

# WAVE OVERTOPPING AT DIKES AND BREAKWATERS UNDER OBLIQUE WAVE ATTACK

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The crest level of coastal structures such as dikes and breakwaters is often based on estimates of the amount of wave overtopping. One of the important parameters affecting wave overtopping is the angle of the incident waves since oblique waves can significantly reduce the amount of wave overtopping compared to perpendicular wave attack. Based on 3D physical model tests on dikes, rubble mound breakwaters and vertical caisson breakwaters, the influence of oblique wave attack has been evaluated. A new expression for oblique waves has been derived that can be applied for all tested structure types.

*Keywords: wave overtopping; dike; rubble mound breakwater; caisson breakwater; wave basin; oblique waves; design formula; discharges; physical model; berm; recurved parapet*

## INTRODUCTION

Estimates of wave overtopping are essential to determine the required crest elevation of coastal structures. With respect to wave overtopping, the mean overtopping discharge traditionally is the main parameter to be determined. Many studies and physical model tests were focussed on these discharges, and the influence of many factors such as the angle of wave attack, the roughness of the slope, the presence of a berm in the seaward slope, the presence of a crest wall, and the presence of swell in combination with wind waves, have been studied for wave overtopping discharges. Other important wave overtopping parameters such as the velocities during wave overtopping events, the flow depth during overtopping events, and the volumes within overtopping events have received less attention. Also, the influence of the earlier mentioned factors on these wave overtopping parameters are less known. For estimates of velocities and flow depths, reference is made to Schüttrumpf (2001), Van Gent (2002), Schüttrumpf and Van Gent (2003) and Van Bergeijk *et al* (2019). For an overview of studies with respect to individual wave overtopping events reference is made to Koosheh *et al* (2021, 2022b). Besides analytical and empirical relations, also numerical modelling and data-driven methods can provide accurate estimates of wave overtopping. Within parameter ranges that are well covered by data, adequate data-driven methods generally outperform empirical expressions. However, for specific wave loading and specific structure configurations empirical relations may still be more suitable, but these are often not accurate well outside the ranges for which the empirical relations have been developed. Due to the large number of parameters affecting wave overtopping, data-driven methods are mainly developed to predict mean overtopping discharges because for other wave overtopping parameters (*e.g.* velocities in overtopping events and overtopping volumes) insufficient data are available to cover the entire ranges of relevant wave loading parameters and structure configurations. For data-driven methods reference is made to the most accurate presently-available neural network for wave overtopping discharges (Van Gent *et al*, 2007) and a more advanced and more accurate data-driven method (Den Bieman *et al*, 2021).

The present paper is focussed on the influence of oblique waves on wave overtopping discharges, both to facilitate empirical design guidelines and to provide input for data-driven methods. Three types of coastal structures are discussed:

- Dikes
- Rubble mound breakwaters
- Vertical caisson breakwaters

The applied data-sets on oblique wave attack on coastal structures are those described in Van Gent (2020) for dikes, Van Gent and Van der Werf (2019) for rubble mound breakwaters, and Van Gent (2021) for vertical caisson breakwaters. In these data-sets the wave directions that were tested were  $\beta = 0^\circ, 15^\circ, 30^\circ, 45^\circ, 60^\circ$  and  $75^\circ$ , where  $0^\circ$  corresponds to perpendicular wave attack. Figure 1 shows images of oblique wave attack on a dike, on a rubble mound breakwater and on a caisson breakwater.

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Figure 1. Pictures of model tests in a wave basin with oblique wave attack on a sloping dike (upper panel), rubble mound breakwater (mid panel), and a vertical caisson breakwater (lower panel); source: Van Gent and Van der Werf (2019) and Van Gent (2020, 2021).

### WAVE OVERTOPPING DISCHARGES

Most empirical expressions to estimate the mean wave overtopping discharge can be rewritten using the following shape:

$$\frac{q}{\sqrt{gH_{m0}^3}} = a \gamma_b s_{m-1,0}^d \cot^e \alpha \exp \left[ -\frac{b}{\gamma_\beta \gamma_b \gamma_f \gamma_p \gamma_v} \left( \frac{R_c - c_{swell} H_{m0-sw}}{H_{m0} \xi_{m-1,0}} \right)^c \right] \quad (1a)$$

with a maximum of

$$\frac{q}{\sqrt{gH_{m0}^3}} = a' s_{m-1,0}^{d'} \cot^{e'} \alpha \exp \left[ -\frac{b'}{\gamma_\beta \gamma_b \gamma_f \gamma_p \gamma_v} \left( \frac{R_c - c_{swell} H_{m0-sw}}{H_{m0} \xi_{m-1,0}^{f'}} \right)^c \right] \quad (1b)$$

where  $q$  is the mean wave overtopping discharge ( $\text{m}^3/\text{s}/\text{m}$ ),  $g$  is the acceleration due to gravity ( $\text{m}/\text{s}^2$ ),  $H_s=H_{m0}$  is the significant wave height of the incident waves of the sea state at the toe of the structure (m),  $s_{m-1,0}$  is the wave steepness ( $s_{m-1,0} = 2\pi H_{m0} / gT_{m-1,0}^2$ ),  $T_{m-1,0}$  is the spectral wave period of the

