Full Scale Wave Overtopping Measurements on the Zeebrugge Rubble Mound Breakwater

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Abstract

Field measurements of wave overtopping over a rubble mound breakwater have been carried out at Zeebrugge, Belgium, during the period 1999-2003. The overtopping volumes of water are collected in an overtopping tank located on the breakwater crest behind the access road. A short side of the tank is made of a weir allowing the collected water to flow out continuously, and allowing measurements to be made for long duration. Calculation of the inflow discharge is based on the outflow discharge over the weir and the volume in the tank. This calculation requires an accurate measurement of the water depth inside the tank. The water depth is measured indirectly using a pressure sensor near the bottom. The measurement set-up for wave overtopping at the breakwater is described in detail. Eight wave records with wave propagation normal to the breakwater have been used in the analysis of the field data. Wave heights range between 2.6 m and 3.9 m for wave records with a typical duration of 1 to 2

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hours. The average overtopping rate has been calculated for all records using 3 methods. Average overtopping rates from the field data have been compared with predicted values from prediction formulae by van der Meer et al. [van der Meer, J.W., Tönjes, P., de Waal, H., 1998. A code for dike height design and examination, Proceedings International Conference on Coastlines, Structures and Breakwaters, Institution of Civil Engineers, London, Thomas Telford, London, pp. 5 - 19], Owen [Owen, M.W., 1980. Design of seawalls allowing for wave overtopping. Hydraulics Research, Wallingford, Report No. EX 924, UK] and Besley [Besley, P., 1999. Wave overtopping of Seawalls. Design and Assessment Manual. Hydraulics Research Wallingford. R&D Technical Report W178, ISBN 1 85705 069 X]. Three crest freeboard values have been used in the analysis, corresponding to the maximum and minimum crest levels and the average crest level in between as the best approximation of the actual crest level for the specific breakwater crest geometry. The calculated reduction factor for surface roughness used in the formulae is 0.51. Best agreement is observed for van der Meer et al.'s (1998) and Besley's (1999) prediction using the actual average crest level. In this case, measured average overtopping rates are within the 95 % confidence intervals of these prediction formulae.

Keywords: wave overtopping; field data; rubble mound breakwater, surface roughness, crest freeboard

1 Introduction

A significant portion of today's population lives in coastal zones as development and recreational activities in these areas increase continuously. Coastal structures are commonly built to protect these coastal zones against storm surge and storms that generate large waves. A combination of storm surge and large waves may cause overtopping of such structures (like breakwaters, dikes, seawalls, etc...) leading to damage to and flooding of the area behind the structure. In this respect, the crest elevation of a coastal structure is an important design parameter as it is directly related to the amount of wave overtopping. Increasing storm intensity (Carter & Draper, 1988), and increasing relative mean sea level rise due to the greenhouse effect or lowering in the land level (National Research Council, 1987, Führböter, 1990, ASCE Task Committee, 1992), are two trends that attract the attention even more on the determination of the crest level of coastal structures, both during design of new structures and during evaluation of existing structures.

In the design of the crest elevation of a seawall or breakwater, usually a tolerable wave overtopping rate is applied (Owen, 1980, Goda, 1985, van der Meer, 1994). For many cases it is sufficient to use the average overtopping rate, or mean discharge, q , expressed as a discharge per meter structure (m^3/s per m), and averaged over the duration T_0 of the storm. Dependent on the function of the structure, different criteria (safety for pedestrians, or damage to the structure itself or buildings behind it) are used and critical values of q for each of these criteria have been presented in literature. Those critical values are regarded as rough approximations, and are still susceptible to improvements. A recent version of tabulated q values is available e.g. in CEM (2003), and more complete discussions on safety levels for people are presented by Allsop et al. (2003).

The prediction of the average overtopping rate for a particular structure geometry, water level, and wave conditions is based on empirical formulae, fitted to hydraulic model test results. Goda et al. (1975) presented design diagrams for estimation of the overtopping rate of vertical revetments and block mound seawalls based on results from small scale model tests. Takayama et al. (1982) derived an empirical formula from Goda's design diagrams to predict the average overtopping rate. Franco & Franco (1999) presented prediction formulae for overtopping at various types of vertical breakwaters based on extensive 2D and 3D model test results. For the case of sloping structures, the most widely used empirical formulas to predict wave overtopping have been published by Owen (1980, 1982) for smooth and rough impermeable slopes, by Bradbury & Allsop (1988) for rock armoured impermeable slopes with a crown wall, by Aminti & Franco (1988) for rubble mound breakwaters with a crown wall and by van der Meer et al. (1998) for smooth and rough impermeable slopes. Each of these studies make use of a dimensionless crest freeboard parameter which is related to a dimensionless overtopping parameter. The most complete summary of empirical overtopping prediction methods is presented by Besley (1999).

Field observations with reported analysis of field data on wave overtopping have been rarely obtained so far. For the case of seawalls and vertical breakwaters, a number of field studies have been reported. Fukuda et al. (1974) reported successful measurements of the average overtopping rates for the case of a seawall covered by 16 ton Tetrapod armour units at Port of Fukui, Japan, 1971 - 1972. Sakakiyama et al. (1997) carried out additional field measurements at the same location in 1996, and obtained time series of average overtopping rates based on storm durations of 2 hours. Comparison of these field data with Takayama et al.'s (1982) formula showed reasonable agreement according to the authors. Herbert (1996) measured overtopping at a vertical seawall and at a rather smooth revetment and obtained

average overtopping rates based on 100 wave sequences. Kimura et al. (1999) commented on hazards resulting from wave overtopping based on field observations, but did not measure overtopping volumes.

