WIND EFFECTS ON WAVE OVERTOPPING DISCHARGES AT DIKES AND BREAKWATERS WITH A CREST WALL

Marcel R.A. van Gent[1,2], Ruben J. van der Bijl[1,2], Sam J. Dijkstra[1,2], Lars F. van Vliet[1,2] and Davide Wüthrich[2]

Estimates of wave overtopping generally determine the required crest level of coastal structures. Wind affects wave overtopping, especially for coastal structures with crest walls because these can cause a vertical wave motion above the crest that is susceptible to wind. Within the relevant range of overtopping discharges, the expected wave overtopping discharge at coastal structures with a crest wall can be up to 5 times larger than for situations without wind. For smaller discharges the influence factor for wind can be significantly larger. In the present study based on physical model tests the maximum influence of wind on wave overtopping discharges has been studied for (impermeable) dikes and (permeable) rubble mound breakwaters with crest walls. The influence of wind on wave overtopping is mainly determined by the magnitude of the overtopping discharge itself and by the height of the crest wall. The result of the study is a guideline to estimate the maximum influence of wind on wave overtopping at dikes and rubble mound breakwaters with a crest wall.

Keywords: wind; wave overtopping; dike; rubble mound breakwater; crest wall; crown wall; recurved parapet; influence factor; design guideline; physical model tests

INTRODUCTION

For the design and climate adaptation of dikes and breakwaters, it is important to limit the amount of wave overtopping to meet the functional requirements of these coastal structures. Due to climate change and the resulting sea level rise, the adaptation of existing dikes and breakwaters has become more important. For dikes and breakwaters in relatively shallow water, sea level rise can increase the wave loading since less wave dissipation occurs before the waves reach the coastal structure. Estimates of future sea level rise are uncertain. Therefore, it may be suitable to design dikes and breakwaters that can be adapted if the sea level rise appears to be more severe than expected. An economically attractive solution to increase the crest level of a dike or a breakwater can be the addition of a crest wall (or crown wall) or to increase the height of an existing crest wall (see for instance Van Gent, 2019, Hogeveen, 2021, or Van Gent and Teng, 2023). For the design and adaptation of dikes and breakwaters with a crest wall accurate predictions of wave overtopping are needed since estimates of wave overtopping generally determine the required crest level. When a dike or breakwater contains a vertical part such as a crest wall, the wave motion is changed, generating an important vertical component. A part of the water volumes reaching levels higher than the crest level can fall back and stay seaward of the crest, while another part can overtop the crest of the coastal structure. Onshore wind can significantly increase the part that overtops. The present study is focused on the maximum effect of onshore wind on wave overtopping discharges at (impermeable) dikes and (permeable) rubble mound breakwaters. For this purpose, physical model tests have been performed with various geometries (*e.g.* different slope angles and different positions and heights of the crest wall). A guideline is developed to account for the maximum effect of wind on wave overtopping discharges.

The presented analysis of the maximum influence of wind on wave overtopping is a continuation of research by Wolters and Van Gent (2007), Van der Bijl (2022), Van Gent *et al* (2023), Dijkstra (2023) and Van Vliet (2023).

LITERATURE REVIEW

Estimates of wave overtopping discharges at coastal structures generally include the influence of wave obliquity (*e.g.* De Waal and Van der Meer, 1992; Napp *et al*, 2004; Van Gent and Van der Werf, 2019; Van Gent, 2020, 2021, 2022), roughness (*e.g.* Bruce *et al*, 2009; Capel, 2015; Molines and Medina, 2015; Chen *et al*, 2020a,b; Van Gent *et al*, 2022), a berm in the seaward slope (*e.g.* De Waal and Van der Meer, 1992; Chen *et al*, 2020a,b, 2022; Van Gent, 2020, Van Gent *et al*, 2022), a crest wall (*e.g.* Molines and Medina, 2016; Van Doorslaer, 2018; Van Gent *et al*, 2022), the presence of swell in combination with sea waves (*e.g.* Van der Werf and Van Gent, 2018; Van Gent, 2021), and the presence of a low-crested structure in front of a dike or breakwater (*e.g.* Van Gent, 2024). Guidance on

[1] Deltares, Dept. of Coastal Structures & Waves, Delft, The Netherlands; marcel.vangent@deltares.nl
[2] Delft University of Technology, Dept. of Hydraulic Engineering, Delft, The Netherlands

the effects of wind on wave overtopping discharges is relatively limited even though wind can significantly increase the amount of wave overtopping (see Ward *et al*, 1994, 1996; De Waal *et al*, 1996; Medina, 1998; Gonzalez-Escriva *et al*, 2004, 2006; Wolters and Van Gent, 2007; Chowdhury *et al*, 2020, and Van Gent *et al*, 2023).

