# Wave overtopping events at dikes

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# WAVE OVERTOPPING EVENTS AT DIKES

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**ABSTRACT:** Wave overtopping over dikes may cause dangerous situations, one of which is breaching of dikes. Wave overtopping can be characterised by *mean* overtopping discharges but this does not provide information on individual wave overtopping *events*. This paper focuses on parameters related to individual wave overtopping *events* with a low probability of exceedance. Measurements of velocities and the thickness of water layers have been performed at the crest and inner slope of dikes. The height, width and roughness of the crest have been varied and also the angle and roughness of the inner slope. Based on these tests, prediction formulae have been derived for velocities, the thickness of water layers, and low-exceedance wave overtopping discharges. The estimates of these quantities can be used to characterise the wave loading on the crest and inner slope of dikes, especially as the characteristic wave loading for the initial phase of breaching of dikes initiated by erosion at the crest or inner slope. Besides above mentioned quantities the measurements were used also to obtain more information on for instance mean wave overtopping discharges and on volumes of water within individual overtopping waves.

## 1. INTRODUCTION

Analyses of dike breaching events in The Netherlands and Germany show that most of them were initiated by failure of the inner slopes of dikes. These failures occurred due to wave overtopping which resulted in slip failure of the inner slope or erosion at the inner slope and crest, or, most likely, a combination of both. To study these processes information is needed on wave overtopping. This concerns both *mean* overtopping discharges and parameters which are related to more *extreme* events for a specific wave condition (*i.e.* “events which have a low probability of exceedance during a storm with certain wave conditions”). Wave run-up levels exceeded by 2% of the incident waves are often used to characterise such extreme events. The

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majority of available information for wave overtopping only concerns *mean* overtopping discharges (*e.g.*, Battjes, 1974; De Waal and Van der Meer, 1992; Van der Meer and Janssen, 1995; Van Gent, 1999-b, 2002). Little information is available on parameters which characterise individual wave overtopping events, among which the *extreme* wave overtopping events. The purpose of this study is to obtain estimates for parameters which characterise wave overtopping events with a rather low probability of exceedance within a certain storm:

- Maximum *water layer thickness* during a wave overtopping event, while for x% of the incident waves this event is exceeded ($h_{X\%}$).
- Maximum *velocity* (depth-averaged) during a wave overtopping event, while for x% of the incident waves this event is exceeded ($u_{X\%}$).
- Maximum *discharge* during a wave overtopping event, while for x% of the incident waves this event is exceeded ($q_{X\%}$).
- *Volume* per overtopping wave, exceeded by x% of the incident waves ($V_{X\%}$).

Events exceeded by 1%, 2% or 10% of the incident waves within a certain storm condition are analysed. Figure 1 shows a typical time signal of wave overtopping discharge where the *mean* wave overtopping discharge is 10 l/s/m. This figure shows that the maximum discharge during a wave overtopping event can be much higher, here approximately 2800 l/s/m. This illustrates that information on the *mean* wave overtopping discharge may not provide sufficient information on *extreme* wave overtopping events.

Based on a study using numerical model results the dependencies of above mentioned parameters on for instance the crest height, the crest width, the angle of the inner slope, and the roughness were studied (Van Gent, 2001-a). In addition, physical model tests were performed to verify results from the numerical model study and to obtain prediction formulae. This paper summarises the latter study, which has been reported in detail in Van Gent (2002-b).

## 2. PHYSICAL MODEL TESTS

The physical model tests were performed in a flume of WL | Delft Hydraulics with a length of 55 m, a width of 1 m and a height of 1.2 m. Second-order wave generation in combination with an active wave absorption system was used.

The characteristics of the crest and inner slope were varied. The foreshore (1:100) and seaward slope (1:4) were kept the same for all configurations tested (Figures 2 and 3). The influence of the foreshore and seaward slope was studied in previous projects with a similar model set-up (*e.g.*, Van Gent, 1999-b, 2001-b). Five of the tested configurations concern dike geometries, of which the main parameters are summarised in Table 1.

For each configuration velocities and the thickness of water layers were measured at five positions (see symbols in Figure 3). The first position was at the seaward side of the crest, the second position was at the landward side of the crest and the other three positions were at different levels on the inner slope. For all five dike

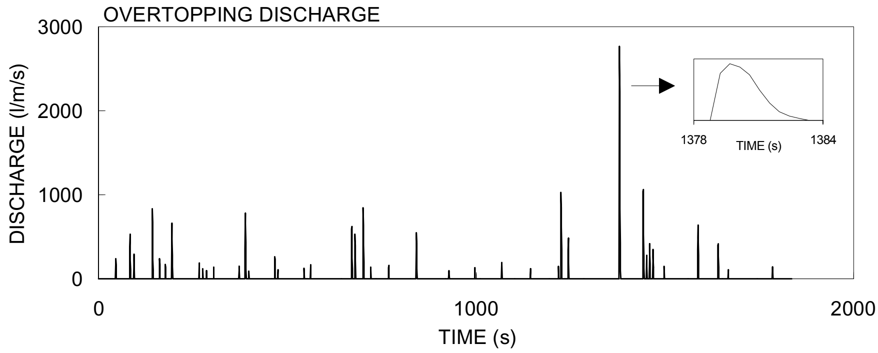


Figure 1 Example of a time signal of overtopping discharge with a *maximum* of about 2800 l/s/m and a *mean* discharge of 10 l/s/m.