For the case of sloping rubble mound breakwaters, the authors are not aware of any full scale overtopping measurements reported in literature. Within the research project CLASH (Crest Level Assessment of coastal Structures by full scale monitoring, neural network prediction and Hazard analysis on permissible wave overtopping, 2002-2004, funded by the European Commission) field measurements on wave overtopping are carried out at three different locations in Europe: a vertical seawall with rubble mound toe protection at Samphire Hoe, United Kingdom (Pullen et al., 2003), a rock armoured rubble mound breakwater in shallow water at Ostia, Italy (Franco et al., 2003) and a rubble mound breakwater armoured with grooved (Antifer) cubes at Zeebrugge, Belgium.

The objective of the present paper is to describe full scale measurements of wave overtopping carried out at Zeebrugge rubble mound breakwater during the period 1999 - 2003. First the measurement site and the specific set-up used for wave overtopping measurements is described in section 2. Section 3 describes the analysis of the field wave overtopping data and discusses methods to derive average overtopping rates. Average overtopping rates from the field data have been compared with predicted values obtained from widely used prediction formulae available in literature (section 4). Finally conclusions are drawn in section 5.

2 Full scale set-up and instrumentation for overtopping measurement

2.1 The Zeebrugge breakwater field site

The Zeebrugge breakwater has been used for more than a decade for full scale measurements of wave interaction with a breakwater. A prototype monitoring system for acquisition of wave characteristics and induced structural response, has been developed since the early nineties. Troch et al. (1998) describes the monitoring system, instrumentation and data-acquisition. The breakwater has been instrumented to measure wave induced pore pressures inside the breakwater core (Troch, 2000), and wave run-up on the breakwater slope (Van de Walle, 2003). More recently the breakwater has been instrumented to capture and measure wave overtopping volumes at full scale. In this paper the wave overtopping over the breakwater crest is analysed.

The Zeebrugge breakwater is a conventional rubble mound breakwater with a relatively low-crested superstructure. The armour layer is made of grooved (Antifer) concrete cubes (25 ton). The breakwater core consists of quarry run (2-300 kg). The filter layer is composed of rock (1-3 ton). An access road behind the wave wall allows access along the breakwater. Design conditions for the breakwater are: return period $R_p = 1:500$ years, significant wave height $H_s = 6.20$ m, maximum peak period $T_p = 9.0$ s and still water level $SWL = Z + 6.75$ ($Z + 0.00$ is chart datum). A tide cycle lasts for 12 hours and 24 minutes, the tidal range varies between $Z + 4.62$ (MHWS) and $Z + 0.32$ (MLWS).

The cross section of the Zeebrugge breakwater at the location of the wave overtopping tank is shown in Fig. 1. The actual slope of the armour layer in this section has been determined as 1 : 1.4, and deviating slightly from the design slope 1 : 1.5. The design crest level of the two-

layer armour layer is $Z + 12.40$, the top of the crest crown wall is at $Z + 10.20$, the access road level is $Z + 8.0$.

2.2 Wave overtopping measurements

2.2.1 The overtopping tank

An overtopping tank, placed behind the access road (Fig. 2), collects water volumes overtopping the crest of the breakwater. This location in Fig. 3 was required in order not to obstruct the traffic on the access road. The section used for construction of the overtopping tank is located 140 m away from the section with the instrumented jetty, where wave characteristics are measured. The reclamation fill behind the breakwater at the section with the instrumented jetty, did not allow space for the tank. The overtopping tank itself is a closed concrete box, with dimensions 7.40 m x 2.0 m x 2.0 m (length x width x height), Fig. 2. The tank is open at the top allowing the overtopping water to flow in, and has a weir in a short side allowing the collected water to flow out of the tank. The inflow discharge in the tank is denoted Q_{in} , the outflow discharge over the weir is denoted Q_{out} .

2.2.2 Measurement of inflow discharge using the weir

In order to obtain average overtopping rates, the instantaneous inflow discharge in the tank $Q_{in}(t)$ has to be determined. The weir allows the water to flow out of the tank continuously, as the tank empties itself. Continuous measurements of the volume in the tank allow overtopping measurements to be made for long duration. The V-shaped weir controls the outflow discharge $Q_{out}(t)$, calculated from the water depth $h(t)$ in front of the weir, defined as the vertical distance between the bottom opening of the weir and the free water surface

inside the tank. Applying continuity to the tank, the net mass inflow inside the tank is equal to the time rate of increase of mass inside the tank:

$$Q_{in}(t) - Q_{out}(t) = A \frac{dh}{dt} \quad (1)$$

where $A = 14.80 \text{ m}^2$ is the horizontal tank surface area. By measuring $h(t)$ continuously inside the tank and calculating $Q_{out}(t)$ over the weir, the instantaneous inflow discharge $Q_{in}(t)$ is calculated using eq. (1). The calculation method is described in section 3.

Overtopping discharges from wind-generated waves are unevenly distributed in space and time because the waves are short-crested and the volume per wave varies considerably in time. In order to have a representative calculation of the discharge per meter crest width, the length of the tank is increased to 7.40 m. As the volumes per wave that enter the overtopping tank vary substantially, the weir has to be able to measure both small and large discharges, so a two-stage weir is used (Fig. 4). The lower part has a very narrow V-shape (width of top opening 0.04 m, height of lower part 0.40 m) for slow evacuation of the small volumes to have sufficient measurement accuracy for small discharges. The upper part of the weir has a top opening of 0.75 m and is 1.35 m high, allowing large outflow volumes (up to $1 \text{ m}^3/\text{s}$ when the tank is completely filled).

