; rubble-mound breakwaters; EurOtop

INTRODUCTION

The crest design of breakwaters is often determined by the allowable wave overtopping discharge. The allowable overtopping discharge may be based on the strength of the rear side to overtopping waves as well as functional requirements for the breakwater and hinterland (people or properties). The overtopping discharges can be estimated from empirical methods obtained from hydraulic model tests from laboratories all around the world. Overtopping test data with all kinds of structures were collected during the CLASH project and included in a publicly available database (Verhaeghe et al. (2003)). This database has afterwards been extended by the EurOtop (2018) team. The database includes at present, more than 13,500 wave overtopping tests and has been used to develop an Artificial Neural Netwok for overtopping predicition. A vast amount of data and prediction methods for overtopping on conventional rubble mound breakwaters exist. An example is EurOtop (2018) which includes empirical overtopping prediction formulae covering various types of structures. For rubble mound breakwaters the front slope is often quite steep (1:1.33 to 1:2) and thus the waves are non-breaking at the structure. The EurOtop (2018) mean prediction formula for these structures is given as:

$$\frac{q}{\sqrt{gH_{m0}^3}} = 0.09 \exp\left[-\left(1.5\frac{R_c}{H_{m0}\,\gamma}\right)^{1.3}\right]C_r \tag{1}$$

$$\gamma = \gamma_{fs}\gamma_{\beta} \tag{2}$$

where q is the mean overtopping discharge per unit width, g is the gravity acceleration, H_{m0} the spectral significant wave height at the toe, R_c the crest freeboard, γ is the total influence factor, γ_{fs} the influence factor for roughness and surging waves, γ_{β} the influence factor for wave obliquity and C_r the crest width discharge correction factor as originally suggested by Besley (1999). In the EurOtop (2018) formula the influence factor for roughness and surging waves (γ_{fs}) has an influence of the wave period for breaker parameters (Iribarren numbers) when it is larger than five ($\xi_{m-1,0} = \cot\alpha \,/\sqrt{s_{m-1,0}}$ > 5). The breaker parameter is calculated from the breakwater front slope angle α and the wave steepness using the deep water wavelength calculated with the energy wave period, $s_{m-1,0} = 2\pi H_{m0}/gT_{m-1,0}^2$. The wave parameters at the toe of the structure are used. Moreover, for rubble mound structures with a permeable core EurOtop (2018) applies an upper limit for γ_{fs} of 0.6.

Other studies have suggested that the wave steepness influence starts at lower breaker parameters and with a higher upper limit. Christensen et al. (2014) and Eldrup et al. (2018) both suggest $\xi_{m-1,0} > 1.8$ as the limit for wave steepness influence and an upper limit of 1.0.

Very recently, an upgrade to the EurOtop (2018) formulae focusing on the influence of the wave steepness and the crest width was proposed by Eldrup et al. (2022). They showed that the wave steepness always has an influence, not only for breaker parameters larger than five as assumed in EurOtop (2018). The influence is though much higher at low wave steepness than at higher wave steepness. Moreover,

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they showed that the breaker parameter is not able to describe the influence of the wave steepness and front slope correctly, but instead the influence of wave steepness and front slope should be described individually. They derived a new influence factor (γ_{fs}) that accounts for roughness, front slope and wave steepness. The upper limit of the factor is 1.0 corresponding to a smooth impermeable slope. Moreover, they showed that the crest width reduction may better be included as an influence factor (γ_{cw}) than by the discharge correction factor (C_r) at the end of the formula, as suggested by Besley (1999). The suggested modifications are given in Eqs. 3-6:

$$\frac{q}{\sqrt{gH_{m0}^3}} = 0.09\exp\left[-\left(1.5\frac{R_c}{H_{m0}\gamma}\right)^{1.3}\right] \tag{3}$$

$$\gamma = \gamma_{fs}\gamma_{cw}\gamma_{\beta} \tag{4}$$

$$\gamma_{\mathrm{fs}} = \min\left(\gamma_f + 0.05 s_{m-1,0}^{-0.5} - 0.07\min(\cot(\alpha), 3) - 0.09, 1\right) \tag{5}$$

$$\gamma_{cw} = \min\left(1.1\exp\left(-0.18\frac{G_c}{H_{m0}}\right), 1\right) \tag{6}$$

where γ_f is the roughness factor as provided in EurOtop ($\gamma_f = 0.40$ for rock) and G_c is the crest width.

The improved influence factors by Eldrup et al. (2022) have focused on conventional rubble mound structures. Their performance for more complex or other types of rubble mound structures is thus unknown.

Overtopping on berm breakwaters has not been as extensively investigated as for conventional rubble mound structures. As part of the CLASH project (De Rouck et al. (2009)) white spots in the overtopping database were identified. Three white spots were selected for additional tests and berm breakwaters was one of them. Based heron Lykke Andersen and Burcharth (2005) performed parametric model tests with hydraulic response of berm breakwaters. The tests covered both fully reshaping, partly reshaping and hardly reshaping berm breakwaters exposed to head-on waves. The tests covered peak wave steepness from 1.0% to 5.4%, but most tests had a peak wave steepness around 4%. The long waves included in Eldrup et al. (2022) was thus hardly covered by the tests. Lykke Andersen and Burcharth (2005) presented an overtopping prediction formula based on their data as well as other data. The formula includes many parameters and fitting coefficients and is not following the EurOtop methodology. The formula is provided in Eq. 7. The description of the various parameters are provided in the original paper, but is not repeated here.

$$\frac{q}{\sqrt{gH_{m0}^3}} = 1.79\cdot 10^{-5}\cdot(f_{H0}^{1.34} + 9.22)\cdot s_{0p}^{-2.52} \cdot \exp[-5.63\cdot R_*^{0.92} - 0.61\cdot G_*^{1.39} - 0.55\cdot h_{b*}^{1.48}\cdot B_*^{1.39}] \tag{7}$$

An alternative prediction method has been given by Van der Meer and Sigurdarson (2017) using a γ_{BB} influence factor in the EurOtop methodology. This influence factor is now also part of the EurOtop (2018) manual. It is applied in Eqs. 1-2 and replaces the influence factor for roughness and surging waves (γ_{fs}). Their formula is based on several data sets, including part of the Lykke Andersen (2006) data as well as data from several design projects. They found that the steepness influence was significant for berm breakwaters, and thus this effect was included in γ_{BB} together with the berm width influence. They give the influence factor dependent on if the berm is hardly reshaping (HR), partly reshaping (PR) or fully reshaping (FR), cf. Eq. 9. For definitions of reshaping see Van der Meer and Sigurdarson (2017). Figure 1 shows the data used to fit Eq. 9 for hardly and partly reshaping berm breakwaters and the comparison to Eqs. 8 and 9.

$$\frac{q}{\sqrt{gH_{m0}^3}} = 0.09 \exp\left[-\left(1.5\frac{R_c}{H_{m0}\gamma_{BB}\gamma_\beta}\right)^{1.3}\right] \tag{8}$$

$$\gamma_{BB} = \begin{cases} 0.68 - 4.5s_{0p} - \dfrac{0.05B}{H_{s,d}} & for\ HR,\ PR \\ 0.70 - 9.0s_{0p} & for\ FR \end{cases} \tag{9}$$