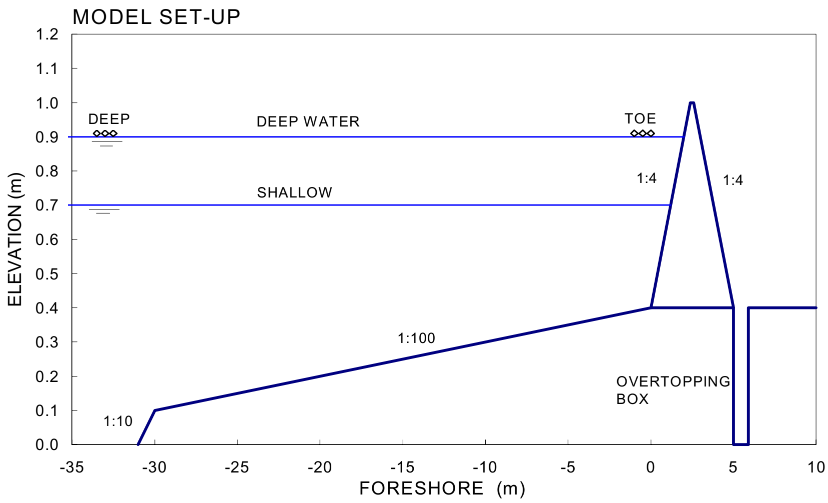


Figure 2 Foreshore 1:100 and a 1:4 slope.

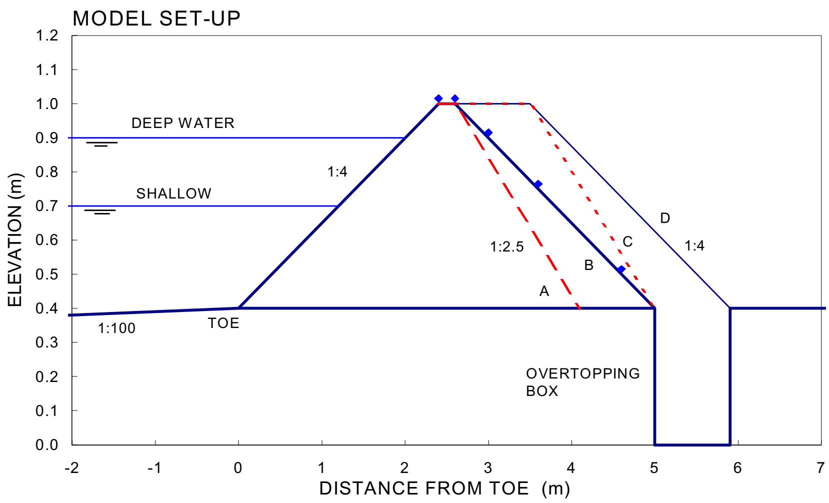


Figure 3 Tested dike configurations (Series A-D).

| Series | Seaward slope | Inner slope | Crest level | Crest width | Roughness (crest and inner slope) |
|---|---|---|---|---|---|
| A | 1:4 | 1:2.5 | 1.0 m | 0.2 m | smooth |
| B | 1:4 | 1:4 | 1.0 m | 0.2 m | smooth |
| C | 1:4 | 1:2.5 | 1.0 m | 1.1 m | smooth |
| D | 1:4 | 1:4 | 1.0 m | 1.1 m | smooth |
| D' | 1:4 | 1:4 | 1.0 m | 1.1 m | rough |

Table 1 Tested dike configurations.

configurations these positions were at elevations of 0.1 m, 0.25 m and 0.4 m below the crest. The velocity meters contain a propeller with a diameter of approximately 10 mm, capable of measuring velocities in the range of 0.5 m/s to 4 m/s for water layers with a thickness larger than 2 mm. Thus, also velocities within layers smaller than the diameter of the propeller can be measured.

At the end of the inner slope an overtopping box was positioned. By measuring the water level elevations inside this box the overtopped volumes of individual overtopping waves could be obtained.

For each dike geometry 18 tests were performed with different wave conditions, including 10 with single-peaked wave energy spectra and 8 with double-peaked wave energy spectra. For each condition 1000 waves were used.

The tests were repeated without the structure in position to obtain the incident waves at the toe of the structure. The method by Mansard and Funke (1980) was used to account for the remaining reflection from the dissipating beach profile at the end of the flume. The wave height $H_s$ (*i.e.*, $H_{1/3}$) and wave period $T_{m-1,0}$ ($T_{m-1,0} = m_{-1}/m_0$ with $m_n = {}_0\int^{\infty} f^n\, S(f)\, df$) of the incident waves at the toe were used for analysis of the tests.