For water depths h smaller than 0.40 m, the active lower part of the weir is a triangular Thompson weir. In this case, the outflow discharge $Q_{out}^{h<0.40}$ is proportional to $h^{\frac{5}{2}}$ (Vardy, 1990):

$$Q_{\text{out}}^{h<0.40} = c_1 h^{\frac{5}{2}} \quad (2)$$

For water depths h larger than 0.40 m, the weir acts as a compound weir and the relation between the outflow discharge $Q_{\text{out}}^{h>0.40}$ and the water depth h is composed of the contributions from a triangular Thompson weir, and a trapezoidal weir (Vardy, 1990):

$$Q_{\text{out}}^{h>0.40} = c_2 h^{\frac{5}{2}} + c_3 h^{\frac{3}{2}} \quad (3)$$

Calibration of the weir in the laboratory was carried out by measuring the outflow discharges for given water depths. The lower part has been calibrated on a scale 1 in 1, the compound weir on a scale 1 in 2. Using least square fitting of equations (2) and (3) to the calibration data, the following coefficients have been obtained: $c_1 = 0.0805$, $c_2 = -0.1691$ and $c_3 = 0.3533$.

2.2.3 Determination of the water depth inside the tank

Measuring the water depth in the overtopping tank is critical for accurate overtopping measurements, as the water depth information is required in both terms of the continuity equation (1). However, due to the non-stationary conditions inside the tank caused by inflow of overtopping waves and the wind, the water is sloshing continuously. The idea to place baffles or porous wave absorbers inside the tank seemed unfeasible. Instead, a spatially averaged measurement of the water depth is carried out. The water depth in the overtopping tank is measured indirectly by pressure sensors near the bottom of the tank. A set of 5 plastic rigid tubes with the same length and diameter is mounted to the bottom. The tubes are open to the water on one end, and are connected to a closed steel tube on the other end. The open ends

are uniformly distributed over the surface area. Inside the steel tube, the pressures from the 5 open end locations are averaged. Two pressure sensors (type DRUCK PTX 1830, pressure range 0 – 350 mbar) mounted on both sides of the steel tube measure the averaged pressure with a sample frequency of 10 Hz (Fig. 5). The pressure from the two sensors is averaged, low-pass filtered and converted to the water depth $h(t) = p(t)/\rho_w g$, where ρ_w is the density of the sea water and g is the gravitational acceleration. If one sensor fails, the measurements are still assured. The rigid tubes and the steel tube are continuously immersed in water preventing air bubbles to affect adversely the measurements. A heating device mounted on the steel tube, prevents damage from freezing water in winter time.

2.2.4 Wave and water level measurements

Wave characteristics and water level in front of the breakwater are required to relate overtopping volumes to the incident sea state and crest freeboard. Wave characteristics are measured along the section of the measurement jetty. A conventional wave rider buoy, located at 200 m from the breakwater, provides wave height and wave period of the incident waves. The water level at the toe of the breakwater is measured by an infra red wave height meter. More detailed information on these devices is given in Troch et al. (1998).

3 Field data for wave overtopping

3.1 Data sets

Wave overtopping measurements have been carried out on the Zeebrugge breakwater using the test set-up presented previously since 1999. Up to now, six storms with waves from North-West (normal to the breakwater section at the overtopping tank) have been measured. Cases with oblique wave attack are not considered in this paper. Summary of the storms, including date and duration of the wave records used in the analysis, is given in Table 1. Incident wave characteristics (significant wave heights H_{m0} from frequency domain analysis and H_s from time domain analysis, peak wave period T_p and mean wave period T_m), the surf similarity parameter $\xi_{m-1,0}$ and the water level (SWL) are listed for all storms in Table 2. Wave characteristics are calculated from the waverider buoy in front of the breakwater. The water level is the mean value of the surface elevations at the toe of the breakwater measured using the IR-meter. The surf similarity parameter or breaker parameter $\xi_{m-1,0}$ is defined as:

$$\xi_{m-1,0} = \frac{\tan \alpha}{\sqrt{\frac{2\pi H_{m0}}{g T_{m-1,0}^2}}} \quad (4)$$

where H_{m0} is the significant wave height, α is the breakwater slope angle and $T_{m-1,0}$ is the wave period based on the zeroth and first negative spectral moments: $T_{m-1,0} = m_{-1}/m_0$.

Analysis of wave overtopping in full scale conditions is carried out for sections of wave records with reasonably constant sea state parameters and water level. The duration of the storm wave (and overtopping) records chosen was 2 hours around high water for storms 1, 2,

3, 4 and 6, when the water level in front of the breakwater and the sea state parameters were assumed constant. Storm 5 was measured during a rising tide, and the water level was assumed constant over 1 hour only. Therefore, storm 5 is divided into three shorter parts 5a, 5b and 5c for further analysis.

The water level in the 6 storms (or the 8 wave records) ranges between $Z + 4.24$ (storm 4) and $Z + 5.28$ (storm 1), yielding water depths between 13 m and 14 m at the waverider buoy respectively. The significant wave height H_{m0} ranges between 2.60 m (storm 2) and 3.86 m (storm 5b), and the peak wave period T_p between 7.34 s (storm 1) and 10.28 s (storm 3).

3.2 Example results

As a typical example, Fig. 6 shows the available field data of wave conditions and wave overtopping, for storm no. 3. Fig. 6.a presents the incident wave field $\eta(t)$ using surface elevations measured from the waverider buoy during 2 hours, from where $H_{m0} = 3.47$ m and $T_{m-1,0} = 8.41$ s are calculated. Water levels from the IR-meter are shown in Fig. 6.b, from where the mean value is calculated as $SWL = Z + 5.01$. Finally, Fig. 6.c presents the time series (during the total duration of the storm) of the measured water depth $h(t)$ inside the wave overtopping tank. This time series exhibits a typical saw tooth pattern, with a steep sudden increase in water depth during an overtopping event when water enters the overtopping tank. The water depth decreases gradually by outflow of the water over the weir when there is no wave overtopping. The water depth $h(t)$ will be used to derive average overtopping rates.