Available guidelines to estimate wave overtopping discharges at dikes and breakwaters are for a large part based on small-scale physical model tests. Modelling wind in small scale physical model tests is affected by scale effects because the interaction of air and water cannot be modelled accurately in a model where Froude scaling is applied. Therefore, small scale test results obtained by direct modelling of wind, cannot be accurately scaled to prototype situations. The effects of wind on wave overtopping are preferably studied on prototype scale or in the field (see for instance Franco *et al*, 2003). Instead of simulating wind on a small or large scale, a method to model the *maximum* effect of wind on wave overtopping discharges was developed by Hans de Waal and applied to study the maximum influence of wind on wave overtopping at vertical breakwaters (De Waal *et al*, 1996). This method is based on using a paddle wheel at the top of the breakwater, see Figure 1. The paddle wheel mechanically transports the water that exceeded the crest level of the structure into an overtopping box behind the structure. The paddle wheel thus allows to assess the maximum amount of overtopped water by measuring all the water that is potentially transported over the crest: This maximum overtopping volume includes the water that would overtop without wind, plus all the additional volumes that reach the crest level of the structure but would fall back to the seaward side of the crest in absence of wind. By comparing the overtopping discharges for the same conditions with and without the paddle wheel, the maximum influence of wind can be assessed, thus for situations where the wind blows all the water that reaches the crest level or higher, over the crest. Based on this principle, Wolters and Van Gent (2007) constructed a paddle wheel, optimised its rotation speed, and studied the maximum influence of wind on wave overtopping for sloped structures with a crest element. The same paddle wheel was applied in the tests described here (see also Figure 2).

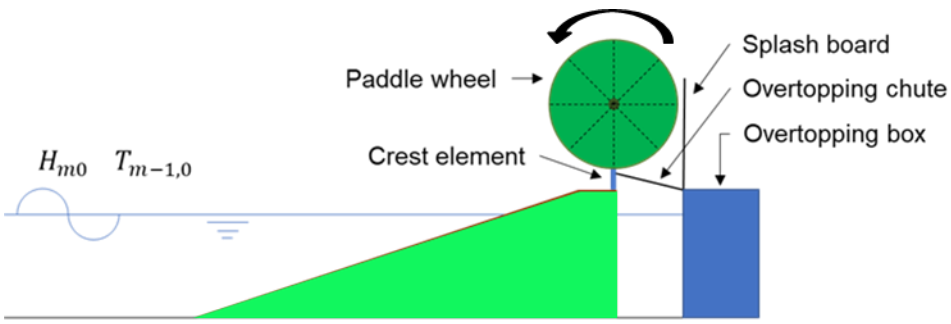

Figure 1. Schematized test set-up with a paddle wheel above the crest of the structure.

The effect of wind depends on the amount of wave overtopping itself (see Van Gent *et al*, 2023), with a large influence of wind for small overtopping discharges and a relatively small influence of wind for large overtopping discharges. For small overtopping discharges it is likely that severe onshore wind transports almost all water that reaches levels higher than the top of the crest wall over the crest. For large overtopping discharges, it is likely that even with severe onshore wind not all water that reaches the top of the crest wall will be transported over the crest. Therefore, the approach to model the maximum effect of wind by using a paddle wheel is expected to be rather close to the actual influence of wind for relatively small overtopping discharges.

In the present study we focus on the maximum effect of wind on wave overtopping discharges but not on the effects of wind on water levels (set-up) and the wave loading itself (*i.e.* the wave conditions at the toe). These effects of wind need to be taken into account in the boundary conditions (water level and wave conditions at the toe) separately.

PHYSICAL MODEL TESTS

Physical model tests were performed in a wave flume within the Pacific Basin at Deltares in Delft (see upper panels of Figure 2). The width of the wave flume was 1.0 m (within the basin with a total width of 14 m). The foreshore was horizontal and no wave breaking occurred on the foreshore.

All tested structures had a crest wall on top. To measure the wave overtopping discharge, a chute was used to guide the discharge to an overtopping box. Each test was performed with and without the paddle wheel (see Figures 1 and 2). The paddle wheel (with a diameter of 1.4 m and 12 paddles) above the crest wall rotated with such a speed (the speed was optimised by Wolters and Van Gent, 2007, to 22 rotations per minute) that the water that reached the level of the crest wall, was transported over the structure into the overtopping box. Based on variations of the rotation speed and visual observations, the effectiveness of paddle wheel to transport all water reaching the crest elevation into the overtopping box was estimated to more than 90%. No correction on the test results was applied for potential water drops reaching levels above the crest elevation but falling back to the seaward side of the structure without being transported by the paddle wheel.

Figure 2. Upper panels: Wave flume in wave basin; Lower panels: Front and side views of paddle wheel above the crest wall.

The following structures were tested:

- Smooth impermeable 1:3 slope with four crest wall geometries (dike)
- Smooth impermeable 1:6 slope with four crest wall geometries (dike)
- Permeable 1:2 slope with two crest wall heights (rubble mound breakwater)
- Permeable 1:6 slope with two crest wall heights (rubble mound breakwater)
- Permeable 1:6 slope with a crest wall with a recurved parapet (rubble mound breakwater)

The four tested crest wall geometries are illustrated in the left panel of Figure 3. The height of the crest wall was either h_c = 0.05 m or h_c = 0.08 m. The position of the crest wall was either at the beginning of the crest (G_c = 0 m) or with a horizontal part in front (G_c = 0.15 m). For the rubble mound structures only the crest wall positions with a horizontal part of the armour layer in front were tested (G_c = 0.15 m). The rubble mound breakwater with a 1:6 slope and a crest wall height of h_c = 0.05 m was also tested with a recurved parapet with a horizontal protruding width of B_r = 0.008 m and exit angle of 90° (see right panel of Figure 3). The rubble mound structures consisted of an armour layer with stones of D_{n50} = 0.038 m with a layer thickness of d_a = 0.075 m, a filter layer of D_{n50} = 0.016 m with a layer thickness of d_f = 0.035 m, and core material of D_{n50} = 0.007 m.

