CHAPTER 172

Wave Overtopping of Vertical Structures including Wind Effect

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ABSTRACT

The influence of both wave breaking and wind on the wave overtopping discharge of vertical sea-walls were assessed. In a qualitative way the influences of wave breaking agreed with the trends as pointed out by Goda (1985). The influence of wind on overtopping appears to be only relevant in specific conditions related to wave breaking. Even in these conditions the influence appeared to be smaller than expected.

1 INTRODUCTION

Intuitively, the influence of wind on wave overtopping is regarded to be relevant, but it is not clear to what extent. There are several types of possible wind influences. In this study the possible influence of wind on wave overtopping was subdivided as follows:

- Wind may cause overtopping of the part of the breaker spray which would have fallen back into the sea in a situation without wind;
- Wind may cause the breaker type to change by deforming the incident waves;
- Wind may cause overtopping by so-called "basic" spray, that is, spray which is generated by the wind at open sea.

Wind influence on the water level at the structure is assumed to be accounted for in the assessment of the design water level. A problem which is closely linked to the different (though coupled) wind influences is the measurability, especially the appropriate interpretation of measurements in a small scale model.

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2 SET-UP OF THE INVESTIGATION

Due to the large number of relevant parameters and the very complex water motion at the structure, a theoretical approach to the wave overtopping problem is hardly feasible. Therefore, it was decided to perform physical model tests in a wave flume in order to develop a set of empirical formulae for overtopping. However, in a physical scale model one should be aware of the following scale effects:

- Water drops in the air are not to scale: they are far too big in the model.
- The drop transport capacity of wind is not known.
- The influence of wind on the wave-form depends on the shear stress on the water surface, therefore the shear velocity of the wind should be to scale.
- The "flip-through" breaker type (see Section 4.4) probably occurs more often than in prototype, because the air entrainment is not to scale. At this point also the salinity of water is of significance, which makes the problem very complicated.

Due to these scale effects the three possible wind influences could not be modelled at one time in a physical scale model. For this reason, it was decided to focus the physical model investigation on an accurate modelling of the influence of wind on the breaker-spray transport only. The other two wind effects were studied theoretically.

The following basic dimensionless parameters are identified:

Mean overtopping discharge $$Q = q/\sqrt{gH_{os}^3} \qquad (2.1)$$

Relative crest height $$R = R_c/H_{os} \qquad (2.2)$$

Wave steepness $$S = H_{os}/L_{op} \qquad (2.3)$$

Relative local water depth $$D = d_t/H_{os} \qquad (2.4)$$

With:

g	=	acceleration due to gravity (= 9.81)	(m/s^2)
d_t	=	water depth at the structure	(m)
H_{os}	=	significant wave height at deep water	(m)
L_{op}	=	wave length in deep water, based on T_p	(m)
q	=	average overtopping discharge per metre structure width	(m^2/s)
R_c	=	crest level with respect to SWL	(m)
T_p	=	wave period at the peak of the spectrum	(s)

These dimensionless parameters will provide a basis both for the set-up of test programmes and for the development of (empirical) overtopping formulae. A generally applicable form of the wave overtopping formula is the basic relationship between the dimensionless overtopping discharge Q and the relative crest height R:

$$Q = c_1 \exp(c_2 R) \tag{2.5}$$

The coefficients c_1 and c_2 are also dimensionless and may be dependent upon all parameters except Q and R. A common way to present a measured or computed relationship between Q and R is a plot of log(Q) (or Q on a logarithmic scale) against R. Formula (2.5) implies that this type of presentation yields a straight line. In some other approaches the lines are curved very slightly downward, as indicated by Battjes (1974, for slopes) and Goda (1985, for vertical walls). However, the proposed straight line has proven to be a very good approximation and is fairly comfortable in computations.

The wave height H_{os} is very important in the basic overtopping formula and should be accurately assessed. If overtopping data are available in which the value of the wave height is uncertain within a band between plus and minus 10%, then the overtopping data lie within a band of which the maximum values are 3 to 6 times the minimum values, even when all other measured parameters are exact!

