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Empirical overtopping law for very shallow foreshores

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Abstract

Wave overtopping is one of the key parameters for designing coastal structures: the crest level is usually determined using admissible overtopping discharges. Several formulae already exist to predict the average overtopping discharge per meter width of the coastal defence, generally for deep or intermediate water depths at the toe of the dike. However, the process of wave overtopping on sea dikes with shallow and very shallow foreshore is not yet fully understood. Gentle foreshores in combination with (very) shallow water conditions lead to heavy wave breaking and a significant change of the wave spectra from offshore to the toe of the dike. The wave steepness is assumed as one of the main criteria to identify cases of severe wave breaking on shallow and very shallow foreshores. For these conditions, Van Gent's formula, generally used for wave overtopping with shallow foreshores, has been implemented and validated against experimental data. It is the purpose of this report to show that Van Gent's formula overestimates the average overtopping discharge for cases of very shallow foreshores. Moreover the existing formula cannot be applied to cases with an emergent toe. The present work introduces a new "equivalent slope" concept to obtain an estimation of average wave overtopping discharges on sea dikes with shallow and very shallow foreshores. This study uses data from CLASH database and experimental campaigns, specifically carried out at Flanders Hydraulics Research (Belgium) and Ghent University, in order to validate this approach. The data have been collected during the last 3 years. Results indicate that this concept shows better performance compared to other empirical formulae, which suggests that the influence of the very shallow foreshore on the average wave overtopping discharge should be included.

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Preface

Initially, the project comprised physical model tests on sea dikes with 1:3 and 1:6 slope characterised by 1:35 foreshore slope. The tests were foreseen specifically to attain the objectives of the project. These objectives also included to investigate a possible correlation between regular wave overtopping and irregular wave overtopping. During the execution of the project, it was decided to leave out this scope from the project and to focus all efforts on collecting more data from different experimental campaigns in order to achieve a more accurate model for the wave overtopping discharge of sea dikes with (very) shallow foreshores. Therefore, the project was several times suspended, waiting for other experimental campaigns to be finalized.

The present report comes after the publication of the results in the Coastal Engineering (CENG) Journal and, therefore, includes the contents of the published paper with few further details.

1 Introduction

Wave overtopping is one of the major issues affecting coastal defenses against flooding. Overtopping occurs when waves run up the seaward face of coastal defenses, reach the crest of the structure and pass over it. Since overtopping is a non-linear and stochastic phenomenon, mean discharges are generally accepted as reliable quantities to be used in the wave overtopping assessment and in the design of coastal structures. The average wave overtopping discharge is defined as the average discharge per meter width of the structure, *q*, expressed in $m^3/s/m$ or in *l/s/m*. The average wave overtopping discharge is calculated in relation to the freeboard of the structure; for sea dikes, wave overtopping can reach the rear part of the structure in cases of limited crest width. For specific conditions, the effects of the width and the slope of the crest on the overtopping discharge have to be properly characterized. The effects of crest width on wave overtopping is not analyzed in the present work.

Unlike most of the Mediterranean countries, which have narrow foreshore widths, Northern European Countries are characterized by large tidal ranges, often with wide and shallow foreshores, where the sea dike is located at the end (Veale et al., 2012). An example from the Belgian coast is depicted in Figure 1, where the foreshore is shaded in yellow. The presence of the foreshore influences the wave processes that occur from offshore to the shoreline (wave transformation, breaking, wave overtopping). The most intensive study on wave run-up and wave overtopping with shallow foreshores has been conducted by Van Gent (Van Gent, 1999; Van Gent, 2001; TAW, 2002; Van Gent and Giarrusso, 2003). However, the foreshore is indirectly considered in run up and overtopping calculations: the effects of the foreshore on wave overtopping phenomena are therefore limited to the investigated range of parameters (incident wave height and wave period, foreshore slope, water depth at the toe, dike slope).

Figure 1 - Former sea dike at Wenduine, Belgium. The foreshore is shaded in yellow.

Considering the important role of sea dikes as both coastal defenses and residential or recreational spaces, it is mandatory to assess the influence of the foreshore on wave overtopping processes, to guarantee the safety of the coastal areas.

For all the aforementioned reasons, physical model tests have been carried out at Flanders Hydraulics Research (Antwerp, Belgium) and Ghent University in the past few years to investigate the wave overtopping of sea dikes with very shallow foreshores (Altomare et al., 2014; Chen et al., 2015; Hohls, 2015). The results have been gathered together with data collected from other different studies reported in literature: in this way a wider and more diverse database has been established.

In this work a new concept is introduced for the "equivalent slope in shallow water conditions" leading to the derivation of a new empirical formula. The accuracy of this novel approach has been verified and compared with existing formulae. The prediction of wave overtopping for sea dike with shallow foreshore is as accurate as in Van Gent (1999). However the new model extends the overtopping calculation outside the range of validity of Van Gent (1999), in particular to cases characterized by very shallow foreshores, for which the new concept gives accurate results. Finally the use of the equivalent slope concept leads to more accurate results than those achieved by applying Goda (2009), which based the analysis using only CLASH data (Verhaeghe et al., 2004). Furthermore the new formula can be also applicable even in case of an emergent toe.

This report is organised as follows: the state-of-art in estimating wave overtopping for shallow foreshores is summarized in Chapter 2, with specific focus on the existing formulae on wave overtopping of sea dikes with shallow foreshores. Chapter 3 describes the experimental data that have been used for the analysis. Chapter 4 analyses the physical model results and compares them with the prediction of existing semi-empirical formulae. A new conceptual model to study wave overtopping in very shallow foreshore is introduced in Chapter 5 and validated against the aforementioned experimental data. Finally, the main conclusions are reported in Chapter 6. The symbols used in the present paper are defined when first used and they are listed in the glossary after the references.

2 State-of-the-art in wave overtopping

The prediction of overtopping rates usually includes both “green-water” and “white-water” overtopping. The former one consists in complete sheets of water passing over the crest of the costal defence; the latter occurs in cases of significant splashes, due to heavy wave breaking on the seaward face of the structure along with strong wind conditions and is characterized by non-continuous overtopping and/or significant volumes of spray (EurOtop, 2007).

The most relevant parameters in wave overtopping that are included in semi-empirical formulae are the incident significant wave height H_{m0} and spectral wave period $T_{m-1,0}$ at the toe of the dike, the crest freeboard, the dike slope, geometrical and structural features of the dike (such as surface roughness, presence of a berm, etc.). The spectral wave period is preferred over either the peak period T_p or the average period T_m in wave overtopping calculations, because it gives more weight to the longer periods in the spectrum. Furthermore, the same $T_{m-1,0}$ along with same wave heights lead to similar overtopping discharges, independently of the spectrum type, even in case of double-peaked or flattened spectra (EurOtop, 2007). A detailed analysis of the use of the spectral period for overtopping predictions is contained in Van Gent (1999) where the author demonstrated that the spectral period shows a better performance than the other wave periods both for wave overtopping and wave run-up predictions.

The overtopping of sea defences due to random waves can be estimated using several methods such as the use of design diagrams (Goda, 1975; Tamada et al., 2012) or empirical formulae (van der Meer and Janssen, 1995; EurOtop, 2007; Goda, 2009; van der Meer and Bruce, 2014; Mase et al., 2013; Van Doorslaer et al., 2015). There is a substantial difference in these approaches. Goda (1975), Tamada et al. (2012) and Mase et al. (2013) employ deep water wave parameters while others use the wave height and period at the toe of the structure (Franco et al., 1994; van der Meer and Janssen, 1995; Van Gent, 1999; EurOtop, 2007; Goda, 2009; van der Meer and Bruce, 2014).

The toe of the structure is here defined as the location where the foreshore meets the slope of the structure (EurOtop, 2007). Hedges and Reis (2004) analyse the role of the location where the wave boundary conditions are specified. The most common locations are offshore, at the toe of the foreshore and at the toe of the structure. Each method has advantages and drawbacks. Offshore location is easy to define and often used in practice; however the effects of wave-current interaction, wave energy spreading and other processes that occur between offshore and the shoreline are missing. The toe of the foreshore in general is difficult to identify and the definition of the wave parameters must account for the wave transformation processes up to the toe of the dike. In both cases of location definition for the hydraulic boundary conditions, offshore and at the toe location of the foreshore, the sea dike and the foreshore must be treated as a single entity in overtopping calculations. Thus it is difficult to take into account the details of the differences in dike structure. On the other hand, when the toe of the sea dike is used for the overtopping calculation, the foreshore must be treated separately from the sea dike: the influence of the wave breaking on the foreshore must be considered. The advantages of the method that uses the toe of the dike are that all the information are included in the wave parameters, such as wave transformation from offshore to the shoreline including wave energy spreading. Even so, it is still an open question to define wave boundary conditions in the case of an emergent toe. Allsop et al. (2005) referred to deepwater conditions to evaluate wave overtopping due to broken waves on vertical and steep seawalls in case of emergent toe: the authors suggest an adaptation of the formula for plunging wave overtopping by van der Meer and Janssen (1995) using the foreshore slope as the characteristic slope and the offshore wave boundary conditions. Mase et al. (2013) proposed new formulations for wave run-up and overtopping discharge in very shallow water by using the deep water wave characteristics.

In this work we refer to wave parameters at the toe of the dike as in most overtopping formulae developed in Europe, even in cases of an emergent toe. The accuracy of this choice is demonstrated later on in the analysis sections (§4). In case wave conditions at the toe of the dike are not available, scale model tests and numerical modelling can be employed to cover this lack of information.

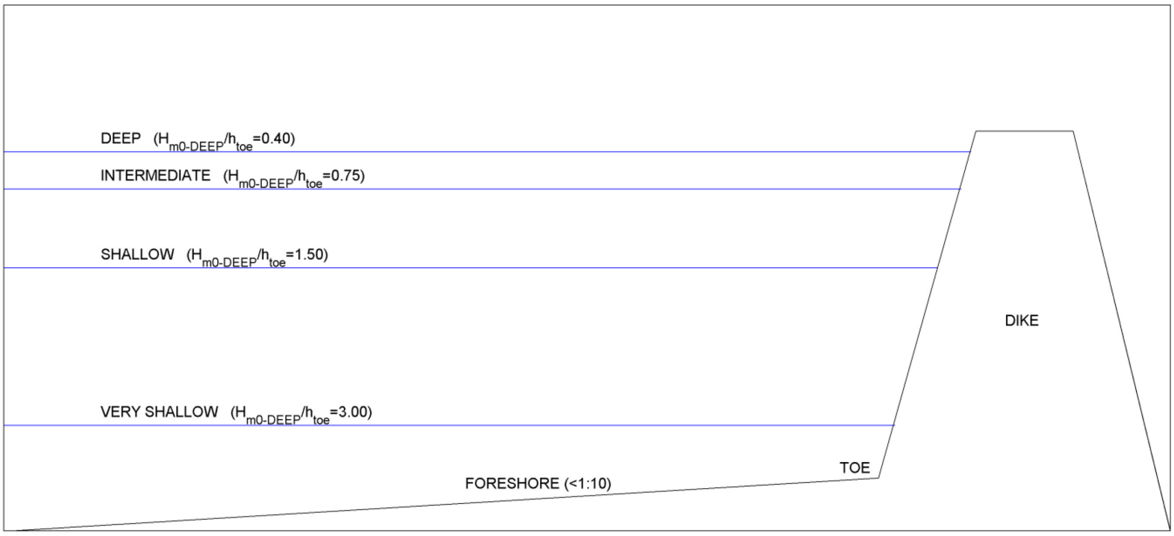
2.1 Definition of shallow and very shallow foreshores

The foreshore is defined as the section in front of the dike and it can be horizontal or up to a maximum slope of 1:10. Goda (2009) discussed the terminology of "foreshore" and its correct use in European references. According to the Rock Manual (CIRIA, CUR, CETMEF, 2007) the foreshore can also be defined as the part of the shore lying between mean high water and mean low water level, or according to the Coastal Engineering Manual (U.S. Army Corps of Engineers, 2002) the foreshore can be defined as part of the shore, lying between the crest of the seaward berm (or upper limit of wave wash at high tide) and the ordinary low-water mark.

Deep, shallow and very shallow foreshores can be distinguished as follows. A shallow and a very shallow foreshore are identical to a deep foreshore, however the waves in those conditions break due to the depth limitation. The wave height on the shallow foreshore and on the very shallow foreshore is therefore much smaller than the offshore wave height. A clear criterion for the differentiation between shallow foreshore and very shallow foreshore does not exist to date.

Van Gent (1999) proposed a criterion to determine if the foreshore is characterized by deep water, intermediate, shallow or very shallow water: if the ratio between the wave height in deep water, $H_{m0\text{-}DEEP}$, and the water depth the toe of the dike, h_{toe}, is greater than 0.75 and smaller than 1.50, then the foreshore can be considered as shallow; if the same ratio is greater than 3.0, then the foreshore can be considered as very shallow. In all the other cases, the foreshore has to be assumed as intermediate or deep. The criteria from Van Gent (1999) is schematized in Figure 2.

Figure 2 - Scheme of foreshore and dike for different water depths



Source: based on Van Gent (1999)

In a shallow foreshore breaking waves and wave height are lower, but there is still a spectrum similar to the original spectrum. At a very shallow foreshore, itis difficult to recognize a spectrum with a peak. Generally speaking, the transition between shallow and very shallow foreshores can be defined as the situation where the original incident wave height has been decreased by 50% or more, due to wave breaking. The effect of a (very) shallow foreshore translates into a high value of the breaker parameter ($\xi_{m-1,0}$>5/7) with relatively gentle dike slopes (e.g. 1:2.5). A large breaker parameter is also found at deep foreshores with a steeper slope (1:2 or steeper). In that case, the ratio of the incident offshore wave height and the wave height at the toe of the dike might be used as further criterion to determine whether there is a (very) shallow foreshore. Quoting TAW (2002), *"it is clear that for heavy breaking waves they show almost no signs of a spectrum with a well-defined peak period (the spectrum has been 'flattened') and that the spectral period $T_{m-1,0}$ is the obvious parameter. Another aspect that plays a role for a very shallow foreshore is that very long waves (surfbeat) can occur due to the breaking. It is possible that this long wave energy is the cause of the relatively high run-up values for large values of the breaker parameter… (omissis)… No study has yet been completed in this area....".*

Recent research has led to the redaction of a second edition of the European Overtopping Manual (EurOtop, 2016, in press). A different criterion, based on wave steepness $s_{m-1,0}$, is proposed in the manual to identify whether or not the foreshore can be considered (very) shallow. In particular it is stated in EurOtop (2016) that: "shallow foreshores may be present if the wave steepness at the toe of the structure becomes smaller than $s_{m-1,0} < 0.01$". This statement has been further checked for the most common sea dikes, the slope of which is between 1:8 and 1:2, within the framework of the redaction of the "Methodology for the Assessment of the Coastal Safety for the Belgian coast" (see Suzuki et al., 2016). The results of this analysis pointed out that the wave steepness is indeed the most optimal criterion for (very) shallow foreshores. It has been verified that for $s_{m-1,0} > 0.01$, the breaker parameter is always less than 5: for sea dikes the formula for breaking/non-breaking waves will always be used (see section 2.2), independently from the dike slope, since the breaker parameter will be always less than 5 for slopes between 1:8 and 1:2. This is the case of breaking and non-breaking waves, where Van Gent's formula is not applicable. Therefore the wave steepness has been selected in the present manuscript as parameter to identify whether the foreshore is shallow or not. Based on the considerations above, a criterion has been set as follows: if $s_{m-1,0} \leq 0.01$, then the foreshore can be considered shallow (or very shallow).

Hereafter, a criterion will be presented to distinguish between shallow and very shallow foreshores, based on the ratio between the water depth at the toe and the wave height at the toe (h_{toe}/H_{m0}).

2.2 Wave overtopping formulae for sea dikes with shallow and very shallow foreshores

The effect of shallow and very shallow foreshore on wave height and period affects the wave overtopping. It is common practice to use empirical formulae for overtopping calculations. Those formulae give a reasonable overtopping discharge for a typical dike configuration based on a large number of physical model tests, not only for deep water conditions. EurOtop (2007), for example, includes a formula for shallow foreshore cases (Van Gent, 1999). Van der Meer and Bruce (2014) reviewed the overtopping formulae contained in the EurOtop manual for low-crested or zero-freeboard sea dikes, proposing a Weibull-structured formula, but they did not include the very shallow foreshore condition in the analysis.

Goda (2009) proposed a set of unified formulae to predict mean wave overtopping discharge at coastal structures with smooth, impermeable surfaces, obtained by the analysis of selected CLASH datasets (Verhaeghe et al., 2004). The formulae are applicable for vertical walls and sea dikes. The effects of the toe depth and of the seabed slope on wave overtopping have been incorporated in the formulae.

The formulae proposed in Mase et al. (2013) and based on deep water wave characteristics are not used in the present work. Two main features of this approach are:

a) the run-up is estimated by a new introduced formula and then the calculated run-up is used to assess wave overtopping;
b) an imaginary slope is defined, and used in overtopping calculations, based on the estimation of the water depth at wave breaking.