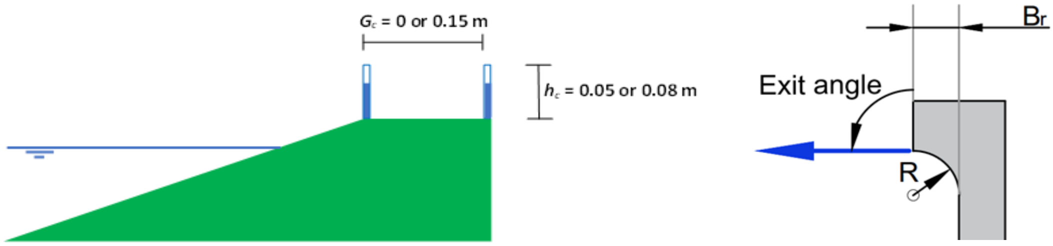

Figure 3. Left panel: Details of crest wall configurations; Right panel: Recurved parapet.

Several combinations of the significant wave height H_{m0} and wave period $T_{m-1,0}$ were tested using 1000 waves to represent a JONSWAP wave spectrum. Three wave gauges were positioned in front of the structure to obtain the incident waves from the measured surface elevations. The wave height and wave steepness of the incident waves varied between H_{m0} = 0.10 m and 0.23 m and $s_{m-1,0}$ = 0.013 and 0.044 respectively. The water depths were in the range between h = 0.6 m and h = 0.9 m which means that wave breaking did not occur before the waves reached the structure. The non-dimensional freeboards were in the range between R_c/H_{m0} = 0.99 and 3.88 for the dike configurations and between R_c/H_{m0} = 0.72 and 2.54 for the rubble mound breakwaters.

Table 1 shows the parameter ranges of the test programme for those tests with an overtopping discharge larger than $q^* = 1.0\cdot10^{-6}$. For the overtopping discharges itself the entire range is shown (also smaller than q^* = $1.0\cdot10^{-6}$). The measured non-dimensional wave overtopping discharges $q^*=q/(gH_{m0}^3)^{0.5}$ were in the range between q^* = $1.5\cdot10^{-8}$ and $2.4\cdot10^{-3}$.

Table 1. Parameter ranges of the test programme

		Dikes	Breakwaters
Parameter	**Symbol**	**Value / Range**	**Value / Range**
Seaward slope angle (-)	cot α	3 and 6	2 and 6
Crest freeboard (m)	R_c	0.15 – 0.48	0.15 – 0.38
Width in front of crest wall (m)	G_c	0 & 0.15	0.15
Level in front of crest wall w.r.t. SWL (m)	A_c	0.10 – 0.40	0.10 – 0.30
Protruding part of crest wall (m)	h_c	0.05 & 0.08	0.05 & 0.08
Water depth (m)	h	0.60 – 0.90	0.70 – 0.90
Incident significant wave (m)	H_{m0}	0.098 – 0.202	0.120 – 0.232
Mean spectral wave period (s)	$T_{m-1,0}$	1.25 – 2.50	1.36 – 3.04
Number of waves (-)	N	1000	1000
Relative crest freeboard (-)	R_c / H_{m0}	0.99 – 3.88	0.72 – 2.54
Relative width of crest in front of crest wall (-)	G_c / H_{m0}	0 – 1.52	0.65 – 1.25
Relative level of crest in front of crest wall (-)	A_c / H_{m0}	0.66 – 3.06	0.48 – 2.08
Relative height of crest wall (-)	h_c / H_{m0}	0.25 – 0.82	0.22 – 0.66
Wave steepness ($s_{m-1,0}=2\pi\, H_{m0}/gT_{m-1,0}^2$) (-)	$s_{m-1,0}$	0.020 – 0.041	0.013 – 0.044
Breaker parameter ($\xi_{m-1,0}$ = tan α /$s_{m-1,0}^{0.5}$) (-)	$\xi_{m-1,0}$	0.82 – 2.35	0.82 – 4.41
Overtopping discharge (without wind) (-)	q^*	4.8 10^{-7} – 2.4 10^{-3}	1.5 10^{-8} – 1.6 10^{-3}
Overtopping discharge (with wind) (-)	q_w^*	1.9 10^{-6} – 2.3 10^{-3}	4.9 10^{-8} – 2.4 10^{-3}
Maximum wind effect for $q^* \geq 1.0\cdot10^{-6}$ (-)	γ_w	1.0 – 5.7	1.0 – 5.2

TEST RESULTS AND ANALYSIS

Each test was performed with and without the paddle wheel, hereafter referred to as “with wind” and “without wind”. The ratio between the two measured discharges is a measure for the maximum influence of wind on wave overtopping:

$$\gamma_w = \frac{q_w}{q} \quad (1)$$

If the influence factor γ_w would be based on the ratio of non-dimensional wave overtopping discharges rather than on dimensional discharges, this would lead to the same factor. The test results show (see for instance Wolters and Van Gent, 2007, and Van Gent *et al*, 2023) that the influence of wind on wave overtopping discharges is small for high overtopping discharges and larger for low wave overtopping discharges. For a subset of the described data, the impermeable structure with a 1:3 slope, the following expression was used by Van Gent *et al* (2023):