3 ANALYSIS OF GODA's GRAPHS

Goda (1985) presents six separate graphs for wave overtopping of a vertical wall at specific combinations of the foreshore slope and the wave steepness. Compared with other information on wave overtopping in literature these graphs have proven to be very well applicable. The dimensionless overtopping discharge is plotted on a logarithmic scale against the relative local water depth, identifying lines for constant values of the relative crest height. Note the small difference between the definition of the dimensionless overtopping discharge used by Goda and the one used in this paper:

$$\text{Goda: } Q^* = q/\sqrt{2gH_{os}^3}; \qquad \text{this paper: } Q = q/\sqrt{gH_{os}^3}$$

The information in the graphs was tabulated for $D \geq 1.0$. This table provided the opportunity to analyze the influences of the relative crest height R, the local water depth D, the wave steepness S and the foreshore slope $\cot\alpha_f$ on the dimensionless overtopping discharge Q. These aspects are discussed below in a qualitative way.

For constant values of $\cot\alpha_f$, S and D the relation between Q and R is well approximated by an exponential relation. A linear regression analysis has yielded the coefficients c_1 and c_2. The value of c_1 appeared to be almost independent of $\cot\alpha_f$, S and D: $c_1 \approx 0.045$. This result proves to be very convenient, because the analysis of Goda's graphs may now be restricted to c_2 as a function of $\cot\alpha_f$, S and D. The optimum values of c_2 have been calculated again, but now with the assumption that $c_1 = 0.045$. The resulting c_2-values have been plotted against D, identifying the different combinations of S and $\cot\alpha_f$, see

Figure 1.

In this Figure the following trends may be identified:

- The influence of S and D are approximately independent of $\cot\alpha_f$ if $D > 3.0$. In other words: the influence of the foreshore slope on wave overtopping is negligible for relatively deep water. This seems quite reasonable.
- If $1.0 \leq D < 3.0$, the c_2-values (read: overtopping discharges) for $\cot\alpha_f = 10$ are greater than those for $\cot\alpha_f = 30$. This also seems reasonable because at a steeper foreshore the waves have less time and space for breaking. For milder foreshore slopes more wave energy is lost on the foreshore before the waves reach the wall.
- With increasing D the c_2-values decrease. This is probably caused by the decreasing peakedness of the wave crests, but this effect is intuitively expected to be smaller than Goda's graphs suggest.
- The influence of S decreases with an increasing D. However, even at very deep water this influence seems fairly significant in Goda's graphs, whereas the influence of the wave steepness is expected to vanish at already intermediate relative water depths.

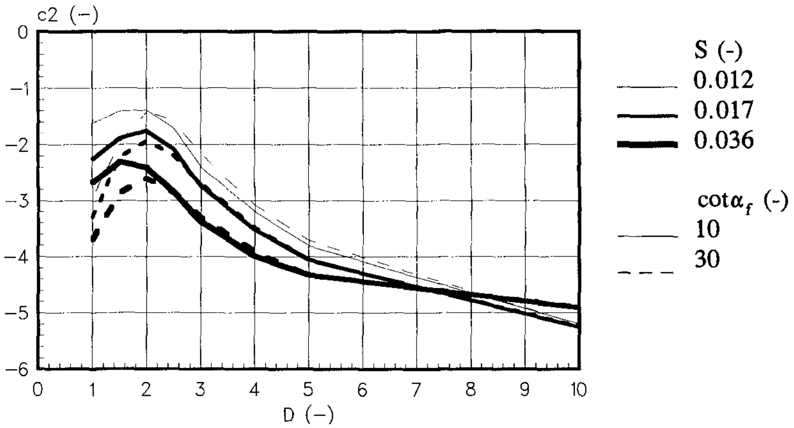

Figure 1. Influence of wave steepness, relative water depth and foreshore slope

4 TESTS ON THE INFLUENCE OF BREAKER SPRAY TRANSPORT

4.1 Model set-up

The objective of this model investigation was to assess the maximum possible influence wind can have on the mean wave overtopping discharge. The experiments were carried out in the Scheldt Flume of DELFT HYDRAULICS, De Voorst location in October 1993.

The foreshore was 8 m long and had a slope of 1:20. The vertical structure was 0.40 or 0.60 m high and had a sharp crest. Five vertical plates were installed perpendicularly to the structure in order to prevent or reduce transverse oscillations in the flume. In order to prevent the loss of spray out of the flume the side walls of the flume were heightened after which a sort of a tunnel was created by covering this part of the flume with a horizontal board.