## 3. PREDICTION FORMULAE

The general methodology to obtain prediction formulae for the earlier mentioned parameters characterising wave overtopping events is schematised in Figure 4 and consists of three steps: (1) from wave conditions to (fictitious) wave run-up levels; (2) from wave run-up levels to wave overtopping parameters at the landward side of the crest; and (3) from wave overtopping parameters at the crest to parameters at the inner slope.

The influence of foreshores, angular wave attack, wave energy spectra, seaward slopes, berms, and roughness at the seaward slopes can all be incorporated in the prediction of wave run-up levels. Wave run-up can be calculated for the situation where the dike is high enough to prevent wave overtopping. For situations where this is not the case, the seaward slope is virtually extended to allow considering a fictitious wave run-up level (Figure 5). The difference between this fictitious wave run-up level ($z$) and the crest level ($R_c$) can be seen as a measure for the energy of the overtopping water. Battjes (1974) used this fictitious wave run-up level to characterise mean overtopping discharges and the total overtopping volumes for

**(1) Seaward side**

| | | | | |
|---|---|---|---|---|
| Foreshore, water depth | → | $H_s$ , $T_{m-1,0}$ | → | Wave run-up $z_{2\%}$ (fictitious) |
| Angular wave attack | → | reduction factor $\gamma_\theta$ | → | |
| Seaward slope, berms | → | tan $\varphi$ | → | |
| Roughness seaward slope | → | reduction factor $\gamma_f$ | → | |

**(2) Crest**

| | | |
|---|---|---|
| Wave run-up $z_{2\%}$ | → | Water layer $h_{2\%}$ at crest |
| Crest level $R_c$ | → | Velocity $u_{2\%}$ at crest |
| Crest width $B_c$ | → | Discharge $q_{2\%}$ |
| Roughness at crest $\gamma_{f\text{-}C}$ | → | Volume $V_{2\%}$ per wave |

**(3) Landward side**

| | | |
|---|---|---|
| Water layer $h_{2\%}$ at crest | → | Water layers $h_{2\%}$ at inner slope as function of position. |
| Velocity $u_{2\%}$ at crest | → | |
| Landward slope tan $\varphi_r$ | → | Velocities $u_{2\%}$ at inner slope as function of position. |
| Roughness landward $\gamma_{f\text{-}L}$ | → | |

Figure 4 Schematisation of methodology.

periodic waves. The difference $z$-$R_c$ along with the influence of the width of the crest, and the roughness at the crest can be used to obtain estimates of parameters at the landward side of the crest. The thickness of the water layer and the velocity at the crest determine, together with the angle of the landward slope with its roughness, the water layer and the velocity at the landward slope of the dike.

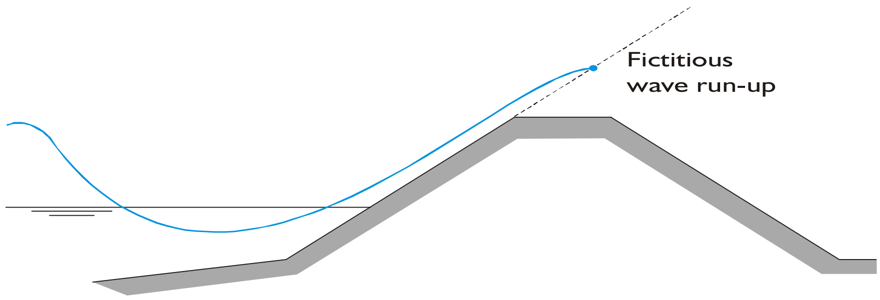


Figure 5 Fictitious wave run-up as measure for wave overtopping parameters.

To predict wave run-up, for situations with deep or shallow water at the toe, the following formula can be used (Van Gent, 2001-b).

$$z_{2\%} / (\gamma\ H_s) = c_0\ \xi_{s,-1} \qquad \text{for } \xi_{s,-1} \leq p$$
$$z_{2\%} / (\gamma\ H_s) = c_1 - c_2 / \xi_{s,-1} \qquad \text{for } \xi_{s,-1} \geq p \qquad (1)$$

where $H_s$ is the significant wave height (*i.e.*, $H_{1/3}$) of the incident waves at the toe of the structure. The reduction factor $\gamma$ $(\gamma = \gamma_f \gamma_\beta)$ takes the effects of angular wave attack ($\gamma_\beta$) and roughness ($\gamma_f$) into account. Continuity of $z_{2\%}$ and its derivative with respect to $\xi_{s,-1}$ determine $c_2 = 0.25\, c_1^2/c_0$ and $p = 0.5\ c_1\ /\ c_0$. The surf-similarity parameter was defined as $\xi_{s,-1} = \tan\varphi\ /\ \sqrt{(2\pi\ /g \cdot H_s\ /T_{m-1,0}^{\ 2})}$. Table 2 provides the values for the coefficients $c_0$ and $c_1$ , not only for 2% exceedance levels but also for the 1% and 10% exceedance levels.

| Parameter | $c_0$ | $c_1$ |
|---|---|---|
| $z_{1\%}$ | 1.45 | 5.1 |
| $z_{2\%}$ | 1.35 | 4.7 |
| $z_{10\%}$ | 1.10 | 4.0 |

Table 2 Coefficients for wave run-up predictions, using $H_s$ and $T_{m-1,0}$ (Equation 1).