$$\gamma_w = 1 + 0.011\ (q^*)^{-0.43} \quad (2)$$

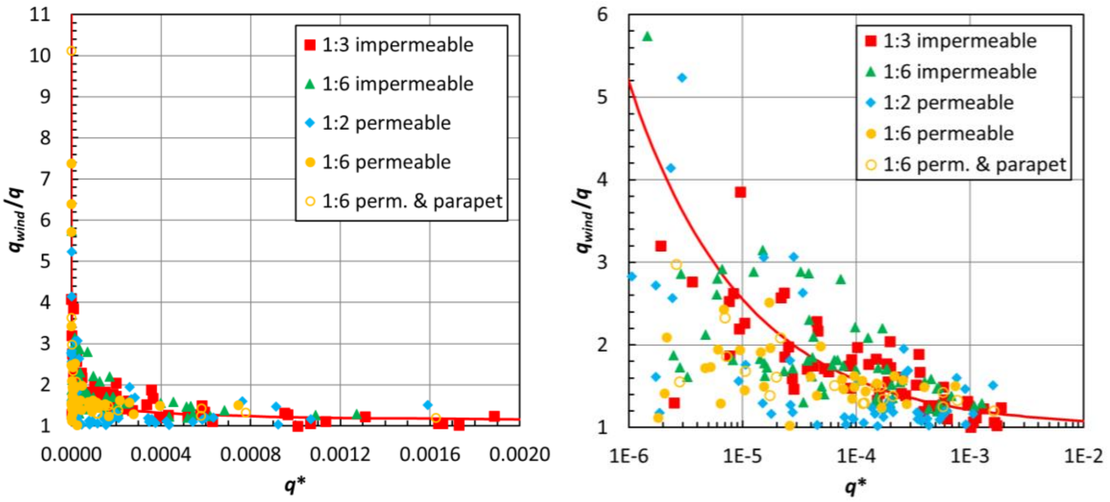

Figure 4. Influence factor for wind (left panel: linear x-axis, right panel: logarithmic x-axis); red curves denote Eq.2 (see also Van Gent *et al*, 2023).

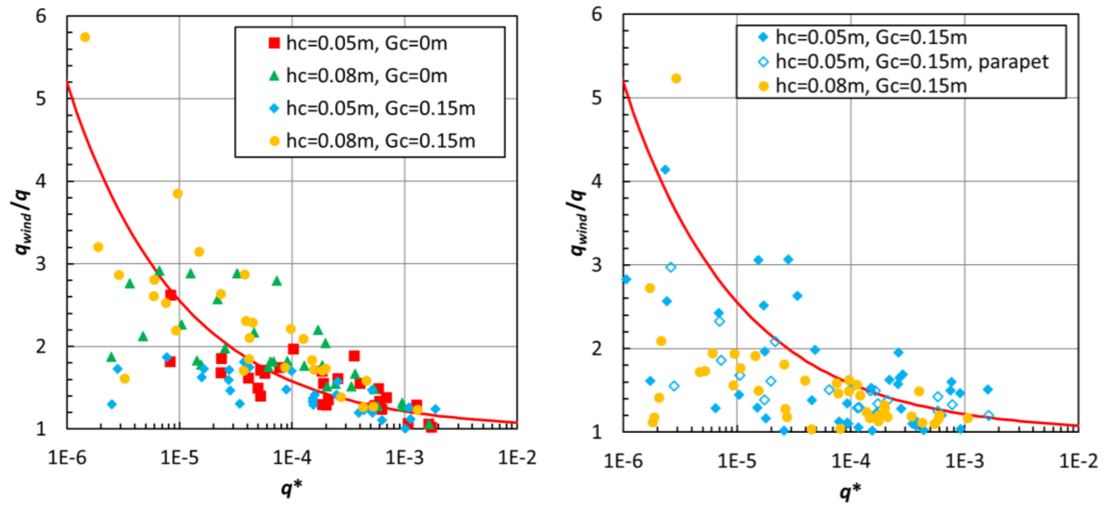

Figure 5. Influence factor for wind for various crest wall configurations. Left panel: Dikes; Right panel: Breakwaters; red curves denote Eq.2.

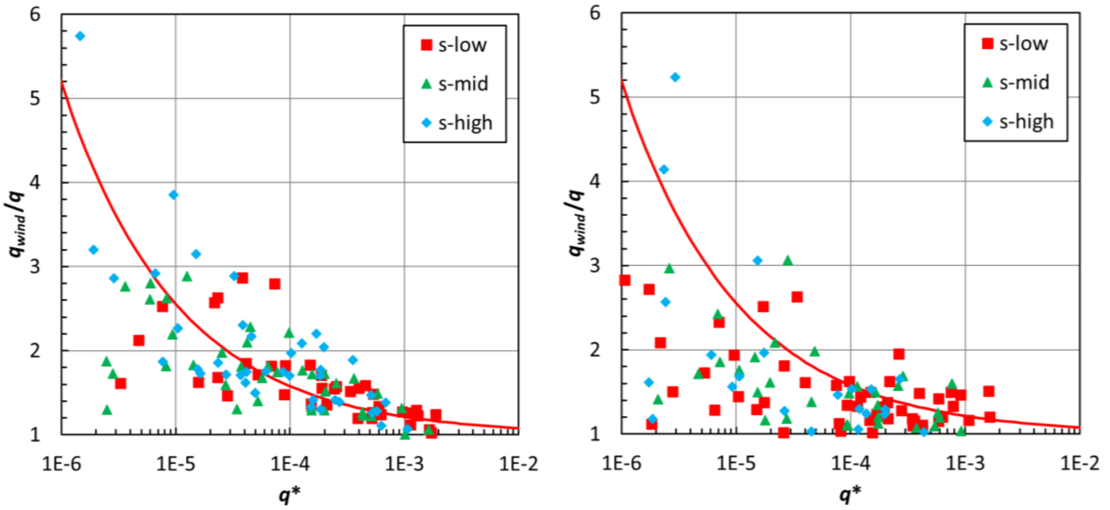

Figure 6. Influence factor for wind per wave steepness. Left panel: Dikes; Right panel: Breakwaters; red curves denote Eq.2.