However, uncertainties lie in the estimation of the wave run-up based on offshore conditions and of the wave breaking depth. Furthermore, directional spreading effects and wave-current interaction are not taken into account for a correct estimation of the waves that finally reach the structures. For the aforementioned reasons, Mase et al. (2013) has not been used for further comparisons.

Van Gent (1999) carried out small scale model tests on a 1:100 and 1:250 foreshores with smooth structure slopes of 1:4 and 1:2.5. Due to the heavy breaking the spectral wave period, $T_{m-1,0}$, can change drastically. One example shows that the spectral wave period was changed from 2-3 s at deep water to 8 s at the toe of the structure. This implies a significant change of spectral shape as well. Wave heights are also reduced from roughly 0.14 m to 0.04 m in this case. With such small wave heights and very long periods at the toe of the structure, the breaker parameter becomes very large, around $\xi_{m-1,0}$ = 14 for a 1:4 slope and $\xi_{m-1,0}$ = 20 for a 1:2.5 slope. The breaker parameter $\xi_{m-1,0}$ is a combination of slope angle and wave steepness. It is defined as:

$$\xi_{m-1,0} = \frac{\tan(\alpha)}{\sqrt{{2\pi H_{m0}}/{gT_{m-1,0}^2}}} \tag{1}$$

where:

- H_{m0} is the significant incident wave height at the toe of the structure [m]
- $T_{m-1,0}$ is the spectral wave period [s]
- α is the dike angle [°]

In the case of (very) shallow foreshore, large breaker parameters may be found for very low wave steepness.

The formula proposed by Van Gent (1999) and also reported in TAW (2002) and EurOtop (2007) is expressed by the following two equations:

$$\frac{q}{\sqrt{gH_{m0}^3}} = 10^c exp\left(-\frac{R_c}{H_{m0}\gamma_f\gamma_\beta(0.33+0.022\xi_{m-1,0})}\right) \tag{2a}$$

$$\frac{q}{\sqrt{gH_{m0}^3}} = 0.21 exp\left(-\frac{R_c}{H_{m0}\gamma_f\gamma_\beta(0.33+0.022\xi_{m-1,0})}\right) \tag{2b}$$

where:

- q is the overtopping discharge per meter width of the structure [m3/s/m]
- R_c is the crest freeboard [m]
- γ_f is the reduction coefficient that considers the effects of the slope roughness [-]
- γ_β is the reduction coefficient that considers the effects of the obliqueness [-]

Equations (2a) and (2b) refer respectively to the probabilistic and deterministic approach as in EurOtop (2007). The c parameter in equation (2a) is assumed as normally distributed parameter with mean value equal to -0.92 and a standard deviation σ equal to 0.24. The 5% upper and under exceedance limits can be calculated as (-0.92)±1.64σ. The value of 10-0.92 is replaced in equation (2b) by 0.21 (≈10-0.92+ σ) which represents a safer evaluation of the mean overtopping discharge. All the data from Van Gent (1999) are also included in CLASH database and they refer to tests from relatively deep water to shallow water conditions (Hm0-DEEP/htoe ≤1.50).

Equations (2a) and (2b) are valid for $\xi_{m-1,0}$ ≥7 (EurOtop, 2007; TAW, 2002; Rock Manual, 2007). In the case of $\xi_{m-1,0}$ ≤5, it is recommended to use the following pair of formulae (for probabilistic approach):

$$\frac{q}{\sqrt{gH_{m0}^3}} = \frac{0.067}{\sqrt{tan\alpha}}\gamma_b\xi_{m-1,0}exp\left(-4.75\frac{R_c}{H_{m0}\gamma_f\gamma_\beta\gamma_v\gamma_b\xi_{m-1,0}}\right)$$

with a maximum of: $$\frac{q}{\sqrt{gH_{m0}^3}} = 0.2exp\left(-2.6\frac{R_c}{H_{m0}\gamma_f\gamma_\beta}\right) \quad (3)$$

The reliability of Eq. (3) is described by taking the coefficients 4.75 and 2.6 as normally distributed parameters with means of 4.75 and 2.6 and standard deviation of 0.5 and 0.35 respectively (EurOtop, 2007).

In the case of 5< $\xi_{m-1,0}$ <7, a linear interpolation is recommended between the two sets of formulae. It must be noticed that the experimental results from Van Gent (1999) show values of $\xi_{m-1,0}$ that are even much lower than 5-7 for several cases. This actually represents an incongruity with the criteria $\xi_{m-1,0}$ ≥7 for the application of Eq. (3). In any case, the breaker parameter will not be used for the following analysis: the criterion adopted by the authors is the wave steepness (as also explained in section 2.1).

Goda (2009) proposed a simple exponential functional form for the overtopping formula, defined as follows:

$$\frac{q}{\sqrt{gH_{m0}^3}} = exp\left[-\left(A + B\frac{R_c}{H_{m0}}\right)\right] \quad (4)$$

with

$$A = A_0\left[(0.956 + 4.44tan\theta)x\left(\frac{h_{toe}}{H_{m0}} + 1.242 - 2.032(tan\theta)^{0.25}\right)\right] \quad (5)$$

$$B = B_0\left[(0.822 + 2.22tan\theta)x\left(\frac{h_{toe}}{H_{m0}} + 0.578 + 2.22tan\theta\right)\right] \quad (6)$$

where ϑ is the angle of the foreshore and h_{toe} is the water depth at the toe of the structure. The coefficients A_0 and B_0 are expressed as functions of the dike slope α:

$$A_0 = 3.4 - 0.734cot\alpha + 0.239cot^2\alpha - 0.0162cot^3\alpha \quad (7)$$

$$B_0 = 2.3 - 0.5cot\alpha + 0.15cot^2\alpha - 0.011cot^3\alpha \quad (8)$$

Eqs. (5-8) were derived by Goda using 715 data points for vertical walls and 1254 data points for sloping dikes respectively, both extracted from CLASH database. To cover a lack of data in shallow foreshore Goda also used data from Tamada et al. (2002). However very few data refer to zero or to very low water depth at the toe of the dike and no data with emergent toes are included. Therefore, in case of very shallow foreshore and emergent toes, the new set of equations proposed by Goda (2009) have to be used carefully and only for a preliminary assessment.

2.3 Research beyond the state-of-the-art

An empirical formula is a functional and simplified representation of the phenomenon that is object of the study. It is functional for a specified aim, since for the same phenomenon there might exist different models to achieve different objectives (e.g. different formulae for wave overtopping discharge, wave overtopping volumes, overtopping flow velocities, etc.). It is simplified because the whole complexity of the real phenomenon is reduced to those aspects that might be essential for the purpose of the model itself. In wave overtopping phenomena, an empirical formula is expressed as an equation, usually in exponential form, that depends on the incident wave height and on the period at the toe of the dike, the dike geometry (freeboard, slope, etc.) and the proper structural characteristics, like the roughness of the seaward face of the dike. The foreshore, despite the influence that it might have on wave transformation and breaking, is not usually taken into account in overtopping calculations. This seems reasonable in cases of deep or intermediate waters at the toe of the structure, where actually the waves do not "feel" the bottom before reaching the structure. In these cases, the waves start to break on the structure itself. Several formulae already exist for wave overtopping assessment which predict the average overtopping discharge per meter width of the coastal defence, generally for deep or intermediate water depths at the toe of the dike.

Gentle foreshores in combination with (very) shallow water conditions lead to heavy wave breaking and a significant change of the wave spectra from offshore to the toe of the dike. The only well-established study on wave overtopping in shallow water conditions is reported in Van Gent (1999). However, compared to that study, the water depth for typical cases from Belgian coast is much shallower. The influence of these very shallow foreshores on wave overtopping is not fully understood yet.

Therefore several experimental campaigns have been carried out at Flanders Hydraulics Research and Ghent University, in Belgium. The aim of these campaigns was to collect new results on wave overtopping of sea dikes with very shallow foreshores. The CLASH database was also analysed in order to extract those data that might correspond to shallow or very shallow water conditions and gentle foreshore. All the aforementioned data have been gathered together with the final objective of defining a new empirical model able to characterize the influence of the foreshore on average wave overtopping discharge.

2.4 Uncertainties in overtopping measurements and calculations

In general, the estimation of overtopping discharge, using either physical or numerical models, involves inherently a scatter of the results, as can be seen in e.g. EurOtop (2007). The scatter generates the model uncertainty that can be considered as the accuracy in the model prediction. The scatter in wave overtopping can be partly explained by the stochastic nature of the waves. For example, Williams et al. (2014) examined the role of offshore boundary conditions in the uncertainty of numerical prediction of wave overtopping. They pointed out that there is a higher level of uncertainty for low overtopping discharges compared to higher discharges and it is within this region that the variability due to different seeding numbers (i.e. wave trains) is higher. When random waves are generated under a specified wave spectrum, several runs of generated waves have different combinations of wave heights and periods, thus producing different volumes of total wave overtopping. The total overtopping volume for a train of random waves is governed by the number and the height of large waves above the threshold of overtopping. Differences become pronounced as the overtopping volume decreases. Romano et al. (2015) also investigated the role of the seeding number and of the test duration for the uncertainties in a physical model of wave overtopping. They clarified, for instance, the level of scatter on the overtopping estimation by using a different number of waves and seeding numbers, depending on crest freeboard.

When data are collected from different experimental campaigns carried out in different facilities around the world, additional differences can be found due to model effects. If a large number of various datasets is used to derive and calibrate wave overtopping formulae, a certain degree of spread of the overtopping rate should be expected and accepted as inevitable (EurOtop, 2007).

When different databases are selected, they must be as homogeneous as possible to reduce the possible sources of uncertainties (the criteria that have been followed to select and use data from CLASH database are summarized in section 3.2).

3 Experimental data

3.1 Scale model tests

Five different experimental campaigns have been carried out between 2012 and 2015, four of which took place at Flanders Hydraulics Research (dataset id.: 00-025, 00-142, 13-168 and 13-116) and one at Department of Civil Engineering of Ghent University (dataset id.: UGent), having as main objectives the characterization of wave overtopping and loading on coastal defences with shallow and very shallow foreshores. The characteristics of each dataset are summarized in Table 1, where *cot* ϑ is the foreshore slope and *cot* α is the dike slope. The dikes are smooth and the wave angle is always 0° (waves approaching perpendicular to the structure). The foreshore length is also reported. For tests 13-168, characterized by 1:50 foreshore slope, a transition slope of 1:15 was constructed between the wavemaker and the start of the foreshore to obtain a sufficient depth at the wavemaker location. The model scale was 1:25 for all tests. Tests 13-116 correspond to the experimental campaign conducted initially for the present project and to which results from the other campaigns have been finally added.

Table 1 - Characteristics of datasets from model tests

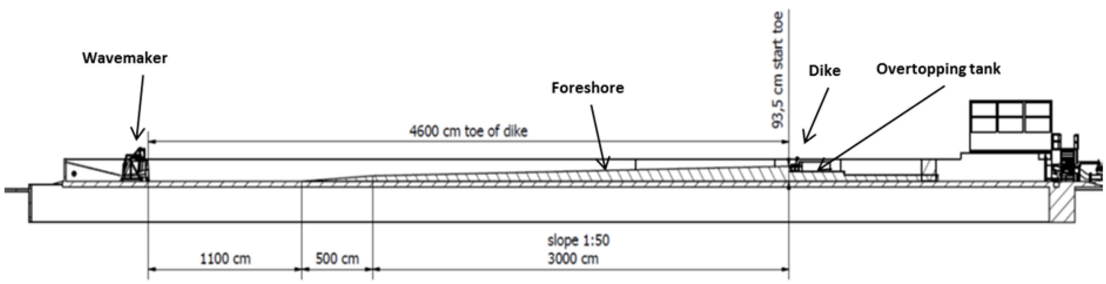
Dataset id.	Number of data	cotθ	foreshore length [m]	cotα	H_{m0} [m]	$T_{m-1,0}$ [s]	R_c [m]	h_{toe} [m]	h_{toe}/H_{m0}
13-116 [(a)]	90	35	~33	3, 6	0.039-0.065	9.66-16.93	0.083-0.118	(-0.01)-0.025	(-0.25)-0.57
00-142	17	35	~33	3	0.053-0.093	5.19-14.11	0.052	0.05-0.052	0.54-0.94
00-025 [(b)]	21	35	~33	2	0.030-0.061	7.22-12.30	0.045-0.127	(-0.026)-0.050	(-0.88)-0.81
13-168	42	50	30	2	0.024-0.070	4.17-11.77	0.015-0.150	0.008-0.087	0.00-1.28
UGent	34	35	~13	2	0.024-0.074	1.76-10.83	0.040-0.120	0.012-0.052	0.26-1.22
TOT	204								

[(a)] the wave height and wave period the at the toe have been estimated using SWASH model for 36 tests.

[(b)] the wave height and wave period at the toe have been estimated using SWASH model

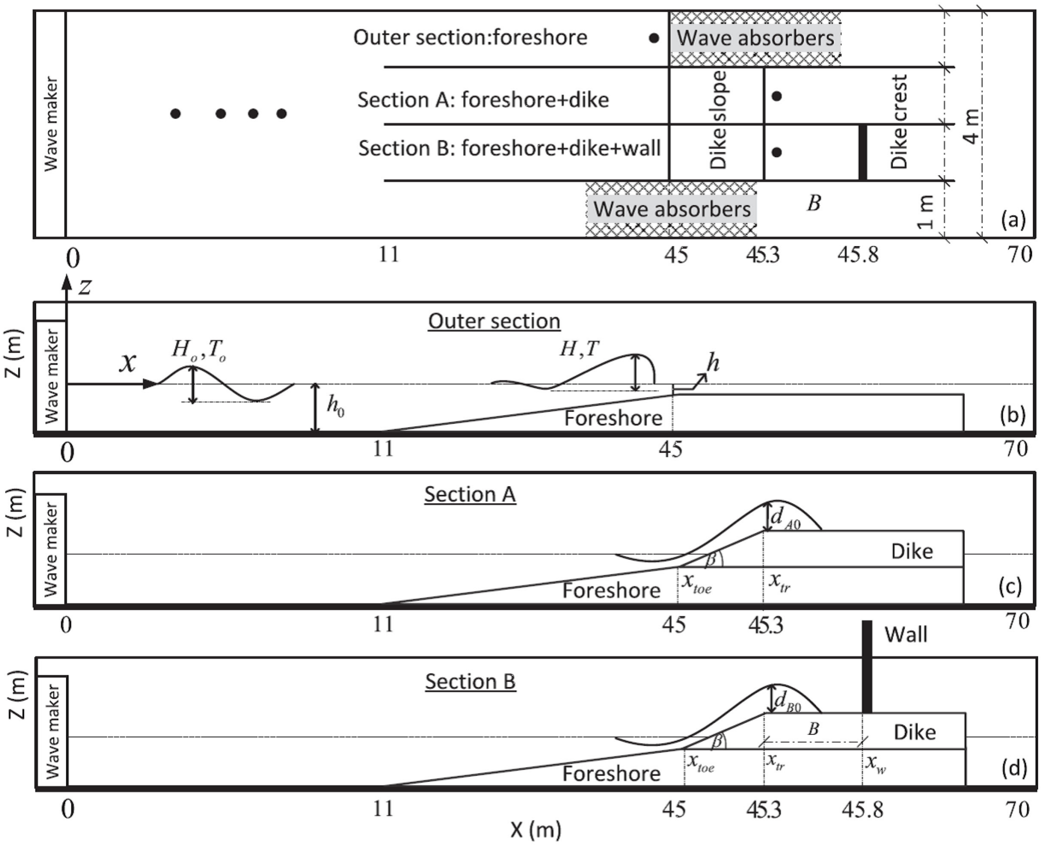
The wave flume at FHR is 70 m long, 4 m wide and 1.5 m high. The maximum water depth at the wavemaker (piston-type) is 1.2 m. Figure 1 shows a 2-D sketch of the wave flume at FHR. The setup corresponds to the dataset 13-168, with 1:50 foreshore slope. The setup in case of the rest of the dataset was similar, despite the different foreshore slope: the distance of the dike from the wavemaker and the toe elevation were the same for all the experimental campaigns.

Figure 3 - Sketch of wave flume at FHR with dike location
(example from experimental campaign 13-168)



The set-up of the experimental campaign 00-142 (also described in Chen et al., 2015) is depicted in Figure 4. The wave overtopping was measured in section A (Figure 4, b) meanwhile the incident wave boundary conditions were measured in the outer section (Figure 4, a). The same set-up of Figure 4 (b) and (c) has been used for the experimental campaign 13-116.

Figure 4. Experimental set-up of the 00-142 project



Source: Chen et al. (2015).

