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## Methodology for safety assessment 2015

Background report on the assessment of  
wave overtopping at sea dikes

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## Background report on the assessment of wave overtopping at sea dikes

Altomare, C.; Willems, M.; Suzuki, T.; Verwaest, T.; Mostaert, F.

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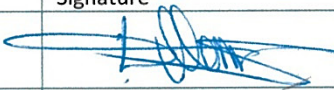
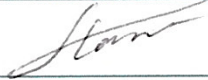
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# Abstract

The present report describes the analysis conducted to improve the overtopping assessment for cases that are typical at the Flemish coast. Sea dikes and quay walls with non-standard geometries (such as storm return walls in combination with wide crests or promenades) are analysed. Furthermore the overtopping reduction for very oblique wave attack is examined and results from physical model tests carried out at Flanders Hydraulics Research are discussed. Finally the overtopping assessment method for a simple dike and in presence of a foreshore with very shallow water is discussed. An overview of numerical models applicable to cases at the Flemish coast is also given.





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# 1 Introduction

## 1.1 Background

In 2015, there will be a safety assessment for all the cross-sections of the Flemish coast. The purpose of this safety assessment is to examine the safety level of the entire coast by a certain methodology and hydraulic boundary condition. The last safety assessment was conducted in 2008.

The methodology used in the last safety assessment (IMDC, 2008) has been updated based on recent discussions, newly proposed empirical equations and new numerical models.

The present document deals with the assessment of mean overtopping discharge at sea dikes. The results from different projects conducted at Flanders Hydraulics Research (FHR) have been used. The aim of the analysis was to evaluate which methodology (i.e. semi-empirical formulae) is suitable for the estimation of wave overtopping of sea dikes at the Flemish coast. Due to the peculiarities of the Flemish coast and coastal defences, the analysis has been focused on three main topics:

- dikes with non-standard geometrical layout (Chapter 2);
- effect of high wave obliqueness on wave overtopping (Chapter 3);
- influence of very shallow foreshore on wave overtopping (Chapter 4).

## 1.2 Methodology for safety assessment 2007

In the last methodology, the estimation of mean wave overtopping was based on semi-empirical formulae (see §8.5 of IMDC, 2008). Mostly the formulae were referring to TAW ([Van der Meer, 2002](#)) and [Den Heijer \(1988\)](#). It has to be noticed that the formulae from van der Meer (2002) are formally the same ones reported afterwards into the European Overtopping Manual ([EurOtop, 2007](#)).

## 1.3 Safety criteria for safety assessment 2015

The safety criterion is based on single standard, developed in the Coastal Safety Master Plan:

- 1000 year storm; no casualties :  $q < 1 \text{ l/s/m}$  @ (veiligheidslijn or “safety line”)



## 2 Wave overtopping discharge over non-standard geometries

### 2.1 Introduction

The present chapter summarizes the available approaches that can be followed to quantify the mean overtopping discharge,  $q$ , over sea dikes and quay walls with complex or non-standard geometries.

For this scope, the following layouts are defined as a non-standard geometry:

1. curve-faced storm return walls;
2. wide dikes with storm return walls (vertical walls, parapets and curve-shaped);
3. stilling wave basins.

The three mentioned cases are not exhaustive of all the possible geometries, but they are typical solutions that have been proposed for and adopted at the Flemish coast. In general, these complex geometries are conceived to reduce the overtopping rates, with respect to the most classical solutions (i.e. sea dikes, vertical quays, etc.). However the effects of such particular layouts on the overtopping discharge is not yet well defined in available equations (e.g. [EurOtop, 2007](#)).

The most recent approaches to quantify the overtopping discharge over such kind of structures are summarized here. A limitation exists: using new approaches is not recommended out of range of the geometrical and hydraulics conditions on which each approach is based. To apply new formulae out of their range introduces uncertainties that cannot be easily quantified. For this reason possible alternatives to assess the overtopping discharge for complex geometries not covered by literature are:

- A. physical model tests in wave flume and wave basin facilities;
- B. numerical modelling;
- C. Artificial Neural Network.

The present chapter is organized in five main sections. Section 2.2 analyses the most recent formulations available in literature for sea dikes. Section 2.3 validates the formulas against physical model test results. The existing formulas for vertical quays are summarized in Section 2.4. Section 2.5 introduces available numerical models, currently in use by the Belgian community of coastal engineers (the list is representative but not exhaustive). Finally, section 2.6 provides a brief description of the Artificial Neural Networks in use for the wave overtopping assessment.

### 2.2 Semi-empirical formulae for sea dikes

The present section analyses the most appropriate formulations, available in literature, to assess the overtopping discharge over complex geometries. In detail:

- [Den Heijer \(1988\)](#): quay walls and dike with wall on top and long crest berms, where the crest is normally submerged (crest berm below the SWL);
- [Verwaest et al. \(2010\)](#): sea dikes with wide crests;
- [Van Doorslaer et al. \(2015\)](#): sea dikes with storm return walls, parapets, long berms and stilling wave basins.

All the aforementioned approaches propose reduction coefficients to account for the effect of the particular layout. These coefficients are introduced in the most classical formulae for overtopping discharge over sea dikes and quay walls (Franco et al., 1994; Allsop et al., 1995; EurOtop, 2007; van der Meer and Bruce, 2013).

### 2.2.1 Sloping dike

Two formulae to assess the mean overtopping discharge for sea dikes have been considered here:

1. Eq. 5.8 from EurOtop (2007) for breaking and non-breaking waves;
2. Van der Meer and Bruce (2013) formula that represents a modification of the EurOtop (2007), more adequate for very low freeboards.

The EurOtop Manual describes the reliability of the formula by taking the coefficients of the overtopping formulas as stochastic parameters with a certain standard deviation (assuming a normal distribution). This approach is defined as “probabilistic”. The “deterministic design or safety assessment” approach in the EurOtop Manual should be termed a probabilistic approach as a partial safety factor of one standard deviation is used. In the present document the formulas are given in terms of probabilistic approach. Meanwhile, the formulas for deterministic approach will be proposed in the latest version of the new methodology for the safety assessment: this choice is motivated by the fact that, for design purposes, the deterministic approach is considered a safe approach because it includes somehow the uncertainties of wave overtopping prediction. The formulae are expressed as follows:

#### EurOtop (2007), Eq. 5.8

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

$$\frac{q}{\sqrt{gH_{m0}^3}} = 0.2 \exp\left(-2.6 \frac{R_c}{H_{m0} \gamma_f \gamma_\beta}\right) \quad \text{for non-breaking waves} \quad (2)$$

where:

- $q$  is the overtopping discharge per meter width of the structure [ $\text{m}^3/\text{s}/\text{m}$ ]
- $H_{m0}$  is the significant incident wave height, measured at the toe of the structure [m]
- $R_c$  is the crest freeboard [m]
- $\xi_{m-1,0}$  is the surf similarity parameter [-]
- $\alpha$  is the dike angle [°]
- $\gamma_f$  is the reduction coefficient that considers the effects of the slope roughness [-]
- $\gamma_\beta$  is the reduction coefficient that considers the effects of the obliqueness [-]
- $\gamma_b$  is the reduction coefficient that considers the effects composite slopes and berms [-]
- $\gamma_v$  is the reduction coefficient that considers the effects of possible storm return walls [-]

#### van der Meer and Bruce (2013)

$$\frac{q}{\sqrt{gH_{m0}^3}} = 0.09 \exp\left(-1.5 \frac{R_c}{H_{m0} \gamma_f \gamma_\beta}\right)^{1.3} \quad (3)$$

Although the presence of a berm or a storm wall is considered in EurOtop (2007), there are a few limitations:

- a) those effects are not considered for non-breaking waves, whereas is demonstrated (Van Doorslaer et al., 2015; Verwaest et al., 2010) that the presence of a wide crest berm is influencing the measured overtopping discharge;
- b) the definition of  $\gamma_v$  is quite limited to very specific cases.

Hence, the approaches proposed by Van Doorslaer et al. (2015) and Verwaest et al. (2010) are considered to study the overtopping discharge over sea dikes with wide crests and storm return walls.

### 2.2.2 Complex geometries

#### Verwaest et al. (2010)

The reduction due to the presence of a very long crest is examined and expressed as a ratio,  $\alpha$ , between  $q$  (overtopping discharge over wide crested dikes) and  $q_0$  (mean overtopping discharge over short crested dikes). The reduction is function of the roughness of the crest (expressed through the Manning coefficient,  $n$ ), the crest slope,  $t$ , and the crest berm width,  $B$ .

Verwaest et al. (2010) proposed the following expression:

$$\alpha = \frac{q}{q_0} = \sqrt{\exp(-22\kappa\beta) - 0.21 \frac{t}{k} (1 - \exp(-22\kappa\beta))} \quad (4)$$

With  $\alpha=0$  if the expression under the root is negative. The dimensionless parameters  $\kappa$  and  $\beta$  are defined as follows:

$$\kappa = \frac{gn^2}{(R_{u2\%} - R_c)^{1/3}} \quad (5)$$

$$\beta = \frac{B}{(R_{u2\%} - R_c)} \quad (6)$$

where,

- $g$  is the gravitational acceleration [ $m/s^2$ ]
- $R_{u2\%}$  is the wave run-up height exceeded by 2% of incoming waves [m]. The expression of  $R_{u2\%}$  is given by EurOtop (see EurOtop, 2007; eq. 5.3):

$$R_{u2\%} = 1.65H_{m0}$$

$$\text{with maximum of } R_{u2\%} = H_{m0} \left( 4 - \frac{1.5}{\sqrt{\xi_{m-1,0}}} \right) \quad (7)$$

#### Van Doorslaer et al. (2015)

The authors investigated the influence of complex geometries on overtopping for non-breaking waves. EurOtop (2007) was in fact lacking of such analysis: the influence of storm walls on wave overtopping, for example, is described in EurOtop (2007) only for breaking waves. Van Doorslaer et al. (2015) define different reduction coefficients  $\gamma$  for different geometrical layouts:

- a. smooth dike + storm return wall ( $\gamma_v$ )
- b. smooth dike + parapet wall ( $\gamma_{par}$ )
- c. smooth dike + promenade ( $\gamma_{prom}$ )
- d. smooth dike + promenade + wall ( $\gamma_{prom\_v}$ )
- e. smooth dike + promenade + parapet ( $\gamma_{prom\_v\_par}$ )
- f. stilling wave basin ( $\gamma_{SWB}$ )

The (quasi) horizontal part at crest level is hereafter called promenade. The promenade usually is characterized by a gentle slope of 1% to 2% and it is different than what is meant by the term “berm” in the

EurOtop manual. A berm is a (quasi) horizontal part in the dike slope and often located around the design water level, to reduce the average slope and thereby reducing wave overtopping discharges.