3.3 Methods for calculation of overtopping discharges

3.3.1 Method 1, using the continuity equation

Fig 7.a (repeated from Fig. 6.c) shows the instantaneous water depth $h(t)$ for storm 3. The instantaneous inflow discharge $Q_{in}(t)$, Fig. 7.c, is calculated using the continuity equation, eq. (1). The terms Q_{out} and $A \frac{dh}{dt}$, Fig. 7.b, are derived from the instantaneous water depth $h(t)$ inside the overtopping tank. The peaks in the function $A \frac{dh}{dt}$ occur during wave overtopping events when water enters the tank, and therefore these peaks are visual detectors of the number of overtopping waves. The average overtopping rate q , per unit crest width, from here on referred to as q_{ceq} , is finally calculated from $Q_{in}(t)$, using:

$$q_{ceq} = \frac{1}{b} \frac{1}{T_0} \int_0^{T_0} Q_{in}(t) dt \quad (5)$$

where $b = 7.40$ m is the tank width parallel to the crest, and T_0 is the duration of the analysed wave record cf. Table 1. Fig. 8 shows the time series for storm 5b, similar to Fig. 7 for storm 3, as a second example.

3.3.2 Method 2, using individual overtopping volumes

A modified calculation of the average overtopping rate q is using the peaks of the overtopping events only from the function $Q_{in}(t)$ and is neglecting the scatter around zero which may affect adversely the time-averaged overtopping rate q_{ceq} . For a single overtopping event, the

individual volume V_i (per m crest width) of the overtopping wave with number i is calculated from (Fig. 9):

$$V_i = \frac{1}{b} \int_{t_0}^{t_1} Q_{in}(t) dt \quad (6)$$

where t_0 and t_1 denote the start and end time of the single overtopping event (typically $t_1 - t_0 \approx 60$ s). The average overtopping rate q_{Vi} using this second method is calculated using:

$$q_{Vi} = \frac{1}{T_0} \sum_{i=1}^{N_{ov}} V_i \quad (7)$$

where N_{ov} is the number of overtopping events during the storm with duration T_0 .

3.3.3 Method 3, using water depth jumps

The previous two methods depend on the accuracy of $Q_{out}(t)$. A third simplified method uses the water depth $h(t)$ directly to calculate the increase in volume inside the tank. The positive jumps Δh_i in the water depth signal $h(t)$ are added as individual volumes contributing to the total volume V inside the tank:

$$V = A \sum_{i=1}^{N_{ov}} \Delta h_i + \sum_{i=1}^{N_{ov}} \Delta V_i \quad (8)$$

During a single overtopping event water is already flowing over the weir out of the tank, and a correction ΔV_i has been added :

$$\Delta V_i = \int_{t_0}^{t_1} Q_{out}(t)dt \quad (9)$$

The average overtopping rate $q_{\Delta h}$ calculated using this third method is:

$$q_{\Delta h} = \frac{1}{b} \frac{1}{T_0} V \quad (10)$$

3.4 Average overtopping rates

The average overtopping rate q has been calculated for all 8 wave records, using the 3 methods described above. The results in Table 3 show average overtopping rates q_{ceq} , q_{Vi} and $q_{\Delta h}$ calculated using the method based on the continuity equation; the method based on the individual overtopping volumes; and the method based on the water depth jumps, respectively. Table 3 also shows the number of overtopping events N_{ov} , equal to the number of peaks (with threshold 0.002 m³/s) in the $Q_{in}(t)$ signal with duration T_0 . Average overtopping rates from Table 3 are presented in Fig. 10, and suggest a number of observations. Storms 1, 2, 4 and 6 have overtopping rates $q < 0.1$ l/sm, showing very little wave overtopping during these storms. Storms 3, 5a, 5b and 5c have overtopping rates between $q = 0.3$ l/sm (for storm 3) and $q = 0.9$ l/sm (for storm 5b), indicating more severe wave overtopping. Storms with small overtopping rates have a smaller number of overtopping

events N_{ov} (≤ 10), whereas the storms with larger overtopping rates have a larger N_{ov} (20 to 30).

In general, the average overtopping rate q_{ceq} , calculated using the first method based on the continuity equation, is smaller than the two other overtopping rates q_{Vi} and $q_{\Delta h}$ (except for storm 2). On average for the storms 1, 2, 4 and 6 with small overtopping rates the deviation between q_{ceq} and q_{Vi} is 28 %, and for storms 3 and 5 with large overtopping rates the deviation is 19 %. The deviation between q_{ceq} and $q_{\Delta h}$ is 17 % and 9 % for the storms with small overtopping rates and large overtopping rates respectively. In general, q_{Vi} is larger than $q_{\Delta h}$ (except storm 3). The deviation between q_{Vi} and $q_{\Delta h}$ is 16 % and 14 % for the storms with small overtopping rates and large overtopping rates respectively. From these observations, the following relationship is obtained:

$$q_{ceq} < q_{\Delta h} < q_{Vi} \quad (11)$$

with

$$q_{ceq} \approx 0.76q_{Vi} \quad (12)$$

and

$$q_{\Delta h} \approx 0.88q_{Vi} \quad (13)$$

4 Comparison of field data with empirical formulae

4.1 Empirical formulae from literature

As mentioned in section 1, several empirical formulae for the prediction of wave overtopping have been derived, in most cases based on small scale model results. The field data on wave overtopping at the Zeebrugge rubble mound breakwater will be compared to two existing prediction formulae from literature. The comparison is discussed in the next section 4.2. First the empirical formulae, applicable for the Zeebrugge breakwater, are given in this section.

The formulae presented by Owen (1980) and van der Meer et al. (1998) are well-known and widely used for breakwater crest level design. These formulae originally are based on extensive datasets from model tests with smooth impermeable sloping structures. By applying a reduction factor for the slope roughness, the formulae are applicable to rock slopes. These two formulae will be used here for comparison with the field data, and are presented hereafter. Bradbury et al. (1988) and Aminti and Franco (1988) present empirical formulae based on model tests with rubble mound breakwaters with a crown wall and are therefore more restricted, consequently these will not be used here.