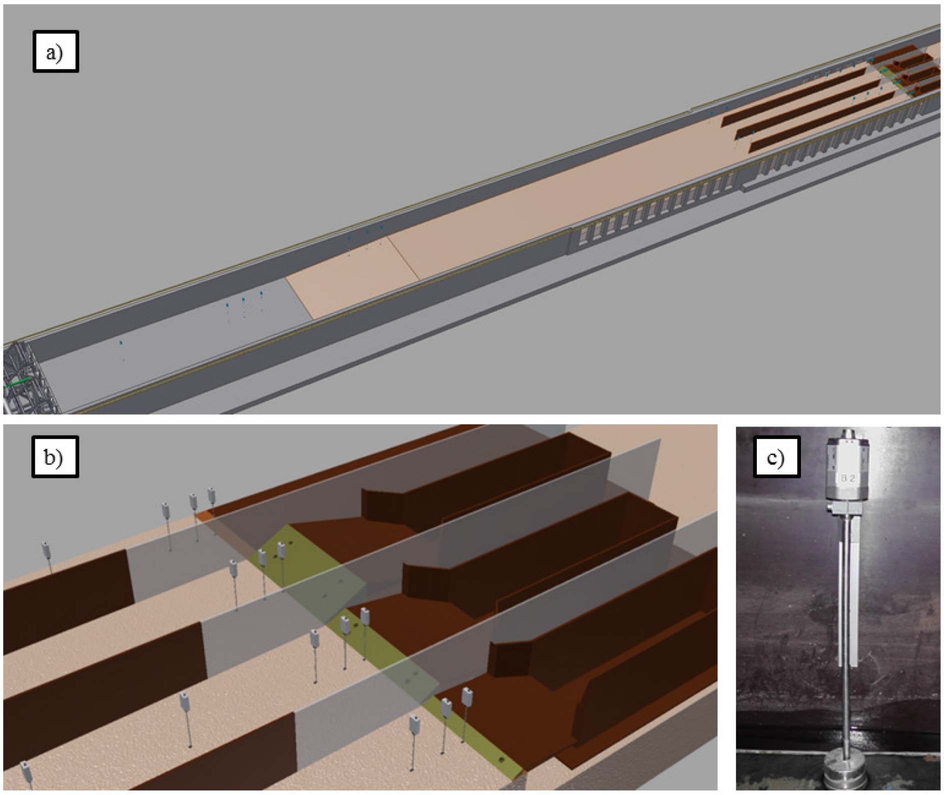
Further details on the experiments conducted at FHR can be found in Altomare et al. (2014) and Chen et al. (2015).

The incident wave height and period at the toe of the dike have been measured by means of resistive wave gauges. Classical reflection analysis methods (e.g. Mansard and Funke, 1980) are not suitable in shallow water conditions because non-linear effects are dominating in the very shallow foreshore cases (Van Gent, 1999). Instead, the measurements of wave height and period have been conducted using wave gauges at the location of the dike toe but, instead of having the sea dike, an horizontal platform has been inserted just after the foreshore and damping material has been located after the platform. The aim is to reduce as much as possible the reflection in order to measure only incident wave characteristics. The waves running over the foreshore dissipate most of their energy during wave breaking and the water that reach the location of the dike toe flows easily towards the back of the flume without any evidence of reflection.

The width of the wave flume at FHR is equal to 4 m and it allows using the flume to be split into different partitions. In Figure 5 a 3-D view of the entire flume (a) and of the zone close to the dikes (b) are depicted. The setup corresponds to the experimental campaign 13-168, being similar for the other experimental campaigns. The four partitions can be observed: one part (without dike) is used for measuring incident wave boundary conditions; the other three parts are used for overtopping measurements. The accuracy of the measurement of the incident wave boundary conditions has therefore been valuated verifying that this particular flume setup did not generate appreciable cross waves in the flume (due to the different reflection of each partition). In fact the waves within each section can be different due to the different end boundaries and reflection. This can cause a difference of the wave phase. However, this influence is not significant: the differences in the wave height measured along the width of the flume at a certain distance from the wavemaker vary between 4.7% (at the foreshore toe) and 6.5% (at the dike toe). Therefore the effect of the flume partition inducing cross waves was neglected.

The wave overtopping has been measured by collecting the water that overtopped the crest in overtopping boxes located downstream of the sea dikes. A short ramp sloped from the seaward edge of the dike crest into the overtopping box. The mean overtopping discharge was obtained by dividing the total volume of water collected during a test by the total duration of the test (usually corresponding to 1000 times the spectral period in deep water conditions). The instantaneous overtopping discharge was measured by the installation of two Baluff "Micropulse" water level sensors inside the overtopping boxes (Figure 5, c).

Figure 5 - 3D view of the wave flume and photo of the Baluff "Micropulse" water level sensor



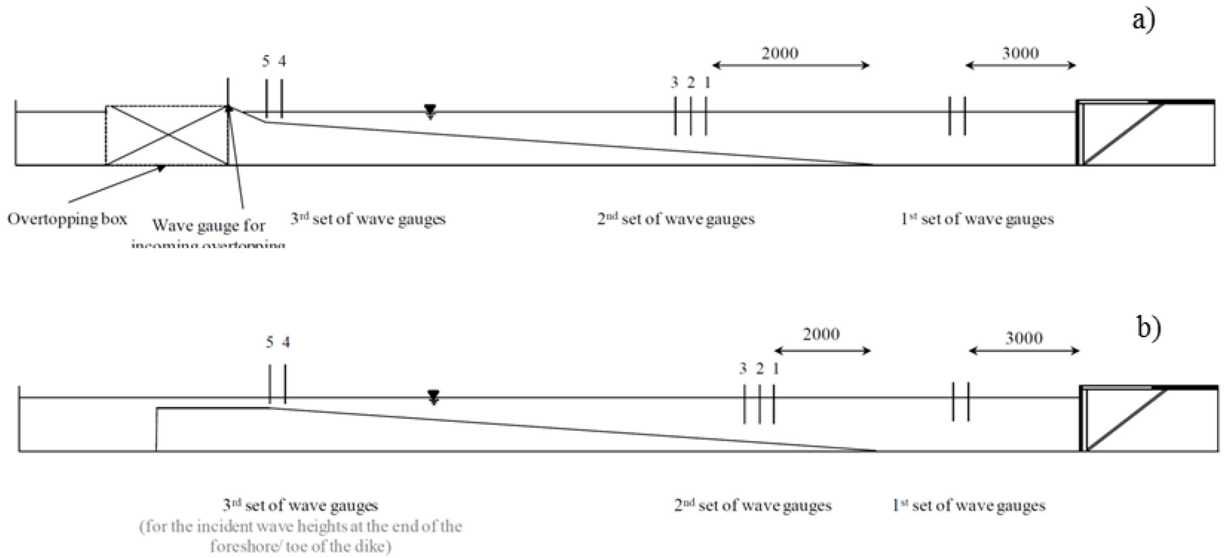
a) and b): 3D view of the wave flume at FHR;

c): photo of the Baluff "Micropulse" water level sensor for overtopping measurement.

A limitation in the experiments is the use of first-order wave generation in the FHR wave flume and the lack of active absorption of long-wave energy. For spurious long waves in the flume, the lack of a second-order wave generation is found to be negligible due to relative large water depth at the wave generator and low steepness (Ottesen-Hansen et al., 1980; Shah and Kamphuis, 1996). The natural frequency of the current flume set-up is around 0.017-0.025 Hz for different test conditions and foreshore slopes, which are found to be outside of the frequency range of the infragravity waves in the flume. Moreover, there is no increasing trend of the observed long-wave energy at the toe of the dike. A passive absorption system was found to be sufficient to reduce the wave reflection, except for the seiching motion around 0.017-0.025 Hz. The seiching frequency band was removed in the analysis of the wave parameters.

The setup used at UGent for wave overtopping measurements is depicted in Figure 6 (a). The wave flume has a width of 1 m, height of 1.2 m and a total length of 30 m. The wave paddle with a length of 3.15 m is included in the total length of the flume. Hence, an effective flume length of approximately 27 m can be used. The overtopping discharge has been measured by one overtopping box. The weight of the overtopping water mass has been measured by means of a weight cell. The signal of the weight cell was connected to the LabVIEW software: the water mass was recorded automatically. The mass divided by the testing time and the width of the plate results in the overtopping rate. The system includes also a pump connected to a reservoir, to be used in case the overtopping volumes exceed the capacity of the overtopping box. The flow out of the pump was also measured automatically. In this way, a measurement was always feasible even in cases of large discharges. All the details on the experiments carried out at Ghent University can be found in Hohls (2015).

Figure 6 - Scheme of the wave flume at UGent

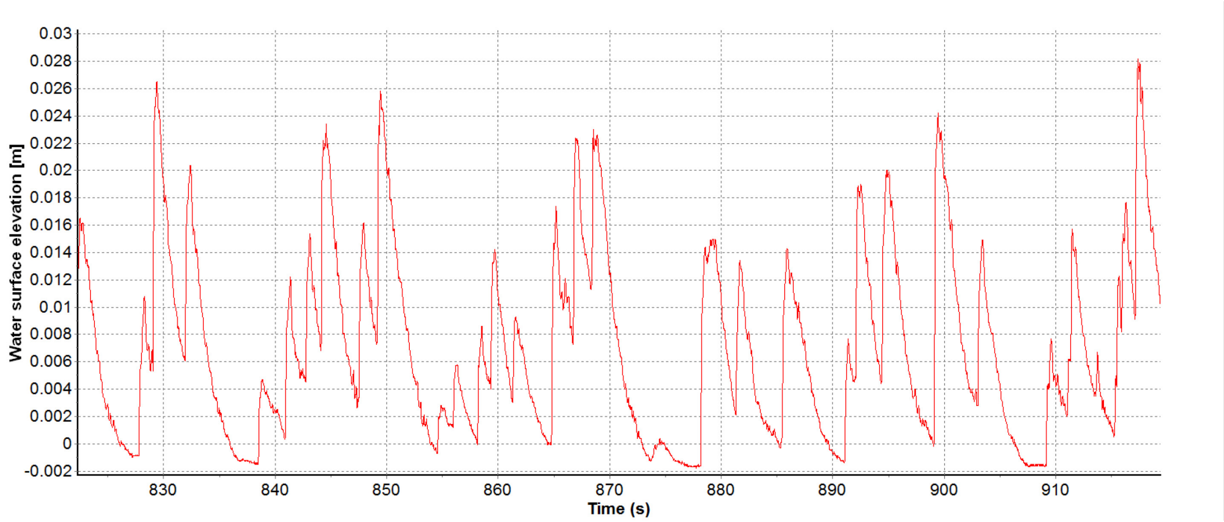


a) cross section for overtopping measurements; b) cross section for wave boundary conditions at the toe. Further details in Hohls (2015).

Similar to the tests conducted at FHR, the wave height and wave period have been measured using wave gauges at the location of the dike toe, but, removing the dike itself and installing an horizontal platform just after the foreshore. The difference in comparison with the FHR tests is that at FHR the incident wave conditions and the wave overtopping were measured simultaneously due to the split of the flume in several partitions as previously discussed. At UGent it was necessary to remove the dike and repeat the overtopping tests without a dike to measure the incident wave heights and period (Figure 6,b). The methods are equivalent and the choice of one (using more partitions simultaneously) or the other (removing the dike) depends on the optimization.

In both cases the novelty of the measurement of the incident wave boundary conditions is that, even though an oscillatory nature of the waves at the toe is no longer recognizable, the spectral momentum is still calculated to define a kind of equivalent significant wave height and the spectral wave period (Figure 7). The wave energy is still well represented in this way, although in many cases the waves reach the dike toe either as previously broken waves or bores. The accuracy of this choice will be demonstrated when showing the overtopping results.

Figure 7- Example of water surface elevation at the toe of the dike



Data from 13-168 dataset.

There were some uncertainties in the measured incident wave height and spectral wave period at the toe of the dike for tests from dataset 00-025 and for 36 of the 90 tests from dataset 13-116. In the case of 00-025 there was no direct measurement of the incident wave conditions at the toe in absence of the dike, instead only wave reflection analysis was performed and only at a distance from the toe equal to 5 times the wave height before breaking. For 13-116, the uncertainties were related to the accuracy of the wave gauges in some cases. Therefore the SWASH model (Zijlema et al., 2011) was used to obtain incident wave information at the toe of the dike. SWASH has been used as described as follows:

- SWASH was previously validated against cases from dataset 00-142 and 13-168 where the wave boundary conditions at the toe were measured during the experimental campaign. The results demonstrated the accuracy of SWASH predictions.
- SWASH was calibrated and validated against cases from datasets 00-025 and 13-116 in term of wave propagation with dike configuration: wave conditions along the flume (expect for those at the toe of the dike) and wave overtopping were compared.
- The dike has been removed in later simulations in SWASH and a sponge layer has been introduced to damp the wave reflection at the end of the domain.
- SWASH has been run and the wave height and period have been calculated in the absence of the dike.
- The results have been compared with the measured ones (when possible).
- The hydraulic boundary conditions calculated at the location corresponding to the toe of the dike have been used for the overtopping calculation.

The uncertainties derived from the use of SWASH model to assess the incident wave boundary conditions at the toe of the dike for datasets 00-025 and partly for dataset 13-116 have been assessed. Eleven cases from dataset 00-142 have been used for this purpose. By using wave boundary conditions from SWASH the predicted mean overtopping discharge was on average 13% lower than predicted using wave height and period measured during the experimental campaign. This is a small difference in terms of overtopping discharge, where usually the scatter in the prediction lead to differences of 2-5 times or greater.

In order to compare with Van Gent (1999) the ratio between the significant wave height in deep water conditions and the water depth at the toe has been calculated and expressed as absolute values since in same case the water depth is negative (emergent toe). This ratio results $1.0 \leq |H_{m0\text{-}DEEP}/h_{toe}| \leq 120.0$ meanwhile in Van Gent was less than or equal to 1.5. The range is therefore totally different from that of Van Gent. The water depth at the toe of the dike is very shallow in most of the cases. In some cases the water level is below the toe level. These differences might cause differences in the wave spectra at the toe of the structure which are mostly characterized by low frequencies (i.e. long periods).

3.2 Datasets from CLASH database

The CLASH database is one of the main results of the CLASH-project (Crest Level Assessment of Coastal Structures by full scale monitoring, neural network prediction and Hazard Analysis on permissible wave overtopping). The database consists of 10,532 measurements, from 163 series of tests conducted in several institutions worldwide (De Rouck et al., 2002; Verhaeghe et al., 2004; van der Meer et al., 2005). A wide range of data is included in the database: small scale tests from 2D and 3D models and field data, simple geometries and very complex structures (e.g. vertical seawalls, sea dikes, rubble mound breakwaters, composite slopes).

The CLASH database has been investigated and data from four different test series have been extracted. The selected data are assumed to be representative of cases with shallow and very shallow foreshores. Table 2 lists the dataset name, the number of data extracted from each dataset and the range of the main hydraulic and geometrical parameters. For dataset DS-226, corresponding to Van Gent's data, a transition slope of 1:10 was constructed before the foreshore to obtain a sufficient depth at the wavemaker location.

The reference of each dataset, when available in CLASH, has been also reported. Data of zero overtopping rates were not used in the present analysis. Only dike slopes between 1:8 and 1:2 have been considered (no vertical walls, no very steep slopes). The tests are characterized by smooth and impermeable slopes (γ_f=1), perpendicular wave attack (β=0°, γβ=1), simple geometries without any berm or storm walls, a crest width equal to zero. As a consequence, a Complexity Factor CF=1 (very simple sections) is assigned to the selected cases, as defined in CLASH. A further criterion has been used to select the data to be employed. As discussed in section 2.1, a foreshore is identified as (very) shallow if $s_{m-1,0} \leq 0.01$. Therefore only cases with wave steepness smaller than 0.01 have been considered. Based on the aforementioned criterion, 75 data have been selected in total and extracted from CLASH. Regarding dataset DS-221, only 1 case was found to satisfy all conditions.