Van Doorslaer et al. (2015) proposed slightly different coefficients from eq. (2), but keeping the same exponential structure. The reduction coefficients  $\gamma_f$  and  $\gamma_\theta$  are assumed equal to 1 (smooth dike and perpendicular attack). The new formula for smooth dikes can be expressed as follows:

$$\frac{q}{\sqrt{gH_{m0}^3}} = 0.2 \exp\left(-2.28 \frac{R_c}{H_{m0}\gamma_f\gamma_\theta}\right) \quad \text{for non-breaking waves} \quad (8)$$

where the coefficient 2.28 has a standard deviation  $\sigma$  equal to 0.15. The range of parameters of the tests carried out by Van Doorslaer et al. (2015) is summarized in Table 1 and must be assumed as the range of applicability of equation 8. Finally Van Doorslaer et al. (2015) recommend to use still the formula for overtopping discharge as in EurOtop but with the new correction factors. Therefore we will refer later on only to eq. (2).

Table 1 - Summary of the characteristics of the tests (non-breaking waves) on a smooth dike slope (scale model values) as in Van Doorslaer et al. (2015)

|                                      |                |               |
|--------------------------------------|----------------|---------------|
| Slope angle of the smooth dike slope | $\cot(\alpha)$ | 2 and 3       |
| Mean spectral wave period            | $T_{m-1,0}$    | 0.91 – 2.45 s |
| Dimensionless freeboard              | $R_c/H_{m0}$   | 1 – 3         |
| Freeboard (top of structure to SWL)  | $R_c$          | 0.12 – 0.27 m |
| Spectral wave height                 | $H_{m0}$       | 0.07 – 0.21 m |
| Water depth at toe of the structure  | $d$            | 0.35 – 0.49 m |
| Wave breaker parameter               | $\xi_{m-1,0}$  | 2.1 – 4.9     |

a) *smooth dike + storm return wall ( $\gamma_v$ )*

The first case (smooth dike + storm return wall) corresponds to the situation where a vertical wall (with height  $h_{wall}$ ) is built on the top of dike (see Figure 1). The reduction coefficient  $\gamma_v$  is defined as follows:

$$\gamma_v = \exp\left(-0.563 \frac{h_{wall}}{R_c}\right) \quad \text{for } h_{wall}/R_c < 1.24 \quad (9)$$

$$\gamma_v = 0.5 \quad \text{for } h_{wall}/R_c \geq 1.24 \quad (10)$$

The range of applicability of the reduction coefficient  $\gamma_v$  has to be related with the experimental conditions used to calibrate it. They can be summarized as in Table 2.

So, eq. (2) can be rewritten as follows:

$$\frac{q}{\sqrt{gH_{m0}^3}} = 0.2 \exp\left(-2.6 \frac{R_c}{H_{m0}\gamma_v}\right) \quad (11)$$

Figure 1 - Smooth dike + vertical storm return wall (source: Van Doorslaer et al., 2015)

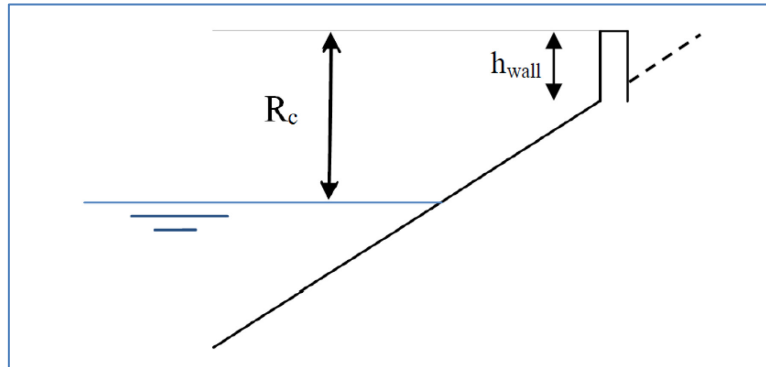


Table 2 - Summary of the characteristics of the tests on a smooth dike slope with storm wall (scale model values) as in Van Doorslaer et al. (2015)

|                 |                     |
|-----------------|---------------------|
| $\cot(\alpha)$  | 2 and 3             |
| $T_{m-1,0}$     | 1.1 – 3.00 s        |
| $R_c/H_{m0}$    | 0.6 – 2.6           |
| $h_{wall}/R_c$  | 0.08 – 1.6          |
| $h_{wall}$      | 2, 4, 5, 6 and 8 cm |
| $R_c$           | 0.05 – 0.29 m       |
| $H_{m0}$        | 0.07 – 0.18 m       |
| $d_{foreshore}$ | 0.36 – 0.57 m       |

b) smooth dike + parapet wall ( $\gamma_{par}$ )

A new coefficient is presented, function of the parapet geometry. The coefficient,  $\gamma_{par}$ , is expressed as follows:

$$\gamma_{par} = 1.8\gamma_\epsilon\gamma_\lambda \quad \text{for } h_{wall}/R_c \geq 0.25 \quad (12)$$

where:

$$\begin{aligned} \gamma_\epsilon &= 1.53 \cdot 10^{-4}\epsilon^2 - 1.63 \cdot 10^{-2}\epsilon + 1 & \text{if } 15 \leq \epsilon \leq 50 \\ \gamma_\epsilon &= 0.56 & \text{if } \epsilon > 50 \\ \gamma_\lambda &= 0.75 - 0.20\lambda & \text{if } 0.125 \leq \lambda \leq 0.6 \end{aligned} \quad (13)$$

and

$$\gamma_{par} = 1.8\gamma_\epsilon\gamma_\lambda - 0.53 \quad \text{for } h_{wall}/R_c < 0.25 \quad (14)$$

where:

$$\begin{aligned} \gamma_\epsilon &= 1 - 0.003\epsilon & \text{if } 15 \leq \epsilon \leq 60 \\ \gamma_\lambda &= 1 - 0.1449\lambda & \text{if } 0.1 \leq \lambda \leq 1 \end{aligned} \quad (15)$$

The quantities  $\lambda$  and  $\epsilon$  are described in Figure 2.

Furthermore Van Doorslaer et al. (2015) demonstrated that the overtopping discharge over a smooth dike + parapet is dependent on the wave period. A coefficient,  $\gamma_{s0,par}$ , is then defined as:

$$\gamma_{s0,par} = 1.33 - 10s_{m-1,0} \quad (16)$$

Finally eq.(2) can be rewritten as follows:

$$\frac{q}{\sqrt{gH_{m0}^3}} = 0.2 \exp\left(-2.6 \frac{R_c}{H_{m0}\gamma_v\gamma_{par}\gamma_{s0,par}}\right) \quad (17)$$

The range of the parameters used by Van Doorslaer et al. (2015) for such kind of geometries is given in Table 3.

Figure 2 - Smooth dike + parapet wall (source: Van Doorslaer et al., 2015)

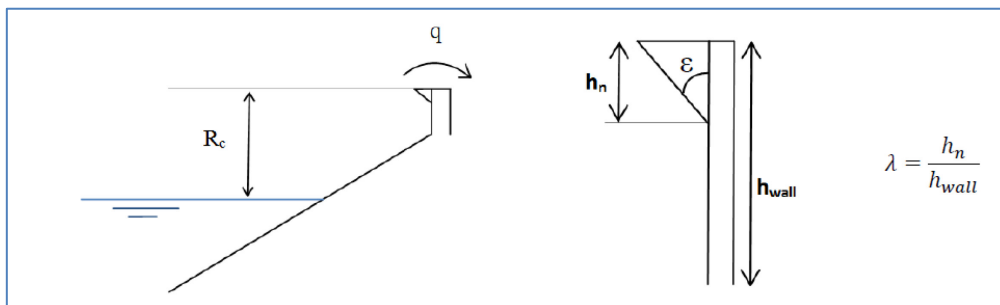


Table 3 - Summary of the test program on a smooth dike slope with storm wall and parapet as in Van Doorslaer et al. (2015)

|                            |                    |
|----------------------------|--------------------|
| $\cot(\alpha)$             | 2 and 3            |
| $T_{m-1,0}$                | 1.1 – 2.3 s        |
| $R_c/H_{m0}$               | 0.6 – 2.4          |
| $h_{wall}$                 | 2, 5 and 8 cm      |
| $h_n$                      | 1, 2 and 3 cm      |
| $\lambda$                  | 0.125 – 1          |
| $\varepsilon$ (in degrees) | 15°, 30°, 45°, 60° |
| $d_{foreshore}$            | 0.36 m – 0.53 m    |
| $\xi_{m-1,0}$              | 2.20 – 4.61        |

#### a) smooth dike + promenade ( $\gamma_{prom}$ )

The case with smooth dike + promenade (c) corresponds to cases with sloping dike with quite wide crest, like the cases studied by Verwaest et al. (2010).

The reduction coefficient is then function of the promenade width  $B$  and the wave length in deep water ( $L_{m-1,0} \approx 1.56T_{m-1,0}^2$ ) and is expressed as follows:

$$\gamma_{prom} = 1 - 0.47 \frac{B}{L_{m-1,0}} \quad (18)$$

So that eq.(2) can be rewritten as follows:

$$\frac{q}{\sqrt{gH_{m0}^3}} = 0.2 \exp\left(-2.6 \frac{R_c}{H_{m0}\gamma_{prom}}\right) \quad (19)$$

Van Doorslaer et al. (2015) performed a total of 62 tests on this geometry. The range of the parameters of the test program is given in Table 4. A scheme of the studied geometry is depicted in Figure 3.