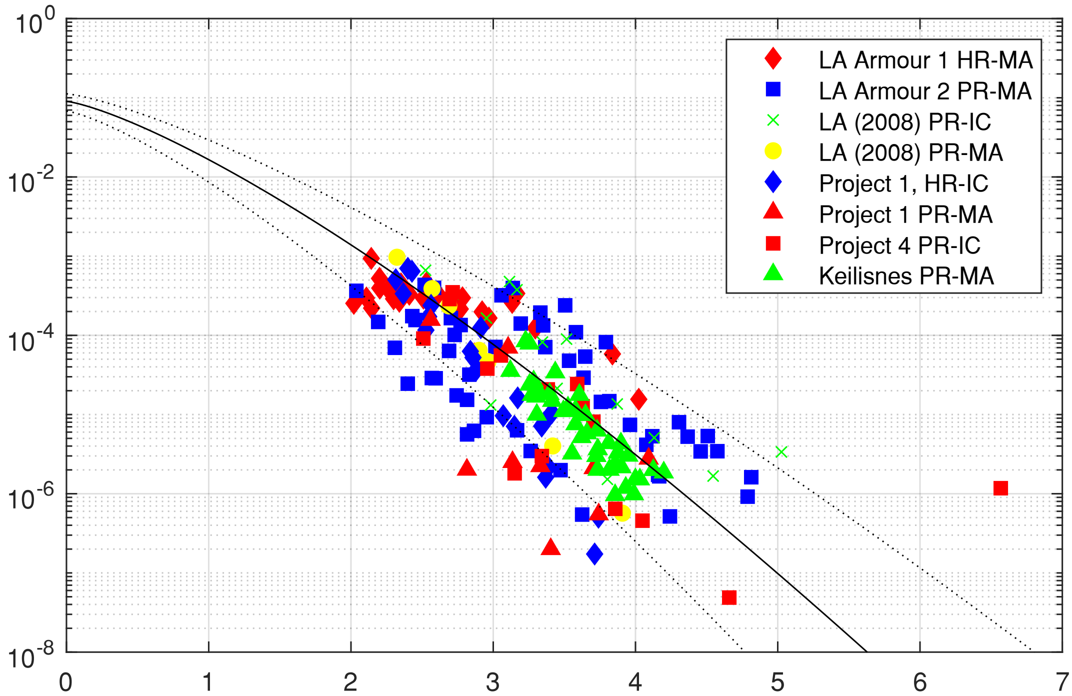

Figure 1. Comparison of Eqs. 8 and 9 with the data for hardly reshaping and partly reshaping berm breakwaters from Lykke Andersen and Burcharth (2005), Lykke Andersen et al. (2008), Project 1 and Project 4 from Van der Meer and Sigurdarson (2017) and Viggosen (1993).

In the present paper the influence factors to be used for hardly and partly reshaping berm breakwaters is re-visited. The reason for the new study is the recent work by Eldrup et al. (2022) on especially the influence factor for roughness and surging waves. The final aim is to develop improved influence factors for overtopping on berm breakwaters by including the factors by Eldrup and an improved influence factor for the berm. The present study is for hardly reshaping berm breakwaters and thus both lower and upper front slope are typical in the range of conventional rubble mound breakwaters and the berm remains largely stable during wave exposure, without significant reshaping. Thus the only difference from the structures described by Eldrup et al. (2022) is the addition of the berm. The assumption is that the influence of the hardly reshaping berm can be described by a new influence factor in addition to those already included by Eldrup et al. (2022). If this is successfully established then the formula becomes applicable for both conventional rubble mound breakwaters and berm breakwaters.

ANALYSIS OF WAVE STEEPNESS INFLUENCE IN EXISTING METHODS

The Van der Meer and Sigurdarson (2017) formula shows that a decreasing wave steepness leads to a larger overtopping discharge and the influence is not only present for large breaker parameters as suggested by EurOtop (2018) for conventional breakwaters. Eldrup et al. (2022) found a similar effect of the wave steepness for conventional rubble mound breakwaters. Therefore, the two approaches are compared in Fig. 2.

The Van der Meer and Sigurdarson (2017) formula for hardly and partly rehaping berm breakwaters is used and shown for various dimensionless berm widths. Even if the cases of no initial berm (B = 0) or very narrow initiall berms have not been included in the fitting of the Van der Meer and Sigurdarson (2017) formula they are included for comparison with the Eldrup et al. (2022) method.

For the conventional rubble mound the Eldrup et al. (2022) formula is shown with two roughness factors. The expected one for rock armour is $\gamma_f = 0.40$, but a better fit with the extrapolated Van der Meer and Sigurdarson (2017) formula for $B = 0$ is obtained for $\gamma_f = 0.45$. The Van der Meer and Sigurdarson (2017) formula is based on data with a peak wave steepness above 1%, which covers the majority of design conditions, but maybe not all. The figure shows that the two formulae give similar influence of the wave steepness in that area. However, the Eldrup et al. (2022) method gives much higher overtopping than the Van der Meer and Sigurdarson (2017) method for a wave steepness lower than the mentioned 1%. Even if the Van der Meer and Sigurdarson (2017) formula has not been validated against conventional rubble mound structures it gives for a hardly reshaping structure without a berm ($B = 0$) and normal wave steepness a prediction similar to Eldrup et al. (2022). Thus, the influence of the wave steepness for berm breakwaters and conventional rubble mound breakwaters might be identical.

A new test programme was established to extend the existing database for hardly reshaping berm breakwaters with tests of lower wave steepness.

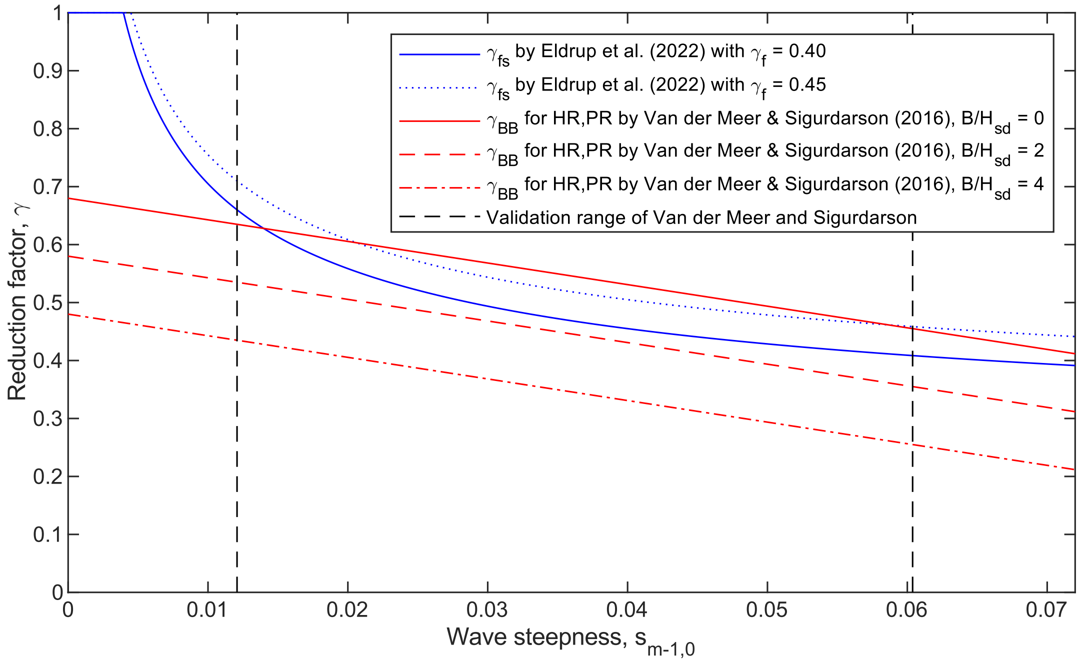

Figure 2. Comparison of Eq. 5 and Eq. 9 for a breakwater with front slope 1:1.5. For Eq. 9 a hardly reshaping structure with three different dimensionless berm widths are considered. The relation between the peak and energy wave steepness is assumed to be $s_{m-1,0} = 1.2\ s_{0p}$.