Table 2 - Characteristics of datasets extracted from CLASH and corresponding to shallow water conditions

Dataset id.	Number of data	cotθ	foreshore length [m]	cotα	H_{m0} [m]	$T_{m-1,0}$ [s]	Rc [m]	htoe [m]	h_{toe}/H_{m0}	Reference
DS-226	24	100, 250	~35	2.5, 4	0.045-0.103	2.45-4.58	0.160-0.310	0.094-0.188	1.62-2.09	Van Gent (1999)
DS-042	6	20, 50	4-20	2, 4	0.111-0.153	3.160-3.647	0.100-0.300	0.200-0.400	1.31-3.65	Coates et al. (1997)
DS-221	1	100	35	4	0.105	2.609	0.210	0.18	1.71	Van der Meer and De Waal (1993)
DS-227	44	100	45	3, 4, 6	0.039-0.119	2.41-10.64	0.066-0.366	0.050-0.199	1.28-2.38	Smith (1999)
TOT	75									

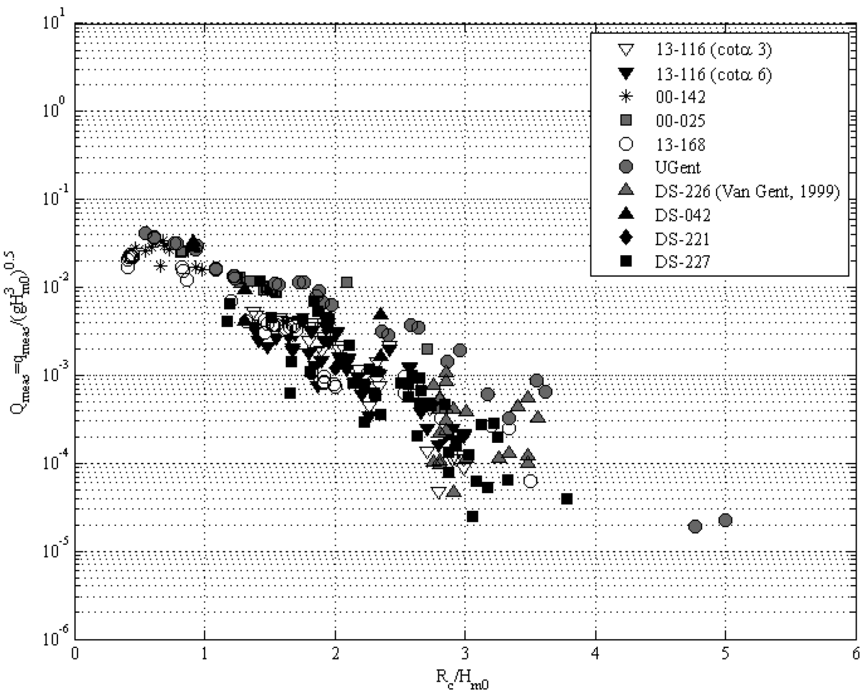
4 Analysis of data sets

This section discusses the dependence of the mean overtopping discharge on the main hydraulic and geometric parameters (section 4.1) where the crest freeboard, wave height and wave period and the water depth at the toe of the dike have been investigated in detail. Sections 4.2 and 4.3 discuss the comparison of existing design formulae to the new datasets for shallow and very shallow foreshores.

4.1 Influence of the main hydraulic and geometrical parameters on wave overtopping

It is well known that the relative freeboard, R_c/H_{m0}, is the most important parameter in wave overtopping: the overtopping rates decrease exponentially if the relative freeboard increases. The variation of the dimensionless measured overtopping discharge, Q_{meas}, with the relative freeboard is also demonstrated for the selected datasets (see Figure 8). The dimensionless overtopping discharge is hereafter calculated as $Q=q/(gH_{m0}^3)^{1/2}$. The data in Figure 8 are plotted using a logarithmic scale on the y-axis: in this way it is clear that the logarithmic of the dimensionless discharge decreases linearly with the increase of the relative freeboard. This means that a certain exponential relationship exists, also for shallow foreshore conditions, between the Q_{meas} and R_c/H_{m0}. Hence any possible selected model to express the relationship between these two quantities must have an exponential form. However, a scatter still exists, due to the influence of other physical or geometrical quantities that has to be therefore determined.

Figure 8 - Variation of the dimensionless overtopping discharge versus with the relative freeboard.

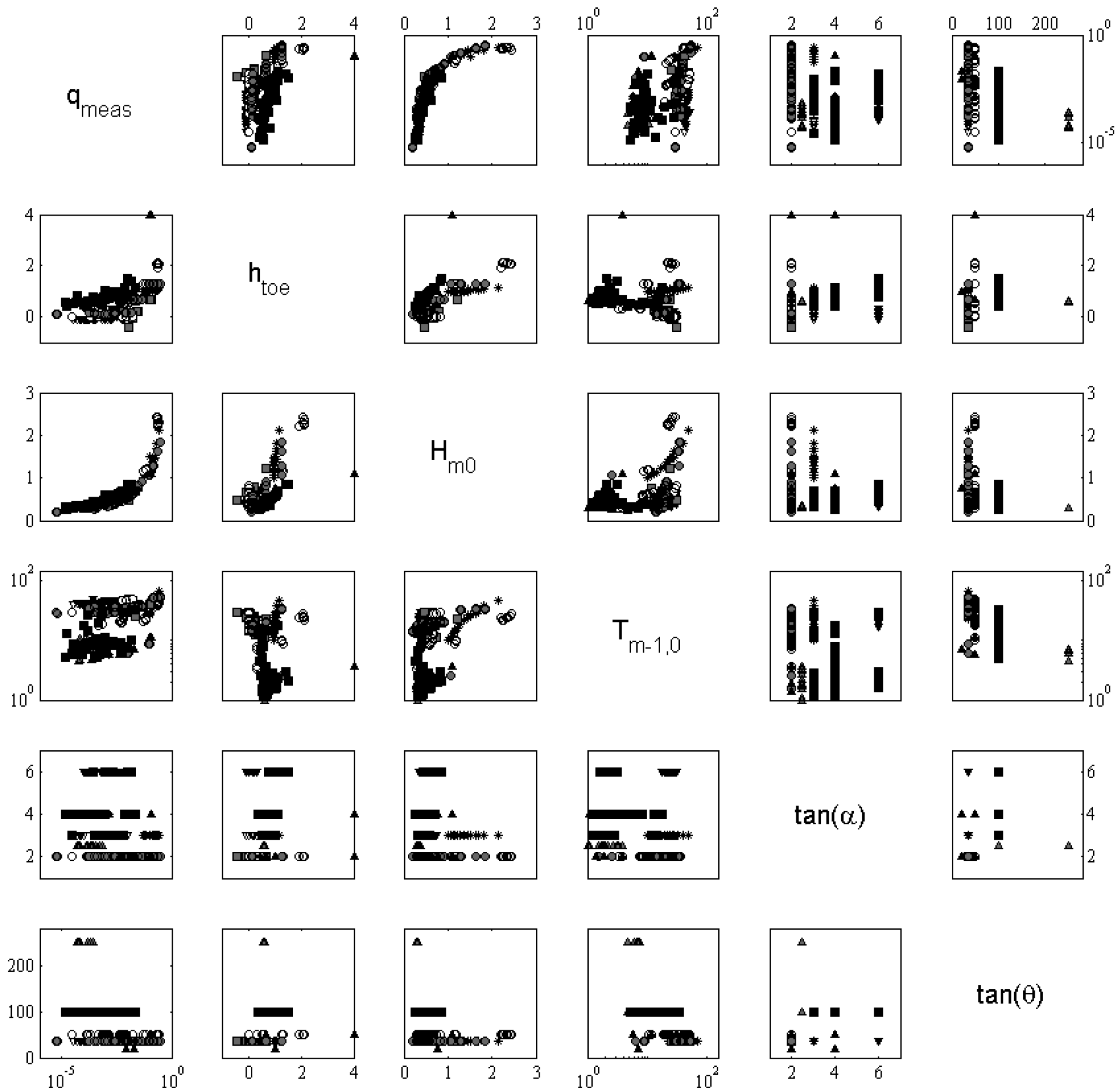


See Tables 1 and 2 for further details on the datasets.

A practical way to determine approximately any possible correlation between variables is to use a scatter-plot matrix. A scatter plot (Chambers et al., 1983) is an instrument widely used among statisticians to reveal relationships between two variables. A scatter plot matrix is an ordered collection of bivariate graphs: given a set of n variables, there are n-choose-2 pairs of variables, and thus the same numbers of scatter plots. In the present study, in order to compare different datasets, each dataset has been scaled to R_c=1m using Froude's similarity law. The selected data are: the measured overtopping discharge, q_{meas}, the incident wave height and period, the water depth at the toe of the dike, the dike slope and the foreshore slope. No correlation is expected between the slope and foreshore slope, however their relationship with the other aforementioned parameter must be verified. The resulting scatter-plot matrix is shown in Figure 9. The overtopping discharge and the incident wave height are highly correlated: the overtopping in fact increases exponentially when the wave height increases. A non-linear correlation is noticeable between the wave height and water depth. A smaller water depth means shallower water conditions at the toe. Under these conditions the wave breaking on the foreshore is severe and, as consequence, the wave height at the toe of the structure is a small fraction of the deep water wave height (smaller for shallower water conditions). A correlation also exists between wave overtopping and water depth; this seems logical because for very shallow water the wave breaking is heavier resulting in smaller wave height and then smaller overtopping rates; for deeper waters, the wave height might be bigger (less breaking) and then the overtopping discharge might results larger. Hence the water depth at the toe has an influence on wave height and wave overtopping that must be considered somehow. At this stage, it was not possible to establish further correlations among variables; however the wave overtopping might be correlated with dimensionless variables defined as a combination of two or more variables. For example, the relationship between wave steepness (defined as combination of wave height and wave period) and wave overtopping discharge is discussed in the following sections.

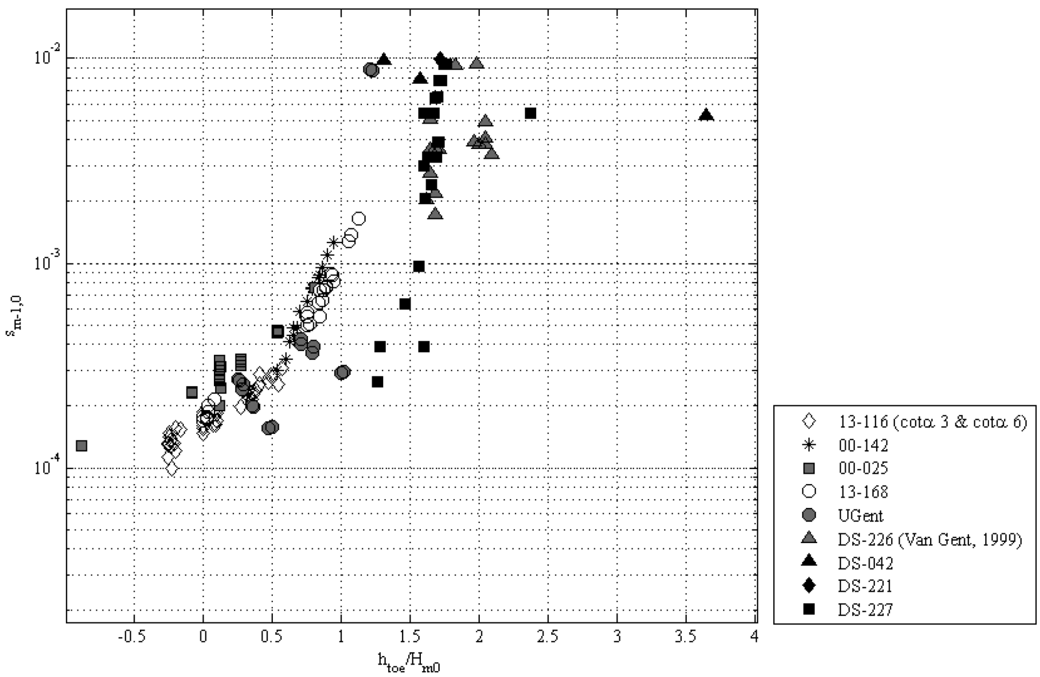
Regarding the wave steepness, it was stated in section 2.1 that the criterion of $s_{m-1,0} \leq 0.01$ has been selected to identify (very) shallow foreshores. Only data with low wave steepness have therefore been analysed. Figure 10 shows the variation of the wave steepness as function of the non-dimensional parameter h_{toe}/H_{m0} for all the cases used in the present analysis. With the exception of a few cases, the plot shows a clear trend that suggests that the water depth at the toe might play an important role in case (very) shallow foreshores. Therefore the influence of water depth at the toe of the sea dike should be considered for wave overtopping assessment. The use of the water depth will be discussed in Chapter 5.

Figure 9 - Scatter-plot matrix of uniformly scaled variables.



See Tables 1 and 2 for further details on the datasets.

Figure 10 - Variation of wave steepness as function of the ratio
between the water depth at the toe and the incident significant wave height.



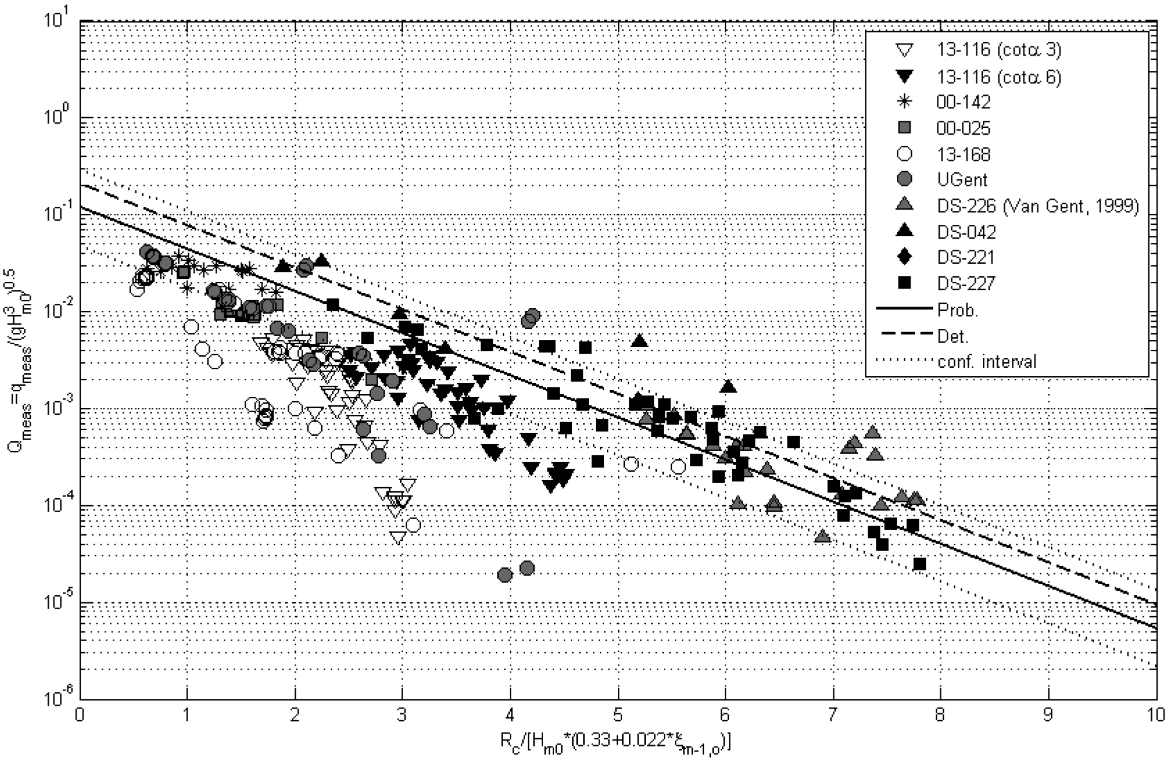
4.2 Comparison to existing approaches

The measured overtopping rates are depicted in Figure 11 versus the dimensionless quantity $R_c/[H_{m0}(0.33+0.022\xi_{m-1,0})]$, and compared with predictions from Eq. 2a (black solid line) and Eq. 2b (dashed line). The dotted lines indicate the 5% under- and upper-exceedance limits. These are calculated by replacing the value of $10^{-0.92}$ in Eq. 2a with $10^{-0.92\pm\sigma}$. Figure 12 shows the comparison of the predicted and measured overtopping discharges. Four lines are added to the graph: a solid line that correspond to a prediction equal to the measurement (using Eq. 2a, probabilistic approach) two dashed lines corresponding to overtopping prediction 10 times and 0.1 times the measured data and a dash-dot line that corresponds to a prediction equal to the measurement in case of use of Eq. 2b (deterministic approach).

The results indicate a tendency of overestimation, especially for low values of $R_c/[H_{m0}(0.33+0.022\xi_{m-1,0})]$. This tendency is examined in the form of the ratio of the dimensionless predicted to the measured overtopping, Q_{pred}/Q_{meas}, (hereinafter defined as the overtopping prediction ratio) versus respectively the wave steepness, $s_{m-1,0}$ and the ratio h_{toe}/H_{m0} as shown in Figure 13 and Figure 14. As soon as the wave steepness is less than $s_{m-1,0} = 0.001$, (that usually corresponds to very long waves with limited wave height), Van Gent formula (see Eq. 2a) overestimates drastically the wave overtopping. For a wave steepness around 0.005, the results are spread around the mean value of the overtopping prediction ratio equal to $Q_{pred}/Q_{meas} = 1$. In general values of the wave steepness lower than 0.001 correspond to situations with very shallow foreshore and severe wave breaking in the examined cases. This suggests that the influence of very shallow foreshores on wave overtopping is not well accounted for in the Van Gent formula due to its application range.

The use of Van Gent's formula clearly overestimates the results for values of h_{toe}/H_{m0} lower than 1-1.5 (Figure 14). It is remarkable that the value of 1.5 for the ratio h_{toe}/H_{m0} corresponds to the breaking limit chosen in EurOtop (2007) to assess the effects of composite slopes on mean overtopping discharge. It is as if the waves that were about to overtop "feel" the bottom if this is located closer than 1.5 times the wave height from the still water level. What if this location corresponds to a point on the foreshore slope? There is no clear recommendation in EurOtop (2007) and no answer in Van Gent (1999), since all his data are characterized by higher values than 1.5. However Figure 14 demonstrates that this ratio might be significant for a proper assessment of mean wave overtopping of sea dikes with very shallow foreshores.