Scale effects in the total volume of water rising above the crest are intuitively considered to be small and the essential assumption in this investigation is that these scale effects may be neglected. In the second half of the test programme all spray rising above the crest was mechanically transported over the crest by means of a rotating paddle wheel, whose dimensions are presented in Figure 2. The optimum rotation speed is the maximum speed at which there is still enough time for all the water collected by a paddle to drain from the paddle and be collected behind the crest. This rotation speed appeared to be 21.4 revolutions per minute, which implied that the average number of strokes (paddles passing the crest) was 4.3 per second.

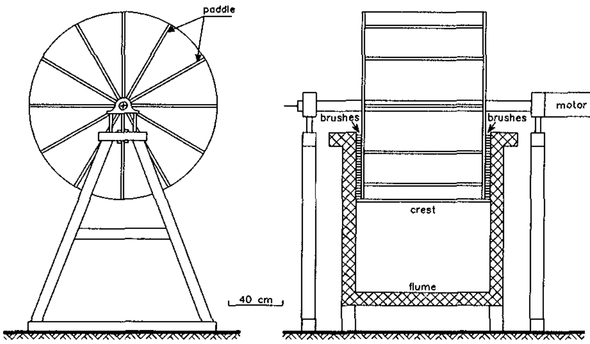

Figure 2. Paddle wheel for mechanical breaker spray transport

Irregular waves were generated according to a JONSWAP spectrum. Two wave gauges at deep water were used for the wave analysis. The signals of the wave gauges were sampled at 25 Hz. The test duration was 30 minutes.

The wave overtopping was characterized by the average overtopping discharge only. The storage capacity of the reservoir was about 300 litres. In addition, pumps with a constant discharge were used at intervals when this was necessary in order to prevent the reservoir from overflowing. The total pumping time was measured with a stopwatch. A water level gauge was installed at the reservoir for the measurement of water levels before and after each test.

The test programme consisted of two series. In the first series the overtopping was measured without the paddle wheel. Table 1 presents the target values of the dimensionless parameters. However, not all the possible combinations were tested. The first test series consisted of 18 tests. After installation of the paddle wheel this series was repeated, yielding the second test series. During the four tests with D = 1.5, video recordings of the water motion at the structure and of the performance of the paddle wheel were made.

Parameter	Notation	Values in test programme
Relative local water depth	D	1.0 1.5 2.0 3.0
Relative crest height	R	1.4 1.7 2.0 2.3
Wave steepness	S	0.02 0.04

Table 1. Target values of dimensionless parameters in overtopping tests

4.2 Analysis method

In order to describe the influence of a specific dimensionless parameter, such a parameter should have been varied while keeping all other relevant dimensionless parameters unchanged. Although the test programme was set up to account for such series, the actually measured wave conditions were not exactly equal to the desired values. Interpolation techniques were applied in order to obtain representative test results at the target values of the dimensionless test conditions.

For the quantitative analysis of the influence of spray transport we introduced a so-called "spray transport factor" W_s, defined as:

$$W_s = \frac{Q_{\text{with spray transport}}}{Q_{\text{without spray transport}}} \tag{4.1}$$

W_s is calculated on the basis of the test results at the target values of the dimensionless parameters.

In the analysis the definition of the significant incident wave height was based on spectral analysis. The incident and reflected wave energy (m_{0i} and m_{0r}) were calculated from a cross-spectrum analysis of the recordings of the two wave gauges. The significant incident wave height was defined as:

$$H_{os} = 4\sqrt{m_{oi}} \tag{4.2}$$

4.3 Observations during tests

The water motion at the structure and the process of wave overtopping vary strongly, even during a single test:

- Ordinary overtopping occurs when the crest of a standing wave reaches higher than the crest of the structure: so-called "green water" flows over the crest.
- Collision of a steep wave crest with the wall results in an upward accelerating compact body of water, which spurts up moderately high and almost completely overtops the crest.
- When air entrainment at the wall is about to occur, the collision of the wave crest with the wall will result in a so-called "flip-through". The sound of the collision is still more like "zip" instead of a bang. A thin water sheet spurts up extremely high, almost vertically. The water sheet rapidly disperses into drops. About half of the upspurting water volume falls back into the flume.