Figure 3 - Scheme of dike with promenade (source: Van Doorslaer et al., 2015)

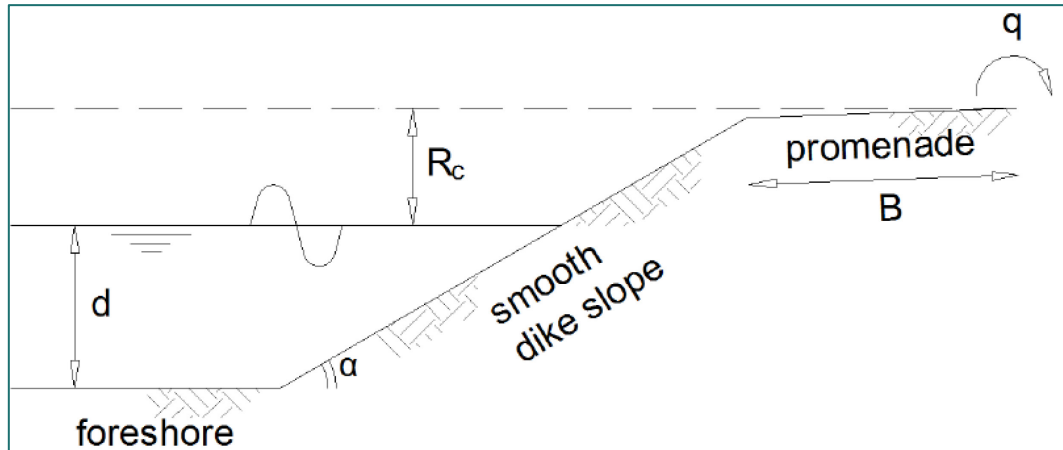


Table 4 - Summary of the characteristics of the tests on a smooth dike slope with promenade as in Van Doorslaer et al. (2015)

|                        |                       |
|------------------------|-----------------------|
| $\cot(\alpha)$         | 2 and 3               |
| slope promenade        | 1% and 2%             |
| $T_{m-1,0}$            | 1.1 – 2.22 s          |
| $R_c/H_{m0}$           | 0.85 – 2.68           |
| $R_c$                  | 0.10 – 0.28 m         |
| $H_{m0}$               | 0.07 – 0.17 m         |
| $B$                    | 33.3, 66.7 and 100 cm |
| $B/L_{m-1,0}$          | 0.045 – 0.5           |
| $\xi_{m-1,0}$          | 2.2 – 4.2             |
| $d_{\text{foreshore}}$ | 0.28 m – 0.53 m       |

b) smooth dike + promenade + wall ( $\gamma_{prom\_v}$ )

When a storm return vertical wall is built on top of the promenade, a new coefficient  $\gamma_{prom\_v}$  is introduced, combination of  $\gamma_{prom}$  and  $\gamma_v$ . This coefficient is equal to:

$$\gamma_{prom\_v} = 0.87\gamma_{prom}\gamma_v \quad (20)$$

So that eq.(2) can be rewritten as follows:

$$\frac{q}{\sqrt{gH_{m0}^3}} = 0.2 \exp\left(-2.6 \frac{R_c}{H_{m0}\gamma_{prom\_v}}\right) \quad (21)$$

The range of the parameters of the test program for such kind of geometry is given in Table 5.

Table 5 - Summary of the characteristics on a smooth dike slope with promenade and storm wall as in Van Doorslaer et al. (2015)

|                 |                       |
|-----------------|-----------------------|
| cot( $\alpha$ ) | 2 and 3               |
| slope promenade | 1% and 2%             |
| $T_{m-1,0}$     | 1.1 – 2.25 s          |
| $R_c/H_{m0}$    | 0.8 – 2.5             |
| $R_c$           | 0.10 – 0.28 m         |
| $H_{m0}$        | 0.075 – 0.17 m        |
| B               | 33.3, 66.7 and 100 cm |
| $h_{wall}$      | 2, 4, 6 and 8 cm      |
| $\xi_{m-1,0}$   | 2.26 – 4.80           |
| $d_{foreshore}$ | 0.36 m – 0.55 m       |

c) *smooth dike + promenade + parapet* ( $\gamma_{prom\_v\_par}$ )

When a parapet vertical wall is built on top of the promenade, a new coefficient  $\gamma_{prom\_v\_par}$  is introduced. This coefficient is equal to:

$$\gamma_{prom\_v\_par} = 1.19\gamma_{prom\_v}\gamma_{par} \quad (22)$$

So that eq.(2) can be rewritten as follows:

$$\frac{q}{\sqrt{gH_{m0}^3}} = 0.2 \exp\left(-2.6 \frac{R_c}{H_{m0}\gamma_{prom\_v\_par}}\right) \quad (23)$$

The range of the parameters of the test program for such kind of geometry is given in Table 6.

Table 6 - Summary of the characteristics of the tests performed on a smooth dike slope with promenade, storm wall and parapet as in Van Doorslaer et al. (2015)

|                 |                       |
|-----------------|-----------------------|
| cot( $\alpha$ ) | 2 and 3               |
| slope promenade | 1% and 2%             |
| $T_{m-1,0}$     | 1.25 – 2.25 s         |
| $R_c/H_{m0}$    | 0.7 – 1.9             |
| $R_c$           | 0.08 – 0.24 m         |
| $H_{m0}$        | 0.08 – 0.17 m         |
| B               | 33.3, 66.7 and 100 cm |
| $h_{wall}$      | 4, 6 and 8 cm         |
| $\varepsilon$   | 30°, 45°              |
| $\lambda$       | 0.25 - 0.375          |
| $\xi_{m-1,0}$   | 2.15 – 4.77           |
| $d_{foreshore}$ | 0.40 m – 0.55 m       |

Differently from Van Doorslaer et al. (2015), Den Heijer (1998) analysed the case when the dike is submerged (SWL above the crest), hence part of the parapet is below the SWL, part of the parapet is above the SWL. In such case the author introduced a reduction coefficient  $\gamma_{pt}$  defined as follows:

$$\gamma_{pt} = 1.0 \quad \text{for } R_c/H_{m0} \leq 0.5$$



$$\gamma_{pt} = 1.4 - 0.8 \left( \frac{R_c}{H_{m0}} \right) \quad \text{for } 0.5 < R_c/H_{m0} \leq 1.0 \quad (24)$$

$$\gamma_{pt} = 0.6 \quad \text{for } R_c/H_{m0} > 1.0$$

d) *stilling wave basin* ( $\gamma_{SWB}$ )

For the last case (stilling wave basin, SWB) [Van Doorslaer et al. \(2015\)](#) discussed the effects of a particular SWB layout on mean wave overtopping (Figure 4). The concept of stilling wave basin is introduced by [Geeraerts et al. \(2006\)](#) and [Beels \(2005\)](#) and is based on the principle of the wave energy dissipation between the two walls forming the basin. For the proposed geometry, [Van Doorslaer et al. \(2015\)](#) proposed a reduction coefficient  $\gamma_{SWB}$  equal to 0.48. [Geeraerts et al. \(2006\)](#) made a distinction between breaking and non-breaking waves with SWB, proposing for the same geometry a value for  $\gamma_{SWB}$  equal to 0.48 and 0.44 respectively for non-breaking and breaking waves.

Eq.(2) can be rewritten then as follows:

$$\frac{q}{\sqrt{gH_{m0}^3}} = 0.2 \exp \left( -2.6 \frac{R_c}{H_{m0} \gamma_{SWB}} \right) \quad (25)$$

Figure 4 - Side view of Stilling Wave Basin (dimensions in mm), Geeraerts et al. (2006)