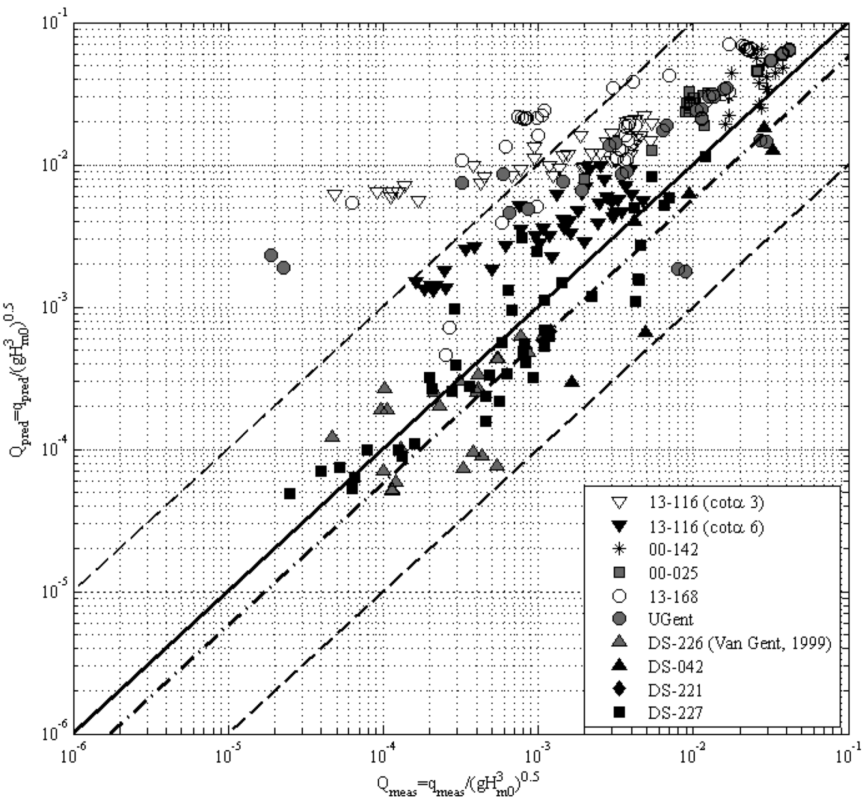
Figure 11 - Wave overtopping data and prediction using Eq.2a and Eq.2b as in Van Gent (1999) and EurOtop (2007) with 5% under and upper exceedance limits.



If the shallow foreshore has a certain influence on the wave overtopping, it is questionable if the dike slope plays still a role in that. This raises the following questions: does the dike slope still play a role in case of very shallow foreshore? Or, is it the only foreshore to affect the wave overtopping? To give an answer to these questions, the results of dataset 13-116 have been selected and analysed separately from the rest of the cases. During this test campaign, two dikes with different slopes (respectively *cotα=3* and *cotα=6*) have been installed in different partitions of the flume oat FHR. This allowed testing the two different dike slopes simultaneously. This means that for each test, the same wave train has been generated by the wavemaker, the same water depth and same freeboard have been used, and same incident wave conditions were obtained at the toe (see section 3.1). In this way, the only difference for each executed test consisted of the dike slope. Hence, if a difference is noticed in wave overtopping, it must depend on the dike slope itself. Figure 15 plots the difference due to the dike slope expressed as ratio of the measured overtopping discharges. This ratio is depicted in function of the dimensionless freeboard R_c/H_{m0} and the parameter $h_{toe}/L_{m-1,0}$. On one hand, the differences between the overtopping rates of the two slopes are negligible for dimensionless freeboards between 2.2 and 2.6. On the other hand, the behaviour of the two dikes seems to be different for positive and negative values of $h_{toe}/L_{m-1,0}$. Negative values of $h_{toe}/L_{m-1,0}$ correspond to negative values of the toe depth (emergent toe). In this case the overtopping on the gentler slope is less than on the steeper slope, as expected. For values of $h_{toe}/L_{m-1,0}$ larger than 0.3-0.5, the ratio tends to an asymptotic value around 1.5. This means that for relative larger toe depths, the toe depth does not longer influence the wave overtopping discharge and only the dike slope might affect the results.

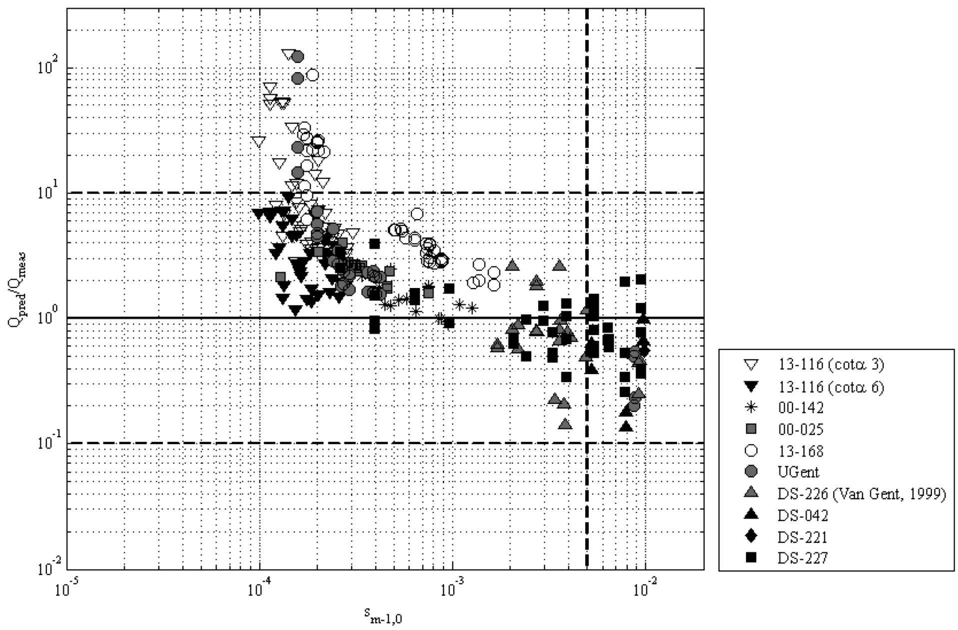
Since it has been demonstrated that Van Gent (1999) tends to overestimate the mean overtopping discharge especially for very shallow foreshores, the more general formula from Goda (2009) has been used and compared with the measured overtopping rates. Figure 15 shows the comparison of the predicted and measured overtopping discharges using Goda's formula, expressed dimensionless as $Q=q/(gH_{m0}^3)^{1/2}$. Three lines are added to the graph: a solid line that correspond to a prediction equal to the measurement (Eq. 4) and two dashed lines that correspond to overtopping prediction 10 times and 0.1 times the measured data. See Table 1 Table 2 for further details on the datasets. The overtopping ratio (Q_{pred}/Q_{meas}) is depicted in Figure 17 versus the ratio h_{toe}/H_{m0}. The performance of using the general approach proposed by Goda (2009) is very poor especially for those datasets characterized by very shallow foreshore conditions (00-025, 13-116, 13-168, UGent) and in general Goda's formula leads to an overestimation of the mean overtopping discharge.

Figure 12 - Comparison of Van Gent and measured overtopping discharge



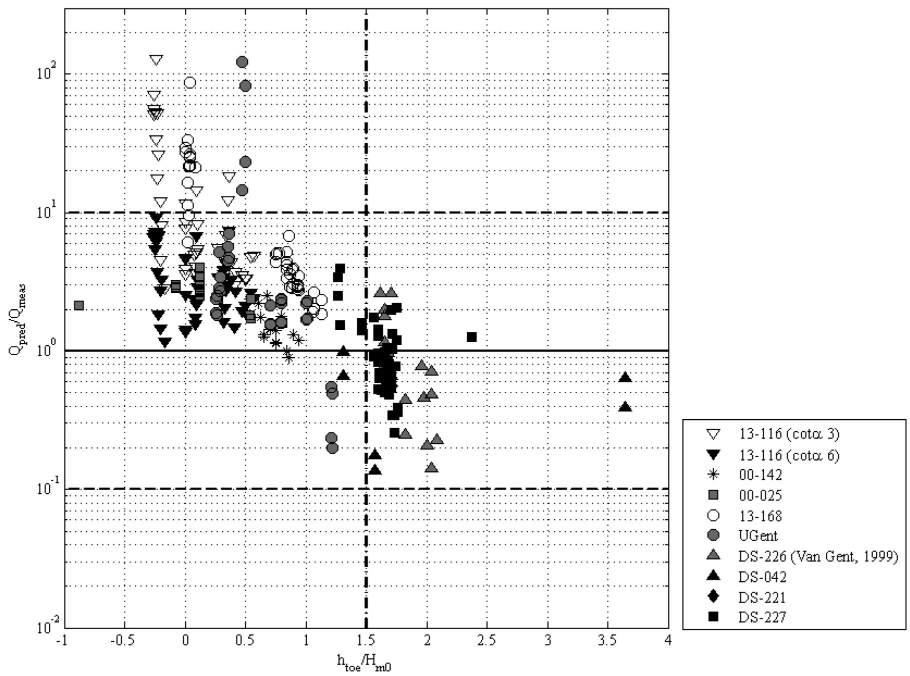
dash-dot line: prediction using Eq. (2b) equal to measured rate; solid line: prediction using Eq. (2a) equal to measured rate; dashed lines: prediction using Eq. (2a) equal to 10 times and 0.1 times the measured rate.

Figure 13 - Ratio of the dimensionless predicted to the measured overtopping versus the wave steepness.



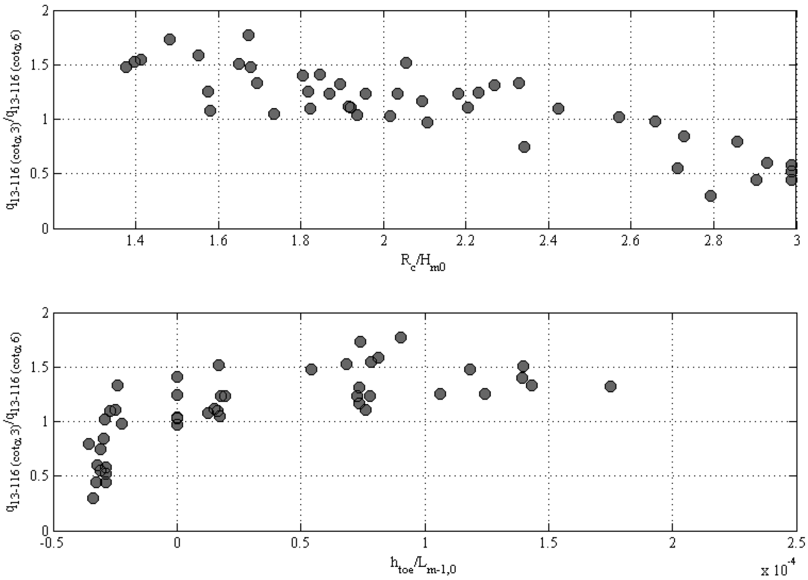
The vertical dashed line indicate a wave steepness of 0.005; the two horizontal dashed lines correspond to predictions 10 times or 0.1 times the measured values; the horizontal solid line corresponds to $Q_{pred}/Q_{meas}= 1$.

Figure 14 - Ratio of the dimensionless predicted to the measured overtopping versus the ratio between the toe depth and the wave height.



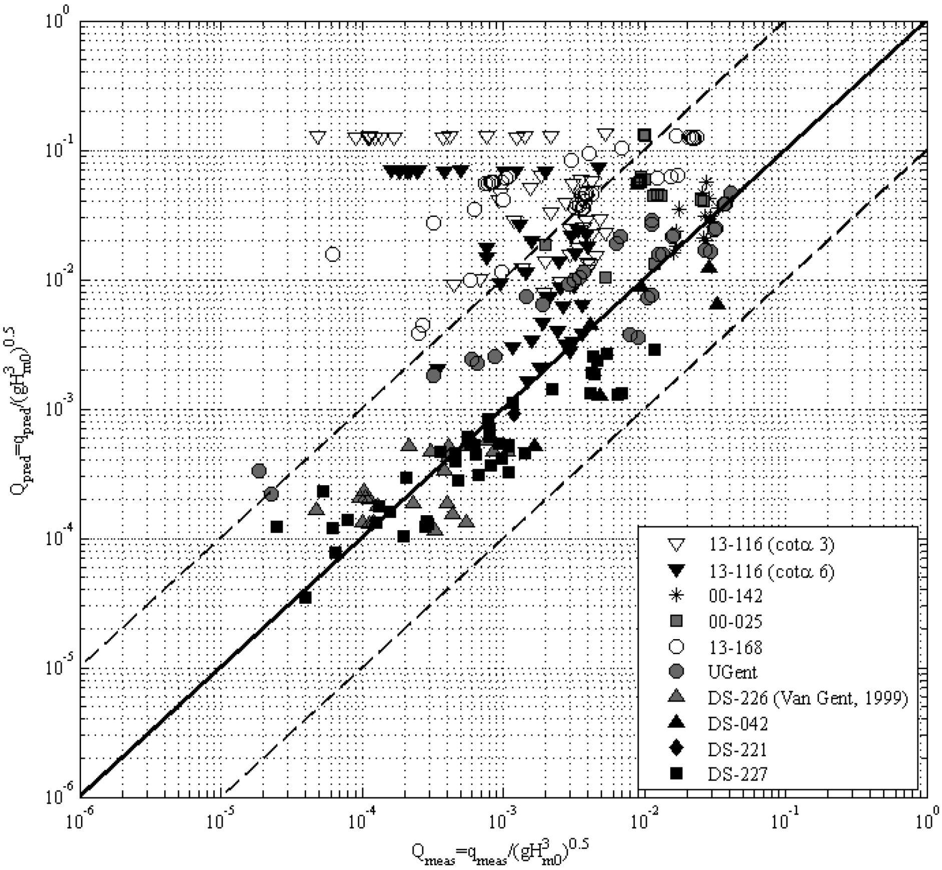
The vertical dashed line indicates a value of h_{toe}/H_{m0} of 1.5; the two horizontal dashed lines correspond to predictions 10 times or 0.1 times the measured values; the horizontal solid line corresponds *to* $Q_{pred}/Q_{meas}= 1$.

Figure 15 - Effect of dike slope on the measured wave overtopping in function of the dimensionless freeboard R_c/H_{m0} and $h_{toe}/L_{m-1,0}$.



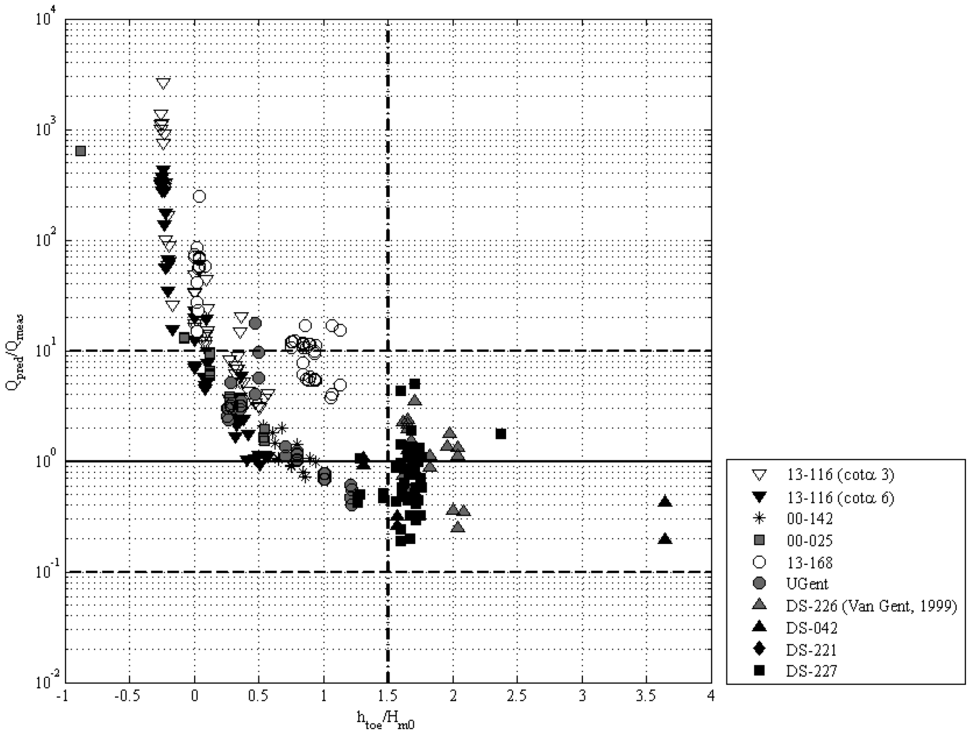
For cases of 13-116 dataset.

Figure 16 - Comparison of Goda formula and measured overtopping discharges.