For the thickness of the water layer $h_{2\%}$ , the velocity $u_{2\%}$ , and the discharge $q_{2\%}$ , the following formulae were derived for the landward side of the crest:

$$\frac{h_{2\%}}{H_s} = c'_h \left( \frac{z_{2\%} - R_c}{\gamma_f H_s} \right) \tag{2}$$

$$\frac{u_{2\%}}{\sqrt{gH_s}} = c'_u \ (\gamma_{f-C})^{0.5} \left( \frac{z_{2\%} - R_c}{\gamma_f H_s} \right)^{0.5} \ / \ \left( 1 + c''_u \frac{B_c}{H_s} \right) \tag{3}$$

$$\frac{q_{2\%}}{\sqrt{gH_s^3}} = c'_q \ (\gamma_{f-C})^{0.5} \left( \frac{z_{2\%} - R_c}{\gamma_f H_s} \right)^{1.5} \ / \ \left( 1 + c''_q \frac{B_c}{H_s} \right) \tag{4}$$

The data (and the calibrated formulae) indicate that the friction at the crest ($\gamma_{f\text{-}C}$) and the width of the crest ($B_c$) have no influence on the thickness of the water layer at the crest. The formula for the maximum discharge is obtained by multiplying the formulae for the thickness of water layers and velocities ($q_{2\%}=h_{2\%}\cdot u_{2\%}$).

Equations 2-4 are related to the landward side of the crest for situations with $z_{2\%} \geq R_c$, in which $z_{2\%}$ is obtained using Equation 1. Estimates of these parameters for the seaward side of the crest are similar, although the reduction factor for roughness at the crest is then omitted and the coefficients in Equation 2-4 are different. The tests provide the values of the coefficients given in Table 3.

| | seaward side of crest | | | landward side of crest | | |
|---|---|---|---|---|---|---|
| Parameter | $c'$ | $c''$ | $\sigma$ | $c'$ | $c''$ | $\sigma$ |
| $h_{2\%}$ | 0.15 | - | 0.010 | 0.1 | - | 0.010 |
| $u_{2\%}$ | 1.3 | 0 | 0.118 | 1.7 | 0.1 | 0.108 |
| $q_{2\%}$ | 0.2 | 0 | 0.020 | 0.17 | 0.1 | 0.012 |

Table 3 Values of coefficients in Equations 2-4 and standard deviations of differences between measured and calculated (non-dimensional) parameters.

Equations 2 and 3 form the boundary conditions for the flow equations from which the thickness of water layers and velocities at the inner slope were obtained. The following expressions form an approximate solution of the shallow water equations for stationary flow conditions (for a friction factor at the inner slope $f_L$>0):

$$h = h_0 u_0 / (\frac{\alpha}{\beta} + \mu \exp(-3\,\alpha\,\beta^2\,s)) \tag{5}$$

$$u = \frac{\alpha}{\beta} + \mu \exp(-3\,\alpha\,\beta^2\,s) \tag{6}$$

with $\alpha = \sqrt[3]{g \sin\varphi}$, $\beta = \sqrt[3]{1/2\, f_L / (h_0\, u_0)}$, $\mu = u_0 - \alpha / \beta$, $s$ is the co-ordinate along the landward slope with $s$=0 at the landward side of the crest, and $\varphi$ is the slope angle. In Equations 5 and 6, $h_0$ and $u_0$ are obtained from the expressions for $h_{2\%}$ and $u_{2\%}$ at the landward side of the crest as given in Equations 2 and 3.

Figure 6, 7 and 8 show the comparison between the test results and the (calibrated) formulae from Equations 2-6. The differences are considered small; the standard deviations ($\sigma$) of these differences are given in Table 3.

For predicting the volumes within an overtopping wave, exceeded by 2% of the incident waves, the following formula was obtained:

$$\frac{V_{2\%}}{H_s^2} = c_V' \,(\gamma_{f-C})^{0.5} \left(\frac{z_{2\%} - R_c}{\gamma_f H_s}\right)^2 \tag{7}$$

Figure 9 shows the comparison between the test results and the (calibrated) formulae from Equation 7 ($c_V'$=1.0). The deviations between both are considered small; the standard deviations of these differences is $\sigma$ = 0.142.

The analysis so far was focused on wave overtopping parameters related to an exceedance level of 2%, *i.e.* 2% of the waves exceed these levels. The approach followed here is however also suitable for other exceedance levels. Table 2 shows the coefficients for wave run-up levels exceeded by 1%, 2%, and 10% of the incident waves. Using the same values for the coefficients as given in Table 3 also predictions can be obtained for wave overtopping parameters exceeded by 1% or 10% of the incident waves. Thus, using $z_{1\%}$ or $z_{10\%}$ in Equations 2-7 provide estimates of $h_{1\%}$, $u_{1\%}$, $q_{1\%}$, and $V_{1\%}$, or $h_{10\%}$, $u_{10\%}$, $q_{10\%}$, and $V_{10\%}$.