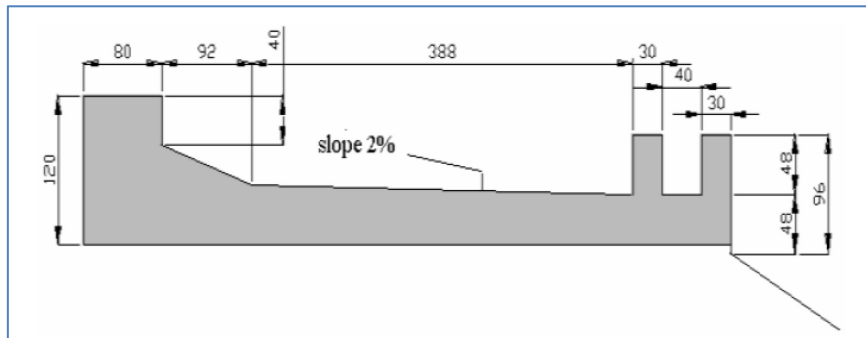


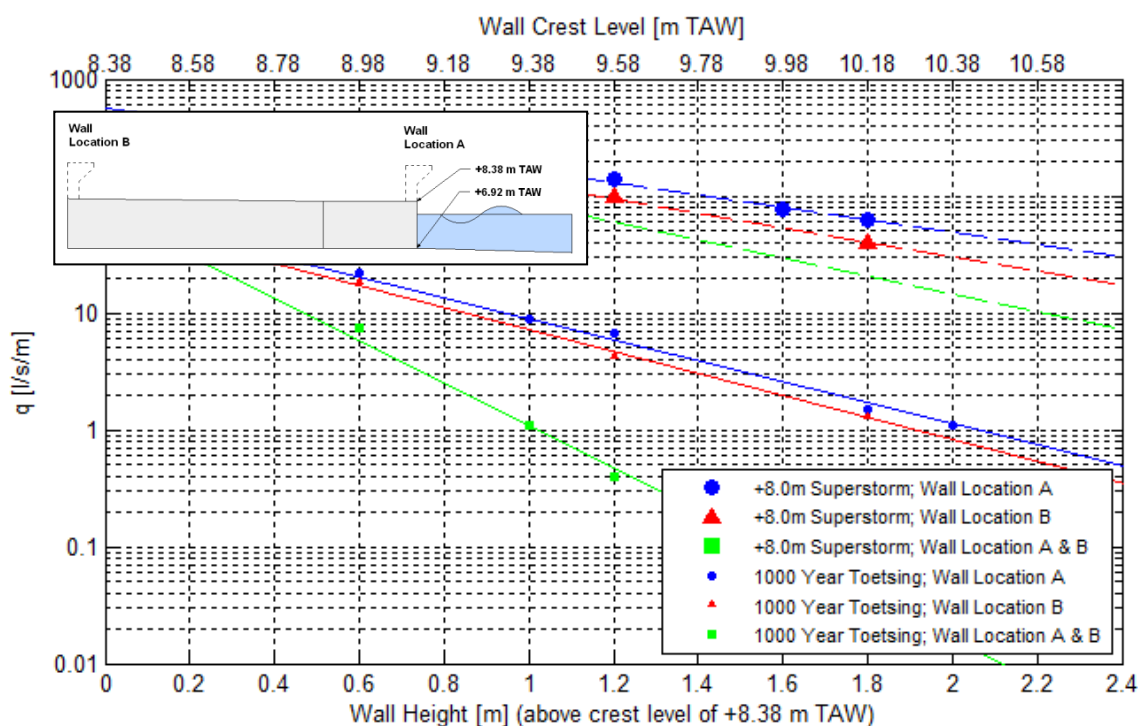
Table 7 -Summary of the characteristics of the tests performed on a smooth dike slope with SWB as in Van Doorslaer et al. (2015)

|                                     |                            |
|-------------------------------------|----------------------------|
| cot( $\alpha$ )                     | 2; 2.5 and 3               |
| slope basin                         | 2%                         |
| $T_{m-1,0}$                         | 1.16 s – 2.33 s            |
| $R_c/H_{m0}$                        | 0.56 – 2.7                 |
| $R_c$                               | 0.10 – 0.27 m              |
| $H_{m0}$                            | 0.08 – 0.18 m              |
| $L_{\text{basin}}$                  | 12, 24, 36 and 48 cm       |
| $h_{\text{front wall}}$             | 48, 72, 96, 120 and 144 mm |
| Distance in between front walls     | 4 cm                       |
| Water depth at toe of structure (d) | 0.30 – 0.52 m              |

SWB concepts have also been analysed during the experimental campaign conducted at Flanders Hydraulics Research (FHR) for assessment of wave overtopping on the sea dike of the coastal town of Wenduine ([Altomare et al., 2014a](#)). A sketch of the analysed geometry with the position of two storm return walls to

form the SWB is depicted in Figure 5. The results of the measured overtopping discharge compared with an only one-wall solution are also plotted. The efficiency of the SWB concept in respect to the one-wall solution is remarkable. However it can be noticed that the effectiveness of the SWB varies between the two different storm conditions, hence it does not only depend on the geometrical layout but also on the hydraulic boundary conditions. Therefore, the effect of a stilling wave basin has to be analysed case per case, depending on the particular situation, through physical and/or numerical modelling.

Figure 5 - Sketch of the layout tested at FHR for the coastal town of Wenduine (SWB effects) and overtopping discharge for two different storm conditions



(for coastal defences with vertical seaward face and wave return walls with parapet)

## 2.3 Validation vs physical model tests

Physical model tests have been recently conducted at the Maritime Engineering Laboratory of the Technical University of Catalonia (LIM/UPC) in Barcelona, Spain. The experimental campaign is part of the HydraLab IV framework. The aim of the experimental campaign was to assess the wave overtopping over sea dikes and vertical walls with storm return walls on top and the wave loadings on such walls. The experiments have been conducted at large model scale (1:6). The tests conditions are summarized in Table 8.

Table 8 - Tests conditions of the UPC experiments (model scale)

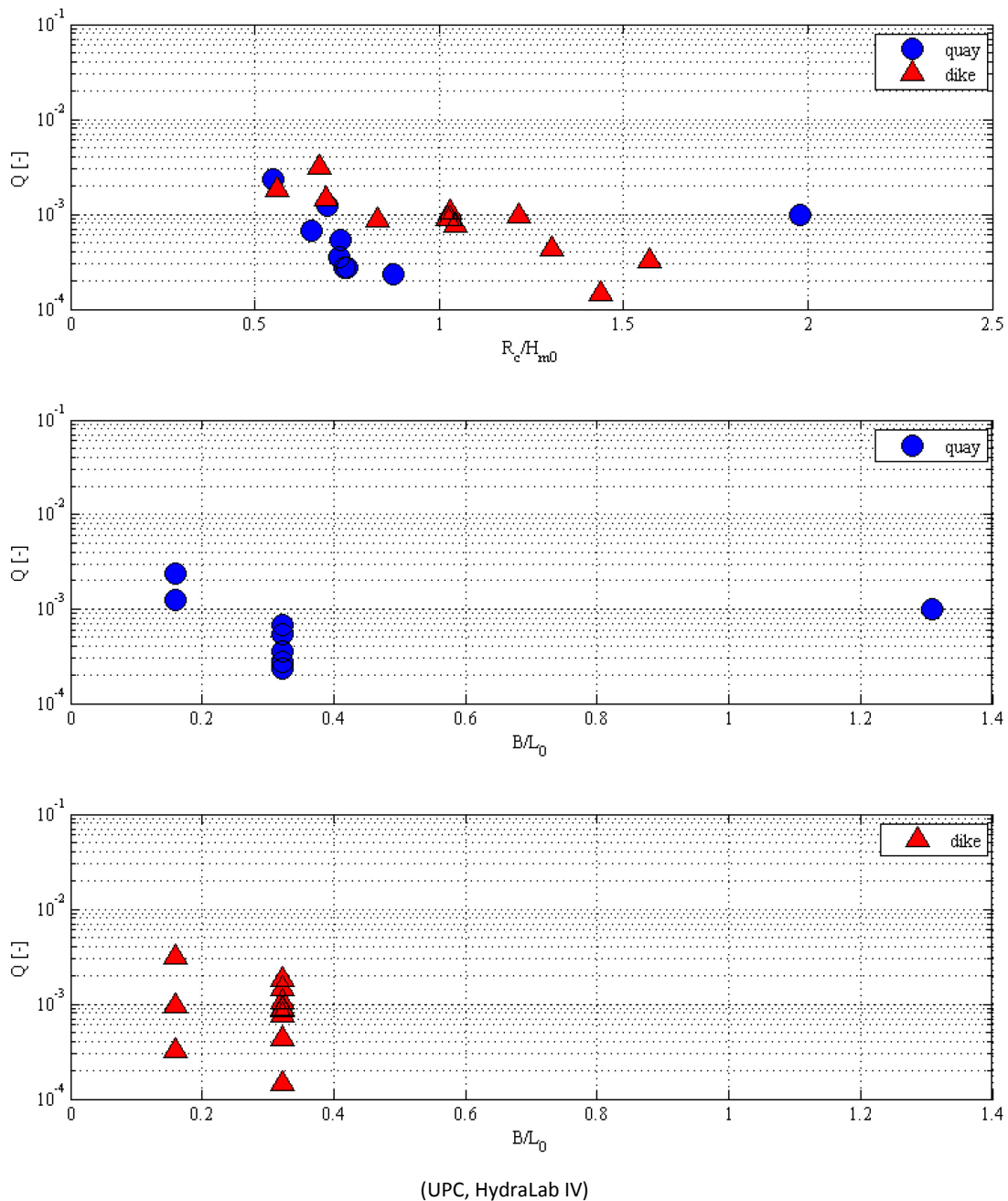
|                                |               |                       |
|--------------------------------|---------------|-----------------------|
| Wave height                    | $H_{m0}$ [m]  | 0.11 - 0.57           |
| Wave period                    | $T_p$ [s]     | 1.42 - 4.08           |
| water depth at the wave paddle | $d$ [m]       | 2.39 - 2.72           |
| Water depth at the toe         | $d_{toe}$ [m] | 1.49 - 1.82           |
| slope                          | $cot$ [-]     | 0 (quay), 2.94 (dike) |

The vertical quay was steel, the slopes was steel painted with sand ( $d_{50} = 0.250$  mm) and the top part of the quay was made of wood (not raw wood, the one that has a smooth cover on top 'marine board'). The plane part of the quay effectively had a little slope (1.5-2 %) in order to remove the water cover from the vertical wall at the end of the quay after each incoming wave.

The experimental results, both for dike and vertical quay, are plotted in Figure 6 in terms of non-dimensional overtopping discharge  $Q = q / \sqrt{gH_{m0}^3}$  against relative freeboard ( $R_c/H_{m0}$ ) and relative berm length ( $B/L_0$ ), where  $L_0 \approx 1.56T_p^2$ , with  $T_p$  the peak period.

As expected the same relative freeboard leads to less overtopping for the vertical quay than for the sloping dike.