NEW MODEL TESTS

A model test programme was carried out at Aalborg University. The main purpose was to investigate the overtopping influence factor as a function of wave steepness and berm width. Hardly reshaping (HR) berm breakwaters were tested, and thus the stability number was in all tests kept below two ($H_0 = H_{m0}/\Delta D_{n50} < 2$).

A conventional rubble mound without a berm ($B = 0$) was included as a reference case. To that structure homogenous berms of widths B = 0.1, 0.2, 0.3 and 0.5 m in model scale were added. A homogenous berm is not a typical configuration for hardly reshaping berm breakwaters. However, a study of the influence of the layer composition would require much more tests and thus for this initial study a single armour class was used. It also significantly simplified the construction of the different berm widths tested. It must be expected that the lower permeability of an Icelandic berm breakwater may have some effect on the overtopping. The cross-section had, in all cases, a front slope of 1:1.5. The seabed level was -0.44 m, berm elevation +0.04 m and crest elevation +0.17 m. The water level tested was +0.0 m and an additional water level of +0.082 m was tested on the reference structure only. The cross-sections tested are provided in Fig. 3 in model scale.

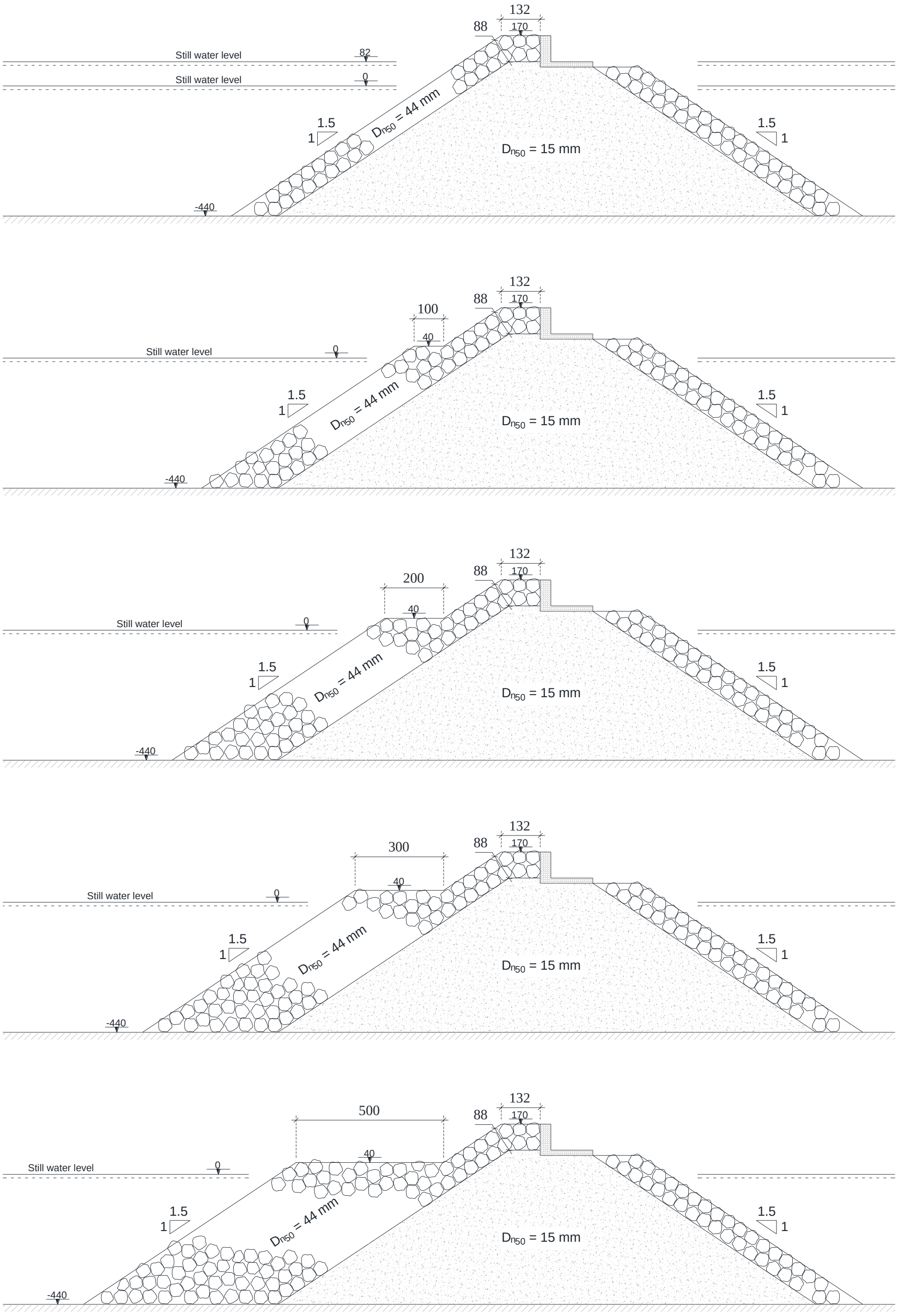

Figure 3. Tested cross-sections. Measurements are in millimetres.

The wave heights tested were 0.08, 0.10, 0.12 and 0.14 m. For the reference structure without a berm, the largest wave height was skipped as it led to too high damage levels. For the structures with a berm, the smallest wave height was skipped as it led to very small overtopping on the reference structure. For every wave height, four wave steepness' were tested corresponding to $s_{m-1,0} = H_{m0} / gT^2_{m-1,0}$ = 0.5%, 1%, 2% and 4.5%. The individual tests contained 3000 waves, ensuring higher reliability for tests with rather low overtopping discharges. For a given sea state, identical wave trains were used for the reference and the four tested berm widths. This eliminates the influence of the natural variability of the wave trains when comparing overtopping as function of berm width. Thus, the data may be used to study the influence of the dimensionless berm width and wave steepness for the given structure. These parameters are the main parameters in the Van der Meer and Sigurdarson (2017) formula for hardly and partly rehaping berm breakwaters. The berm level might also be of importance, but as only a single berm and crest level was tested the influence of the dimensionless berm elevation cannot be studied based on the present data.

The structure was not rebuilt in the course of the test programme. However, as the structure was hardly reshaping, the damage was so low that it is expected to have only a minor influence on the overtopping discharge.

Due to the very long waves in shallow water, the waves were very nonlinear. Thus normal wave generation and wave analysis may lead to unreliable results. Thus, the wave generation was performed following the recommendations of Eldrup and Lykke Andersen (2019a) and using the state-of-the-art software AwaSys. Therefore, only the least nonlinear cases were generated by second order wavemaker theory. The waves in the remaining tests were generated by a combination of numerical and physical modelling, as originally suggested by Zhang et al. (2007). In order to apply that method, a Boussinesq numerical model was used to shoal the waves. The output of that model was used to drive the wavemaker in the physical model using ad-hoc unified wave generation. As waves were nonbreaking on the foreshore, it was possible to use a horizontal foreshore in the physical model. The foreshore in the numerical model was a constant slope of 1:100. This makes it possible to generate highly nonlinear waves on a horizontal bottom with insignificant spurious harmonics. Active absorption on the wave paddle was applied using the method of Lykke Andersen et al. (2016). Lykke Andersen et al. (2018) showed that the method is also highly effective in absorbing nonlinear irregular waves.

The incident waves were estimated from a wave gauge array with six gauges. Separation methods assuming linear waves may lead to very wrong results for the highly nonlinear waves. Therefore, the nonlinear method of Eldrup and Lykke Andersen (2019b) was used. The alternative option would be to calibrate waves without the model in place, but as waves were non-breaking on a horizontal foreshore the estimates by Eldrup and Lykke Andersen (2019b) were assumed to be accurate.