The three lines indicate the conditions that the prediction is 10 times, equal to, and 0.1 times the measured rate.

Figure 17 - ratio of the dimensionless predicted to the measured overtopping versus the ration between the toe depth and the wave height.



Source: Goda (2009).

The vertical dashed line indicates a value of h_{toe}/H_{m0} of 1.5; the two horizontal dashed lines correspond to predictions 10 times or 0.1 times the measured values; the horizontal solid line corresponds to $Q_{pred}/Q_{meas}=1$.

4.3 Error analysis

A qualitative comparison between predictions of formulae and measured overtopping discharges has been shown in the previous section. However the accuracy of each formula has not been quantified. For cases when the scatter of the data is quite large, it is necessary to quantify the error of the prediction to evaluate the performance of each implemented formula.

Here the same method proposed in Goda (2009) is used. The geometric mean (Geo) is introduced and defined as:

$$\bar{x}_G = exp\left[\frac{1}{N}\sum_{i=1}^{N} lnx_i\right] \text{ with } \quad x_i = \frac{q_{est,i}}{q_{meas,i}} \tag{9}$$

where N is the number of data points, $q_{est,i}$ and $q_{meas,i}$ are the *i-th* predicted and measured mean overtopping discharge, respectively. The scatter of the data is assessed through the geometric standard deviation (GSD) that is calculated as the exponential value of the standard deviation of the logarithm:

$$\sigma(x_G) = exp\left\{\left[\frac{1}{N}\sum_{i=1}^{N}\left((lnx_i) - (ln\bar{x}_G)\right)^2\right]^{0.5}\right\} \tag{10}$$

Considering a quantity normally distributed, 90% of the data will be contained in the range between the mean divided by 1.64 times GSD and the mean multiplied by 1.64 times GSD. The geometric mean and geometric standard deviation of each dataset when Van Gent’s formula is used are listed in the third and fourth columns of Table 3, respectively.

The geometric mean and the geometric standard deviation calculated on the entire dataset of 279 tests are equal to 2.612 and 3.439, respectively. This means that, if the overtopping rate is assumed to be normally distributed, 90% of the predicted overtopping rate is to be located in the range between 0.463 and 14.732 times the measured overtopping discharge. In general the tendency is to overestimate the overtopping for cases characterized by very shallow foreshores, while the overtopping is slightly underestimate for datasets DS-042 and DS-221. The GSD for DS-221 is equal to 1 since only 1 test case has been used. The application of Van Gent's formula to data only from Van Gent (1999) with $s_{m-1,0}$≤0.01, underestimates the wave overtopping of about 32% in average.

The geometric mean and geometric standard deviation have been also calculated in case of use of Goda's approach. The results are reported in the second to last and last columns respectively of Table 3. The geometric mean and the geometric standard deviation calculated on the entire dataset of 279 tests are equal to 4.659 and 7.335, respectively. This means that, if the overtopping rate is assumed to be normally distributed, 90% of the predicted overtopping rate is to be located in the range between 0.387 and 56.045 times the measured overtopping discharge.

Table 3 - Results of prediction performance using Van Gent (1999) and Goda (2009) formulae

Dataset id.	N	Van Gent (1999) q_{est}/q_{pred}		Goda (2009) q_{est}/q_{pred}	
		Geo	GSD	Geo	GSD
13-116 (cotα=3)	45	8.919	2.764	33.374	7.533
13-116 (cotα=6)	45	3.184	1.744	12.760	7.255
00-142	17	1.429	1.350	1.154	1.364
00-025	21	2.578	1.235	6.183	3.455
13-168	42	6.539	2.675	16.734	2.799
UGent	34	2.749	3.645	1.765	2.403
DS-226 (Van Gent)	24	0.684	2.081	0.997	1.952
DS-042	6	0.395	2.047	0.433	1.887
DS-221	1	0.559	1.000	0.770	1.000
DS-227	44	0.860	1.854	0.694	1.983
TOT	279	2.612	3.439	4.659	7.335

4.4 Discussion

In the previous section, the results of the application of Van Gent (1999) and Goda (2009) were shown. Van Gent's equation was expected to be used for the estimation of overtopping discharge on the dikes with shallow and very shallow foreshores. However, the Van Gent formula overestimates the physical model results here presented. The main reason lies in the conditions at the toe of the structure. If the water depth at the toe of structure is much shallower than that one used in Van Gent (1999) and the wave steepness reduces to values lower than or equal to 0.001-0.005 then the effect of the foreshore on the wave overtopping is not properly taken into account. The spectral wave period in these conditions is much longer than the ones in the calculation of Van Gent's equation. This led to an expected overestimation, up to two orders of magnitude, of the mean wave overtopping discharge. It is also demonstrated that this overestimation occurs particularly for values of the water depth at the toe smaller than 1.5 times the incident wave height.

Goda (2009) aimed to define a set of unified formulae able to provide a preliminary, but accurate, estimation of wave overtopping rates, however the application of his formula to cases with (very) shallow foreshores is far from accurate. In particular, the accuracy of Goda (2009) prediction is very low for those cases characterized by very low water depth at the toe and very long period. The reason of the inaccuracy lies in the expressions of coefficients A and B in Eqs. 5-8. There was a significant lack of data for very low water depths in the datasets analysed by Goda which makes the data fitting to calibrate Eqs. 5-8 less reliable for such conditions. On the other hand, the performance of Goda formula for data with higher water depth and bigger wave steepness results is equal to or better than the Van Gent formula. Hence, the Goda formula is not recommended to assess the wave overtopping discharge on sloping dikes with very shallow foreshores.

The results underline that there is still a lack of knowledge in this field and it is necessary to investigate in more detail the wave overtopping on sea dikes with very shallow foreshores. It is evident that the foreshore slope has an influence on the wave transformations. The wave spectra at the toe of the structure are characterized by low frequencies (i.e. long periods), the wave height has usually undergone a significant reduction from deep water conditions (less than 50%). In many cases the waves are already broken when they reach the dike making it difficult to identify an oscillatory pattern in the water surface elevation. The Van Gent (1999) formula has been widely used for prediction of wave overtopping in shallow foreshores and the formula is included in EurOtop (2007) as well. However, if the water depth at the toe is shallower than that used in Van Gent (1999), and the wave steepness is lower than in datasets used by Van Gent (1999), then it is expected that the Van Gent formula would overestimate the mean overtopping rates.

An important question concerns the definition of the slope in the Van Gent equation. In fact, the equation is a function of the breaker parameter, which contains the tangent of the slope angle. However, in cases where the water is very shallow at the toe of the dike, wave breaking does not occur on the dike itself but on the foreshore. It might be more correct to use the foreshore slope instead of the dike slope in the Van Gent (1999) equation or to calculate an average slope starting from the foreshore and dike slope. This modification in the input for Van Gent (1999) is discussed in the next sections.

It should be noted that the wave boundary conditions (significant wave height and spectral wave period) are still assumed at the toe of the dike when modifications to Van Gent formula are introduced (as it will be described afterwards). This can appear in contrast to the choice to use the foreshore slope or an equivalent slope (which can include the foreshore effects) to define the breaker parameter. In fact, if the foreshore is assumed in Van Gent equations, the wave boundary conditions should be referred to the toe of the foreshore itself and not to the toe of the dike, since the foreshore is assumed together with the coastal structure as a single entity. However it is often difficult to define precisely where the toe of the foreshore is located. Therefore, in the following approach, the incident wave boundary conditions are considered at the toe of the dike but the breaker parameter is calculated considering both contributions of foreshore slope and dike slope. The accuracy of the results that will be shown in the next sections (especially when the new concept of “equivalent slope in shallow foreshore” is introduced) supporting the aforementioned assumption.

5 The equivalent slope concept

The analysis in the previous section has demonstrated that well-known existing formulae like Van Gent (1999) and Goda (2009) are not accurate to predict the mean wave overtopping discharge of sea dikes with (very) shallow foreshores. The main reason is the lack of data with very shallow foreshores in both studies. In fact, despite the fact that those formulae cover cases similar to the ones that are object of study, both Van Gent (1999) and Goda (2009) are applied mostly outside their range. In particular it has been demonstrated that the quality of the prediction is extremely poor for values of the wave steepness lower than 0.001 and for water depth at the toe lower than 1.5 times the incident wave height. Both results show the important role that the foreshore plays, triggering severe wave breaking that modifies the wave spectral shape from offshore to nearshore. The wave height is therefore drastically reduced and a large amount of the wave energy shifts to very low frequencies (i.e. long periods). Thus it is mandatory to use a model that is able to take into account the influence of the foreshore and the toe depth on the mean overtopping discharge.

The authors' intention here is not to propose a totally new formula, since the existing exponential equations have been largely proven in the past to be accurate models of the mean overtopping discharge within their range of applicability. The new conceptual model presented in the next section, aims to correct and extend the application of the Van Gent (1999) to cover a wide range of cases with shallow and very shallow foreshores, also in case of an emergent toe (equivalent to negative water depths at the toe of the dike).

5.1 Conceptual model

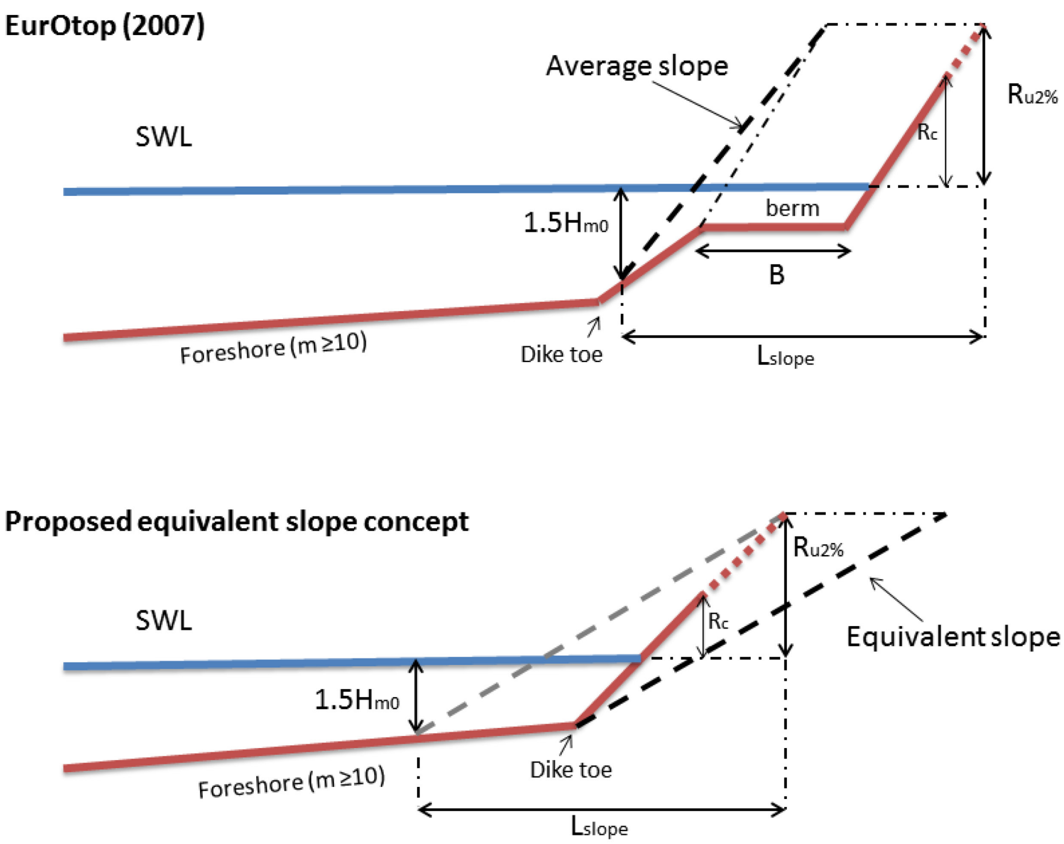
A new concept of equivalent slope is introduced here. Saville (1958) first proposed a concept of an imaginary slope, corresponding to a line connecting two points: the first one on the seabed at the wave breaking point and the second one at the limit of the wave uprush. Later, de Waal and van der Meer (1992) proposed an average slope considering two different points: the first on the seabed where the water depth is equal to one time the significant wave height and the second one on the dike at a vertical distance above the still water level equal to one time the significant wave height. Wang and Grüne (1996) derived an approach which takes the wave steepness into account. A totally different approach for the calculation of an imaginary slope is presented by Mase et al. (2013), where the imaginary slope takes into consideration the configuration of the cross section of the foreshore and sea dike, following the approach by Nakamura et al. (1972).

Herein, the "equivalent slope in shallow foreshore" concept is introduced (hereafter defined only as equivalent slope). The equivalent slope is defined as the average slope, *tan δ*, in the zone between $SWL\text{-}1.5H_{m0}$ and $SWL+R_{u2\%}$, where *SWL* is the still water level, H_{m0} is the significant wave height at the toe of the structure and $R_{u2\%}$ is the wave run-up exceeded by 2% of the incident waves. The average slope is calculated as follows:

$$tan\delta = \frac{(1.5H_{m0}+Ru_{2\%})}{L_{slope}} \tag{11}$$

where the quantity L_{slope} is the horizontal length of the zone between $SWL\text{-}1.5H_{m0}$ and $SWL+R_{u2\%}$ (see Figure 18). The equivalent slope defined here is similar to the average slope as reported in TAW (2002) and EurOtop (2007) to evaluate the effect of composite slopes and berms on wave overtopping. The differences between the present approach and the one in EurOtop (2007) are depicted in Figure 18. As in the aforementioned works, the influence is recognized that a change of the slope from the breaking point to the maximum wave run-up height has on the run-up process and overtopping phenomenon. The main difference with TAW (2002) and EurOtop (2007) is that we extend the average slope concept to very gentle, long and shallow foreshores but still the wave heights and wave periods are calculated at the toe of the dike and not at the toe of the foreshore or at the breaking point. The assumption has already been discussed above.

Figure 18 - Parameter definition to calculate the equivalent slope
and comparison with scheme from EurOtop (2007) for composite slopes and berms.



Wave height is determined at the toe of the dike.

To calculate the wave run-up the following formula is used (eq. 5.4, EurOtop, 2007):

$$Ru_{2\%} = 1.65\gamma_b \mathrm{H_{m0}}\xi_{m-1,0} \quad \text{with maximum of}$$

$$Ru_{2\%} = \left(4.0 - \frac{1.5}{\sqrt{\xi_{m-1,0}}}\right) H_{m0} \tag{12}$$

Eq. (12) corresponds to the calculation of the wave run-up height following a probabilistic (i.e. using an average) approach as defined in EurOtop (2007). Even though the reduction coefficient γ_b is present in Eq. 12, there is no evidence of the influence of an intermediate berm on the wave overtopping in (very) shallow foreshores. Therefore γ_b must be assumed always equal to 1 for the calculation of the equivalent slope. If a berm is really present, it is recommended to investigate further its effects on the wave overtopping in shallow foreshore conditions.

The wave run-up is not known a priori because its calculation depends on the slope used to calculate the surf similarity parameter. Hence, the application of equation (12) requires an iterative procedure that is stopped when the result for wave run-up is starting to converge. The iterative procedure can be summarized as follows:

1. The first run-up value can be estimated as $1.5 \cdot H_{m0}$.
2. With this first run-up value, the new slope is calculated using Eq.11.
3. This slope is reintroduced in Eq.1 to calculate the new value of the breaker parameter.
4. Then equation (12) is used for the 2nd run-up calculation.
5. These steps have to be iterated until the wave run-up value starts to converge: the difference between 2 consecutive values is smaller than 1%.
6. The final calculated slope will be used as the equivalent slope *tan δ*.

This iterative procedure has been applied to the tests from all the datasets. It is clear that the foreshore is not taking part in the calculation when the water depth at the toe of the dike is larger than $1.5*H_{m0}$. This is, for example, the case for some tests from Van Gent (1999). Hence, for $h_{toe}>1.5*H_{m0}$, the Van Gent formula (Eq.2a or Eq.2b) is used with the calculated breaker parameter using only the dike slope. However, as the water depth becomes progressively smaller, the proportion of the foreshore in L_{slope} is increasingly large. In fact, L_{slope} can be calculated as:

$$L_{slope} = \frac{(Ru_{2\%}+h_{toe})}{tan\alpha} + \frac{(1.5H_{m0}-h_{toe})}{tan\theta} \tag{13}$$

In this way, the average slope can be calculated also in cases with a water level below the toe level, (i.e. for an emergent toe where $h_{toe}<0$). The influence of the foreshore will be consequently more important than in cases with a wet toe.

5.2 Application of the model

The present section describes the use of the equivalent slope to calculate the breaker parameter that will be used in Eq. 2a or Eq. 2b in order to improve the wave overtopping prediction. At the present stage, this represents the only modification in the use of the formulae proposed by Van Gent (1999). Later, in section 5.3, the final wave overtopping formula will be presented, being slightly different from Van Gent's one.