Figure 6. Measured non-dimensional overtopping discharge for quay and dike layout with storm return walls

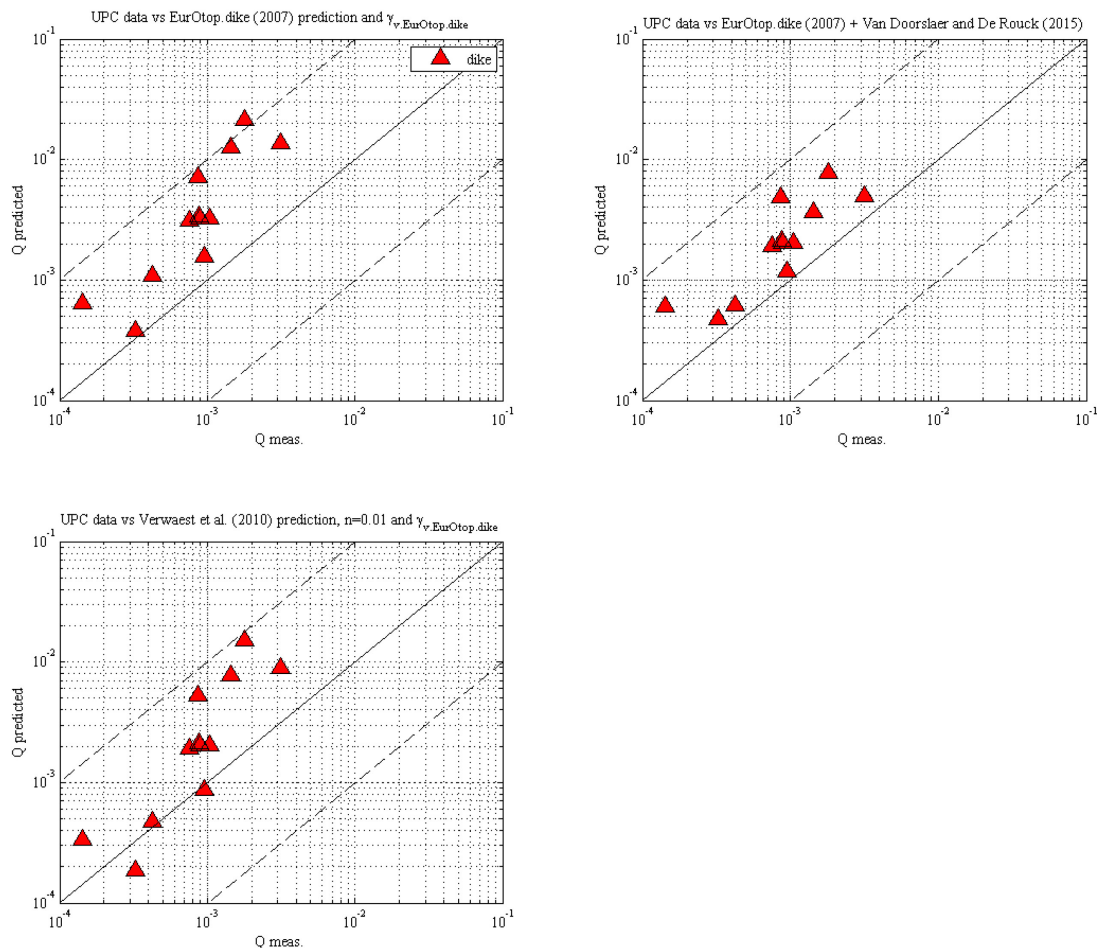


The physical model results for the sloping dike have been used to judge the performance of the approaches proposed in [Verwaest et al. \(2010\)](#) and [Van Doorslaer et al. \(2015\)](#). [Den Heijer \(1998\)](#) cannot be used because his work is mainly focused on cases where the SWL is already above the dike crest level.

The predicted non dimensional overtopping discharge against the measured one is plotted in Figure 7 using three different approaches:

- A. Eq. 5.8 of [EurOtop \(2007\)](#) for non-breaking waves +  $\gamma_{v,EurOtop,dike}=0.65$  (see section 5.3.5 of EurOtop).
- B. Eq. 5.8 of EurOtop (2007) for non-breaking waves +  $\gamma_{prom\_v}$  as proposed by [Van Doorslaer et al. \(2015\)](#)
- C. Eq. 5.8 of [EurOtop \(2007\)](#) for non-breaking waves +  $\alpha$  reduction as proposed by [Verwaest et al. \(2010\)](#) +  $\gamma_{v,EurOtop,dike}=0.65$  (see section 5.3.5 of EurOtop)

Figure 7 - Comparison of UPC test results with formulas predictions (dike layout)



A Manning coefficient,  $n$ , of 0.010 for wood (material as used in the physical model tests) is used in the third approach. It can be noticed that meanwhile approach (A) gives clear overestimation of the overtopping discharges, both approaches (B) and (C) lead to better predictions (below or close to 1 order of magnitude), even though it cannot be considered still satisfactory. Approach (C) seems to be the one that best describes the effects of the examined complex geometry, however it contains some uncertainties:

- The choice of the Manning coefficient  $n$  may be sometimes quite subjective and not really representative of the particular situation and the reduction is demonstrated really sensitive to the chosen value for  $n$ .

- To take into account the presence of the storm return wall, the reduction coefficient from [EurOtop \(2007\)](#) even though the application of such reduction is quite limited and might be not suitable for this specific case (this represents a drawback of the EurOtop methodology for cases with storm walls).

On the other hand, [Van Doorslaer et al. \(2015\)](#) do not consider any roughness of the crest whereas this should have a significant effect on the propagation of the overtopping flows.

At this stage, it is evident that the only formulations proposed in the [EurOtop \(2007\)](#) cannot be applied to the cases of complex dikes with wide berms and storm return walls. [Den Heijer \(1998\)](#) is discarded because, on one hand, his work is mainly focused on cases where the SWL is already above the dike crest level, and on the other hand, there is only a rough estimation of the influence of the berm length in the overtopping discharge.

Due to the aforementioned uncertainties in the input of [Verwaest et al. \(2010\)](#), considering that the roughness plays a significant role, for many cases where concrete sea dikes are studied (i.e. considered like smooth dikes) it is suggested to use the approach described in [Van Doorslaer et al. \(2015\)](#) however it must be compared with results from physical and/or numerical modelling for each specific case.

## 2.4 Vertical quay

To study the overtopping discharges over vertical quays it is recommended to refer to [EurOtop \(2007\)](#), [Goda \(2009\)](#) and [van der Meer and Bruce \(2013\)](#).

Here only one formula is considered:

- Eq. 7.3 from [EurOtop \(2007\)](#) that corresponds to [Allsop et al. \(1995\)](#).

[Van der Meer and Bruce \(2013\)](#) demonstrated that this formula is the most appropriate for relative freeboards ( $R_c/H_{m0}$ ) minor than 0.91. The choice of this formula is justified by the fact that its predictions are compared against the UPC physical model test results where the relative freeboard is always less than 0.91.

The formula is expressed as follows:

**EurOtop (2007), Eq. 7.3**

$$\frac{q}{\sqrt{gH_{m0}^3}} = 0.04 \exp\left(-2.6 \frac{R_c}{H_{m0}}\right) \quad (26)$$

where:

- $q$  is the overtopping discharge per meter of width of the structure [ $\text{m}^3/\text{s}/\text{m}$ ]
- $H_{m0}$  is the significant incident wave height, measured at the toe of the structure [m]
- $R_c$  is the crest freeboard [m]

Eq. (26) corresponds to the formula proposed in [EurOtop \(2007\)](#) for only non-impulsive conditions.

[Goda \(2009\)](#) argued that the set of formulations contained in the [EurOtop \(2007\)](#) manual presumes that different regimes of non-breaking, impulsive breaking and broken waves may produce differences in the overtopping. Although the wave loadings on vertical walls by individual waves are certainly affected by the wave regime, there are doubts that the overtopping is affected in the same way. He stated that in fact, the overtopping discharge is a mean value where many non-breaking, breaking and broken waves can contribute in the same wave train. Hence, [Goda \(2009\)](#) assumed only the structure of eq.(26) for both non-

impulsive and impulsive wave conditions. The use of Goda's formula instead of EurOtop ones is left to expert judgment case per case. Only eq.(26) will be used in the following discussions.

Eq. (26) doesn't include the effects of a storm return wall on top of the vertical quay. When the SWL is below the crest of the quay or equal to its elevation, no formulation is available in literature. In cases when the quay is submerged (SWL above the quay but still below the elevation of the storm return wall), the correction factors proposed by [Den Heijer \(1988\)](#) can be used for a preliminary estimation of the overtopping discharge reduction.

In particular [Den Heijer \(1988\)](#) proposed a reduction coefficient  $\gamma_k$  to assess the effects of the presence of a vertical storm return wall on top a vertical quay and when the quay is completely submerged ( $h_b > 0$ , with  $h_b$  the water depth on top of the quay) the reduction factor is function of  $h_b$  and  $H_{m0}$  as follows:

$$\begin{aligned}\gamma_k &= 0.6 && \text{for } h_b/H_{m0} \leq 0.5 \text{ and } h_b/H_{m0} > 3.0 \\ \gamma_k &= 0.6 + \frac{0.4}{1.5} \left( \frac{h_b}{H_{m0}} + 3 \right) && \text{for } 1.5 < h_b/H_{m0} \leq 3.0 \\ \gamma_k &= 0.6 - \frac{0.6}{1.5} \left( \frac{h_b}{H_{m0}} \right) && \text{for } 0.5 < h_b/H_{m0} \leq 1.5\end{aligned}\quad (27)$$

The proposed coefficient has been derived from experimental tests, therefore its applicability should be limited to similar conditions:

- relative freeboard between 1.5 and 3.0
- relative water depth on the dike  $h_b/H_{m0}$  between 0.5 and 2.0 (quay always under water)
- wave steepness between 1% and 4%
- relative crest berm length ( $B/L_0$ ) between 0.25 and 0.50