In the applied methodology the difference between the fictitious wave run-up level and the actual crest elevation is used to predict overtopping parameters. If the difference between the fictitious wave run-up level and the actual crest elevation becomes large, the slope angle in the prediction of the fictitious wave run-up level becomes less meaningful because a larger part of the slope does in reality not exist. Therefore, the approach becomes less suitable if a larger part of the wave tongue becomes "fictitious". To study to what extent the described approach can still be applied two series of tests with a smooth low-crested structure have been performed.

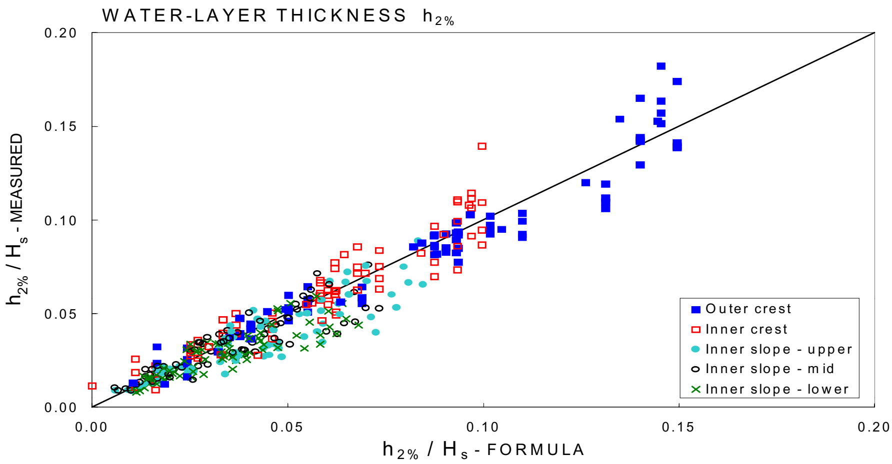


Figure 6 Water layer thickness at all positions (Series A-D'); data compared with formulae (Eq. 2 and Eq. 5-6).

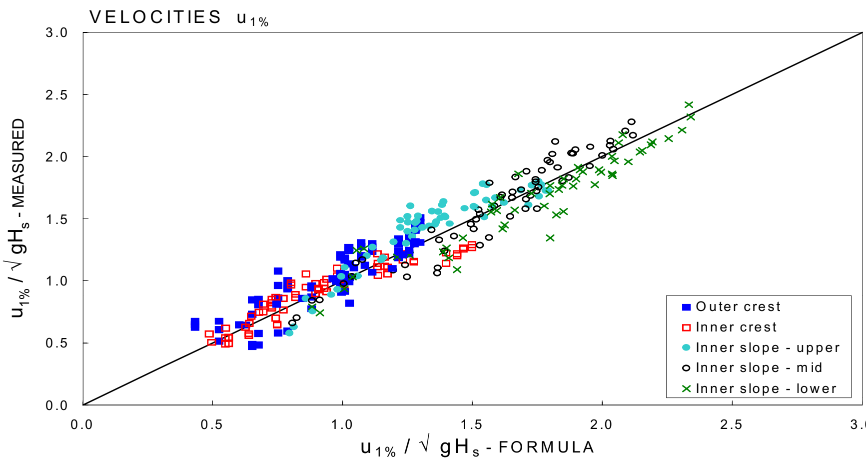


Figure 7 Velocities at all positions (Series A-D'); data compared with formulae (Eq. 3 and Eq. 6).

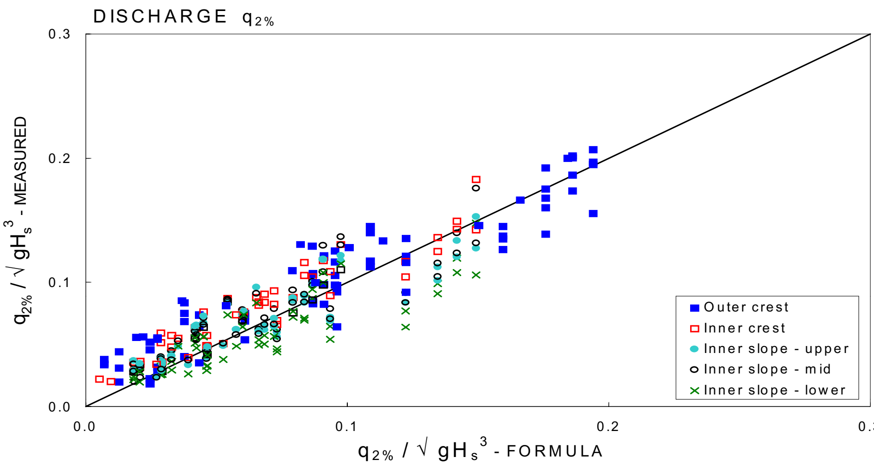


Figure 8 Discharges at all positions (Series A-D'); data compared with formulae (Eq. 4 and Eq. 5-6).