incident waves of the sea state at the toe of the structure ( $s$ ),  $\xi_{m-1,0}$  is the surf-similarity parameter or breaker parameter ( $\xi_{m-1,0} = \tan \alpha / (s_{m-1,0})^{0.5}$ ),  $R_c$  is the crest height relative to the still water level including the height of a crest wall, if present (m),  $H_{m0\text{-swell}}$  is the significant wave height of the incident waves of the swell at the toe of the structure (m),  $\alpha$  is the slope angle of the structure ( $^\circ$ ),  $\gamma$  denotes influence factors for various effects such as those by the angle of wave attack of the sea state ( $\gamma_\beta$ ), the presence of a berm ( $\gamma_b$ ), the roughness of the slope ( $\gamma_f$ ), the presence of a recurved parapet ( $\gamma_p$ ), and the presence of a crest element ( $\gamma_v$ ), and  $a$ ,  $a'$ ,  $b$ ,  $b'$ ,  $c$ ,  $c_{swell}$ ,  $d$ ,  $d'$ ,  $e$ ,  $e'$  and  $f'$  are coefficients.  $Q = q / (gH_{m0}^3)^{0.5}$  is the non-dimensional wave overtopping discharge. Eq.1a can be referred to as the expression for breaking waves and Eq.1b as the expression for non-breaking waves.

For dikes, which generally have slopes of 1:3 or gentler, the expression for breaking waves (Eq.1a) is often more relevant than the expression for non-breaking wave conditions (Eq.1b) since most sea states without severe wave breaking on the foreshore result in conditions with breaking waves on the slope due to the gentle slopes of dikes. On the other hand, for vertical caisson breakwaters only the expression for non-breaking waves (Eq.1b) is relevant. For caisson breakwaters the influence factor for roughness is not applicable ( $\gamma_f=1$ ) and if the front face of the caisson and crest wall on top of the caisson are aligned, also the influence factor for the vertical crest wall is not applicable ( $\gamma_v=1$ ).

For the influence of berms on wave overtopping discharges ( $\gamma_b$ ) reference is made to Chen *et al* (2020a,b, 2021, 2022) and to Van Gent (2020) for dikes with a berm under oblique wave attack. For the influence of roughness ( $\gamma_f$ ) reference is made to TAW (2002), Capel (2015) and Chen *et al* (2020a,b, 2021, 2022) for dikes and to Bruce *et al* (2009) and Molines and Medina (2015) for rubble mound breakwaters. For the influence of crest walls ( $\gamma_v$ ) reference is made to Van Doorslaer (2018) for crest elements on dikes and to Van Gent *et al* (2022) and Irías Mata and Van Gent (2023) for crest elements on rubble mound breakwaters. For the effects of a recurved parapet on caisson breakwaters ( $\gamma_p$ ) reference is made to Franco and Franco (1999) and Martinelli *et al* (2018), while the influence of recurved parapets in combination with oblique waves is discussed in Napp *et al* (2004) for impulsive wave loading and in Van Gent (2021) for non-impulsive wave loading. For the influence of the swell in combination with sea conditions, reference is made to Van der Werf and Van Gent (2018) for dikes and to Van Gent (2021) for caisson breakwaters.

Note that the reduction factors depend on the equation in which it is used. Here, Eq.1 is used with the value  $c = 1$ . Although different values than  $c = 1$  have been proposed, several studies have shown that either  $c = 1$  is the optimal value or that a different value for  $c$  does not really improve the results (see for instance Gallach-Sánchez, 2018; Gallach-Sánchez *et al*, 2021; Van Gent and Van der Werf, 2019; Van Gent, 2020, 2021; Koosheh *et al*, 2022a). For the mentioned data-sets  $c = 1$  is used here, since higher values do not lead to clearly better results.

In Etemad-Shahidi *et al* (2022) a method to estimate wave overtopping discharges is applied that cannot be rewritten as in Eq.1 because the fictitious wave run-up level is used to estimate wave overtopping discharges. Their method to account for oblique waves on wave overtopping discharges at rubble mound breakwaters accounts for the amount of directional spreading in the sea state. In the present study the influence of the amount of directional spreading of the incident waves on the effects of oblique waves has not been accounted for (lack of data for some of the studied structure types).

## PHYSICAL MODEL TESTS

The three data-sets used here were obtained from physical model tests performed in the Delta Basin at Deltares (Van Gent and Van der Werf, 2019, and Van Gent, 2020, 2021). The multi-directional wave generator consists of 100 paddles and is equipped with active reflection compensation. This means that the motion of the wave paddles compensates for the waves reflected by the coastal structures, preventing them to re-reflect at the wave paddles and propagate towards the model, causing an unrealistic amount of wave energy and an unrealistic wave field in the wave basin. The active wave absorption system accounts for 3D effects. This means that the direction of the reflected waves propagating towards the wave board is accounted for. For coastal structures that cause a significant amount of wave reflection, this system is essential to prevent that the measurements are disturbed by unphysical re-reflected waves. The applied wave generation and wave absorption method is based on Wenneker *et al* (2010). Accounting for the mentioned 3D effects in the described active wave absorption technique is a feature that has not been available during earlier research on oblique wave attack. Therefore, the quality of the present tests on oblique waves is considered superior to earlier data-sets on the effects of oblique wave attack on coastal structures, especially for highly reflecting structures such as caisson breakwaters.