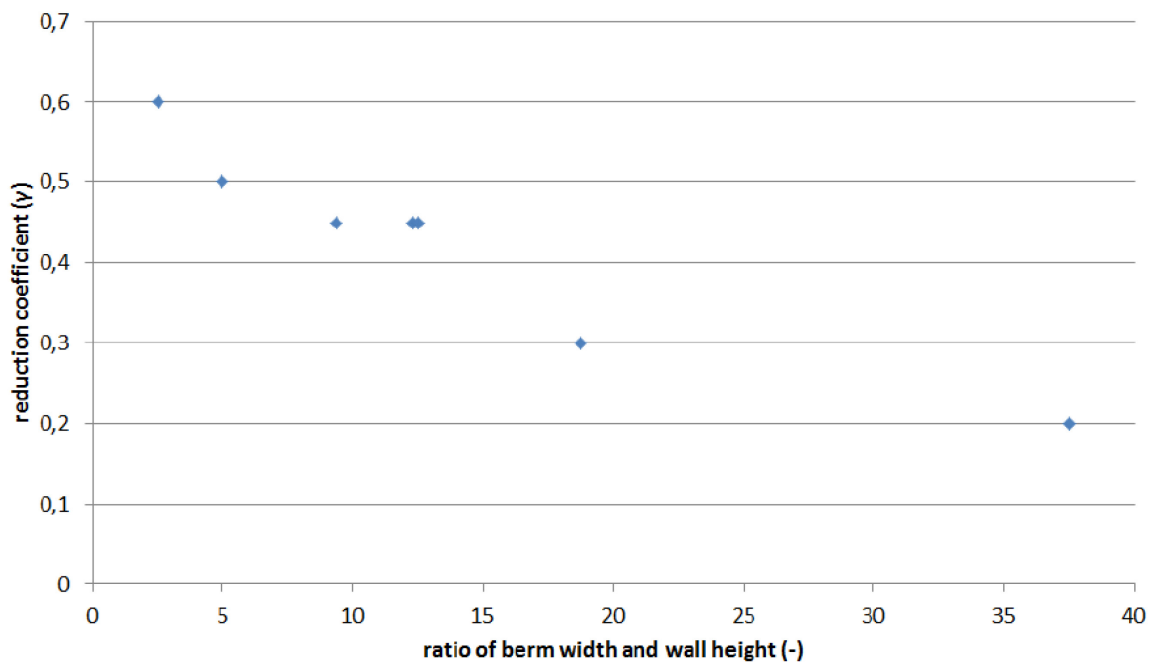
If a parapet wall is installed on the top of the quay, a new reduction coefficient is introduced. The new coefficient  $\gamma_{pk}$  is expressed as follows:

$$\begin{aligned}\gamma_{pk} &= 1.0 && \text{for } R_c/H_{m0} \leq 0.5 \\ \gamma_{pk} &= 1.3 - 0.6 \left( \frac{R_c}{H_{m0}} \right) && \text{for } 0.5 < R_c/H_{m0} \leq 1.0 \\ \gamma_{pk} &= 0.7 && \text{for } R_c/H_{m0} > 1.0\end{aligned}\quad (28)$$

Eq. (27) and eq. (28) are applicable only in cases where the top of the quay wall is below the SWL.

Further research on vertical quays with storm return walls on top is described in [Trouw et al. \(2012\)](#). The authors described the effects of different wall heights and crest widths on the mean overtopping discharge. The dependence of the reduction coefficient to be used in equation (26) in function of the ratio between crest width and wall height is plotted in Figure 8. The value of this ratio is equal to 8.45 for all the UPC tests, so a reduction coefficient of approximately 0.45 can be estimated using [Trouw et al. \(2012\)](#).

Figure 8 - Reduction coefficient in relation to the ratio of crest width and wall height



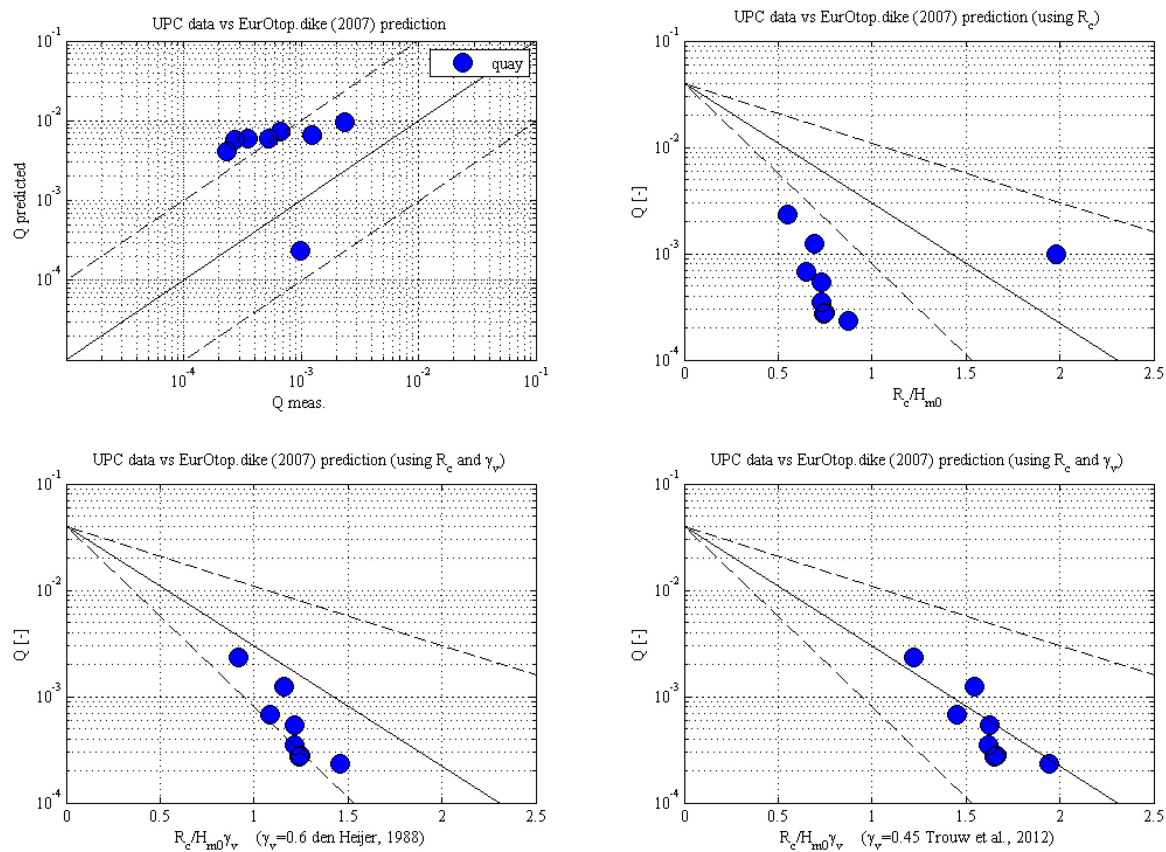
(Trouw et al., 2012)

The UPC test results for vertical quays have been compared with the prediction made using eq. (26). Figure 9 plots the predicted non-dimensional mean overtopping discharge against the measured one and analyses the influence of the crest freeboard. Indeed the overtopping is mainly dependent on the non-dimensional freeboard but eq.(26) is not enough to achieve good predictions. Although we stated that Den Heijer's approach can be used only for cases with the quay crest under water, it is proposed to assume the minimum value for  $\gamma_k$  (0.6) as reduction coefficient for the presence of the storm return wall. The results are shown in Figure 9, where it is noticeable that [Den Heijer \(1998\)](#) is overestimating the results. Finally, using a reduction coefficient of 0.45, based on [Trouw et al. \(2012\)](#), leads to the best prediction of the overtopping results.

For all the above mentioned reasons, the reduction coefficient of [Trouw et al. \(2012\)](#) together with the [EurOtop \(2007\)](#) approach can be used for preliminary design of quay walls with storm return walls on top; however physical model tests and numerical models are advisable for a final design of the structures ([Suzuki et al., 2013](#); [Vanneste and Gallach, 2013](#); [Vanneste et al., 2014](#); [Altomare et al., 2015a](#); [Altomare et al., 2015b](#)).



Figure 9 - UPC results for vertical quay with storm return wall vs formulas predictions



## 2.5 Outlines of numerical models suitable for wave overtopping assessment

Semi-empirical formulae are easy and useful models to be used for the wave overtopping assessment. However their use is limited to certain geometrical layouts and hydraulic boundary conditions. It is recommended to perform physical or numerical modelling in order to avoid wrong extrapolations and predictions. The present section provides an overview of possible numerical models to be used for overtopping assessment and wave-structure interaction analysis. The list of the proposed models is not exhaustive but it is based on the recent works carried out for the Flemish coast where those numerical models were used ([Suzuki et al., 2013b](#); [Altomare et al., 2015a](#); [Altomare et al., 2015b](#)).

To simulate wave overtopping in a realistic way, the numerical models must simulate properly the following wave processes and features: wave propagation, shoaling, breaking, effects of structural permeability, impulsive action of sea waves on the structures for heavy breaking processes and violent impacts, 3D effects. In general a numerical model capable to satisfy all these requirements does not exist. Each numerical model has to be used for a specific purpose accounting for its proper limitations and capabilities. Three different numerical models are introduced in the present section:

- SWASH (Eulerian, NLSWE method)
- Flow-3D (Eulerian, Navier-Stokes based method)
- DualSPHysics (Lagrangian, Navier-Stokes based method)

A technique that aims to couple SWASH and DualSPHysics is also briefly described.



### 2.5.1 SWASH

SWASH (an acronym of Simulating WAVes till SHore) is a non-hydrostatic wave-flow model and is intended to be used for predicting transformation of dispersive surface waves from offshore to the beach for studying the surf zone and swash zone dynamics, wave propagation and agitation in ports and harbours, rapidly varied shallow water flows typically found in coastal flooding resulting from e.g. dike breaks, tsunamis and flood waves, density driven flows in coastal waters, and large-scale ocean circulation, tides and storm surges ([Zijlema et al., 2011](#)). The governing equations are the nonlinear shallow water equations for one layer calculation (depth-averaged mode), and it is including non-hydrostatic pressure. Optionally the equations for conservative transport of salinity, temperature and suspended sediment can be modeled. In addition, the vertical turbulent dispersion of momentum and diffusion of salt, heat and sediment load are modelled by means of the standard k- $\epsilon$  turbulence model. This model can be employed to resolve the dynamics of wave transformation, buoyancy flow and turbulent exchange of momentum, salinity, heat and suspended sediment in shallow seas, coastal waters, estuaries, reefs, rivers and lakes. Examples are small-scale coastal applications, like waves approaching a beach, wave penetration in a harbour, flood waves in a river, oscillatory flow through canopies, salt intrusion in an estuary, internal waves, and large-scale ocean, shelf and coastal systems driven by Coriolis and meteorological forces to simulate tidal waves and storm surge floods (SWASH user manual, 2014).