The results of using the equivalent slope in Eq.2a and Eq.2b to calculate the mean overtopping discharge are shown in Figure 19-Figure 22. The accuracy of the prediction is immediately noticeable. All the cases are now between 0.1 and 10 times the average predicted value (Figure 10) and the prediction improves significantly with respect to Figure 12. The overtopping prediction ratio is no longer dependent on the wave steepness: the results are scattered uniformly around the value $Q_{pred}/Q_{meas}=1$ (see Figure 21 in comparison with Figure 13). The same can be observed in Figure 22 where the results are plotted versus h_{toe}/H_{m0} (see Figure 14 for comparison). Thus the model (i.e. Eq.2a that implements the equivalent slope concept) can be considered valid.

Figure 19 - Wave overtopping data and prediction
using the equivalent slope into the equations 2a and 2b with 5% under and upper exceedance limits.

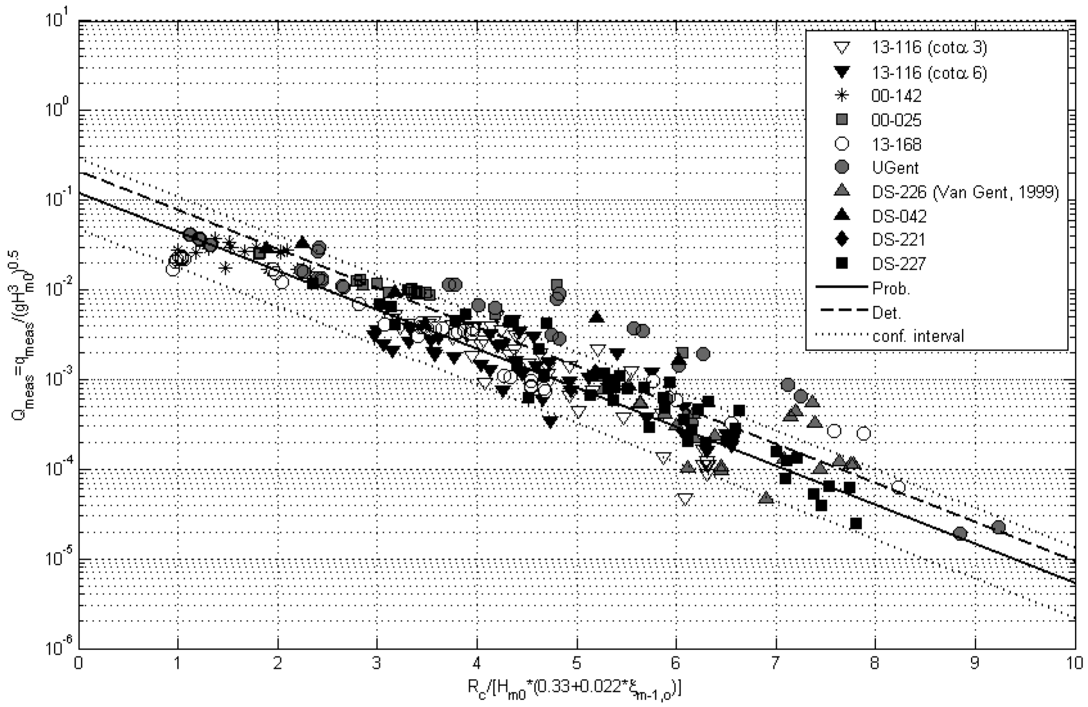
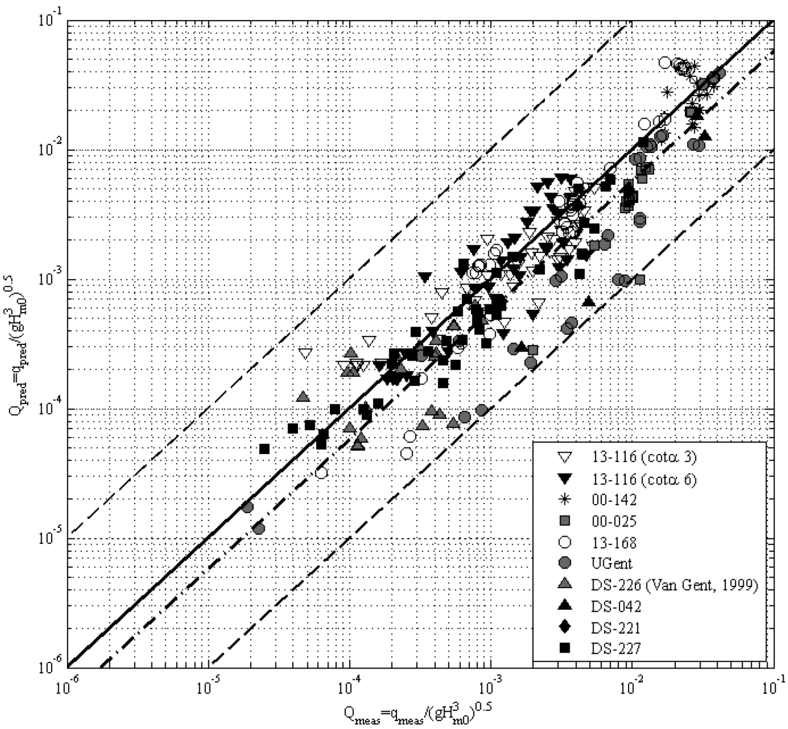
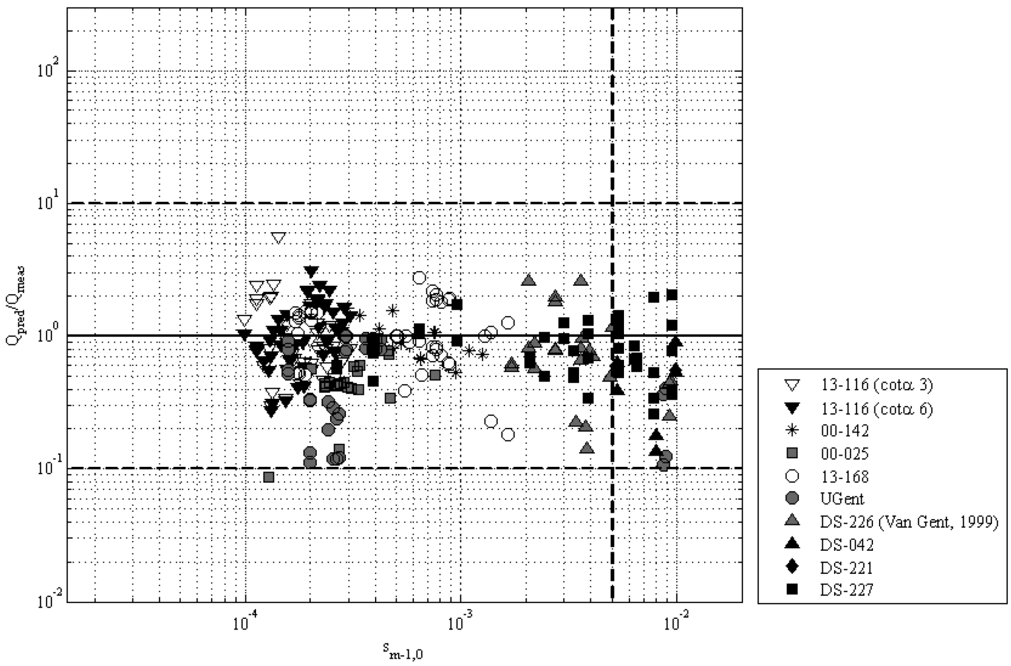


Figure 20 - Measured versus predicted non-dimensional overtopping discharge
using the equivalent slope into the equations 2a and 2b



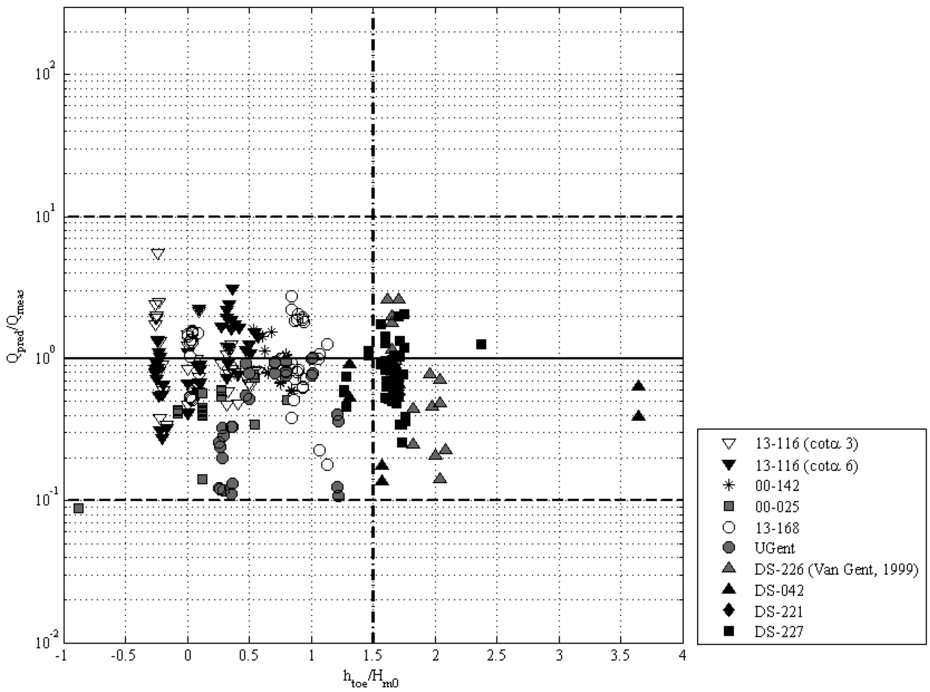
Dash-dot line: prediction using Eq. 2b equal to measured rate; solid line: prediction using Eq. 2a equal to measured rate; dashed lines: prediction is 10 times and 0.1 times the measured rate.

Figure 21 - Equivalent slope: ratio of the dimensionless predicted to the measured overtopping versus the wave steepness.



The vertical dashed lines indicate a wave steepness of 0.005; the two horizontal dashed lines correspond to predictions 10 times or 0.1 times the measured values; the horizontal solid line corresponds to $Q_{pred}/Q_{meas}=1$.

Figure 22 - Equivalent slope: ratio of the dimensionless predicted to the measured overtopping versus the ration between the toe depth and the wave height.



The vertical dashed line indicates a value of h_{toe}/H_{m0} of 1.5; the two horizontal dashed lines correspond to predictions 10 times or 0.1 times the measured values; the horizontal solid line corresponds to $Q_{pred}/Q_{meas}=1$.

The geometric mean (Geo) and geometric standard deviation (GSD) of each dataset have been calculated and are listed in Table 4. The geometric mean and the geometric standard deviation calculated on the entire dataset of 279 tests are equal to 0.743 and 1.968, respectively. This means that if the overtopping rate is assumed to be normally distributed, 90% of the predicted overtopping rate is to be located in the range between 0.230 and 2.398 times the measured overtopping discharge, with a significant improvement of the mean overtopping discharge prediction.

Table 4 - Results of prediction performance using “equivalent slope”
to calculate the breaker parameter to be introduced in Van Gent (1999) formula

Dataset id.	**N**	**“Equivalent slope” in Van Gent (1999) q_{est}/q_{pred}**	
		Geo	**GSD**
13-116 (cotα=3)	45	0.945	1.780
13-116 (cotα=6)	45	0.964	1.745
00-142	17	0.877	1.386
00-025	21	0.418	1.614
13-168	42	0.998	1.802
UGent	34	0.423	2.208
DS-226 (Van Gent)	24	0.684	2.081
DS-042	6	0.377	1.972
DS-221	1	0.559	1.000
DS-227	44	0.738	1.620
TOT	279	0.743	1.968

For the sake of completeness, the foreshore slope, $cot\ \vartheta$, was used also to calculate the breaker parameter $\xi_{m-1,0}$ to be introduced in Eq. 2a and Eq. 2b. The results are not here reported but they show a clear underestimation of the mean overtopping discharge with respect to the measured values.

So far, the equivalent slope concept has been demonstrated to give more accurate overtopping prediction than Van Gent (1999) and Goda (2009). The limitation of Van Gent (1999) and Goda (2009) lies in the ranges of wave parameters that characterize the databases on which the authors have based their analysis. In fact sea dikes with very shallow foreshore represent a case that is not properly cover by the both aforementioned authors.

Despite the limitations to use Van Gent’s formula, it still represents the most widely-used and well-known formula for wave overtopping in shallow water conditions. However, the results here show that Eq.2a together with the equivalent slope still underestimate slightly the overtopping. It appears that the results lie between the prediction of Eq.2a and Eq.3b (dash-dot line in Figure 20 corresponding to the deterministic approach defined in the EurOtop manual). Thus, an improved fitting of the overtopping data is proposed in the next section, keeping the formula structure as in Van Gent (1999).

The results demonstrate the performance of using the equivalent slope instead of the dike slope to define the breaker parameter that will be used for overtopping calculations. The accuracy is improved and the error no longer depends on the wave steepness and presents a random distribution around the value $Q_{pred}/Q_{meas}=1$. This condition is defined as homoscedasticity and it is violated in the case of the simple use of Eq. 2a or Eq.2b. However, it is no longer violated when the equivalent slope is implemented. Nevertheless, as clear from Figure 19 and Table 4, the application of the equivalent slope to Van Gent (1999) leads to a slight underestimation of the measured overtopping discharge for most of the datasets. This aspect requires a further adjustment of the overtopping prediction formula for a sea dike with (very) shallow foreshores. The new formula is shown in the next section.

5.3 Empirical formula based on the equivalent slope concept

Many different functional forms of overtopping formulae can be found in literature. The present study employs an exponential expression for wave overtopping of smooth and impermeable sea dikes with very shallow foreshore, that has the same form of Eq. 2a:

$$\frac{q}{\sqrt{gH_{m0}^3}} = 10^{c\_new} exp\left(-\frac{R_c}{H_{m0}(0.33+0.022\xi_{m-1,0})}\right) \tag{14}$$

where the new *c_new* parameter is calculated based on the analysis of all the datasets both from FHR, UGent? and CLASH already described in the present paper. The new fitting is based on the assumption to use the "equivalent slope in shallow foreshore" concept and uses the incident significant wave height and spectral period defined at the toe of the sea dike. With respect to Eq. 2a and Eq. 2b, the influence factors γ_β and γ_f have been omitted since the effects of wave obliqueness and surface roughness have not been analysed in the present study.

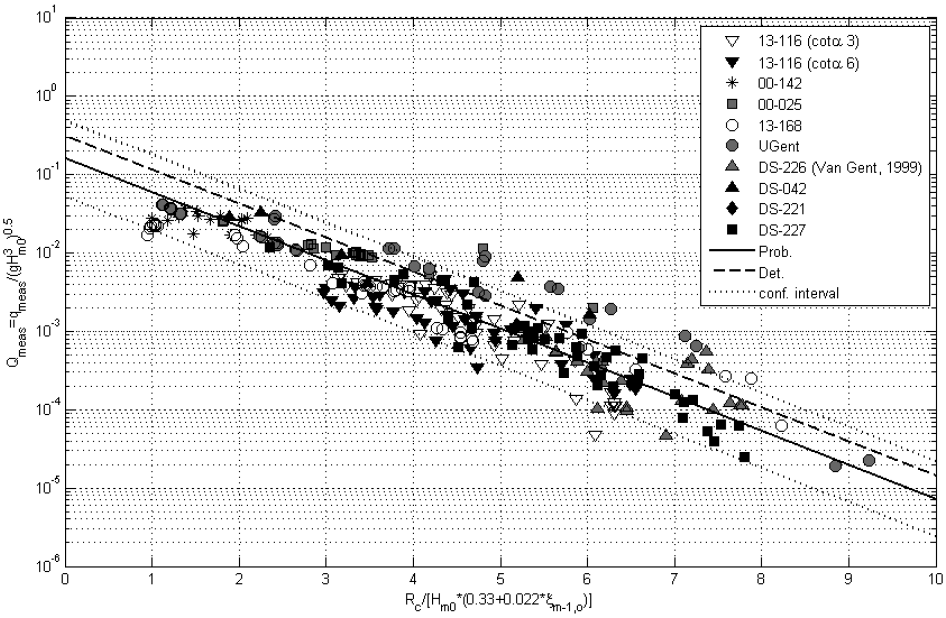
The choice to maintain the same structure as Eq. 2a with the use of the equivalent slope lies in the results shown in the previous sections, where this combination has been demonstrated to reduce the inaccuracies in overtopping prediction over the entire dataset (i.e. homoscedasticity). However, still there was a slight underestimation of the overtopping results (see Table 4). This underestimation might depend on the value assumed by the *c* parameter in Eq. 2a, that defines y-intercept of the black solid line in Figure 19. Therefore a new fit has been conducted to determine the new mean value of such parameter and its standard deviation.