### Dikes

The tests on dikes as described in Van Gent (2020) have been performed for a smooth impermeable dike with a berm and a rough impermeable dike with a berm. Figure 2 shows the tested cross-section. The slopes were 1:3. These tests were performed with long-crested waves. Three water depths (thus three different freeboards) were tested and three different levels of the berm relative to the still water level were used. For each water depth, wave direction and structure geometry the tests were performed with two different wave steepnesses. A maximum of three wave height conditions per wave steepness and wave direction were tested. This resulted in the following ranges: Wave steepness of  $0.017 \leq s_{m-1,0} \leq 0.042$ , wave angles of  $0^\circ \leq \beta \leq 75^\circ$ , a surf-similarity parameter of  $1.6 \leq \xi_{m-1,0} \leq 2.6$ , a berm width of  $B/H_{m0} \leq 2.5$ , a berm depth of  $-0.42 \leq d_b/H_{m0} \leq 0.42$  and a (non-dimensional) crest level of  $R_c/H_{m0} \leq 3$ .

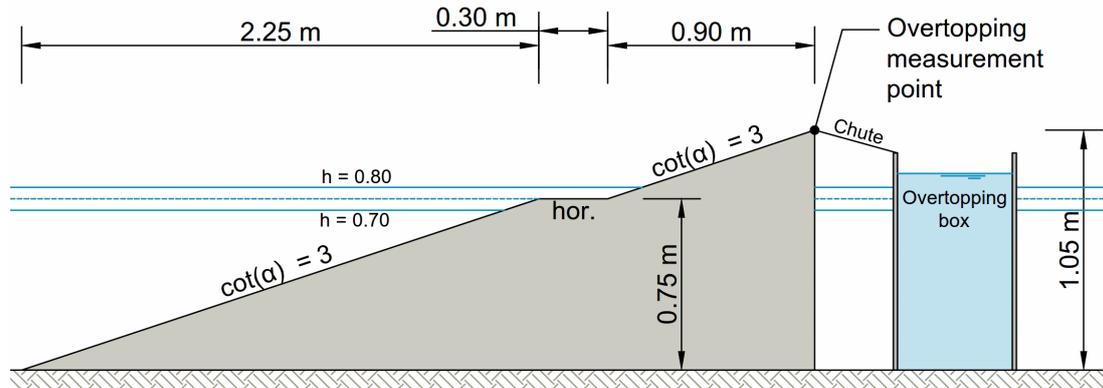


Figure 2. Dikes: Geometry of tested structure (derived from Van Gent, 2020).

With respect to the empirical expression shown in Eq.1a the following values were proposed:  $a = 0.067$ ,  $b = 4.75$ ,  $c = 1$ ,  $d = -0.5$  and  $e = -0.5$  ( $c_{swell} = 0.5$  was proposed by Van der Werf and Van Gent, 2018, but in the tests on dikes as applied here no swell component was present). The influence of the berm ( $\gamma_b$ ) depends on the berm width, berm level, and the wave steepness, see also Chen *et al* (2020a,b, 2022) and Van Gent (2020). The influence of the roughness can be accounted for using  $\gamma_f = 1 - 0.16 R_c / (H_{m0} \xi_{m-1,0})$ . Since no crest wall and no recurved parapet were used the reduction factors for those effects are not relevant here ( $\gamma_v = 1$  and  $\gamma_p = 1$ ). The tests indicated that the influence of oblique waves increases for wider berms (limited to  $B/H_{m0} \leq 2.5$ ). For the influence of oblique waves Eq.2 was proposed, but this will be discussed in more detail in the corresponding section.

### Rubble mound breakwaters

The tests on rubble mound breakwaters as described in Van Gent and Van der Werf (2019) have been performed for permeable rock-armoured breakwaters with crest elements. Although the tests were mainly focused on forces on crest elements, also overtopping discharges were measured. Figure 3 shows the tested cross-section. The slopes were 1:2. These tests were performed with long-crested waves and with short-crested waves. Three water depths (thus three different freeboards) were tested. For each water depth, wave direction and structure geometry the tests were performed with two different wave steepnesses. About three wave height conditions per wave steepness and wave direction were tested. This resulted in the following ranges: Wave steepness of  $0.015 \leq s_{m-1,0} \leq 0.046$ , wave angles of  $0^\circ \leq \beta \leq 75^\circ$ , a surf-similarity parameter of  $2.3 \leq \xi_{m-1,0} \leq 4.2$ , and a (non-dimensional) crest level of  $0.8 \leq R_c/H_{m0} \leq 2.2$ .