SWASH is not a Boussinesq-type wave model. The vertical structure of the flow is a part of the solution. In fact, SWASH may either be run in depth-averaged mode or multi-layered mode in which the computational domain is divided into a fixed number of vertical terrain-following layers. SWASH improves its frequency dispersion by increasing this number of layers rather than increasing the order of derivatives of the dependent variables like Boussinesq-type wave models do. Good feature of SWASH is that it can deal with wave breaking in a good manner due to its shock capturing feature in NLSW equation (e.g. [Smit et al., 2013](#)) whereas a Boussinesq-type model cannot avoid to use artificial dissipation term. Even though the detailed breaking process such as overturning and turbulence are not resolved, SWASH can reliably predict surf zone wave statistics ([Smit et al., 2014](#)).

There is only a limited amount of papers regarding wave overtopping simulated by SWASH. [Suzuki et al. \(2011\)](#) have first demonstrated an overtopping calculation in SWASH with shallow foreshore conditions by one layer and the result shows a good accuracy compared to a physical model result. [Martínez Pés \(2013\)](#) applied the SWASH model and got reasonably good overtopping results for a dike configuration. On the other hand SWASH output discharges compared with [EurOtop \(2007\)](#) are smaller than the 5% lower limit. [St-Germain et al. \(2014\)](#) demonstrated the SWASH model for overtopping by regular waves in a dike configuration. The result shows a good agreement with the measured data.

### 2.5.2 Flow3D

FLOW-3D® is a 3D model based on Navier-Stokes equations with finite volume approximations on a structured grid and Volume-Of-Fluid method ([Hirt and Nichols, 1981](#)). This is an intensive model which solves pressure and velocity at each cell and at each time step, and gives wave transformation and wave overtopping with a good accuracy. The wave generation in Flow3D mimics the physical wave flumes: a wavemaker is implemented which generates the desired waves, and simultaneously absorbs the waves reflected from the tested structure. For further details of Flow3D and its application to coastal structures, see [Vanneste \(2012\)](#). There are several models having the same concepts such as CADMAS-SURF/3D ([Arikawa et al., 2004](#)) and IHFORM ([Lara et al., 2008](#)) and the performance of those models are promising. One of the known disadvantages of this type of models is the computational cost. However, recent development of computational techniques (e.g. parallel computation and efficient solvers) and computational resources (e.g. development of CPU and memory) make it possible to access those models.

Flow3D has been recently applied to cases from the Flemish coast as described by [Vanneste and Troch \(2012\)](#), [Suzuki et al. \(2013a\)](#) and [Vanneste et al. \(2014\)](#).

### 2.5.3 DualSPHysics

Smoothed Particle Hydrodynamics (SPH) is a Lagrangian meshless method that is increasingly used for an extensive range of applications within the field of Computational Fluid Dynamics (CFD). The method uses particles to represent a fluid and these particles move according to the governing dynamics. When simulating free-surface flows, the Lagrangian nature of SPH allows the domain to be multiply-connected, with no need of a special treatment of the surface, making the technique ideal for studying violent free-surface motion. SPH has been used to describe a variety of free-surface flows (wave propagation over a beach, plunging breakers, impact on structures and dam breaks).

The DualSPHysics code (Crespo et al., 2011; Gómez-Gesteira et al., 2012a, 2012b) has been developed to use SPH for real engineering problems with software that can be run on either CPUs or GPUs (graphics cards with powerful parallel computing). GPUs offer now a higher computing power than CPUs and they are an affordable option to accelerate SPH with a low economic cost. DualSPHysics is open source and can be freely downloaded from [www.dual.sphysics.org](http://www.dual.sphysics.org). The first rigorous validation of the GPU implementation of DualSPHysics code was presented in Crespo et al. (2011).

The DualSPHysics code has been developed to simulate real-life engineering problems using SPH models such as the computation of forces exerted by large waves on the urban furniture of a realistic promenade (Barreiro et al., 2013) or the study of the run-up in an existing armour block sea breakwater (Altomare et al., 2014b). Other recent examples of the study of wave-structure interaction, by means of the DualSPHysics model, are the works of Ren et al. (2014), where the SPH model is validated against other available numerical results and against experimental data for wave damping over porous seabed with different levels of permeability. Another recent example is the work of St-Germain et al. (2014) to investigate the hydrodynamic forces induced by the impact of a rapidly advancing tsunami like hydraulic bores. Finally the recent work of Altomare et al. (2015c) proved the reliability of an SPH approach for studying wave impact phenomena of typical cases from the Flemish coast.

### 2.5.4 Coupling technique (SWASH+DualSPHysics)

A coupling technique has been implemented between the aforementioned SWASH and DualSPHysics models. The scope of this kind of strategy is to join the strengths and to overcome the limitations of both models, such as:

- Fast computations with large domains can be performed with SWASH, while simulating large domains with SPH implies huge computation times even when using hardware acceleration.
- SWASH is suitable for calculations where statistical analysis is necessary such as computing wave heights, and good accuracy is obtained for wave propagation (results comparable with Mike21 BW), whereas wave decay is observed with standard SPH formulation for large domains.
- SWASH is not suitable for calculation of wave impact, while SPH can easily compute wave impact, pressure load and exerted force onto coastal structures.
- Detailed bathymetries and complex geometries cannot be represented with SWASH and computation stability problems may appear when applied to rapidly changing bathymetry. Using SPH, any complex geometry or varying bathymetry can be simulated.

The coupling has been implemented and validated for regular and irregular waves running over a sandy beach. A complete description of the technique can be found in Altomare et al. (2014c). The results have also been presented in international conferences (Dominguez et al, 2014; Altomare et al., 2014d, 2014e). Further development and application to cases for the Flemish coast are planned to be executed by mid 2015.

## 2.6 Artificial neural network for wave overtopping assessment

Artificial neural networks (ANNs) are computational models inspired by human brain, which is capable of machine learning as well as pattern recognition. Artificial neural networks are generally presented as systems of interconnected "neurons" which can compute values from inputs. ANNs are simple to apply as predictive methods for structures characterized by complex geometries that are not covered by existing formulations. In the last decade the ANN concept has been applied successfully in coastal engineering. For wave overtopping, the works of [Van Gent et al. \(2008\)](#) and [Verhaeghe et al. \(2008\)](#) are worthy to be mentioned. Recently [Zanuttigh et al. \(2014\)](#) presented a novel approach that aims to extend the dataset on which the ANN for overtopping is trained; at the same time the ANN proposed by Zanuttigh et al. (2014) is capable to provide response not only in terms of wave overtopping but also wave reflection and transmission.

The use of an ANN is a possible option to quantify the mean overtopping discharge over a complex structure, however some recommendations have to be considered:

- Once the ANN is trained with a certain amount of data, it results a sort of “black box” tool for an external user, hence the schematization of the structures by means of the ANN input variables has to be carefully conducted;
- A neural network is just an approximation of the exact solution and errors are inevitable.
- An ANN has a certain proneness of overfitting (problem quite common to all the very complex models).

Therefore the use of one of the aforementioned ANNs is advisable only for a preliminary estimation of the overtopping discharge, whereas physical or numerical modelling is preferred for a final design of a coastal structure.

## 3 Oblique wave overtopping

### 3.1 Introduction

The effects of wave obliqueness on wave overtopping are analysed, both for sea dikes and vertical walls. The attention is mainly focused on very oblique wave attack (wave angle  $\beta \geq 60^\circ$ ). The most used formulae are collected and reported: in general, each formula is derived from a semi-empirical approach, where its calibration is based on physical model measurements. However most of the studies from literature do not investigate very oblique wave overtopping and are limited to angles between  $0^\circ$  and  $50^\circ$ .

Recently, physical model experiments have been carried out at Flanders Hydraulics Research (FHR), Danish Hydraulic Institute (DHI) and at the Coastal Research Center (FSK) of NLWKN, in Hannover. These three different experimental campaigns are focused on very oblique attacks on coastal structures. The data from the FHR test cases have been analysed and the results are reported here. The results from DHI and FSK are respectively contained in [Bornschein et al. \(2014\)](#) and [Kerpen and Schlurmann \(2012\)](#) and will be mentioned later on.

### 3.2 FHR model tests

To investigate the influence of the wave angle on the overtopping discharge a physical model was designed and built at Flanders Hydraulics Research (FHR).

The experiments were carried out in the wave basin at FHR. The wave basin (dimensions 17.50 m x 12.20 m x 0.45 m) is equipped with a piston - type wave generator. The wave generator has a width of 12 m and generates only long-crested waves. Both regular and random waves can be generated. Since the wave generator is displaceable, there are different angles of potential wave incidence (between  $-22.5^\circ$  and  $+22.5^\circ$  with respect to the wave normal incidence).