- Wave slamming together with a loud bang will occur when a wave crest hits the wall about horizontally and some air is entrained. The water spurts up high, in big drops, in a great variety of directions.
- When the wave crest tip is already moving downward as it collides with the wall and much air is entrained, there is no clear "bang" heard any more. There is a lot of turbulence and the water spurts up chaotically and moderately high.
- When the wave is already broken as it reaches the wall almost no "bang" will be heard any more, partially due to the fact that the breaking wave itself already produces noise. The water does not spurt up very high and most of the big drops fall back into the flume.

The sequence of the phenomena described above corresponds with the transition from relatively deep local water to relatively shallow local water at the structure. In a single test with irregular waves this development corresponds with the transition from relatively small waves to relatively large waves. In tests with an intermediate relative local water depth (D = 1.5 to 2.0) this whole variation of phenomena could be observed.

The effectiveness of the mechanical spray transport was assessed by means of extensive visual inspections during the tests. These inspections led to the estimation that at least 90% of the volume of water which should have been transported by the paddles actually was transported. (Of course one can see some water falling back into the flume but it is very hard to conclude whether or not this water actually had risen higher than the crest). This estimated effectiveness is considered to be amply sufficient for this investigation.

4.4 Tests results

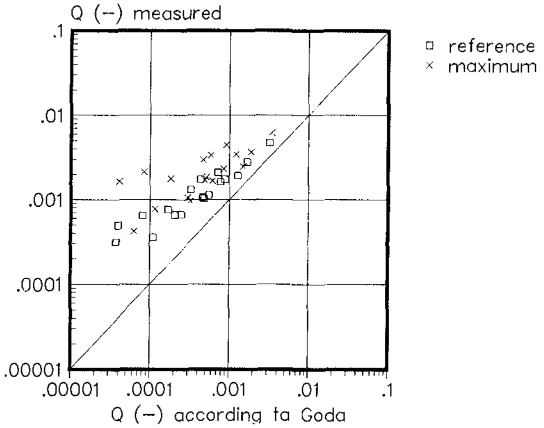

Figure 3. Comparison of test results with values based on Goda's graphs

Figure 3 shows the comparison between the measured overtopping and the calculated overtopping on the basis of Goda's graphs. The calculated values were based on the measured hydraulic conditions. Most of the measured overtopping discharges appear to be considerably greater than the values according to Goda's lines.

The results for tests without spray transport are presented in Figure 4 and 5.

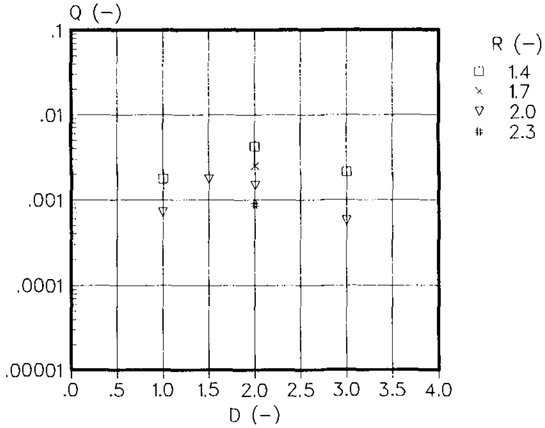

Figure 4. Influence of relative local water depth and relative crest height without mechanical breaker spray transport for S = 0.02

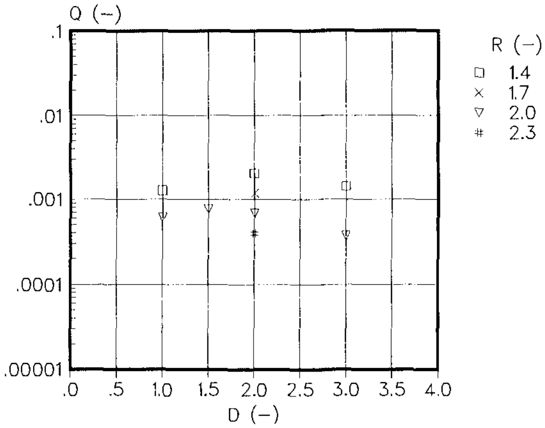

Figure 5. Influence of relative local water depth and relative crest height without mechanical breaker spray transport for S = 0.04

From these Figures the following conclusions may be drawn:

- The overtopping discharge clearly decreases with an increasing relative crest height. The four measurement points from the test series with D = 2.0 are at equidistant positions. This confirms the assumed linear relationship between log(Q) and R for tests without spray transport. (From a closer analysis it is concluded that the c_1 and c_2 values are not constant for the different values of

D. The c_1 values may be approximated by a single constant of about 0.050 for D = 2, which agrees with the value based on Goda's graphs, but for D = 1 the measurements show a different tendency.)