OVERVIEW OF RESULTS AND COMPARISON TO EXISITING METHODS

The new model test results are plotted in the EurOtop (2018) style graph in Fig. 4. It appears that the data corresponds to a total influence factor (γ) of 0.30 to 0.65. It also appears that the main influence is not the berm width as the influence seems low compared to the scatter.

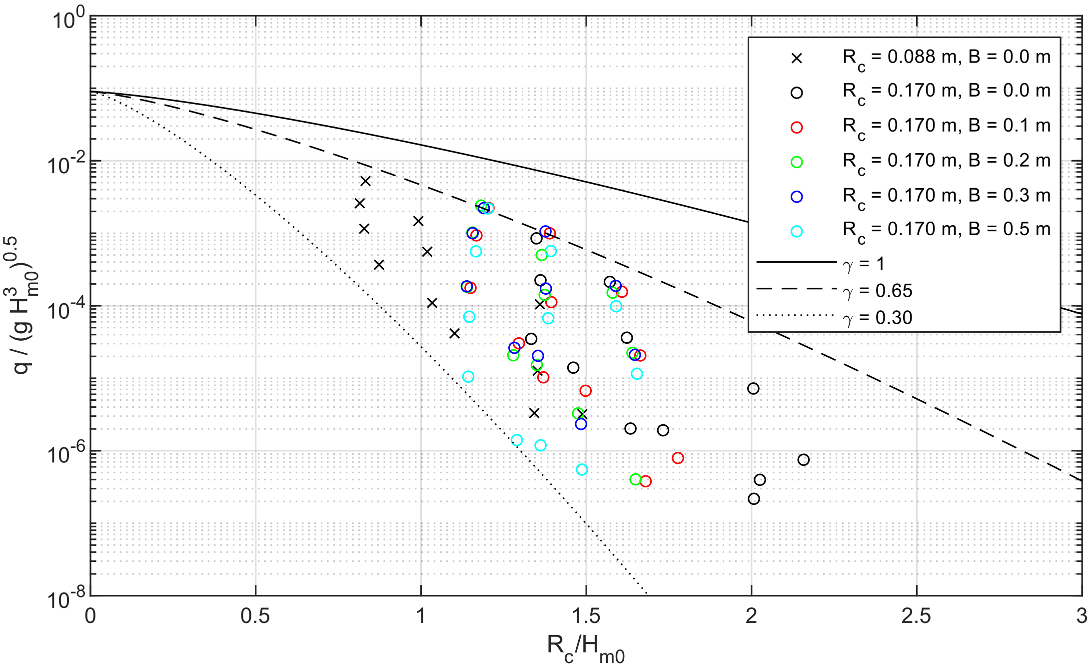

Figure 4. Overview of new data in the dimensionless overtopping plot. Comparison to EurOtop (2018) prediction formula. Eq. 1 is used without crest reduction, i.e. C_r = 1.

Inclusion of the γ_{BB} influence factor by Van der Meer and Sigurdarson (2016) for hardly and partly reshaping berm breakwaters decreases the scatter significantly, as demonstrated by Fig. 5. In their method, the design wave height is taken as $H_{s,D}$ = 0.12 m and thus the tests with the wave height 0.14 m are considered as overload conditions. Most of the data for berm widths of 0.2 m and up are now inside the confidence band. Only the structure without a berm and with a very narrow berm of around two rock diameters (B = 0.1 m) is overpredicted, but this is outside the validation range of their formula.

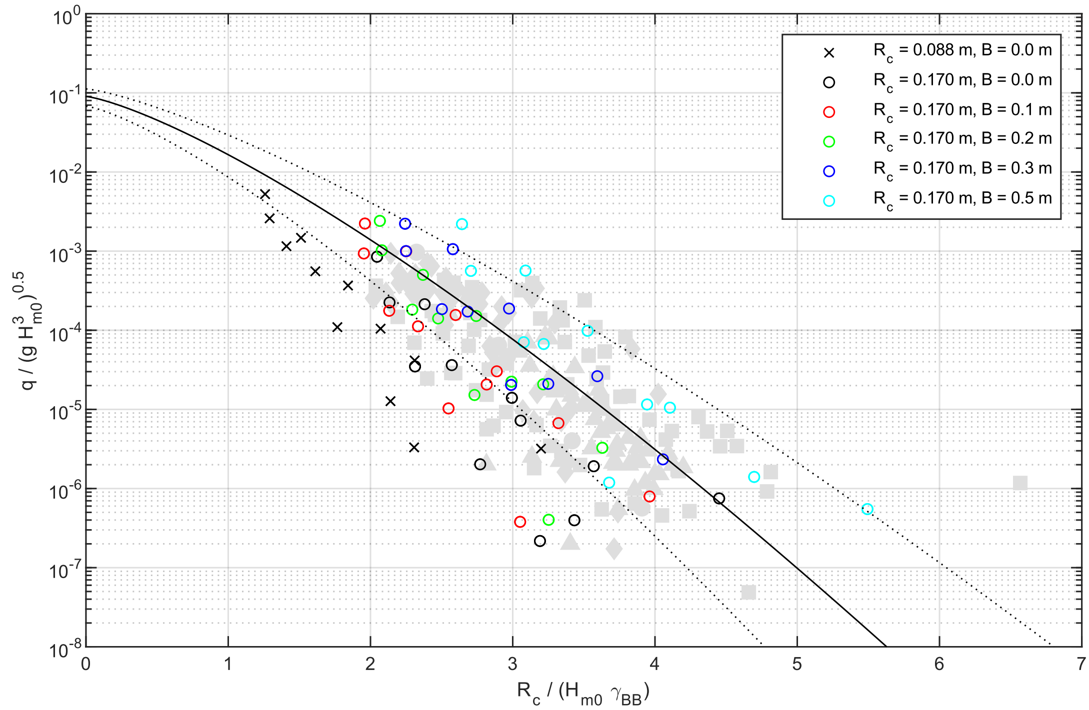

Figure 5. Comparison of present data to Van der Meer and Sigurdarson (2016) predictions according to Eq. 8. 90% confidence band according to EurOtop (2018) is also given. Existing data from Fig. 1 is shown in grey.

The reduced scatter compared to Fig. 4 is mainly caused by the inclusion of the wave steepness influence, which was not included in Fig. 4. However, Eldrup et al. (2022) showed that also, for conventional breakwaters, an influence of the wave steepness is present. They gave this influence in the γ_{fs} influence factor. In Fig. 6 the results are compared to the Eldrup et al. (2022) prediction method, i.e. without considering the effect of the berm.