The *c_new* parameter in Eq.14 is assumed to be normally distributed: following this hypothesis the mean value of *c_new* results equal to -0.791 and the standard deviation σ is 0.294. The new coefficient of Eq.14 is then $10^{-0.791}$≈0.16 that is between Eq. 2a and Eq. 2b. The 5% upper- and under-exceedance limits can be calculated as (-0.791)±1.64·0.294.

Assuming a more conservative approach, as in Eq. 2b, the value of $10^{-0.791}$ can be replaced by the value 0.32 (≈$10^{-0.791+\sigma}$), which corresponds to the mean value of *c_new* plus one standard deviation. The results of the new fitting are shown in Figure 23 and Figure 24. Figure 24 shows the comparison of the predicted and measured overtopping discharges using Eq. 14.

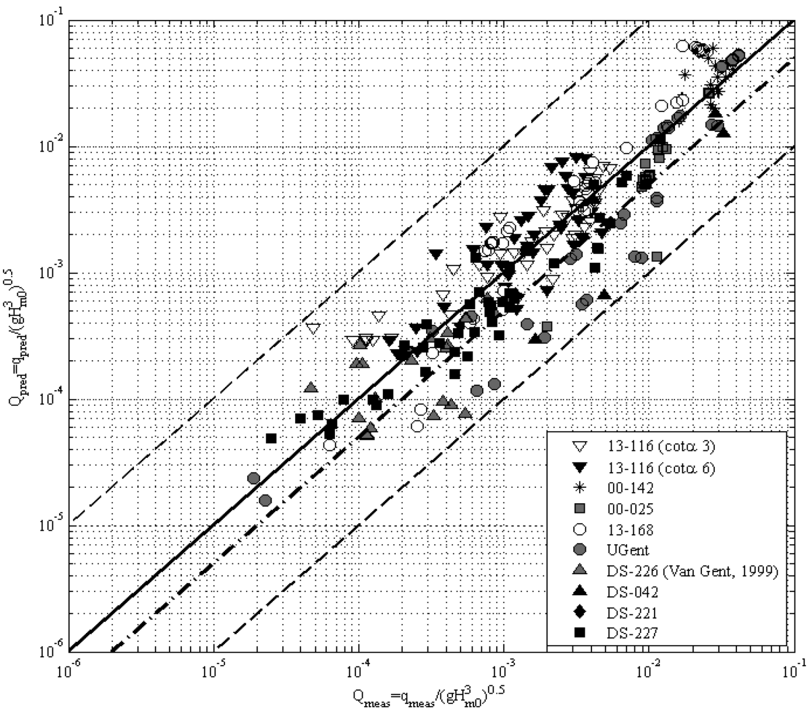
The results of the overtopping ratio are depicted in Figure 25 and Figure 26 that confirm the assumption of homoscedasticity as demonstrated in the previous paragraph (see Figure 21). The performance of the new approach for cases with emergent toe is also demonstrated (Figure 26).

Figure 23 - Wave overtopping data and prediction using Eq.14 with 5% under and upper exceedance limits



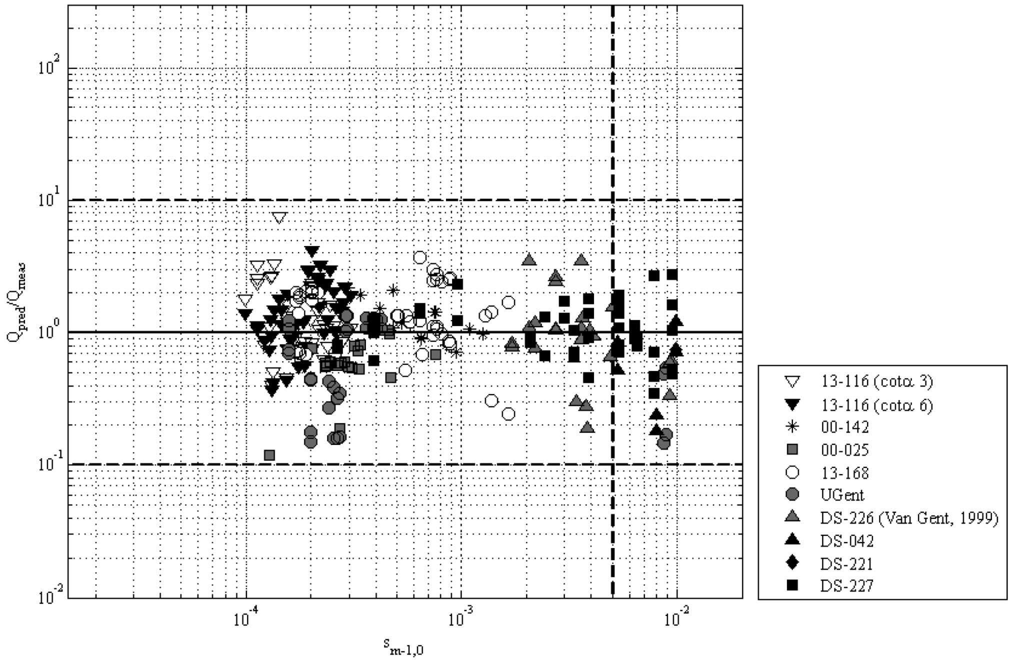
The dash-dot line represents the conservative approach (still called deterministic using definitions from EurOtop, 2007). The solid line represents the predictions using Eq. 14, meanwhile the two dot lines the upper and lower 5% exceedance probability. See Tables 1 and 2 for further details on the datasets.

Figure 24 - Measured versus predicted non-dimensional overtopping discharge using Eq.14 and equivalent slope.



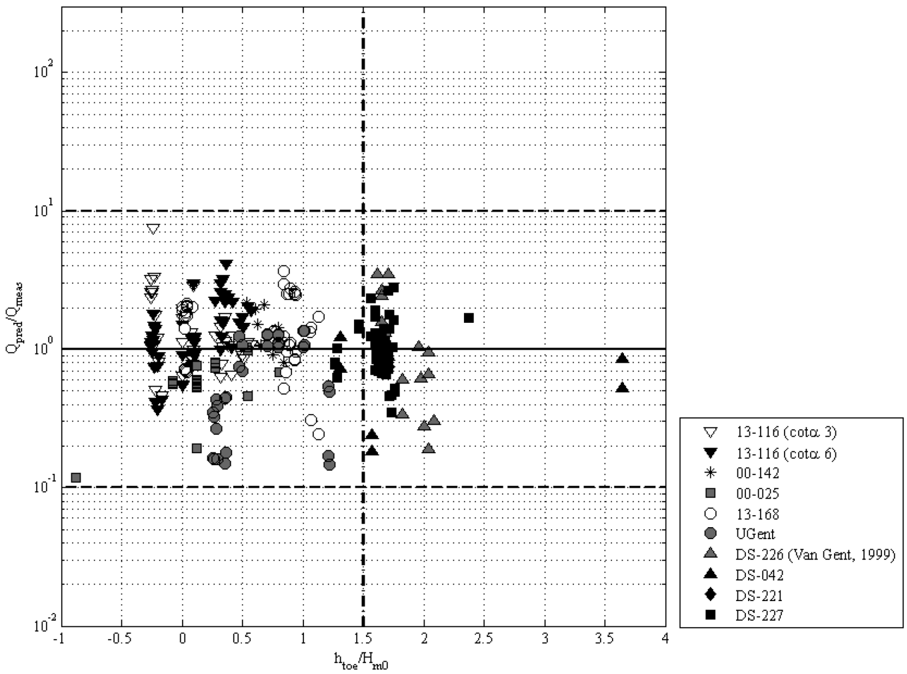
Three diagonal lines are added to the graph; a solid line that corresponds to a prediction equal to the measurement and two dashed lines corresponding to overtopping prediction 10 times and 0.1 times the measured data. Solid line: prediction using Eq. 14 equal to measured rate; dash-dot line: prediction using Eq.14 with *c_new* plus one standard deviation; dashed lines: prediction using Eq. 14 equal to 10 times and 0.1 times the measured rate.

Figure 25 - Eq.14: ratio of the dimensionless predicted to the measured overtopping versus the wave steepness.



The vertical dashed line indicate a wave steepness of 0.005; the two horizontal dashed lines correspond to predictions 10 times or 0.1 times the measured values; the horizontal solid line corresponds to $Q_{pred}/Q_{meas}=1$.

Figure 26 - Eq.14: ratio of the dimensionless predicted to the measured overtopping versus the ration between the toe depth and the wave height.



The vertical dashed line indicates a value of h_{toe}/H_{m0} of 1.5; the two horizontal dashed lines correspond to predictions 10 times or 0.1 times the measured values; the horizontal solid line corresponds to $Q_{pred}/Q_{meas}=1$.

The geometric mean (Geo) and geometric standard deviation (GSD) of each dataset using Eq.14 have been also calculated and are listed in the second to last and last columns in Table 5, respectively. Considering the overtopping rate as normally distributed, 90% of the predicted overtopping rate is to be located in the range between 0.310 and 3.227 times the measured overtopping discharge. Looking in detail at each dataset it can be noticed that:

- The overtopping prediction of DS-226 data (Van Gent's data) is on average 8% lower, meanwhile using the same Van Gent formula (Eq. 2a) was 32% lower than the measured data.
- All the datasets characterized by very low water depth or emergent toe and very low values of the wave steepness are well predicted (e.g. Geo_DS-227=0.993, Geo_13-168=1.343, Geo_13-116(cotα=3)=1.272, Geo_13-116(cotα=6)=1.300, Geo_00-142=1.181) being the prediction more accurate than using Eq. 2a (Geo_DS-227=0.860, Geo_13-168=6.539, Geo_13-116(cotα=3)=8.919, Geo_13-116(cotα=6)=3.184, Geo_00-142=1.429).
- The 00-025 data prediction seems to lead to an underestimate of mean wave overtopping if compared with Eq. 2a. However there are uncertainties in the wave boundary conditions due to the fact that the wave height and period at the toe of the dike have been calculated using numerical model in this dataset where incident waves are not measured during the experimental campaign.
- The prediction of UGent results is 43% underestimated on average. However, the prediction is more accurate than using Eq. 2a only which led to an overestimation of 170%.

Overall the results demonstrate the performance of using Eq.14 together with the equivalent slope to define the breaker parameter that will be used for overtopping calculations. If the water depth at the toe is larger than $1.5 \cdot H_{m0}$ then only the dike slope has to be used in Eq.14. Moreover, the use of the wave conditions estimated at the toe of the dike, both for still water level below and above the toe level of the dike, is demonstrated to give accurate results.

It is recommended to use Eq. 14 within the range of the datasets contained in the present work (for example, no intermediate berm, perpendicular wave attack, no storm walls, wave steepness lower than or equal to 0.01). Outside that range, Eq.14 can be used only for a very preliminary wave overtopping calculation.

Table 5 - Results of prediction performance using the "equivalent slope" concept applied to Eq.14.

Dataset id.	N	"Equivalent slope" in Eq.(14) q_{est}/q_{pred}	
		Geo	**GSD**
13-116 (cotα=3)	45	1.272	1.780
13-116 (cotα=6)	45	1.300	1.745
00-142	17	1.181	1.386
00-025	21	0.562	1.614
13-168	42	1.343	1.802
UGent	34	0.570	2.208
DS-226 (Van Gent)	24	0.920	2.081
DS-042	6	0.507	1.972
DS-221	1	0.752	1.000
DS-227	44	0.993	1.620
TOT	279	1.000	1.968

6 Conclusions

Many different formulae exist to predict the mean overtopping discharge per meter width of the coastal defence. Nevertheless, the process of wave overtopping on sea dikes with shallow and very shallow foreshore is still not yet fully understood. Gentle foreshores in combination with (very) shallow water conditions lead to heavy wave breaking and cause drastic changes of the wave spectra from offshore to the toe of the dike. The waves that reach the toe of the dike and that then overtop the dike are characterized, under such conditions, by very long periods (larger than 40s and up to 100s or more) and by limited height. These broken waves present an unusual shape, more similar to a bore rather than to an oscillatory phenomenon around the mean water level.

The present study aims to offer a new insight in wave overtopping of sea dikes with shallow and very shallow foreshores introducing the concept of “equivalent slope in shallow foreshore”. The works revises and integrates the work of Van Gent (1999) on overtopping of sea dikes in shallow water conditions and the work of Goda (2009). In detailed, much shallower conditions than in Van Gent (1999) and cases with emergent toe are considered. By using Van Gent (1999) formula the overtopping discharge is overestimated, especially for low steepness values for values of the h_{toe}/H_{m0} quantity smaller than 1.5. The new “equivalent slope in shallow foreshore” concept is therefore introduced, defined as an average slope between the foreshore and the dike slope. The influence of the foreshore slope on overtopping discharge is taken into account in this way.

The poor performance of Van Gent’s and Goda’s formulae is justified by the fact that sea dikes with very shallow foreshore represent a limit case that was not fully analysed by the authors. Therefore, their formulae are often used outside their natural range of application, in this case. Nevertheless, they represent a typical sample of the existing approaches in literature that can be used for the purpose of this study.

A final formula has been calibrated with the CLASH, FHR and UGent datasets, showing the overall agreement with the measured data without any bias. The formula has the same form of the Van Gent’s one but includes the equivalent slope and a different coefficient. This new approach is applicable for sea dikes with smooth and impermeable surfaces, without intermediate berm and storm walls.

Nevertheless, it is recommended to use the present approach within the range of the selected datasets: foreshore slope, cot θ, between 20 and 250; dike slope, cot α, between 2 and 6: range of h_{toe}/H_{m0} between (-0.88) and 2.38.

The equivalent slope concept along with the new fitting for the overtopping formula are recommended for those cases characterized by wave steepness $s_{m-1,0} \leq 0.01$. However, if the water depth at the toe of the dike, h_{toe}, is bigger than 1.5 times the incident significant wave height, H_{m0}, then the equivalent slope is equal to the dike slope: in that case the user can choose between Van Gent (1999) or the new Eq. 14.

It should be noted that there is still a gap between formulae for breaking and non-breaking waves in deep or intermediate waters and methods for shallow and very shallow waters. Further research must identify a proper transition between these two different sets of formulae.

Finally, it is noteworthy that the prediction of mean wave overtopping discharges usually has a broad range of uncertainties, one of which due to the stochastic nature of wave overtopping of random waves. The wave boundary conditions, the number of waves, the individual wave height and the combination of wave heights and periods, might produce different overtopping discharges even in case of the same specified wave spectra (Williams et al., 2015; Romano et al. 2015). This variability is high in cases of low overtopping rates (Williams et al., 2014). Furthermore the overtopping is a fully three-dimensional phenomenon, but often represented with two-dimensional models. Indeed, in many models it is assumed that the overtopping discharge is uniformly distributed along the coastal defence.

Uncertainties also lie in the choice of the model used to represent a phenomenon. For example, in the case of mean overtopping discharge, a certain structure of the formula is assumed and its coefficients are usually evaluated through some form of regression. From a purely practical point of view, depending on the model that has been selected, a formula for wave overtopping should be employed for preliminary assessment only. A physical scale model test or a numerical modelling might be employed in any case of detailed design to confirm the overtopping prediction by any formula.

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Glossary

A: intercept coefficient for overtopping rate formula given by Eq. 5

A_0: intercept constant given by Eq. 7

B: gradient coefficient of overtopping rate formula given by Eq. 6

B_0: gradient constant given by Eq. 8

g: gravitational acceleration

Geo: geometric mean as defined by Eq.9

GSD: geometric standard deviation as defined by Eq. 10

$H_{m0\text{-}DEEP}$: significant wave height at deep water calculated from spectral analysis as $4m_0^{1/2}$

H_{m0}: significant wave height at the toe of the structure calculated from spectral analysis as $4m_0^{1/2}$

h_{toe}: water depth at the toe of the structure

L_{slope}: horizontal length of the zone between $SWL\text{-}1.5H_{m0}$ and $SWL+Ru_{2\%}$

mn: n-th spectral moment

m0: zero-th spectral moment

N: number of data

q: mean wave overtopping rate per unit width per second

q_{meas}: measured value of dimensionless overtopping rate

q_{pred}: predicted value of dimensionless overtopping rate

Q: dimensionless overtopping rate defined by $Q=q/(gH_{m0}^{3})^{1/2}$

Q_{meas}: measured value of dimensionless overtopping rate

Q_{pred}: predicted value of dimensionless overtopping rate

R_c: freeboard or the crest height above the design water level

$Ru_{2\%}$:is the wave run-up exceeded by 2% of the incident waves

$s_{m-1,0}$: wave steepness defined as $2\pi H_{m0}/(gT_{m-1,0}^{2})$

SWL: still water level

$tan(\delta)$: equivalent slope as defined in Eq. 11

$T_{m-1,0}$: wave period defined with m_{-1}/m_0

α: angle between the surface of inclined seawall (e.g. sea dike) and the horizontal plane

γ_b: influence factor of a berm

γ_f: influence factor of roughness elements on a slope

γ_β: influence factor of oblique wave attack

ϑ: angle that the foreshore makes with the horizontal plane

$\xi_{m-1,0}$: Irribarren number or breaker parameter defined by Eq. 1

σ: standard-deviation (-)

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