In Figures 4, 5 and 6 the measured influence factors are shown together with the curve (in red) of Eq.2. Figure 4 shows the influence factor versus the non-dimensional discharge on the horizontal axis, using an *x*-axis with a linear scale in the left panel and an *x*-axis with a logarithmic scale in the left panel. The left panel also includes the discharges smaller than $q^* = 1.0 \cdot 10^{-6}$ (without wind), while the right panel only shows the data with discharges larger than $q^* = 1.0 \cdot 10^{-6}$. Note that for the smaller discharges ($q^* < 1.0 \cdot 10^{-6}$), which are considered either irrelevant and/or potentially affected by scale effects, an influence factor up to about 10 was measured. For larger discharges ($q^* > 1.0 \cdot 10^{-6}$) the influence factor remains below 6 for all structure slopes. Not only the influence of wind on wave overtopping increases for lower discharges, also the spreading in the test results increases for small discharges. Note that the curve (Eq.2) is only based on data from the 1:3 impermeable slope (red squares), although it also provides a reasonable estimate for the other sloped structures. The tests with a recurved parapet (permeable 1:6 slope) do not show a clear influence on the discharges. Similar results were also indicated in numerical model computations described by Irías Mata and Van Gent (2023), showing that a recurved parapet on the crest wall of a rubble mound breakwater has no effect on the discharges, except for very low discharges. In the present test ranges no systematic effect of the recurved parapet was found, neither for the tests with wind, nor for the tests without wind.

In Figure 5 the same data as presented in the left panel of Figure 4 is shown, but now grouped by the slopes representing a dike (impermeable 1:3 and 1:6, left panel) and a rubble mound breakwater (permeable 1:2 and 1:6, right panel). The symbols denote the various crest wall configurations, either without a horizontal part in front of the crest wall (G_c=0, for the impermeable slopes only) or with a horizontal part in front of the crest wall (G_c=0.15 m). The protruding part of crest wall was either $h_c = 0.05$ m ($h_c = R_c - A_c$) or $h_c = 0.08$ m. The left panel for dikes clearly shows that the larger values of the influence factor for wind γ_w correspond to the higher crest walls, while in the right panel for breakwaters this is less clearly visible. Comparing the data for dikes (left panel of Figure 5) with the data for breakwaters (right panel of Figure 5) indicates that the spreading around the main trend is larger for the data from the tests with breakwaters than those from tests with dikes.

Figure 6 again shows the same data, but now grouped by wave steepness. The highest values of the influencing factors γ_w often correspond to conditions with the highest wave steepness. However, both panels, for dikes on the left and for breakwaters on the right, show that the data are not stratified with respect to wave steepness. There appears to be no clear influence of wave steepness on the influence factor for wind (as also discussed in Van Gent *et al*, 2023), although wave steepness does influence the discharges themselves. The higher wave steepness generally leads to less wave overtopping, but this applies to both conditions with and without wind. The observations of the largest influence factors (factors of about 3 or greater) for the highest steepness are because the discharges are lower for conditions with a higher steepness, not because the influence factor itself depends on the wave steepness, since the data for the different values of wave steepness follow on average more or less the same trend.

The above analysis and that described in Van Gent *et al* (2023) indicate that the influence factor for wind mainly depends on the magnitude of the wave overtopping discharge and that most of the parameters that affect the discharges (*e.g.* slope angle, roughness and permeability of the slope, freeboard, wave height, wave steepness) do not affect the influence of wind other than via the discharge itself. This led to the expression in Eq.2. Without a crest wall there hardly is a vertical component of the water such that the direct influence of wind on the discharge is likely to be small or negligible. This is not expressed by Eq.2. In addition, the tests with dikes indicate that for higher crest walls (larger h_c / H_{m0} but with equal crest height R_c/H_{m0}) the influence of wind increases slightly. This is also consistent with the visual observations that for higher crest walls the vertical motion of the water in front of the crest wall is more evident. Therefore, a modified expression for the influence factor for wind that leads to no effect of wind for the situation without a protruding crest wall ($\gamma_w = 1$ for $h_c = 0$) and larger influence factors for higher crest walls is examined. The following expression has been calibrated based on the present data:

$$\gamma_w = 1 + 0.075 \frac{h_c}{H_{m0}} (q^*)^{-0.3} \tag{3}$$

The left panel of Figure 7 shows all data (with $q^* > 1.0 \cdot 10^{-6}$) with the influence factor for wind on the vertical axis but now with $(h_c / H_{m0})^{1/(-0.3)}\, q^*$ on the horizontal (logarithmic) axis, together with Eq.3.