- The overtopping decreases with increasing wave steepness, which agrees with Goda's graphs. This influence is the most obvious for tests with D = 2.0.
- The overtopping discharge seems to reach a maximum at D = 1.5 to 2.0, which agrees with Goda's graphs. For S = 0.02 this maximum is clearer than for S = 0.04.

Figure 6 and 7 show the results for the spray transport factor.

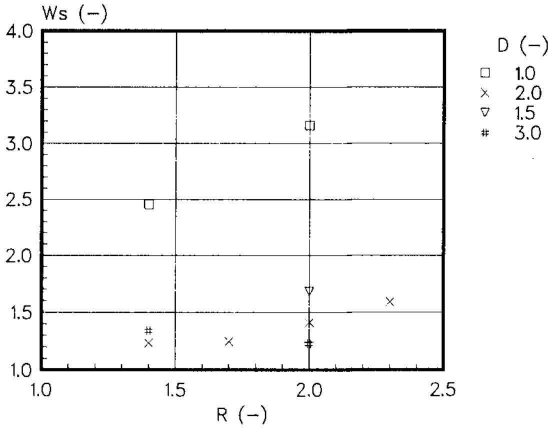

Figure 6. Measured spray transport factor for S = 0.02

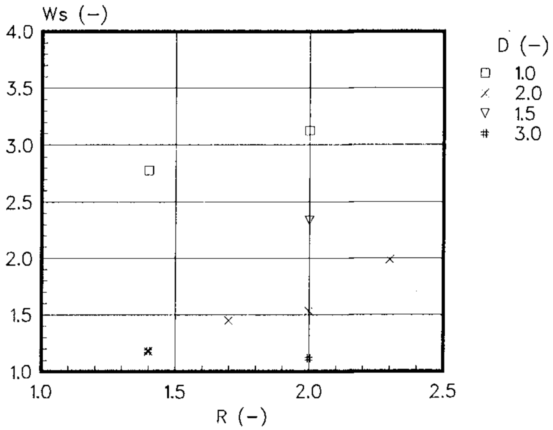

Figure 7. Measured spray transport factor for S = 0.04

All tests with spray transport resulted in more overtopping than the corresponding tests without spray transport ($W_s > 1$). This seems a trivial conclusion, but a greater scatter in the measurement results could have caused less consistency. The maximum value of the spray transport factor was about 3.2. The overtopping discharge in tests with spray transport still decreased with an increasing relative crest height, but the linear relationship between log(Q) and R appeared to be no longer valid.

From Figure 6 and 7 it may be concluded that the influence of spray transport increases with increasing relative crest height. This may be explained by the fact that the horizontal velocity of the water rising above the crest is smaller for a higher crest. This implies that for a higher crest a larger portion of the water falls back into the flume and the effect of spray transport may therefore be larger.

The spray transport factor sharply decreases with increasing relative local water depth. The explanation of this trend is similar to the explanation of the relative crest height influence. In shallower water, more waves have already broken when they reach the structure. The spray, resulting from collision of broken (or breaking) waves and the wall, has a great variety of directions. A relatively large portion of the water volume (spray) rising above the crest of the structure falls back into the flume. The influence of spray transport is considerable.

A comparison between Figure 6 and 7 leads to the conclusion that the influence of spray transport hardly depends on the wave steepness. In only a few tests the spray transport factor appears to be greater for the steeper waves.

5 INFLUENCE OF WIND ON WAVE OVERTOPPING

5.1 Additional breaker-spray transport due to wind

The physical model investigation has yielded fairly accurate quantitative information about the maximum additional breaker-spray transport due to wind. The upper limit of this wind occurs for breaking or broken waves and appears to be about 3 times the overtopping discharge for a vertical wall without wind. This factor may be regarded to be relatively small; it is of the same order as the scatter in overtopping data from other investigations. Therefore, the assessment of the relation between the actual wind speed and the additional portion of transported breaker spray is not considered to be very useful. The relatively small factor also implies that the error in past investigations due to the absence of additional spray transport was small.