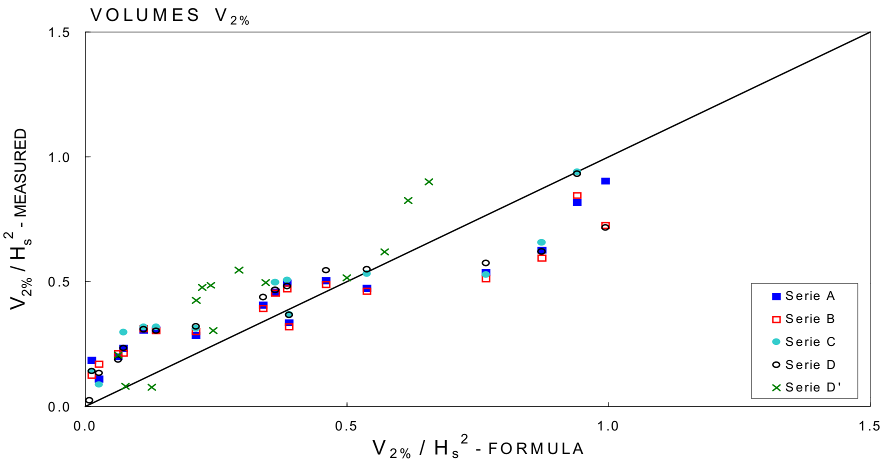


Figure 9 Volumes per overtopping wave (Series A-D'); data compared with formula (Eq. 7).

The conditions for tests with dikes were in the range of $0 < (z_{2\%} - R_c) / \gamma_f H_s < 1.0$ while the two series of tests with low-crested structures were in the range between $1.6 < (z_{2\%} - R_c) / \gamma_f H_s < 2.3$. This means that in the latter two series the fictitious part of the computed wave run-up level is much larger. It appeared that the applied approach is not appropriate for low-crested structures; the deviations increased significantly compared to the conditions with (much higher) dikes.

Schüttrumpf (2001) obtained formulae for the thickness of the water layer and the velocity on the seaward slope, the crest, and the inner slope. The approach is similar to the one followed by Van Gent (2001-a, 2002-b). The formulae by Schüttrumpf (2001) for the crest are considered here, in combination with predictions of the fictitious wave run-up level as given in Equation 1. These formulae are then applied to provide boundary conditions for the expressions for the inner slope (Equations 5 and 6). The formulae by Schüttrumpf (2001) for the water layer thickness and velocity at the crest can be re-written as (where $x$ is the co-ordinate along the crest with $x$=0 at the seaward side of the crest):

$$\frac{h_{2\%}}{H_s} = c'_h \left( \frac{z_{2\%} - R_c}{\gamma_f H_s} \right) \cdot \exp\left(-c''_h \; x / B_c\right) \tag{8}$$

$$\frac{u_{2\%}}{\sqrt{g H_s}} = c'_u \left( \frac{z_{2\%} - R_c}{\gamma_f H_s} \right)^{0.5} \cdot \exp\left(-c''_u \; x \; f_c / h_{2\%}\right) \tag{9}$$

The coefficients $c'_h$ and $c'_u$ in Equations 8 and 9 as proposed by Schüttrumpf (2001) cannot directly be used since a different wave run-up formula is used. The values obtained based on the present data set (Equations 2 and 3) can be used: $c'_h$=0.15 and $c'_u$=1.3. Schüttrumpf (2001) calibrated $c''_h$=0.75 for the thickness of water layers and derived $c''_u$=0.5 for velocities. For the present data set the coefficient $c''_h$=0.75 is not optimal; the optimal value for the present data set is $c''_h$=0.4. This re-calibration also affects the calculated velocities because the thickness of the water layer is present in the formula for velocities. Nevertheless,

$c''_u$=0.5 can be used to calculate velocities. Predictions of discharges can be obtained by multiplying Equations 8 and 9. The standard deviations of the differences between the measured (non-dimensional) parameters $h_{2\%}$, $u_{2\%}$, and $q_{2\%}$ at the landward side of the crest and those calculated with Equations 8 and 9 are 0.010, 0.122 and 0.034 respectively (compared to 0.010, 0.108 and 0.012 using Equations 2-4). The re-calibrated formulae Equations 8 and 9 are used as boundary condition to obtain estimates of the thickness of water layers, velocities and discharges at the inner slope using Equations 5 and 6. The standard deviations of the differences between measured and calculated (non-dimensional) parameters $h_{2\%}$, $u_{2\%}$ and $q_{2\%}$ for all positions are 0.010, 0.143 and 0.021 respectively (compared to 0.009, 0.132 and 0.018 using Equations 2-6). Thus, the comparison between the results using either Equations 2-4 or Equations 8-9 shows that the differences are not significant; the differences between the tests and Equations 8-9 are relatively large for the tests with increased roughness.