The wave overtopping formula presented in van der Meer et al. (1998) is:

$$\frac{q}{\sqrt{gH_{m0}^3}} = \frac{a_v}{\sqrt{\tan \alpha}} \gamma_b \xi_0 \exp \left(-b_v \frac{R_c}{H_{m0}} \frac{1}{\xi_{0,p} \gamma_b \gamma_f \gamma_\beta \gamma_v} \right) \quad (14.a)$$

with a maximum of:

$$\frac{q}{\sqrt{gH_{m0}^3}} = 0.2 \exp\left(-2.6 \frac{R_c}{H_{m0}} \frac{1}{\gamma_f \gamma_\beta}\right) \quad (14.b)$$

where R_c is the crest freeboard, defined as the vertical distance between SWL and crest level of the breakwater. The coefficients $a_v = 0.06$ and $b_v = 5.2$ are mean values based on the average of all test data. Reduction factors γ_b , γ_f , γ_β and γ_v include effects of a berm, surface roughness, oblique wave attack, and a vertical wall on top of the slope, respectively. The breaker parameter $\xi_{0,p}$ used by van der Meer et al. (1998) is calculated using the peak wave period T_p :

$$\xi_{0,p} = \frac{\tan \alpha}{\sqrt{\frac{2\pi H_{m0}}{gT_p^2}}} \quad (15)$$

The Dutch guideline on wave run-up and overtopping on dikes (TAW, 2002) suggests to replace T_p by $T_{m-1,0}$ (in eq. 15, resulting in eq. 4) at the toe of the structure to account for the effects of double-peaked wave spectra related to wave transformation and breaking in shallow foreshores (based on results presented by Van Gent, 2001). In this case, the values for the coefficients in eq. (14.a) are replaced by $a_v = 0.067$ and $b_v = 4.75$.

The wave overtopping formula by Owen (1980) reads:

$$\frac{q}{gH_s T_m} = a_o \exp\left(-b_o \frac{R_c}{T_m \sqrt{gH_s}} \frac{1}{\gamma_r}\right) \quad (16)$$

where a_o and b_o are dimensionless empirically derived coefficients, whose values depend on the breakwater slope, and γ_r is a surface roughness reduction factor.

4.2 Comparison of the Zeebrugge field data with empirical formulae

4.2.1 Crest freeboard for Zeebrugge breakwater

Both empirical formulae, eq. (14) and eq. (16), include the crest freeboard parameter R_c , defined as the vertical distance between SWL and crest level. For the actual Zeebrugge breakwater case the crest level of the breakwater is not straightforward, and has to be defined here. Fig. 11 shows the geometry of the breakwater crest at the location of the overtopping tank. The photo is taken from the land ward side before the overtopping tank has been constructed. The actual crest level is located between $Z + 10.20$ (top of crest wall on service road), assumed as minimum crest level; and $Z + 12.40$ (theoretical design crest level for upper layer of armour units), assumed as theoretical maximum crest level.

The theoretical maximum crest level is not realistic and a more practical maximum crest level is proposed. The maximum crest level used in the wave overtopping prediction formulae is $Z + 12.02$, determined by calculating the average value of the crest level of all points from the bold line in Fig. 11, representing the local level of the armour unit faces in front of the overtopping tank.

Since the armour units of the crest do not have an idealized wall profile with the same wall height along the crest, the actual crest level experienced by the waves is somewhere in

between the minimum level $Z + 10.20$ and maximum level $Z + 12.02$. For the calculations, the average level between minimum and maximum level is taken as the best representation of the actual crest level for wave overtopping: $Z + 11.11$. The crest freeboard is denoted R_{c1} for the maximum crest level $Z + 12.02$, R_{c2} for the minimum crest level $Z + 10.20$, and R_{c3} for the actual average crest level $Z + 11.11$. The crest freeboard R_{c3} is considered to be the most accurate estimation with upper limit R_{c1} and lower limit R_{c2} .

4.2.2 van der Meer et al. (1998) formula

For the application of eq. (14) to the Zeebrugge breakwater, the reduction factors γ_b , γ_β and γ_v are set to unity. There is no berm present ($\gamma_b = 1$), for the reported storms in Table 1 perpendicular wave attack is assumed ($\gamma_\beta = 1$), and there is no vertical wall on the breakwater slope ($\gamma_v = 1$). The reduction factor for surface roughness, γ_f , for the case of the Zeebrugge breakwater with an armour layer with grooved cubes, is determined using the following methodology, for lack of more appropriate values for γ_f for this armour layer. For rock slopes with a double layer of rock, van der Meer & Stam (1992) suggest to calculate γ_f as the ratio of the wave run-up level exceeded by two percent of the incoming waves $R_{u2\%}$ on a rough slope to $R_{u2\%}$ on a smooth slope for the same ξ_0 . The general prediction formula used for the calculation of the run-up level exceeded by two percent of the incident waves $R_{u2\%}$ is (TAW, 2002):

$$\frac{R_{u2\%}}{H_{m0}} = 1.75 \gamma_b \gamma_f \gamma_\beta \xi_{m-1,0} \quad (17.a)$$

with a maximum of:

$$\frac{R_{u2\%}}{H_{m0}} = \gamma_f \gamma_\beta \left(4.3 - \frac{1.6}{\sqrt{\xi_{m-1,0}}} \right) \quad (17.b)$$

Eq. (17) is valid for the range $0.5 < \gamma_b \xi_{m-1,0} < 8$ to 10. In case of the Zeebrugge breakwater, again $\gamma_b = 1$, and $\gamma_\beta = 1$. The average breaker parameter $\xi_{m-1,0}$ for the field data, calculated from the values for all storms in Table 2, is $\xi_{m-1,0} = 3.67$. For this value, the maximum (eq. (17.b)) is applicable, resulting in $\frac{R_{u2\%}}{H_{m0}} = 3.46$ for a smooth slope with $\gamma_f = 1$. Using the average value for $\xi_{m-1,0}$ is justified since $\xi_{m-1,0}$ varies only in a very small range for the field storms ($3.46 < \xi_{m-1,0} < 4.03$). The dimensionless two percent run-up level for the Zeebrugge breakwater with a rough slope has been determined based on extensive full scale measurements of wave run-up, resulting in $\frac{R_{u2\%}}{H_{m0}} = 1.76$ (Van de Walle et al., 2002). The reduction factor γ_f is calculated using the wave run-up level ratio: $\gamma_f = 1.76/3.46 = 0.51$.