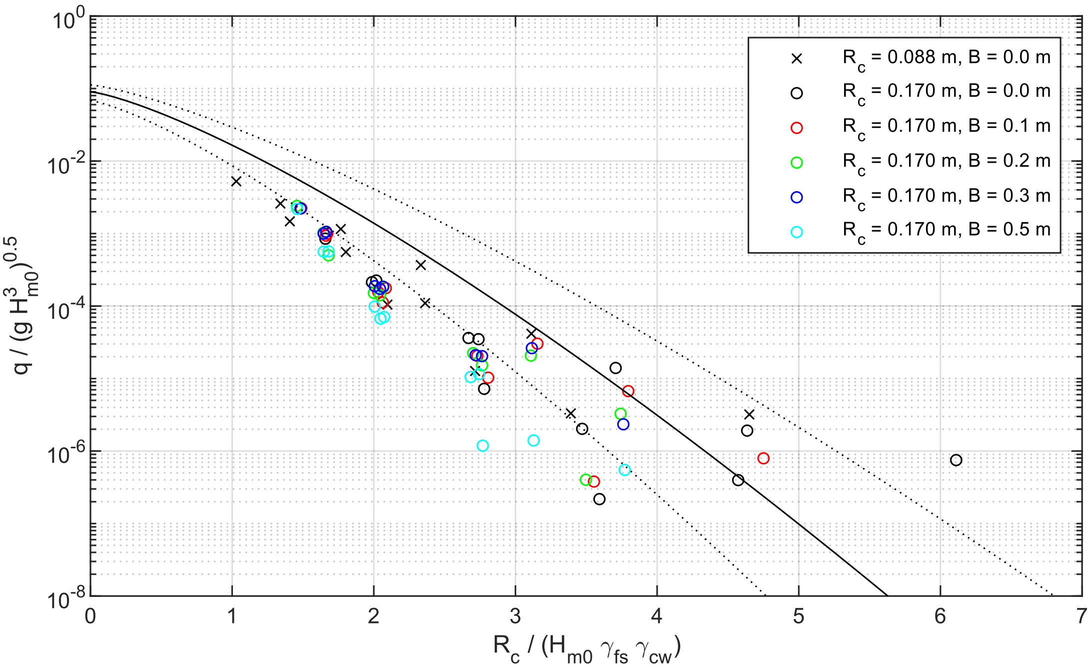

Figure 6. Comparison of present data to Eldrup et al. (2022) method, i.e. Eqs. 3-6 with γ_f = 0.40. 90% confidence band according to EurOtop (2018) also given.

The scatter has been reduced enormously compared to Fig. 4, but the overtopping is in many cases overpredicted even for the reference structure. Anyway, it appears that the influence of the berm is not very significant as most of the test results with a berm are not deviating significantly from the tests without a berm. In order to understand the overprediction, the reference structure is analysed in detail by following the methodology of Eldrup et al. (2022). Afterwards, an influence factor for the berm may be established.

ANALYSIS OF THE REFERENCE STRUCTURE

The effect of the berm width needs to be studied separately from the other effects. Thus, the results from the reference structure are studied in detail following the Eldrup et al. (2022) methodology. For the crest width influence, the γ_{cw} proposal by Eldrup et al. (2022) is utilized directly. However, for the influence of the wave steepness, the present data deviate a bit from the trendline of Eldrup et al. (2022). This is shown in Fig. 7 where γ_{fs} is plotted as function of the wave steepness. Here γ_{fs} is calculated by first finding γ by isolation in Eq. 3 which leads to:

$$\gamma = \frac{1.5R_c}{H_{m0}}\left(-ln\left(\frac{q}{0.09\sqrt{gH_{m0}^3}}\right)\right)^{-1/1.3} \tag{10}$$

Afterwards, γ_{fs} is found from $\gamma = \gamma_{fs}\,\gamma_{cw}\,\gamma_\beta$ with $\gamma_\beta = 1$ and γ_{cw} calculated from Eq. 6. Based heron γ_{fs} may be calculated and plotted as function of the wave steepness. In the figure, the dimensionless overtopping rates $\left(q/\sqrt{gH_{m0}^3}\right)$ below 10^{-6} were not plotted because low overtopping rates may lead to large scatter on the calculated influence factor.

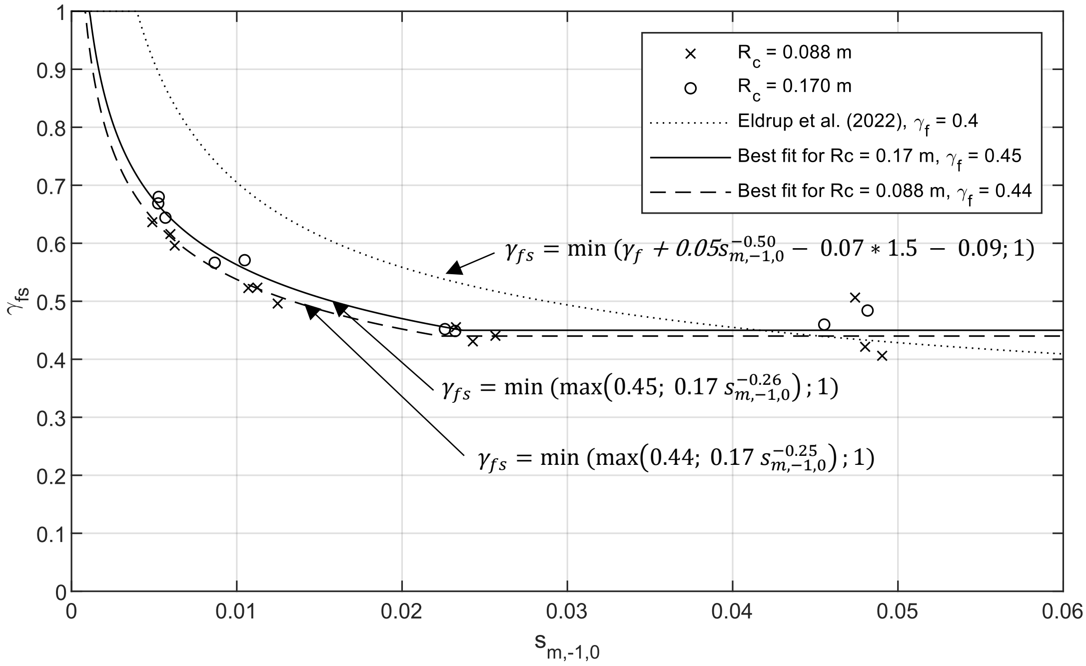

Figure 7. Comparison of results from the reference structure (*B* = 0) to Eq. 5. The crest influence factor given in Eq. 6 is assumed valid. An alternative fit to the present data is also shown and given in Eq. 11.

In order to study the effect of the berm separately from other effects, a curve is fitted for γ_{fs} for the reference structure of the present tests. The fitted curve is given by:

$$\gamma_{fs} = \min\left(\max\left(\gamma_f;\ 0.17\ s_{m,-1,0}^{\alpha}\right); 1\right) \tag{11}$$

where α = -0.26 and γ_f = 0.45 is chosen as best fit parameters for the tests with a freeboard of 0.17 m. Fitting only to this freeboard was chosen because it was the only one tested on the berm breakwaters. However, the other freeboard freeboard tested on the reference structure does not lead to significantly different value. The best fit parameters for the freeboard of 0.088 m is given by α = -0.25 and γ_f = 0.44 as also shown in the figure.

DEVELOPMENT OF NEW BERM INFLUENCE FACTOR

The fitted γ_{fs} function for the reference structure may now be included and used for the structures with a berm. This should be valid as all structures were exposed to exactly the same wave trains. Thus, the total influence factor is assumed to be given by:

$$\gamma = \gamma_{fs}\gamma_{cw}\gamma_{BB}\gamma_{\beta} \tag{12}$$

Note that this definition is different from the one of Van der Meer and Sigurdarson (2016) that included the influence of the wave steepness in γ_{BB} and they did not apply γ_{fs}. Fig. 8 shows the results with this γ influence factor in Eq. 3, but with γ_{BB} = 1 and using the fit in Eq. 11 for γ_{fs}. The scatter has been reduced enormously compared to Figs. 4-6. It appears now that the influence of the berm is not very significant, as most of the test results are already inside the confidence band. Except for the widest berm tested the scatter is already less than with Van der Meer and Sigurdarson (2016) , comparing Figs. 5 and 8. This is even without considering the effect of the berm. However, for the widest berm it is clear that the effect of the berm is significant in some cases with a few points far below the confidence band while for other conditions the effect is not so significant even if the berm is wide. This is to be analysed further in the following in order to establish a berm influence factor. It should though be noted that above results are based on the γ_{fs} fitted to this specific structure. Eldrup et al. (2022) showed quite some scatter on the γ_{fs}. Thus, if the γ_{fs} is not fitted to the specific structure the scatter will be larger. A main issue in reducing scatter on overtopping prediction methods will thus be to better understand the origin of the scatter on γ_{fs}.