Although most existing and available data sets do not originate from a systematic variation of parameters as in the present test programme, existing data can be used for comparison with the prediction formulae obtained based on the present data set. Three data sets obtained by tests at WL | Delft Hydraulics were used:

- Small-scale tests for a dike with a smooth 1:4 seaward slope and a smooth 1:2.6 inner slope (Van der Meer, 1987). Data on the thickness of water layers $h_{2\%}$ and velocities $u_{2\%}$ could be extracted for the crest and inner slope.
- Small-scale tests for dikes with smooth 1:3 and 1:4 slopes, and slopes with a berm (Van der Meer and De Waal, 1993). Data on the thickness of water layers $h_{2\%}$ (crest) and the volumes per overtopping wave $V_{2\%}$ could be extracted.
- Large-scale tests for a dike with a smooth 1:4 slope with grass (Smith, 1994). Data on volumes per overtopping wave $V_{2\%}$ could be extracted.

Figure 10 shows a comparison between the thickness of water layers as measured in the first two projects mentioned above, and those calculated using Equation 2. The coefficients in Equation 2 were however set at $c'_h$=0.21 for the seaward side of the crest and $c'_h$=0.15 for the landward side of the crest; the values obtained from the present test programme ($c'_h$=0.15 and $c'_h$=0.10) lead to a systematic difference of about 30%. Nevertheless, the shape of the formula is confirmed by these data sets since the influence of the different wave conditions and structure parameters are represented well, as can be concluded from the small amount of scatter around the main trend.

From the first data set also velocities and discharges could be derived. Figure 11 shows the comparison between measured and calculated velocities (Equation 3 and 6). No re-calibration was required ($c'_u$=1.7, $c''_u$=0.1). For velocities at the crest the comparison is good, at the inner slope the deviations are larger. Figure 12 shows the comparison between measured and calculated volumes within overtopping waves; there is a rather good agreement between the formula and the data from these large-scale tests, using the same coefficient ($c'_V$=1.0) as obtained from the present tests.

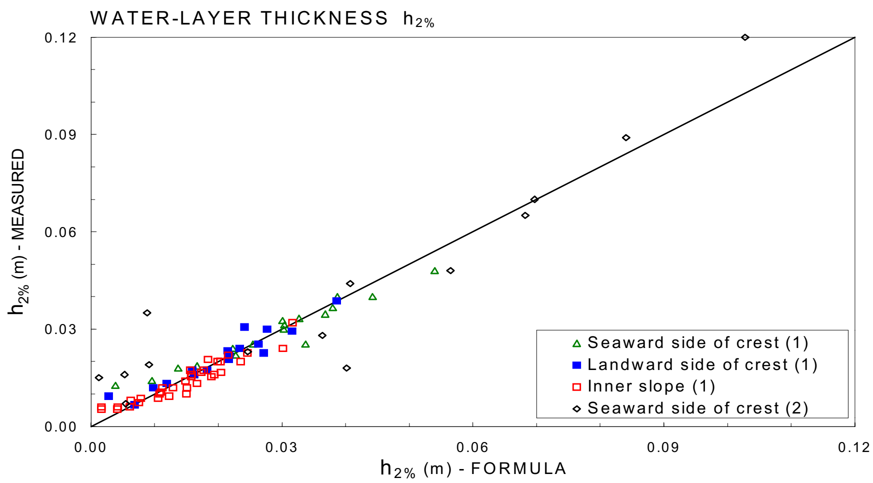


Figure 10 Water layer thickness at crest and inner slope.

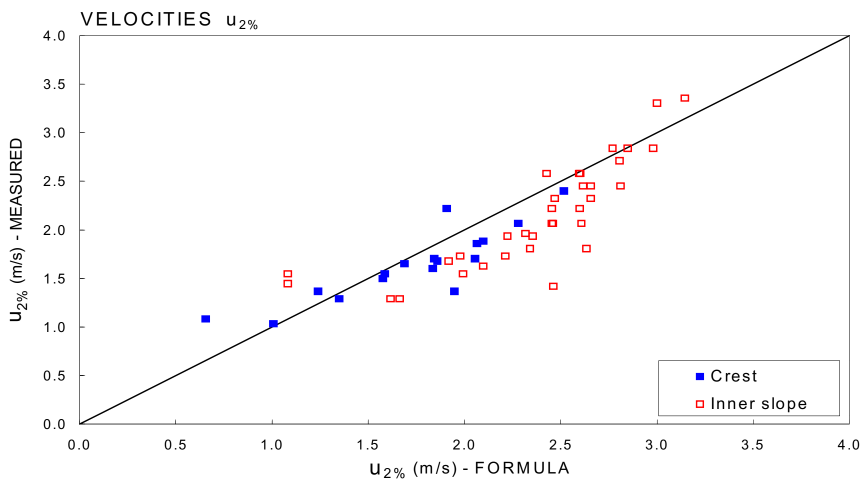


Figure 11 Velocities at crest and inner slope.