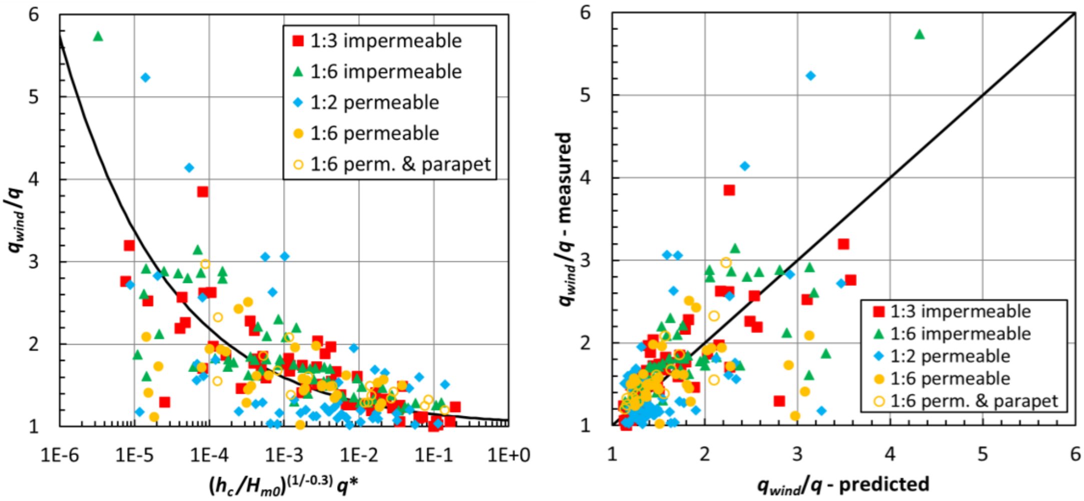

Figure 7. Influence factor for wind. Left panel: Data with newly proposed black curve of Eq.3. Right panel: Measured versus calculated influence factors for wind.

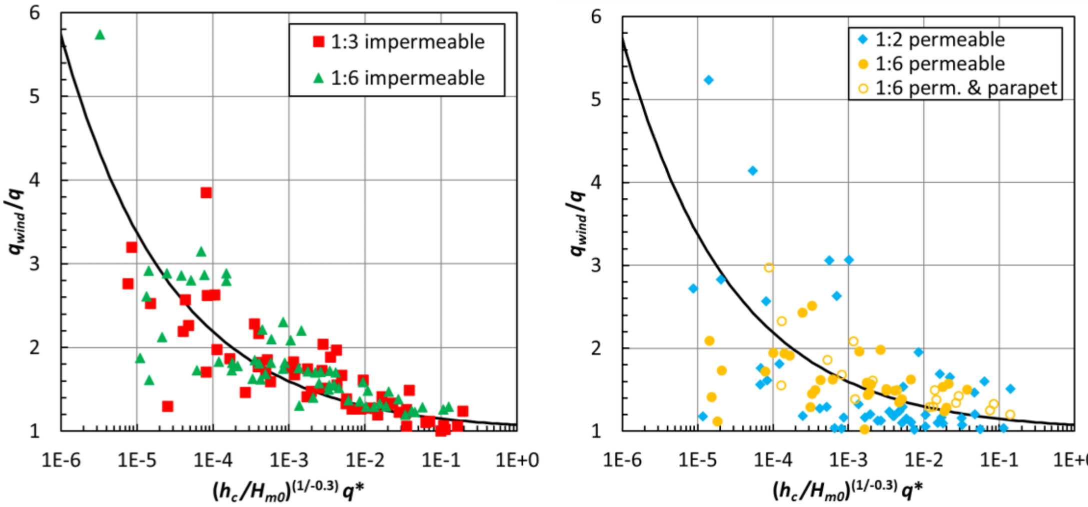

Figure 8. Influence factor for wind for various slopes. Left panel: Dikes; Right panel: Breakwaters; black curves denote newly proposed Eq.3.

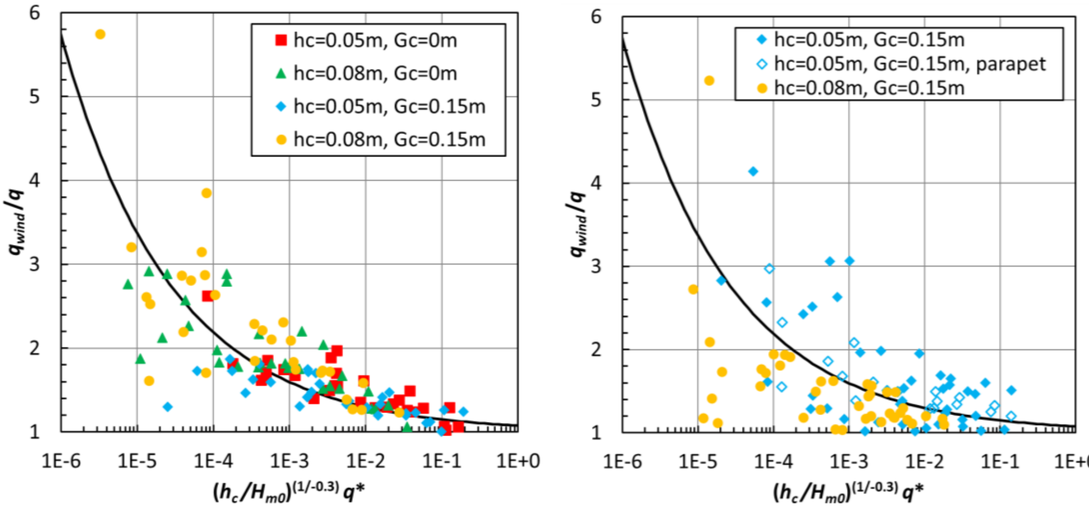

Figure 9. Influence factor for wind for various crest wall configuration. Left panel: Dikes; Right panel: Breakwaters; black curves denote newly proposed Eq.3.