This calculated value is compared with other suggested values from literature. De Waal and van der Meer (1992) suggest to use γ_f in the range 0.55 – 0.60 for two or more layers of rock. Owen suggest to use γ_f in the range 0.5 – 0.6 for two or more layers of rubble. Updated values provided in CEM (2003) estimate γ_f in the range 0.50 – 0.55 for two or more layers of rock. Good agreement is obtained between the calculated reduction factor $\gamma_f = 0.51$ and the reduction factors indicated in literature. Consequently, $\gamma_f = 0.51$ is used for the determination of the predicted average overtopping rates.

The predicted average overtopping rates q_{vdm} calculated using the van der Meer et al. (1998) formulae, eq. (14), are summarised in Table 4. Again, the maximum value, eq. (14.b) is applicable.

4.2.3 Owen (1980) formula

The application of the Owen formula, eq. (16), to the Zeebrugge breakwater requires the determination of the coefficients a_o and b_o for the slope $\tan \alpha = 1/1.4$. Interpolation between values given by Besley (1999) yields $a_o = 0.00866$ and $b_o = 19.96$. Owen suggests to use a value in the range 0.50 – 0.55 for the surface roughness reduction factor γ_r for the case of rough rock slopes. The range is very similar to the range proposed by De Waal and van der Meer (1992), and therefore the same value for the reduction factor $\gamma_r = 0.51$ for use in the Owen formula is maintained. The predicted average overtopping rates q_{owen} calculated using the Owen (1980) formula, eq. (16), are summarised in Table 5.

4.2.4 Correction for permeable crest berm: Besley (1999) formula

To take account of a permeable crest, Besley (1999) suggests to multiply the predicted overtopping rate from the Owen (1980) formula, eq. (16), with a reduction factor:

$$C_r = 3.06 \exp \left(-1.5 \frac{C_w}{H_s} \right) \quad (18)$$

where C_w is the crest berm width. For the Zeebrugge breakwater, the permeable crest berm width is located in front of the access road, and is estimated around $C_w = 5.0$ m. The

predicted average overtopping rates q_{besley} calculated using the Besley (1999) reduction factor, eq. (18), on q_{owen} , are summarised in Table 6.

4.2.5 Comparison and discussion

A comparison between predicted average overtopping rates by Owen (1980) or Besley (1999) and van der Meer et al. (1998) formulae for all storms is given for the three crest freeboard values R_{c1} , R_{c2} and R_{c3} in Fig. 12.a, 12.b and 12.c respectively, on a linear scale. For the Zeebrugge field data, Owen consistently predicts higher average overtopping rates than van der Meer et al. Application of the reduction factor C_r due to the presence of a permeable crest berm reduces the over prediction significantly and much better agreement is observed between Besley and van der Meer, especially for the actual average crest level (denoted R_{c3}) and for the minimum crest level (R_{c2}) with best agreement.

The measured average overtopping rates from Table 3 have been compared to the predicted average overtopping rates from van der Meer et al., Owen and Besley, in Fig. 13, for the three crest levels and associated crest freeboards. The range of the measured q - values obtained using the three methods (cf. section 3.3) is indicated in Fig. 13 using a vertical bar. Linear scales have been used in the graphs. For the maximum crest level at $Z + 12.02$ (using R_{c1} , cf. Fig. 13.a), the van der Meer et al. prediction underestimates the measured average overtopping rates, especially for higher q - values. Owen's prediction overestimates the overtopping rates, and the reduction of Besley shows good agreement for smaller q - values and underestimation for larger q - values. Using the minimum crest level at $Z + 10.20$ (using R_{c2} , cf. Fig. 13.b), the predicted average overtopping rates from all formulae overestimate the measured q - values. The van der Meer et al. and Besley predictions have the same

magnitude (cf. Fig. 12), and the Owen prediction is twice as large. Finally, using the actual average crest level at $Z + 11.11$ (using R_{c3} , cf. Fig. 13.c), the predicted q - values by van der Meer et al. and Besley are in good agreement with the measured q - values, and the Owen prediction still overestimates the measured average overtopping rates. In general best agreement between measured and predicted overtopping rates is observed using van der Meer et al.'s and Besley's prediction formulae for the actual average crest level $Z + 11.11$ and R_{c3} .

A more traditional presentation of the prediction formulae is given in Fig. 14, where a dimensionless overtopping discharge (using a logarithmic scale) is plotted versus a dimensionless crest freeboard. The thick solid line is the prediction formula itself, the thin solid lines are the 95 % confidence intervals of the formula (based on variation coefficients provided by the author of the formula). The measured field data have been plotted in the same graph using vertical bars to indicate the scatter from using the three calculation methods. Fig. 14.a, 14.b and 14.c show the resulting graphs for van der Meer et al., Owen and Besley, respectively. For each prediction formula, the graphs are shown for the three crest freeboards.

Obviously, the same observations as from Fig. 13 are made. Using R_{c1} (with crest level at $Z + 12.02$), the van der Meer et al. and Besley prediction formulae underestimate the measured overtopping rates, and the Owen prediction formula overestimates the measured average overtopping rates. Measured overtopping rates are within the 95 % confidence intervals of all prediction formulae, and therefore reasonable agreement is found between measured and predicted values.

Using R_{c2} ($Z + 10.20$), the previously observed overestimation of the measured overtopping rates is clearly visible in Fig. 14, where most of the measured overtopping rates are even

outside the 95 % confidence intervals for Owen and Besley. In this case, bad agreement is found.