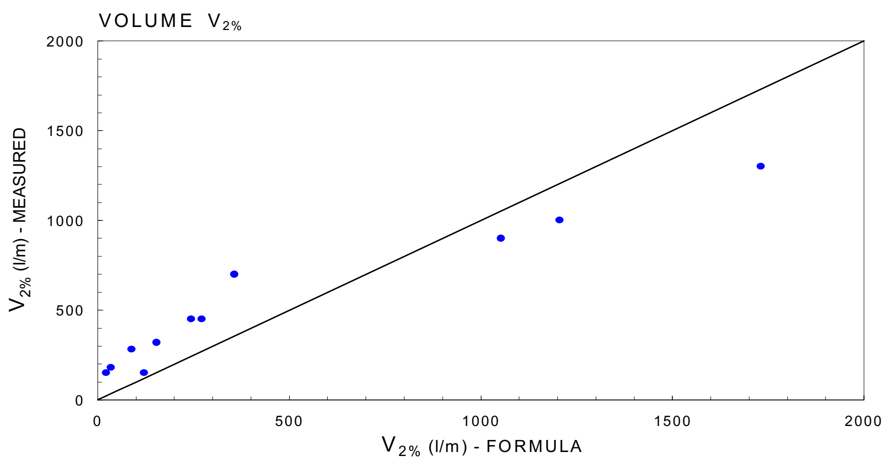


Figure 12 Comparison between measured (large-scale) and calculated volumes per overtopping wave.

## 4. CONCLUSIONS

The described study led to the following conclusions:

- For four parameters characterising wave overtopping events formulae have been calibrated based on physical model tests. These parameters concern the maximum water layer thickness within wave overtopping events, the maximum velocity, the maximum discharge, and the volume within an overtopping wave, exceeded by a certain percentage of the incident waves. Here the 2%-values are used. The formulae for the landward side of the crest are:

$$h_{2\%} = c'_h \; ((z_{2\%} - R_c)/\gamma_f) \tag{10}$$

$$u_{2\%} = c'_u \, (g\,\gamma_{f-C})^{0.5} \, ((z_{2\%} - R_c)/\gamma_f)^{0.5} \; / (1 + c''_u \, B_c / H_s) \tag{11}$$

$$q_{2\%} = c'_q \, (g\,\gamma_{f-C})^{0.5} \, ((z_{2\%} - R_c)/\gamma_f)^{1.5} \; / (1 + c''_q \, B_c / H_s) \tag{12}$$

$$V_{2\%} = c'_V \;\; \gamma_{f-C}{}^{0.5} \; ((z_{2\%} - R_c)/\gamma_f)^2 \tag{13}$$

  for situations with $z_{2\%} \geq R_c$, in which $z_{2\%}$ is obtained using Equation 1. The present tests provide the calibration given in Table 3. The configurations tested concern conditions where the crest width is within the range $1 < B_c / H_s < 7.5$ and the crest height $0 < (z_{2\%} - R_c) / \gamma_f H_s < 1$; the set of equations is considered valid only within this range. A few other exceedance levels than the applied 2%-exceedance levels have been examined. If the corresponding predictions of wave run-up levels are used ($z_{1\%}$ or $z_{10\%}$) the same coefficients (see Table 3) can be used to obtain estimates of $h_{1\%}$ $u_{1\%}$,, $q_{1\%}$ and $V_{1\%}$ , or $h_{10\%}$ , $u_{10\%}$,, $q_{10\%}$ and $V_{10\%}$.

- The thickness of water layers (perpendicular to the slope) and velocities along the inner slope of dikes can be estimated based on an approximate solution of the shallow-water equation for stationary conditions:

$$h_{2\%-LANDWARD\ SLOPE} = (h_{2\%-CREST} \cdot u_{2\%-CREST}) / (\frac{\alpha}{\beta} + \mu \exp(-3\,\alpha\,\beta^2\, s)) \tag{14}$$

$$u_{2\%-LANDWARD\ SLOPE} = \frac{\alpha}{\beta} + \mu \exp(-3\,\alpha\,\beta^2\, s) \tag{15}$$

  with $\alpha = \sqrt[3]{g \sin\varphi}$ , $\beta = \sqrt[3]{1/2\, f_L / (h_{2\%-CREST} \cdot u_{2\%-CREST})}$ , $\mu = u_{2\%-CREST} - \alpha / \beta$ and $s$ is the co-ordinate along the landward slope with $s$=0 at the landward side of the crest. The standard deviations of the differences between measured and calculated (non-dimensional) parameters $h_{2\%}$, $u_{2\%}$ and $q_{2\%}$ ( $= h_{2\%} \cdot u_{2\%}$ ) at the inner slope are $\sigma = 0.008$, $\sigma = 0.139$ and $\sigma = 0.018$ respectively.

- Formulae by Schüttrumpf (2001), Equations 8 and 9, for the thickness of water layers and velocities at the crest of dikes (alternative formulae for Equations 10 and 11) were used for comparison with the results from the present tests; after re-calibration of these formulae a rather good match was found. If predictions of the thickness of water layers and velocities along the crest are needed, and not only at the seaward edge and landward edge of the crest, Equations 8 and 9 can be used.

The results from the present investigations can be used to characterise the flow during wave overtopping events at dikes. It is recommended to study the relation between the hydrodynamic loading and the sensitivity to erosion of (cohesive) material at the crest and inner slope of dikes. This would provide insight into the strength of dikes and its capability to withstand conditions with overtopping.

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