Table 1 shows the RMSE values for the data shown in Figure 7 (right panel shows the measured versus calculated influence factors using Eq.3), calculated using the following expression:

$$\text{RMSE} = \sqrt{\frac{\sum_{i=1}^{n_{tests}} (log(\,q^*_{measured}) - log(q^*_{calculated}))^2}{n_{tests}}} \tag{4}$$

where n_{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. Table 1 shows that the RMSE clearly reduces using Eq.3 compared to Eq.2 (RMSE from 0.726 to 0.511 for all tests). Preference is given to Eq.3 not only because the RMSE values reduce, but also because Eq.3 leads to no effect of wind in absence of a crest wall.

Table 1. RMSE per structure type using Eq.2 or Eq.3.

Structure type	Eq.2	Eq.3
Dikes	0.560	0.430
Rubble mound breakwaters	0.882	0.592
Both	0.726	0.511

In Figure 8 the same data as in the left panel of Figure 7 is shown but now grouped by the type of structure. The left panel of Figure 8 shows the data obtained for the dike slopes and the right panel shows the data from the breakwater slopes. Comparing these two panels shows that spreading for data obtained for dikes is somewhat less than for the data from breakwaters. Both panels indicate that there is no dependency of the slope angle on the influence factor for wind.

Figure 9 shows the data grouped by position (G_c) and height (h_c) of the crest wall. The left panel for dikes clearly shows a clear improvement compared to the left panel of Figure 5. The data in the right panel for breakwaters is somewhat closer to the curve (Eq.3) than in the corresponding right panel of Figure 5, but the data for the highest crest wall is more often lower than the expression, while the data for the lower crest wall is on average somewhat higher than the expression. Nevertheless, for both structure types, Eq.3 is closer to the data than Eq.2 (compare panels of Figure 5 with those of Figure 9). It is concluded that based on physical reasoning and comparison with the data, Eq.3 is an improvement compared to Eq.2.

The measured influence factors have been obtained by comparing the measured discharges from tests with the same conditions, but with and without a paddle wheel. This makes the analysis rather insensitive to the applied wave generation technique, model set-up, and measurement techniques for waves and overtopping discharges. Nevertheless, it is useful to verify how close the measured discharges are to those obtained from other test programmes. For this purpose, use is made of the following empirical expressions to predict wave overtopping discharges (q in m^3/s/m) for dikes and breakwaters, based on TAW (2002), Van der Werf and Van Gent (2018) and Van Gent *et al* (2022).

$$\frac{q}{\sqrt{gH_{m0}^3}} = 0.067\ \gamma_b\ \xi_{m-1,0}\ \sqrt{cot\alpha}\ \ exp\left[-\frac{4.75\ (R_c - 0.5H_{m0-swell})}{\gamma_\beta\ \gamma_b\ \gamma_f\ \gamma_v\ \gamma_p\ \ H_{m0}\ \xi_{m-1,0}}\right] \tag{5}$$

with a maximum of:

$$\frac{q}{\sqrt{gH_{m0}^3}} = 0.016\ s_{m-1,0}^{-1}\ \ exp\left[-\frac{2.4\ (R_c - 0.4H_{m0-swell})}{\gamma_\beta\ \gamma_b\ \gamma_f\ \gamma_v\ \gamma_p\ \ H_{m0}}\right] \tag{6}$$

where the wave height for swell $H_{m0\text{-}swell}$ (in addition to sea state with wave height H_{m0}) was zero in the present tests since only one sea state was present per storm condition. The influence factor for oblique waves (γ_β) and the influence factor for a berm (γ_b) are equal to one since the present tests were with perpendicular waves while the structures did not contain a berm. A recurved parapet is mainly effective for vertical breakwaters (caissons) and not for crest walls on rubble mound breakwaters and therefore the influence factor for a recurved parapet (γ_p) was set to one as well. For the dike configurations with a smooth impermeable slope, the influence factor for roughness (γ_f), was also one. For the breakwater configurations the expression for roughness as applied in Van Gent *et al* (2022) has been used, although the coefficient 0.7 from the original expression was calibrated to reduce the bias between data from the breakwater tests and the expression (coefficient modified from 0.7 to 0.63):

$$\gamma_f = 1 - 0.63 \left(\frac{D_{n50}}{H_{m0}}\right)^{0.1} \quad (7)$$

If a constant value for the roughness factor would be applied, the optimal value would be $\gamma_f = 0.45$, but this would lead to a larger value for the RMSE. Using Eq.7, the influence factor for roughness varies between $\gamma_f = 0.429$ and $\gamma_f = 0.474$ for the tested breakwater configurations.

For the influence of the crest wall (γ_v) on a dike a fixed value was applied ($\gamma_v = 0.85$) while for the crest walls on a breakwater, the expression as applied in Van Gent *et al* (2022) has been used:

$$\gamma_v = 1 + 0.45 \left(\frac{R_c - A_c}{R_c}\right) \quad (8)$$

Using Eq.8, the influence factor for the crest wall varies between $\gamma_v = 1.064$ and $\gamma_v = 1.2$ for the tested breakwater configurations.