5.2 Influence of wind on the wave breaker type

The effect of wind on the wave height at the structure should be accounted for in the substitution of the significant wave height term in the overtopping formula. However, wind may also change both the wave shape and the wave breaker type at the structure. As yet, there are no computation methods to assess this effect. Qualitatively, the influence of onshore wind on wave breaking can be summarized as follows (Douglass, 1989):

- Waves tend to break earlier (in deeper water).
- The type of wave breaking tends towards a spilling breaker type.

Douglass (1989) discusses only waves breaking on a mild slope, such as a beach. By a first approximation, the above tendencies may also be assumed to be valid for a mild slope ending in a vertical wall. However, the wave reflection with a vertical wall differs completely from that without a wall. The influence of wave reflection on the wave deformation by wind may be significant but is as yet unknown.

The wind effects on wave breaking are similar to the influences of a decrease in local water depth together with a decrease in foreshore slope. A decrease in local water depth causes the wave-breaking location to change: a decrease in foreshore slope the wave breaker type to change. This implies that an approximation of the influence of wind on wave breaking may consist of a reduction in both the local water depth and the foreshore slope in the overtopping formulae. This approximation is first roughly quantified and substituted afterwards into the overtopping formula.

On the basis of information from literature, Douglass suggests that the water particle velocity in the wave crest be increased by 3% of the wind speed to account for the onshore wind. The wave is assumed to break as soon as this water particle velocity exceeds the wave phase velocity. The reduction of the water depth at which wave breaking occurs may be approximated by:

$$\frac{d_b \text{ (with wind)}}{d_b \text{ (without wind)}} = \left(1 + 0.03\frac{u_{a,10}}{\sqrt{g d_b}}\right)^2 \qquad (5.1)$$

with:

d_b = water depth at which wave breaking occurs (m)

$u_{a,10}$ = wind speed at 10 m height (m/s)

For a numerical example of this wind effect a situation is considered in which waves break at a water depth of 5.0 to 10.0 m if there is no wind. Now a heavy onshore storm is considered in which the wind speed reaches 30 m/s. From Equation (5.1) it is concluded that the wind effect causes the same waves to break in 1.27 to 1.19 times deeper water than without wind. This change in location of wave breaking would also be caused by a reduction in water depth by about 20%.

The wave breaker type is characterized by the breaker parameter ξ_{op}, which is in this case determined by the slope of the foreshore instead of the slope of the structure:

$$\xi_{op} = \tan\alpha_f / \sqrt{S} \qquad (5.2)$$

However, the type of wave breaking is very difficult to quantify and the influence of wind on the breaker type is only qualitatively described. For a first indication, the influence of wind on the breaker type is approximated arbitrarily by a reduction in foreshore slope by 20%. This effectively decreases the value of the breaker parameter, thereby representing a transition towards a more spilling breaker type. In view of this arbitrary decision, the small influence of the reduction of the foreshore slope on the wave-breaking location has been neglected.

In order to estimate the order of magnitude of this wind effect on overtopping the influence of the representative changes in the foreshore on overtopping is assessed using Goda's lines. Figure 8 presents the wave-deformation factor due to wind, defined as:

$$W_d = \frac{Q_{\text{with wave deformation due to wind}}}{Q_{\text{without wind}}} \tag{5.3}$$

From this Figure it can be concluded that due to this wind effect, the overtopping discharge increases by a factor of about 3.0 for a relatively high crest and an intermediate relative local water depth ($D \approx 3$). On the other hand, the overtopping discharge decreases for a smaller relative local water depth ($1 \leq D \leq 2$).

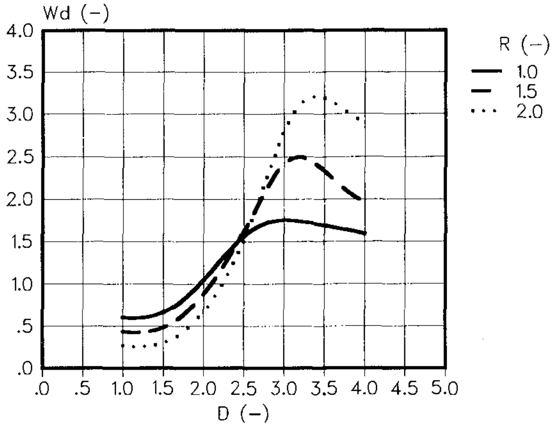

Figure 8. Wave deformation factor due to wind for $\cot\alpha_f = 20$ and $S = 0.03$

The wind effect in question seems to have the same order of magnitude as the influence of wind on the breaker-spray transport, although it reaches a maximum under different circumstances. However, the verification tests showed that the influence of the relative water depth on overtopping is less pronounced than Goda's graphs suggest. This implies that the maximum factor of increase in overtopping due to wave deformation is probably less than 3. Still, one should be aware of the rough approximations which form the basis of this conclusion. This wind effect requires further investigation before it can be quantified accurately.