The same data may further be plotted with γ as function of the wave steepness, cf. Fig. 9. Here only the crest width influence factor is included. Again, the influence of the narrow berms is quite minor, but it is clear that the same berm width is more effective in reducing overtopping for high wave steepness than for low. For the wave steepness of 0.5% hardly any difference between the different berm widths is observed, while for the highest steepness, the points are clearly grouped after berm width. Also it appears that the Van der Meer and Sigurdarson (2017) formula is fitting well to the data with a wave steepness between four and five percent.

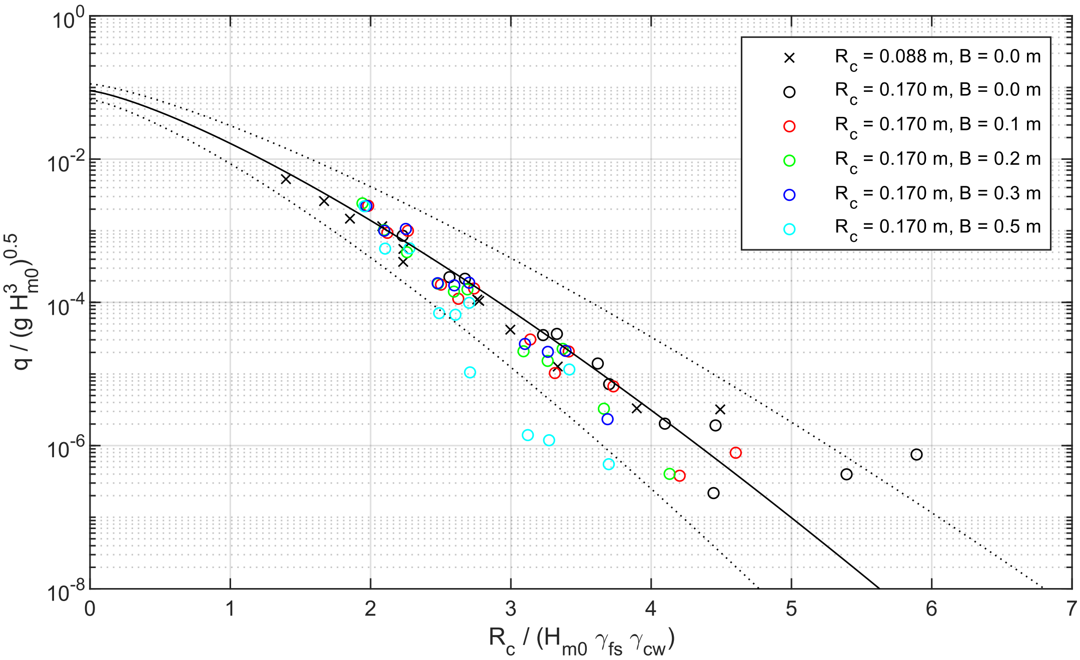

Figure 8. Comparison of present data to Eqs. 3 and 4 with γ_{fs} given by Eq. 11 and γ_{cw} by Eq. 6. 90% confidence band according to EurOtop (2018) is also given.

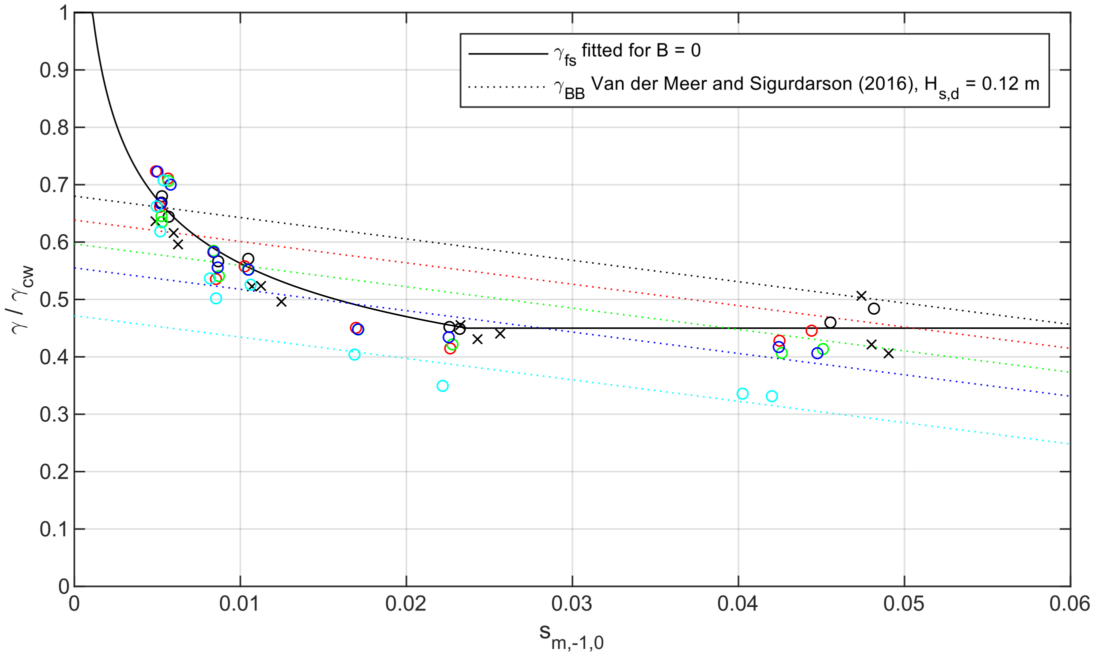

Figure 9. Calculated total influence factor corrected by the crest width influence. Colours identical to Fig. 8.

Based on Eq. 12, the berm influence factor (γ_{BB}) may be calculated from the total influence factor calculated from Eq. 10, the fitted function for γ_{fs}, in Eq. 11, the function for γ_{cw} in Eq. 6 and $\gamma_{\beta} = 1$. This is plotted in Fig. 10 against the dimensionless berm width B/H_{m0}. Note that here the actual wave height is used and not the design wave height as in Van der Meer and Sigurdarson (2017), cf. Eq. 9. The actual wave height was chosen as the overtopping in the individual tests, do not depend on how large the design wave height is in comparison to the tested wave height. Tests with dimensionless overtopping discharges below 10^{-6} are also ignored in Fig. 10. The dependency on both the wave steepness ($s_{m-1,0}$) and dimensionless berm width (B/H_{m0}) is clearly observed.

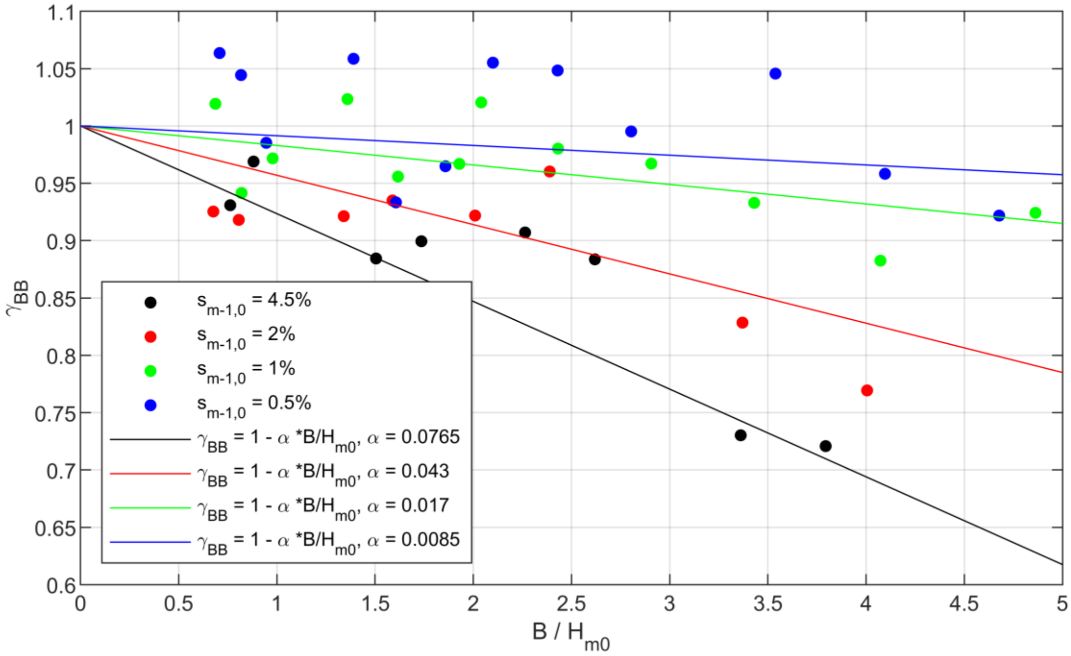

Figure 10. Calculated berm influence factor for each test.