Figure 10 shows the comparison between the measured wave overtopping discharges (left panel: without wind; right panel: with wind/paddle wheel) and the calculated overtopping discharges. The values for the RMSE are 0.385 for the dike configurations and 0.379 for the breakwater configurations for conditions without wind and 0.405 and 0.381 respectively for those with wind (with paddle wheel). Figure 10 indicates that the expressions on average lead to underestimates for the 1:6 dike configurations and overestimates for the 1:3 configurations. For the breakwater configurations the errors do not show this clear slope dependency. Although the values for RMSE are almost twice as large as those obtained by Van Gent *et al* (2022) for breakwater configurations with crest walls and berms, the differences between the calculated and measured values generally stay within a factor 10. This indicates that in the present tests the mean overtopping discharges are generally in line with those calculated with the described prediction method, but the spreading is larger than in earlier tests with crest walls by Van Gent *et al* (2022).

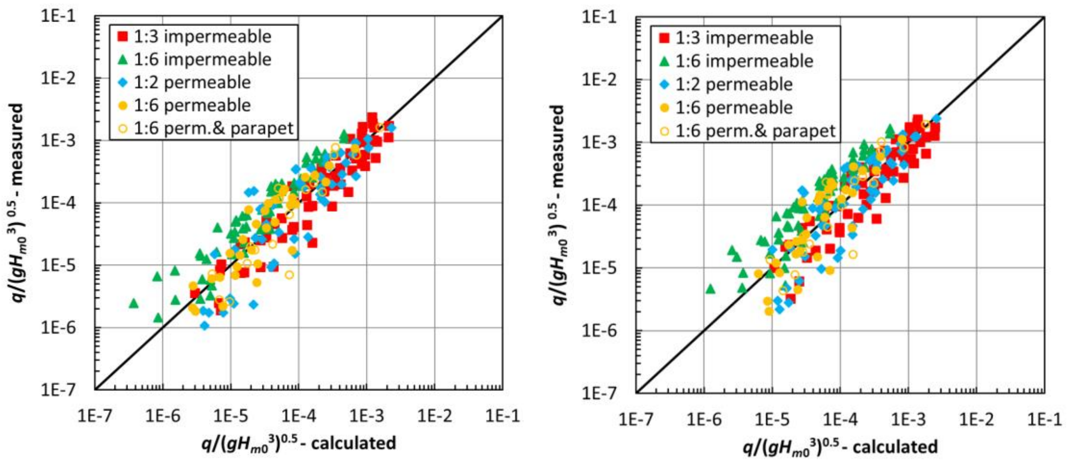

Figure 10. Measured versus calculated overtopping discharges (left panel: without wind; right panel: with wind / paddle wheel).

CONCLUSIONS AND RECOMMENDATIONS

Physical model tests have been performed to assess the maximum influence of wind on wave overtopping discharges at dikes and rubble mound breakwaters. Assuming that all water above the tip of a crest wall on top of a dike or rubble mound breakwater will be blown over the structure by strong onshore wind, tests with and without a paddle wheel above the crest wall to collect all water exceeding the top, provide the influence factor for wind on wave overtopping discharges. Dike configurations with two different slope angles and four configurations of a crest wall (two heights and two positions) were studied. Rubble mound breakwater configurations with two different slope angles and three configurations of the crest wall were tested (two heights, one with a recurved parapet/bullnose). Based on the tests the following conclusions can be drawn:

- Using the approach with a paddle wheel to study the maximum influence of wind (rather than to simulate wind itself), is likely to provide estimates that are close to the actual influence of wind for small overtopping discharges because it is realistic that during a storm with onshore wind all water that exceeds the crest level is blown over the structure.
- The influence of wind on wave overtopping discharges at coastal structures with a crest wall on top depends strongly on the overtopping discharge itself (characterized here by the overtopping discharge without wind); for large overtopping discharges the influence of wind is small to negligible; for small overtopping discharges the influence is large and not negligible.
- The hydraulic boundary conditions (*e.g.* wave height, wave steepness) and structure configurations (*e.g.* slope angle, roughness and permeability of the slope, freeboard) affect the wave overtopping discharge but once these effects are accounted for in the estimates of the overtopping discharge without wind, the maximum influence of wind is dominated by the discharge itself and these parameters hardly show additional effects on the maximum influence of wind.
- Within the relevant range of overtopping discharges ($q^* > 1.0 \cdot 10^{-6}$) the expected wave overtopping discharge at coastal structures with a crest wall can be up to a factor of about 5 times larger than for situations without wind. For smaller discharges this factor can be much higher; a factor of about 10 has been measured, but the lowest discharges are less relevant and/or potentially affected by scale effects.
- The influence of wind on wave overtopping depends also on the height of the crest wall, with a larger effect for higher crest walls. Taking this effect into account improves the estimates for the maximum influence of wind, especially for the tested dike configurations.
- The present study provides the following updated empirical expression to estimate the influence of wind on wave overtopping discharges (equal to Eq.3):

$$\gamma_w = 1 + 0.075 \frac{h_c}{H_{m0}} (q^*)^{-0.3}$$

  where h_c is the height of the protruding part of the crest wall and q^* is the non-dimensional wave overtopping discharge without the presence of wind. The estimate of wave overtopping discharges in presence of onshore wind can be obtained by $q_w^* = \gamma_w\, q^*$ while estimates of the overtopping discharges at dikes and rubble mound breakwaters without wind (q^*) can be obtained using Eqs.5-8. The expression leads to no effect of wind on the overtopping discharges in absence of a protruding crest wall.

It is recommended to take the influence of wind into account in estimates of wave overtopping at dikes and breakwaters because the effects of wind can be large and are not negligible. Furthermore, it is recommended to also analyse the influence of wind on wave overtopping parameters other than the overtopping discharge.

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