With respect to the empirical expression shown in Eq.1b the following values were used by Van Gent and Van der Werf (2019):  $a' = 0.2$ ,  $b' = 2.06$ ,  $c = 1$ ,  $d' = 0$ ,  $e' = 0$  and  $f' = 0$  ( $c_{swell} = 0.4$  was proposed by Van Gent, 2021, based on tests with caisson breakwaters but in the tests on rubble mound structures as applied here no swell component was present). The influence of the roughness is accounted for using  $\gamma_f = 0.45$ . Since no recurved parapet was used the corresponding reduction factor is not relevant here ( $\gamma_p = 1$ ). Note that in the performed analysis no value for  $\gamma_v$  was derived since the calibrated  $b$ -value includes the effect of the crest wall (thus in Eq.1b,  $b/\gamma_v = 2.06$ ). The tests showed no important differences between discharges measured for long-crested waves and for short-crested waves. For the influence of oblique waves Eq.3 was proposed, but this will be discussed in more detail in the corresponding section.

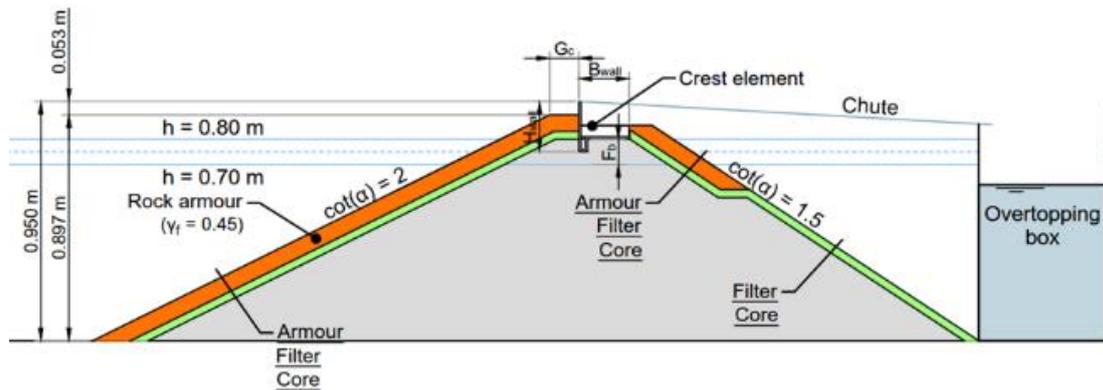


Figure 3. Rubble mound breakwater: Geometry of tested structure (derived from Van Gent and Van der Werf, 2019).

### Caisson breakwaters

Tests on vertical caisson breakwaters as described in Van Gent (2021) have been performed for structures with and without a recurved parapet. Figure 4 shows the tested cross-section. These tests were performed with long-crested waves and with short-crested waves, both leading to similar results except for the most oblique waves  $\beta = 75^\circ$ . The spreading in the results was smaller for the conditions with short-crested waves. Various water depths (thus different freeboards) were tested. For each water depth, wave direction and structure geometry the tests were performed with two different wave steepnesses. Three wave height conditions per wave steepness and wave direction were tested. This resulted in the following ranges: Wave steepness of  $0.015 \leq s_{m-1,0} \leq 0.041$ , wave angles of  $0^\circ \leq \beta \leq 75^\circ$ , and a (non-dimensional) crest level of  $1.2 \leq R_c / H_{m0} \leq 2.9$ .

In addition, bimodal wave conditions were tested with sea from one direction and swell from another direction. All conditions can be seen as non-impulsive wave conditions. Non-impulsive wave conditions can be characterised, based on EA manual (1999), as  $(h/H_{m0})^2 s_{m-1,0} > 0.25$ . This expression to distinguish between impulsive and non-impulsive wave loading may have to be revisited since it can be assumed that non-impulsive wave loading is more likely to occur for conditions with a low wave steepness than for conditions with a higher wave steepness; the expression suggests the opposite.

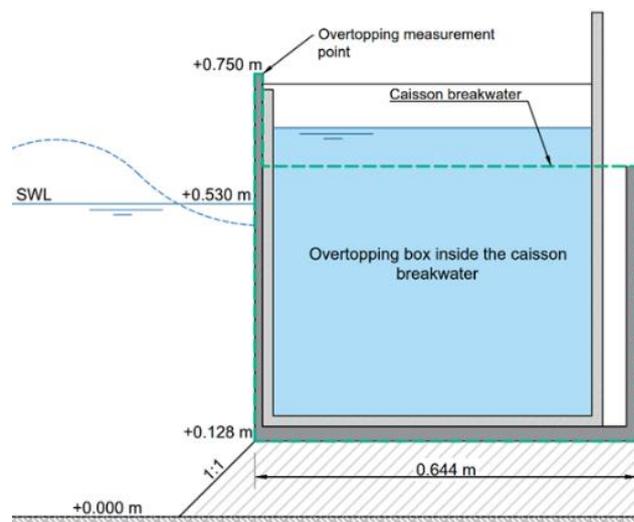


Figure 4. Caisson breakwaters: Geometry of tested structure (derived from Van Gent, 2021).