Finally, for the case using R_{c3} ($Z + 11.11$), good agreement is observed for the van der Meer et al. and Besley predictions, with all measured overtopping rates between the 95 % confidence intervals.

The prediction of van der Meer and Besley finally show better agreement than Owen's formula, and best agreement is found using the actual average crest level at $Z + 11.11$. Comparing results from the three crest levels used in this analysis, shows that the actual average crest level and the associated crest freeboard R_{c3} yield best agreement between predicted and measured average overtopping rates, whereas the minimum crest level yields overestimations outside the 95 % confidence intervals.

5 Conclusions

In this paper, full scale wave overtopping measurements carried out at the Zeebrugge rubble mound breakwater during 1999 – 2003 have been presented and analysed. The measurement set-up, using a waverider buoy to measure incident waves and an overtopping tank to catch the volumes of overtopping water, and equipped with an outflow weir on a short side and water depth measurements, has been used successfully to obtain reliable field data.

Eight storm records have been used in the analysis of the field data, with duration between 1 and 2 hours, and with significant wave heights ranging between 2.6 m and 3.9 m. Although the measured storm conditions are considerably lower than the design storm conditions, average overtopping rates close to 1 l/sm have occurred.

Three methods for deriving the average overtopping rate inside the overtopping tank have been used, based on measurements of outflow discharge over the weir and instantaneous water depth inside the overtopping tank. These methods yield results that are in good agreement, with deviations between 12 % and 24 % on average.

Average overtopping rates from the field data have been compared with predicted values from the widely used prediction formulae from van der Meer et al. (1998), Owen (1980) and Besley (1999). For application of these formulae to the case of a rubble mound breakwater, the value of the reduction factor for the surface roughness of the armour layer has to be determined. This reduction factor has been calculated according to the methodology reported in van der Meer & Stam (1992), and the value 0.51 has been used. The actual value for the crest freeboard parameter R_c for the case of the Zeebrugge breakwater is not obvious, and

therefore a sensitivity analysis has been carried out, varying the crest freeboard between a maximum value R_{c1} (with crest level at $Z + 12.02$), and a minimum value R_{c2} (with crest level at $Z + 10.20$), and the average value R_{c3} (with crest level $Z + 11.11$) between maximum and minimum values.

Using R_{c1} , the van der Meer et al. and Besley prediction formulae underestimate the measured overtopping rates, and the Owen prediction formula overestimates the measured average overtopping rates. Measured overtopping rates are within the 95 % confidence intervals of all prediction formulae. Using R_{c2} , all prediction formulae overestimate the measured average overtopping rates, with values outside the 95 % confidence interval. Best agreement between measured and predicted values is observed using the average crest level (with R_{c3}) and van der Meer et al's and Besley's prediction formulae. In general very good agreement between the prototype average overtopping rates and the prediction formulae of van der Meer and Besley is achieved, taking into account the right choice of surface roughness and crest freeboard parameters.

Acknowledgements

The Coastal Division of the Flemish Community owns the prototype monitoring system and provided funds for developments within national and international research projects. Presented results are obtained partly within the European Community funded project CLASH ('Crest Level Assessment of Coastal Structures by full scale monitoring, neural network prediction and Hazard analysis on permissible wave overtopping' – contract N°: EVK3-CT-2001-00058) within the Fifth Framework Programme (2002-2004). The financial support of the Flemish Community and the European Community is very much acknowledged. The authors thank Tom Versluys (Ghent University) for his contribution in the design and installation of the measurement infrastructure.

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TABLES

Table 1. Overview of available data sets of full scale wave overtopping measurements, with storm identification number, date and duration of the wave records.

Storm No.	Date	Time	Duration T_0 (s)
1	6 November 1999	11h30 – 13h30	7200
2	6-7 November 1999	23h45 – 01h45	7200
3	8 November 2001	16h15 – 18h15	7200
4	26 February 2002	12h30 – 14h30	7200
5a	27 October 2002	17h00 – 18h00	3600
5b	27 October 2002	18h00 – 19h00	3600
5c	27 October 2002	19h00 – 20h15	4500
6	29 January 2003	10h00 – 12h00	7200

Table 2. Wave characteristics, surf similarity parameter and water level for the storms.

Storm No.	H_{m0}	H_s	$T_{m-1,0}$	T_p	T_m	$\xi_{m-1,0}$	SWL
	(m)	(m)	(s)	(s)	(s)	(-)	(m Z)
1	3.04	2.89	6.88	7.34	5.70	3.52	5.28
2	2.60	2.44	6.93	7.34	5.36	3.84	5.11
3	3.47	3.31	8.41	10.28	6.35	4.03	5.01
4	2.63	2.48	6.49	7.91	5.32	3.57	4.24
5a	3.74	3.61	7.50	8.57	6.21	3.46	4.40
5b	3.86	3.71	7.64	8.57	6.35	3.47	4.60
5c	3.71	3.55	7.98	8.57	6.45	3.70	4.35
6	3.16	3.03	7.28	7.91	5.94	3.66	4.71

Table 3. Average overtopping rates q_{ceq} , q_{Vi} and $q_{\Delta h}$ for all storms, calculated using the 3 methods based on the continuity equation, the individual overtopping volumes and the water depth jumps, respectively, and the number of overtopping events N_{ov} .