5.3 Contribution of basic spray to overtopping

In storm conditions the air above the sea surface is filled with spray. This spray is called "basic spray" in this paper. There is a dynamic equilibrium of the spray distribution over the height above the sea surface. This equilibrium is maintained by spray generation and turbulence in the air on the one hand and gravity on the other hand. The equilibrium is disturbed at the transition from sea to land, where the spray starts to be deposited. The amount of this type of overtopping was compared with the overtopping by waves (including breaker spray) and rainfall.

The following simple exponential equation for the spray concentration is applied:

$$c = c_0 \exp\{-\frac{w}{D_t}(z - z_0)\} \tag{5.4}$$

With:

c	=	volumic spray concentration (-)
c_0	=	average spray concentration at $z = z_0$ (-)
t	=	time (s)
z	=	vertical coordinate, positive upward and $z=0$ at mean sea level (m)
z_0	=	reference level for spray concentration distribution (m)
w	=	average fall velocity of spray (drops) in air (m/s)
D_t	=	turbulent diffusion coefficient (m^2/s)

We introduce a horizontal coordinate x, being zero at the structure crest and negative at the open sea. We assumed a stationary situation. We assumed that the vertical distribution of spray concentration is transported with the horizontal wind velocity u_a, which was assumed to be uniform (independent of z). The total spray discharge (per m crest width) over the crest level R_c is given by:

$$q_b = \int_{R_c}^{\infty} c u_a \, dz \tag{5.5}$$

As soon as this spray distribution passes the structure crest (at $x = 0$), the production of basic spray at $z = 0$ stops. For $x > 0$ we neglect the influence of diffusion and simply assume that all spray falls down, having a velocity w. The total discharge of basic spray falling on the first L metre behind the crest (per metre crest width) is now given by:

$$q_b = \frac{D_t}{w} u_a c_0 \exp\{-\frac{w}{D_t}(R_c - z_0)\} \left(1 - \exp\{-\frac{w}{D_t}\frac{w}{u_a}L\}\right) \tag{5.6}$$

The governing parameters in this formula were chosen as follows. The amount of spray strongly depends on the wind velocity. In the calculations a wind velocity u_a of 30 m/s is applied, representing wind force 11 on the Beaufort scale. The spray fall velocity w strongly depends on the drop size. Since the variety of drop sizes in the spray is very great, the representative fall velocity is difficult to assess. Therefore, two values have been applied, namely 0.5 and 2.5 m/s, representing a low and a high estimate respectively. The turbulent diffusion coefficient D_t is unknown but is at first approximation regarded to be related to the average wind velocity. Two values have been applied, namely 5.0 and 50 m/s. The average spray concentration c_0 at the reference level $z = z_0$ is based on a rough estimation. The concentration of water in air during intensive rainfall is estimated at 10 ml/m^3. This yields $c_0 = 10^{-5}$. The level of z_0 is assumed to be equal to the level of R_c. For the length L behind the structure crest in which basic spray is deposited two values were applied: 50 m and 100 m.

An estimation of the contribution of rainfall to the average overtopping was based on an intensity of 10 mm/hr. A representative value for the wave overtopping discharge was based on $H_{os} = 5$ m, $S = 0.04$, $D = 2.0$ and $R = 1.7$. This yields $Q \approx 0.001$, which results in $q = 0.035$ m^2/s. The results of the computations are presented in Table 2.

	w (m/s)	D_t (m^2/s)	q (l/m/s)	
			L = 50 m	L = 100 m
Basic Spray	0.5	5	0.2	0.5
	0.5	50	0.2	0.5
	2.5	5	0.5	0.6
	2.5	50	1.1	2.0
Rainfall			0.1	0.3
Wave overtopping			35	

Table 2. The contribution of basic spray to overtopping

From these results we concluded that the contribution of basic spray to the overtopping discharge is only slightly greater than the contribution of rainfall and is still negligibly small in comparison with representative wave overtopping discharges.

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