With respect to the empirical expression shown in Eq.1b the following values were used by Van Gent (2022):  $a' = 0.2$ ,  $b' = 3.9$ ,  $c = 1$ ,  $c_{swell} = 0.4$ ,  $d' = 0$ ,  $e' = 0$  and  $f' = 0$ . For caissons the reduction factor for friction is not relevant ( $\gamma_f = 0$ ). Also, the reduction factor for crest elements is not relevant for the tested structure where the height of the parapet is included in the crest level ( $\gamma_v = 1$ ). The influence

of the recurved parapet applied in the tests can be accounted for by  $\gamma_p = 0.89$ . The effect of a recurved parapet on the overtopping discharge can be large but reduces for larger angles of wave attack. For the influence of oblique waves Eq.4 was proposed where the reduction due to oblique waves depends on the presence of a recurved parapet. This will be discussed in more detail in the following section.

#### INFLUENCE OF OBLIQUE WAVE ATTACK

The analysis of the tests with dikes, rubble mound breakwaters and caissons under oblique wave attack resulted in prediction formulae to incorporate the effects of oblique waves.

For dikes:

$$\gamma_\beta = (1 - c_\beta) \cos^2 \beta + c_\beta \quad \text{with} \quad c_\beta = 0.35 \left( 1 + \frac{B}{H_{m0}} \right)^{-1} \quad (2)$$

For rubble mound breakwaters:

$$\gamma_\beta = (1 - c_\beta) \cos^2 \beta + c_\beta \quad \text{with} \quad c_\beta = 0.35 \quad (3)$$

For caisson breakwaters:

$$\gamma_\beta = (1 - c_\beta) \cos^2 \beta + c_\beta \quad \text{with} \quad c_\beta = 0.75 / \gamma_p \quad (4)$$

For dikes with a berm it appeared that the reduction due to oblique wave attack depends on the berm width (see Eq.2). For caisson breakwaters it appeared that this reduction depends on the (presence of a) recurved parapet (see Eq.4). Also, the values of the coefficient  $c_\beta$  in each of the above expressions are not always the same. For the tested 1:3 dikes with a berm, the maximum value (for  $B=0$ ) is  $c_\beta = 0.35$  using  $c = 1$  in Eq.1 (note: for long-crested waves). For the tested rubble mound breakwaters with 1:2 slopes plus a crest wall, the value was  $c_\beta = 0.35$  using  $c = 1$  in Eq.1 (note: for long-crested and short-crested waves). The vertical caisson breakwaters (vertical, with or without a recurved parapet) shows the highest values:  $c_\beta = 0.75$  for short-crested waves using  $c = 1$  in Eq.1 (note: for long-crested waves a slightly different value was found  $c_\beta = 0.8$ ). Figure 5 illustrates the expressions (Eqs.2 to 4) for the various types of structures.

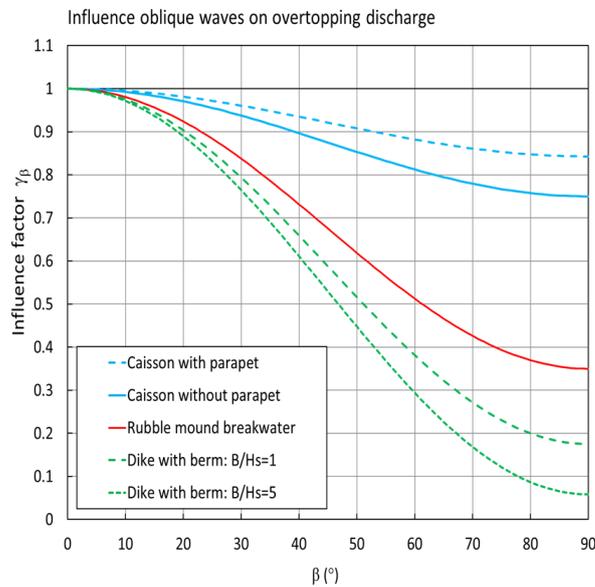


Figure 5. Influence of oblique waves on wave overtopping discharges for various types of coastal structures, as obtained from original studies.

Note that the expressions shown in Figure 5 are for the reduction factor  $\gamma_\beta$  in the exponential part of expression of Eq.1 such that the actual influence of the wave angle on the overtopping discharge is much larger than the linear scale shown in Figure 5 could suggest.

The dikes with a berm have the most gentle slopes, while the vertical caissons are of course the steepest. This is an indication that the influence of oblique wave attack increases for gentler slopes (*i.e.* lowest value of  $c_\beta$ ). Although a systematic test programme with various slopes up to vertical has not been performed using the same structure configuration (*i.e.* more parameters than only the slope have been varied in the described tests), an attempt is made to include the structure slope in the expression to account for the effects of oblique wave attack such that the same expression can be used for the three studied structure types. This resulted in the following expression for the coefficient  $c_\beta$ :

$$\gamma_\beta = (1 - c_\beta) \cos^2 \beta + c_\beta \quad \text{with} \quad c_\beta = \left( 0.75 - 0.26 (\cot \alpha)^{0.5} \left( 1 + \frac{B}{H_{m0}} \right)^{-1} \right) / \gamma_p \quad (5)$$

for  $0 \leq \cot \alpha \leq 6$ . For the applied recurved parapet with an exit angle of  $90^\circ$   $\gamma_p = 0.89$ . To illustrate the accuracy of Eq.5 in combination with Eq.1, the left panel of Figure 6 shows the comparison between measured and calculated overtopping discharges. Figure 6 clearly shows that the accuracy of Eq.1 in combination with the influence factor for oblique waves is the highest for the caisson breakwaters. Although Figure 6 is made using Eqs.1 and 5, this is also the case if Eqs.1 to 4 are used. Figure 7 illustrates the Eq.5 for the various types of structures.