Lines are fitted to each wave steepness of the form:

$$\gamma_{BB} = 1 - \alpha \frac{B}{H_{m0}} \tag{13}$$

In Fig. 11, the fitted α values are plotted as function of the wave steepness. The α values are well represented by a linear function of the wave steepness, as given in Eq. 14.

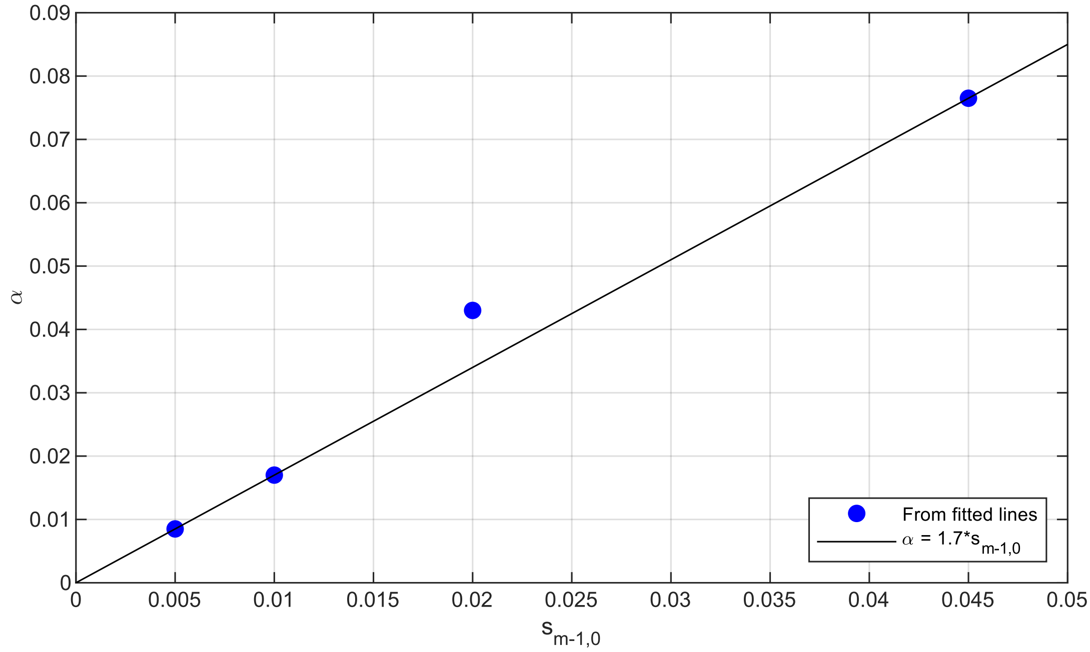

Figure 11. Plot of the fitted α values from Fig. 10 as function of the wave steepness.

$$\alpha = 1.7 s_{m-1,0} = 1.7 \frac{H_{m0}}{L_{m-1,0}} \tag{14}$$

Thus, by inserting Eq. 14 into Eq. 13, the berm influence factor becomes:

$$\gamma_{BB} = 1 - \alpha \frac{B}{H_{m0}} = 1 - 1.7 \frac{H_{m0}}{L_{m-1,0}} \frac{B}{H_{m0}} = 1 - 1.7 \frac{B}{L_{m-1,0}} \tag{15}$$

Thus, it seems more appropriate to make the berm width dimensionless by the wavelength instead of the wave height.

EVALUATION OF THE PROPOSED INFLEUNCE FACTOR

In Fig. 12 the data are compared to the fitted function for the influence factor, i.e. Eq. 15. The figure shows that the data are well fitted by the new formula. The width of the confidence band seems independent on both the value of the dimensionless berm width and the wave steepness and corresponds to approximately $\gamma \pm 0.07$. The band is also plotted in Fig. 13 together with data from the reference structure and the various berm widths. Here the calculated γ_{fs} values are plotted under the assumption that the estimated γ_{cw} and γ_{BB} values by Eqs. 6 and 15 are correct.

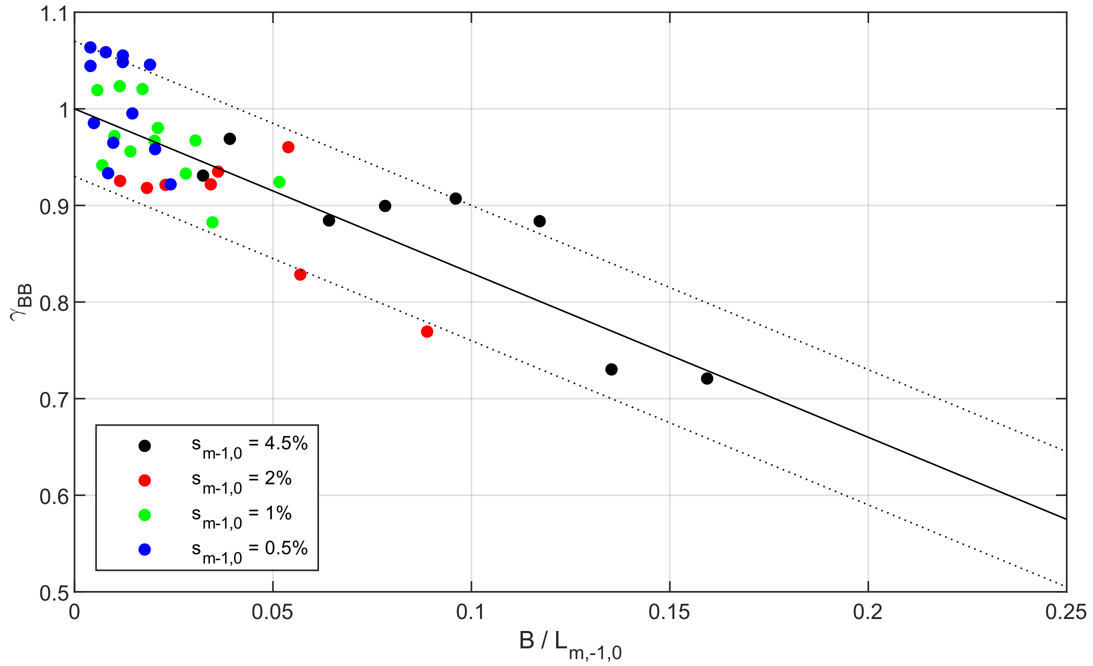

Figure 12. Evaluation of proposed berm influence factor as function of *B*/*L*$_{m\text{-}1,0}$.

In Fig. 14, the data are plotted in the traditional EurOtop style graph but including all the influence factors. Note that the γ_{fs} function fitted to the reference structure (Eq. 11) is used. The other influence factors are based on Eqs. 6 and 15.