Storm No.	q_{ceq} (l/sm)	q_{Vi} (l/sm)	$q_{\Delta h}$ (l/sm)	N_{ov} (-)
1	3.161E-02	5.709E-02	4.677E-02	10
2	2.299E-02	2.211E-02	1.842E-02	3
3	2.825E-01	3.310E-01	3.588E-01	29
4	3.919E-03	1.010E-02	9.031E-03	1
5a	4.037E-01	5.158E-01	4.404E-01	19
5b	5.919E-01	8.585E-01	5.963E-01	30
5c	6.296E-01	7.036E-01	6.780E-01	31
6	8.479E-02	9.620E-02	8.646E-02	9

Table 4. Predicted average overtopping rates q_{vdm} calculated using the van der Meer et al. (1998) formula (eq. 22), for all storms, for the Zeebrugge breakwater with $\tan \alpha = 1/1.4$, $\gamma_b = 1$, $\gamma_f = 0.51$, $\gamma_\beta = 1$, $\gamma_v = 1$. Crest freeboards calculated using crest level $Z + 12.02$ (R_{c1}), $Z + 10.20$ (R_{c2}) and $Z + 11.11$ (R_{c3}), respectively.

Storm No.	H_{m0} (m)	SWL (m Z)	$\xi_{m-1,0}$ (-)	Crest level $Z + 12.02$			Crest level $Z + 10.20$			Crest level $Z + 11.11$		
				R_{c1} (m)	R_{c1}/H_{m0} (-)	q_{vdm1} (m ³ /sm)	R_{c2} (m)	R_{c2}/H_{m0} (-)	q_{vdm2} (m ³ /sm)	R_{c3} (m)	R_{c3}/H_{m0} (-)	q_{vdm3} (m ³ /sm)
1	3.04	5.28	3.52	6.74	2.22	4.10E-05	4.92	1.62	8.67E-04	5.83	1.92	1.88E-04
2	2.60	5.11	3.84	6.91	2.66	3.43E-06	5.09	1.96	1.22E-04	6.0	2.31	2.04E-05
3	3.47	5.01	4.03	7.01	2.02	1.36E-04	5.19	1.50	1.98E-03	6.10	1.76	5.19E-04
4	2.63	4.24	3.57	7.78	2.96	7.11E-07	5.99	2.28	2.42E-05	6.90	2.62	4.15E-06
5a	3.74	4.40	3.46	7.62	2.04	1.40E-04	5.80	1.55	1.67E-03	6.71	1.79	4.83E-04
5b	3.86	4.60	3.47	7.42	1.92	2.63E-04	5.60	1.45	2.92E-03	6.51	1.69	8.76E-04
5c	3.71	4.35	3.70	7.67	2.07	1.18E-04	5.85	1.58	1.44E-03	6.76	1.82	4.14E-04
6	3.16	4.71	3.66	7.31	2.31	2.66E-05	5.49	1.74	5.01E-04	6.40	2.03	1.15E-04

Table 5. Predicted average overtopping rates q_{owen} calculated using Owen (1980) formula (eq. 24), for all storms, for the Zeebrugge breakwater with $a_o = 0.00866$ and $b_o = 19.96$, and $\gamma_r = 0.51$. Crest freeboards calculated using crest level $Z + 12.02$ (R_{c1}), $Z + 10.20$ (R_{c2}) and $Z + 11.11$ (R_{c3}), respectively.

Storm No.	H_s (m)	T_m (s)	SWL (m Z)	Crest level Z + 12.02			Crest level Z + 10.20			Crest level Z + 11.11		
				R_{c1} (m)	$\frac{R_{c1}}{T_m \sqrt{gH_m}}$ (-)	q_{owen1} (m ³ /sm)	R_{c2} (m)	$\frac{R_{c2}}{T_m \sqrt{gH_m}}$ (-)	q_{owen2} (m ³ /sm)	R_{c3} (m)	$\frac{R_{c3}}{T_m \sqrt{gH_m}}$ (-)	q_{owen3} (m ³ /sm)
1	2.89	5.70	5.28	6.74	0.22	2.32E-04	4.92	0.16	2.44E-03	5.83	0.19	7.52E-04
2	2.44	5.36	5.11	6.91	0.26	3.63E-05	5.09	0.19	5.52E-04	6.0	0.23	1.42E-04
3	3.31	6.35	5.01	7.01	0.19	9.00E-04	5.19	0.14	6.47E-03	6.10	0.17	2.41E-03
4	2.48	5.32	4.24	7.78	0.29	1.36E-05	5.99	0.22	1.92E-04	6.90	0.26	5.11E-05
5a	3.61	6.21	4.40	7.62	0.21	5.89E-04	5.80	0.16	4.06E-03	6.71	0.18	1.55E-03
5b	3.71	6.35	4.60	7.42	0.19	1.01E-03	5.60	0.15	6.51E-03	6.51	0.17	2.57E-03
5c	3.55	6.45	4.35	7.67	0.20	7.23E-04	5.85	0.15	4.71E-03	6.76	0.18	1.85E-03
6	3.03	5.94	4.71	7.31	0.23	2.20E-04	5.49	0.17	1.99E-03	6.40	0.20	6.62E-04

Table 6. Corrected average overtopping rates q_{besley} on Owen (1980) formula (eq. 24) for influence of permeable crest berm, for all storms, for the Zeebrugge breakwater with $C_w = 5.0$ m, using crest level $Z + 12.02$ (R_{c1}), $Z + 10.20$ (R_{c2}) and $Z + 11.11$ (R_{c3}), respectively.

Storm No.	H_s (m)	T_m (s)	C_r (-)	q_{besley1} (m ³ /sm)	q_{besley2} (m ³ /sm)	q_{besley3} (m ³ /sm)
1	2.89	5.70	0.23	5.30E-05	5.57E-04	1.72E-04
2	2.44	5.36	0.14	5.14E-06	7.81E-05	2.00E-05
3	3.31	6.35	0.32	2.86E-04	2.05E-03	7.66E-04
4	2.48	5.32	0.15	2.02E-06	2.85E-05	7.59E-06
5a	3.61	6.21	0.38	2.26E-04	1.56E-03	5.93E-04
5b	3.71	6.35	0.41	4.10E-04	2.64E-03	1.04E-03
5c	3.55	6.45	0.37	2.67E-04	1.74E-03	6.83E-04
6	3.03	5.94	0.26	5.66E-05	5.13E-04	1.70E-04