In studies on rubble mound breakwaters more recent than the one applied here (Van Gent and Van der Werf, 2019), other values for the coefficients in Eq.1b have been proposed. In Van Gent *et al* (2022) it was found that the overtopping discharges depend on the wave steepness, leading to other coefficients in Eq.1b for their tests ( $a' = 0.016$ ,  $b' = 2.4$ ,  $c = 1$ ,  $c_{swell} = 0.4$ ,  $d' = -1$ ,  $e' = 0$  and  $f' = 0$ ). Eq.5 appears to be slightly more accurate in combination with their expression (their Eq.3), see also the right panel in Figure 6.

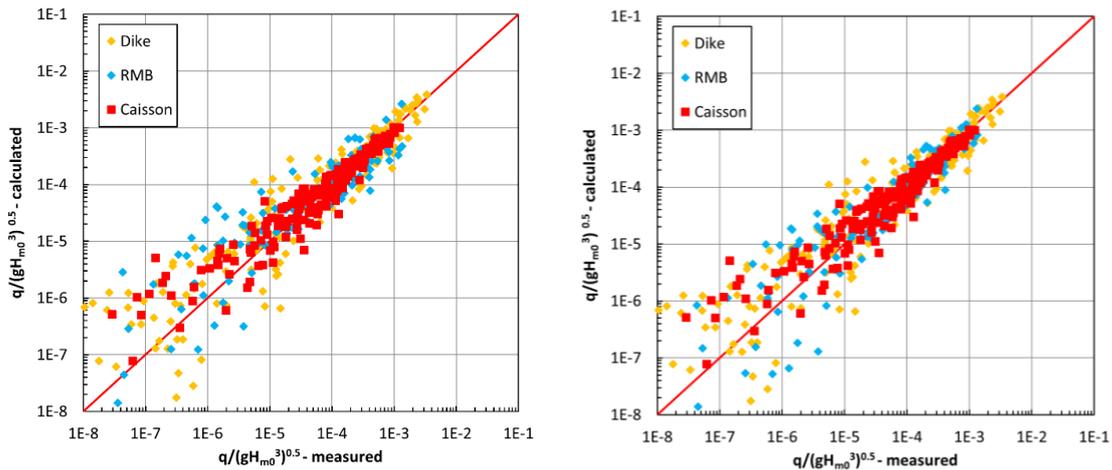


Figure 6. Comparison between measured and calculated non-dimensional wave overtopping discharges using Eqs.1 and 5, for dikes, rubble mound breakwaters and caisson breakwaters (left panel for original coefficients in Eq.1; right panel using Eq.5 and for rubble mound breakwaters the coefficients in Eq.1 as proposed by Van Gent *et al* (2022)).

Based on numerical model computations by Irías Mata and Van Gent (2023), it was found that the wave overtopping discharges at rubble mound breakwaters do not only depend on the wave steepness but also on the slope angle, leading to another set of optimal values ( $a' = 0.03$ ,  $b' = 4$ ,  $c = 1$ ,  $c_{swell} = 0.4$ ,  $d' = -0.5$ ,  $e' = 1$  and  $f' = 0.5$ ). Nevertheless, their numerical results still need to be verified based on physical model tests in which the slope angle is systematically varied.

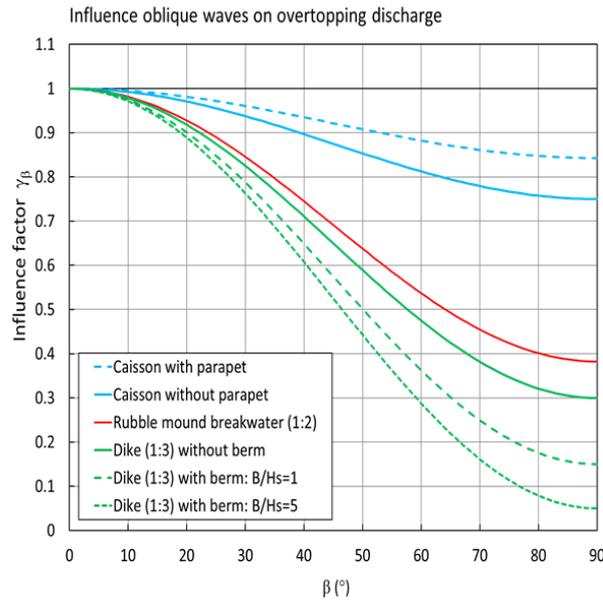


Figure 7. Influence of oblique waves on wave overtopping discharges for various types of coastal structures using Eq.5.