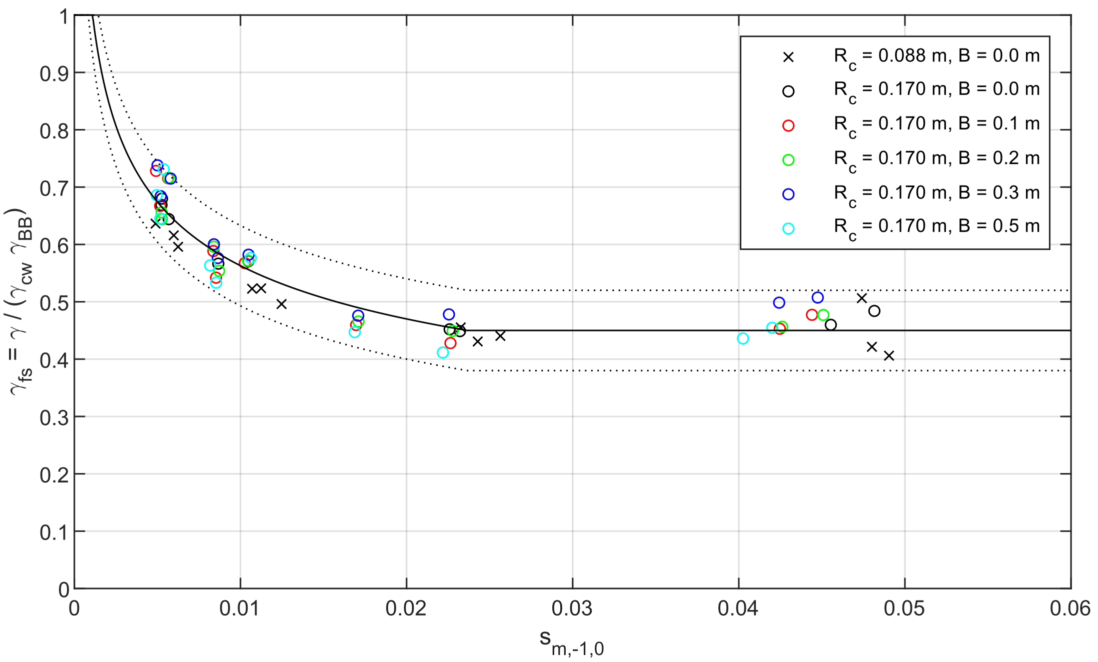

Figure 13. Calculated total influence factor corrected by the crest width and berm width influence factors calculated by Eqs. 6 and 15. Confidence band given by $\gamma \pm 0.07$.

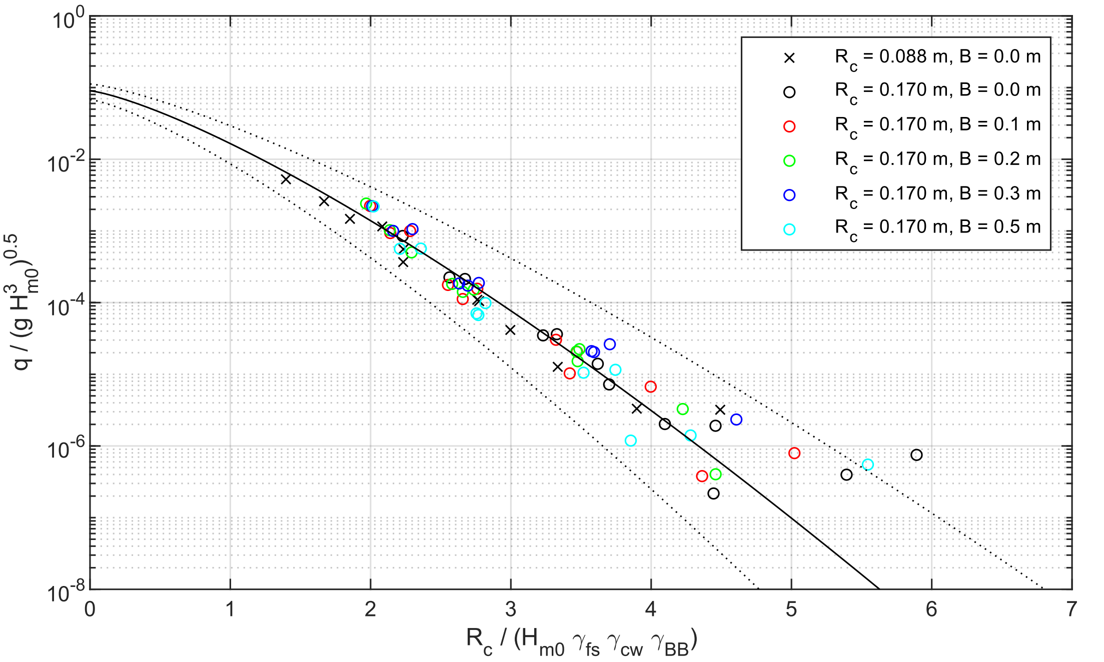

Figure 14. Comparison of present data to Eqs. 3 and 12 with γ_{fs} given by Eq. 11, γ_{cw} by Eq. 6 and γ_{BB} by Eq. 15. 90% confidence band according to EurOtop (2018) is also given.

Fig. 14 shows that by including the berm influence factor, the scatter is reduced significantly compared to Fig. 8. The scatter is much less than given by the confidence band, but that is also partly caused by using the fitted γ_{fs} function from the reference structure. It appears though that the scatter for the berm breakwater data is not higher than for the reference structure.

CONCLUSIONS

The present paper presents overtopping results from a new model test study with hardly reshaping berm breakwaters. In order to reduce the complexity of the structure, the berm was homogenous in all tests and with a fixed berm level. Multi-layer berm breakwaters (Icelandic type) will have lower permeability and thus the effect of the berm is expected to be slightly lower and thus slightly higher overtopping than predicted with the present method must be expected. The tests were performed with a berm elevation of 30-50% of the spectral significant wave height (d_b/H_{m0} = 0.3 - 0.5). This covers only the lower end of the interval typically applied for Icelandic type berm breakwaters. The influence of berm elevation is still to be studied, but a higher berm elevation is expected to lead to lower overtopping. Lykke Andersen and Burcharth (2005) studied the influence of the berm elevation, but this was based mainly on fully reshaping berm breakwaters, where after reshaping no berm is left anymore.

A reference structure without a berm was also studied, and four different berm widths were added to the reference structure. The results from the reference structure deviate a bit from the Eldrup et al. (2022) formula for the wave steepness influence through γ_{fs}. Further studies are needed to better understand the scatter in this influence factor. The focus for the present paper was solely the berm influence factor and thus a formula was fitted for γ_{fs} for the reference stucture. This factor was then applied also for the tests with a berm.

The results showed that the main parameter for the berm influence factor is the berm width relative to the wavelength ($B/L_{m-1,0}$). This is different from earlier studies that made the berm width dimensionless with the significant wave height. A linear relation was found between the berm influence factor (γ_{BB}) and $B/L_{m-1,0}$. The results also showed that the influence of a berm is rather small unless the berm is wide, i.e. only when $B/L_{m-1,0} > 0.05$ the influence on γ_{BB} is larger than 10%. When taking the dimensionless berm width as $B/L_{m-1,0}$ then no additional wave steepness influence was needed in γ_{BB}. Thus, the wave steepness influence in γ_{fs} also describes the influence of the wave steepness for berm breakwaters.

The developed berm influence factor is applied in the modified EurOtop formula by Eldrup et al. (2022). By including the developed berm influence factor, it was shown that a similar scatter is obtained for berm breakwaters as for the reference structure.

Further work is needed to verify the developed influence factor on a larger database and study the effects of berm elevation and more stone classes (Icelandic type). The influence of the berm elevation is still to be studied, but the EurOtop formula with the present berm influence factor is expected to provide conservative results for higher berms.

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