Table 1 shows the RMSE values for the expressions per structure type (Eqs.2 to 4) and for the generic expression (Eq.5) for all structure types. Use is made of the following error-measure, referred to as RMSE:

$$\text{RMSE} = \sqrt{\frac{\sum_{i=1}^{n_{\text{tests}}} (\log(Q_{\text{measured}}) - \log(Q_{\text{calculated}}))^2}{n_{\text{tests}}}} \quad (6)$$

where  $n_{\text{tests}}$  is the number of tests on which the RMSE is based,  $Q$  are the non-dimensional values of the measured and calculated overtopping discharges [ $Q = q/(gH_{m0}^3)^{0.5}$ ]. The RMSE is only based on measured overtopping values larger than  $Q = 10^{-6}$  since smaller values are often less relevant and scale effects may be present. If the expression by Van Gent *et al* (2022) to estimate wave overtopping is applied for rubble mound breakwaters, the RMSE value using Eq.5 becomes 0.417.

Table 1. RMSE for formulas per structure type (Eqs.2-4) and RMSE for the generic formula (Eq.5).		
Structure type	RMSE Eqs.2-4	RMSE Eq.5
Dikes	0.424	0.428
Rubble mound breakwaters	0.439	0.435
Caisson breakwaters	0.220	0.220
<b>All</b>		<b>0.350</b>

Note that for caisson breakwaters the coefficient 0.75 in Eq.5 is used for both the conditions with long-crested waves and the short-crested waves, while in Van Gent (2020b) the coefficient 0.75 was used for the conditions with short-crested waves and 0.8 was used for long-crested waves. Nevertheless, the generic expression leads to comparable RMSE for all structures.

Both Figure 6 and the RMSE values shown in Table 1 indicate that there is a rather good match between the data and the expressions (Eqs.1 and 5), indicating that the influence of the slope of the structure on the influence of oblique waves can be accounted for as proposed in Eq.5. Nevertheless, data based on a systematic variation of the slope angle without changing other structure parameters would be preferable to obtain a better insight into the accuracy of Eq.5.

## CONCLUSIONS AND RECOMMENDATIONS

The influence of the angle of wave attack on wave overtopping discharges has been studied for dikes, rubble mound breakwaters and caisson breakwaters. Compared to perpendicular wave attack, the reducing effects of oblique waves are large. For dikes with a berm these reducing effects appear to be the largest while the reducing effects are much smaller for caisson breakwaters. The effect of a recurved parapet on a caisson breakwater on the overtopping discharge can be large but reduces for larger angles of wave attack.

The derived expressions to account for oblique waves have been merged into one expression to account for oblique waves on dikes, rubble mound breakwaters and caisson breakwaters. The match between the data (for wave angles of  $0^\circ \leq \beta \leq 75^\circ$ ) and the expression (Eq.1 with  $c=1$  plus Eq.5) appears to be good. Nevertheless, it is advisable to validate the expression for other than the tested structure types.

The present study and analysis are focussed on the influence of oblique wave attack on wave overtopping discharges. The influence of oblique waves on other wave overtopping parameters is not necessarily the same. Therefore, it is recommended to study the influence of oblique wave attack on volumes per overtopping wave, on percentages of overtopping waves, and on flow velocities and flow depths during overtopping events.

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**APPENDIX**

Eq.1 shows that the influence of various effects on wave overtopping discharges can be accounted for by using influence factors denoted by  $\gamma$ , where  $\gamma_\beta$  denotes the influence of the angle of wave attack,  $\gamma_b$  the presence of a berm,  $\gamma_f$  the roughness of the slope,  $\gamma_p$  the presence of a recurved parapet, and  $\gamma_c$  the presence of a crest element. Two reasons are given below why caution is advised when using influence factors in combination with expressions from EurOtop (2018), Manual on wave overtopping of sea defences and related structures, J.W. van der Meer, N.W.H. Allsop, T. Bruce, J. de Rouck, A. Kortenhaus, T. Pullen, H. Schüttrumpf, P. Troch, B. Zanuttigh (Eds.), [www.overtopping-manual.com](http://www.overtopping-manual.com):

a) EurOtop (2018) refers to a definition of influence factors as follows (Eq.5.22):

$$\gamma = \frac{\ln q_{\text{no influence}}}{\ln q_{\text{influence}}} \quad (\text{A-1})$$

This expression does not match with almost all expressions given in EurOtop (2018) for the mean overtopping discharge (e.g. Eq.4.2, 4.3, Eq.4.4, Eq.5.10, Eq.5.11, Eq.6.5, Eq.7.1). To illustrate this Eq.6.5 and Eq.7.1 from EurOtop (2018) are used here:

$$\frac{q}{\sqrt{gH_{m0}^3}} = a \exp \left[ - \left( b \frac{R_c}{\gamma H_{m0}} \right)^{1.3} \right] \quad (\text{A-2})$$

Substituting Eq.A-2 in Eq.A-1:

$$\gamma = \frac{\ln q_{\text{no influence}}}{\ln q_{\text{influence}}} = \frac{\ln \left( a \sqrt{gH_{m0}^3} \exp \left[ - \left( b \frac{R_c}{H_{m0}} \right)^{1.3} \right] \right)}{\ln \left( a \sqrt{gH_{m0}^3} \exp \left[ - \left( b \frac{R_c}{\gamma H_{m0}} \right)^{1.3} \right] \right)} = \frac{\ln(a \sqrt{gH_{m0}^3}) + \ln \left( \exp \left[ - \left( b \frac{R_c}{H_{m0}} \right)^{1.3} \right] \right)}{\ln(a \sqrt{gH_{m0}^3}) + \ln \left( \exp \left[ - \left( b \frac{R_c}{\gamma H_{m0}} \right)^{1.3} \right] \right)} \quad (\text{A-3})$$

if (for simplicity) it is assumed that:

$$\left| \ln \left( a \sqrt{gH_{m0}^3} \right) \right| \ll \left| - \left( b \frac{R_c}{H_{m0}} \right)^{1.3} \right| \quad (\text{A-4})$$

then the ratio simplifies to:

$$\gamma = \frac{\ln q_{\text{no influence}}}{\ln q_{\text{influence}}} = \frac{\ln(a \sqrt{gH_{m0}^3}) + \ln \left( \exp \left[ - \left( b \frac{R_c}{H_{m0}} \right)^{1.3} \right] \right)}{\ln(a \sqrt{gH_{m0}^3}) + \ln \left( \exp \left[ - \left( b \frac{R_c}{\gamma H_{m0}} \right)^{1.3} \right] \right)} \approx \frac{- \left( b \frac{R_c}{H_{m0}} \right)^{1.3}}{- \left( b \frac{R_c}{\gamma H_{m0}} \right)^{1.3}} = \frac{\left( b \frac{R_c}{H_{m0}} \right)^{1.3}}{\left( \gamma^{-1.3} \right) \left( b \frac{R_c}{H_{m0}} \right)^{1.3}} = \gamma^{1.3} \quad (\text{A-5})$$

Eq.A-5 ( $\gamma = \gamma^{1.3}$ ) is only valid for  $\gamma = 1$ , thus the expressions proposed in EurOtop (2018) to estimate wave overtopping discharges do not match with Eq.A-1 to take the influence of effects (e.g. oblique waves or roughness) into account. Note that Eq.A-4 is not always valid, but irrespective whether Eq.A-4 is valid or not, Eq.A-1 does not match with Eq.A-2. Thus, Eq.5.22 (Eq.A-1) from EurOtop (2018) manual should not be used in combination with the equations to estimate mean overtopping discharges as proposed in EurOtop (2018). Note that the expression to estimate mean overtopping discharges as applied in the present paper (Eq.1) matches with Eq.A-1 (i.e. Eq.5.22 from the EurOtop 2018), only if  $|\ln(a \gamma_b s_{m-1,0}^d \cot^e \alpha \sqrt{gH_{m0}^3})| \ll |(b R_c / (\gamma \xi_{m-1,0}^f H_{m0}))|$ . Thus, there are also conditions for which Eq.A-1 does not match with the mean wave overtopping expression as applied in the present paper (Eq.1).

b) EurOtop (2018) proposes to use estimates for the mean overtopping discharge where influence factors  $\gamma$  in the exponential part of the expressions are applied to the power  $c=1.3$  (e.g. Eq.4.3, Eq.4.4, Eq.5.10, Eq.5.11, Eq.6.5, Eq.7.1). If influence factors are derived based on expressions for the mean overtopping discharge where influence factors  $\gamma$  in the exponential part of the expressions are applied to the power  $c=1$  (like in the expression proposed in TAW, 2002, and in the earlier version of the EurOtop manual, referred to in EurOtop, 2018, as Eq.4.2), applying the same influence factor in an expression where these influence factors are applied to the power  $c=1.3$  can lead to significant overestimates of the influence, especially for influence factors that show a significant effect like those for roughness and oblique waves. For instance, an influence factor for the roughness  $\gamma_f=0.5$  that is derived based on expressions with the power  $c=1$ , while applying this in the expressions with the power  $c=1.3$  as proposed by EurOtop effectively leads to a roughness factor of  $\gamma_f=0.5^{1.3}=0.406$ , leading to an overestimate of the influence of roughness. EurOtop (2018) refers to several roughness factors, for instance in Table 6.2, that have been derived based on expressions where the original roughness factors were derived based on expressions using the power  $c=1$ , while in EurOtop (2018) they are applied to the power  $c=1.3$ , leading to overestimates of the influence of roughness.

Also for the influence of oblique waves an expression is used (Eq.5.29 in EurOtop, 2018) that was derived using an expression with the power  $c=1.0$  while in EurOtop (2018) the same influence factor is applied to the power  $c=1.3$ . Obviously, this is incorrect but fortunately the mentioned expression to account for oblique waves (Eq.5.29) underestimates the influence of oblique waves, such that the two flaws counteract each other. For the influence of roughness, that is not the case.

The expression to account for the influence of oblique waves as derived in the present paper (Eq.5) should not be applied directly in expressions proposed in EurOtop (2018) since in the present paper the influence of oblique waves is derived using an expression (Eq.1) where the power of the influence factors is 1.0. Applying Eq.5 in expressions proposed by EurOtop (2018) with  $c=1.3$  (like Eq.4.3, Eq.4.4, Eq.5.10, Eq.5.11, Eq.6.5, Eq.7.1) requires rewriting the overtopping expression, or estimating the influence of oblique waves by applying the influence factor from Eq.5 to the power  $c=1/1.3=0.77$  if the influence factor is used in an overtopping expression to the power  $c=1.3$ .