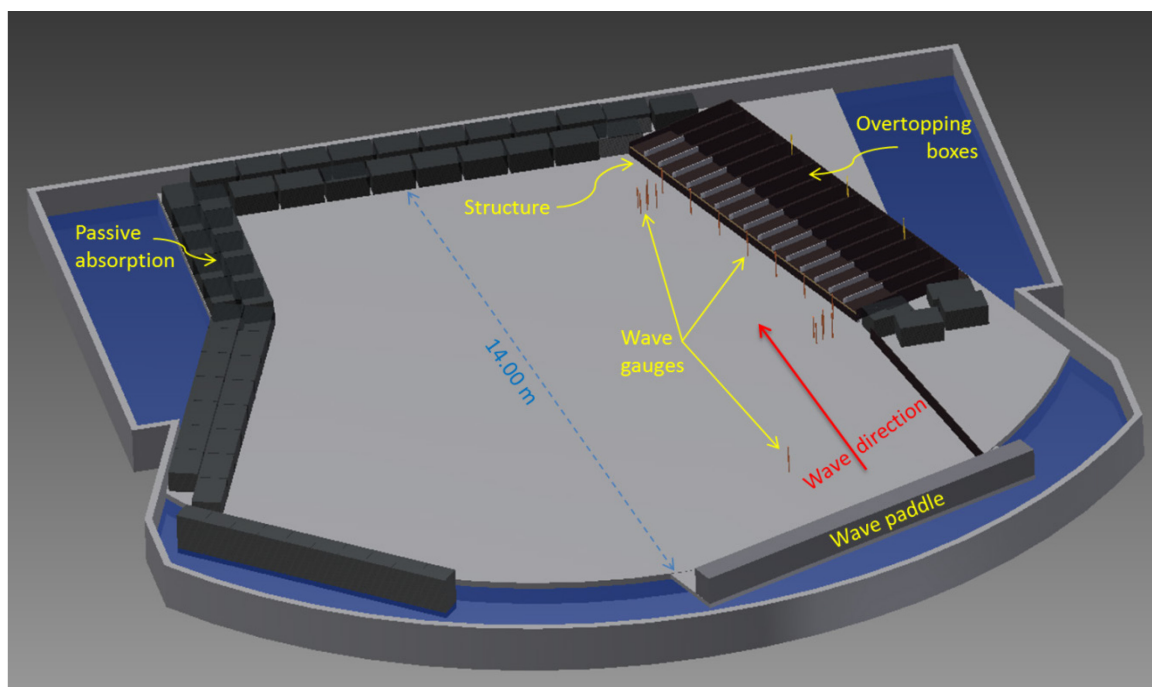
A structure of 8 m long and 1 m wide was built. Attached to this structure, at its backside, there are 16 boxes to collect the overtopping water during the experiment. Each box is 1.5 m long, 0.48 m wide and 0.18 m deep. Every overtopping box has a mechanism to read the water level using a needle touching the water surface and a metallic ruler. The water level is determined before and after the experiment, the difference is the overtopped volume. Additionally, three of the boxes have a second instrument (so-called Balluff) which records automatically and in real time the water level. A number of 17 resistance wave gauges were used to obtain the wave characteristics (height, period and direction). One wave gauge was permanently situated in front of the wave maker to check the generated waves. Two wave gauges arrays have been built, each existing of 5 wave gauges: the wave gauges were located in such way to use the arrays to perform a directional spectral analysis. The incident wave height has been measured using these two 3D-arrays of five wave gauges. The WaveLab software package, which utilizes the BDM method by [Hashimoto et al. \(1987\)](#), has been used for the analysis. Using the BDM method, the user generally indicates a circular sector around the expected incident and reflected wave direction: the analysis will then be limited to this sector. It is possible to choose a very narrow circular sector, so excluding from the analysis wave directions far from the main one. On the other hand, it's also possible to choose  $\pm 90^\circ$  around the main direction, so that finally the entire  $360^\circ$  will be covered in the analysis. In this work, the analysis for the case with perpendicular wave attack has used  $\pm 30^\circ$  around the main expected directions ( $0^\circ$  for the incident and  $180^\circ$  for the reflected waves respectively). Hence spurious transversal effects are removed from the results. Differently, for the oblique wave cases, it has been preferred to extend the analysis to the entire  $360^\circ$ , because in such case the main reflected direction can be presumed but the effects on wave re-reflection on the sides of the basin (even though passive absorption was installed) has to be checked.

The last 6 wave gauges were mounted equidistantly, just close to the structure to provide information about the variation of the total wave height along the structure. Wave analysis is done with the software WaveLab 3.39 (Aalborg University, 2012) and advice regarding the distribution of the wave gauges in the basin was received directly from Thomas Lykke Andersen, the leader of the team which developed WaveLab software.

Every overtopping box has a mechanism to read the water level using a needle touching the water surface and a metallic ruler. The water level is determined before and after the experiment, the difference is the overtopped volume. Additionally, three of the boxes have other instruments which record automatically and continuously the water level, so-called Balluff.

A sketch of the wave basin and instrumentations is depicted in Figure 10.

Figure 10 - Sketch of the wave basin layout for the 80° test case



A comprehensive test program was elaborated in order to investigate the reduction of overtopping related to the wave angle increase. A total number of 377 tests were realized out of which 230 were used to analyze the overtopping reduction. The remaining of 147 tests were: 1) tests used to properly set up the model and the wave conditions into the basin; and 2) tests which were not usable due to the failure of waves and/or overtopping measurements.

In order to quantify the reduction of overtopping four wave angles were used for both configurations, the vertical and the sloping dike: 0° - 34 tests, 45° - 54 tests, 60° - 45 tests, 70° - 26 tests and 80° - 71 tests. The wave angle is defined as the angle between the wave direction and the line normal to the quay structure so that 0° defines perpendicular wave attack. For all the wave angles a configuration with vertical wall was used and for the wave directions 45°, 60° and 80° a slope was also used during the tests.

In all the tests only long-crested waves were generated.

The water level has been varied around the crest level: the so defined “dike freeboard” ( $A_c$ ) assumes from negative (i.e. Still Water Level, SWL, above the dike crest) to positive values (SWL below the dike crest).

Two different wall heights have been used (1 m and 2 m in prototype scale). Note that tests without storm wall (i.e. wall height = 0 m) were also performed and compared. The wall elevation with respect to the SWL

defines the crest freeboard  $R_c$ . The wall distance from the edge of the dike or quay crest has been varied between 0 m and 100 m (in prototype). The influence of the berm length has been investigated.

The wave conditions can be summarized as follows (in prototype scale):

- significant wave height  $H_{m0}$ : 0.96÷3.39 m;
- peak period  $T_p$ : 5.1÷12.6 s;
- crest freeboard  $R_c$  between 0 and 2.75 m.

A number of 191 tests for the vertical quay and a number of 39 tests for sloping dikes have been used in the analysis. Tests with no reliable measured wave conditions and zero overtopping have been excluded. Sometimes the measured overtopping has been larger than the overtopping box volumes: these tests are also not taken into account.

The results have been compared with existing and most used formulations. The analysis of the physical model test results helped to judge whether or not existing formulas can describe the reduction of overtopping discharge for wave angles between 60° and 80°. If not, new reduction factors are proposed.

To analyse the reduction due to the obliqueness, only tests without crest berm or with very short crest berm (5 m in prototype) have been considered. The influence of long crest berms has been analysed afterwards.

Physical model test results included in the CLASH database (Verhaeghe et al., 2004) have been used for comparison. Only tests with similarities to the present experimental campaign have been extracted from the database. In detail:

- SLOPING DIKE: only CLASH data with slope between 1:4 and 1:2 with gentle or no foreshore have been considered;
- VERTICAL QUAY: only tests with gentle or without foreshore have been considered.

The results for both layouts are discussed in the following sections. All the results from tests carried out at Flanders Hydraulics Research are labelled as “FHR”.

### 3.3 Sloping dike

A 1:2.5 sloping dike has been tested during the experimental campaign. A horizontal bottom has been used in the wave basin (no foreshore in front of the dike). Hence only CLASH data with the same dike slope have been used for comparison. The selected cases have gentle or no foreshore slope.

Only three wave angles have been tested: 45°, 60° and 80°.

Two formulae from literature have been considered:

1. Eq. 5.8 from [EurOtop \(2007\)](#) for non-breaking waves
2. [Van der Meer and Bruce \(2013\)](#) formula that represent a modification of the EurOtop formula.

The formulae are expressed as follows:

**EurOtop (2007), Eq. 5.8**

$$\frac{q}{\sqrt{gH_{m0}^3}} = 0.2 \exp\left(-2.6 \frac{R_c}{H_{m0} \gamma_f \gamma_\beta}\right) \quad (29)$$

where:

- $q$  is the overtopping discharge per meter of width of the structure [ $\text{m}^3/\text{s}/\text{m}$ ]
- $H_{m0}$  is the significant incident wave height, measured at the toe of the structure [m]



- $R_c$  is the crest freeboard [m]
- $\gamma_f$  is the reduction coefficient that considers the effects of the slope roughness [-]
- $\gamma_\beta$  is the reduction coefficient that considers the effects of the obliqueness [-]

The coefficient  $\gamma_\beta$  is expressed in EurOtop (eq. 5.24) as:

$$\gamma_\beta = 1 - 0.0033|\beta| \quad \text{for: } 0^\circ \leq \beta \leq 80^\circ \quad (30)$$

For wave angles larger than  $80^\circ$  a constant value of 0.736 is proposed. It has been noticed that the eq. (30) is based on the analysis of short-crested wave data, meanwhile the tests conducted at Flanders Hydraulics Research are only using long-crested waves.

#### Van der Meer and Bruce (2013)

$$\frac{q}{\sqrt{gH_{m0}^3}} = 0.09 \exp\left(-1.5 \frac{R_c}{H_{m0}\gamma_f\gamma_\beta}\right)^{1.3} \quad (31)$$

In all cases  $\gamma_f$  has been assumed equal to 1 (smooth slope). Following [Van Doorslaer et al. \(2015\)](#), a reduction factor  $\gamma_{prom\_v}$  has been considered to take into account the presence of a storm return wall on top of the dike. This coefficient considers both the effect of the wall height and the position. The values of  $\gamma_{prom\_v}$  are calculated for each case based on the approach described in [Van Doorslaer et al. \(2015\)](#). Hence the final formulae include both  $\gamma_{prom\_v}$  and  $\gamma_\beta$ .

The selected CLASH data are plotted in Figure 11. The predictions based on [EurOtop \(2007\)](#) and [Van der Meer and Bruce \(2013\)](#) are included. The CLASH data have been divided between wave attacks with a wave angle larger than  $45^\circ$  (magenta dots in Figure 11) and with a wave angle smaller than or equal to  $45^\circ$  (yellow dots). A certain scatter can be observed. The data are plotted against both the non-dimensional freeboard  $R_c/H_i$  and the equivalent non-dimensional freeboard  $R_c/H_i\gamma_\beta$  where the reduction coefficient has been preliminarily calculated using eq. (30).  $H_i$  stands for the incident significant wave height  $H_{m0}$ .

The measured overtopping discharges for FHR data are plotted in Figure 12 against the values predicted using eq. (29) and eq. (30). Again, the formula seems to overestimate the overtopping discharge for very oblique wave attacks.

Figure 11 - CLASH data vs EurOtop (2007) and van der Meer and Bruce (2014) predictions.

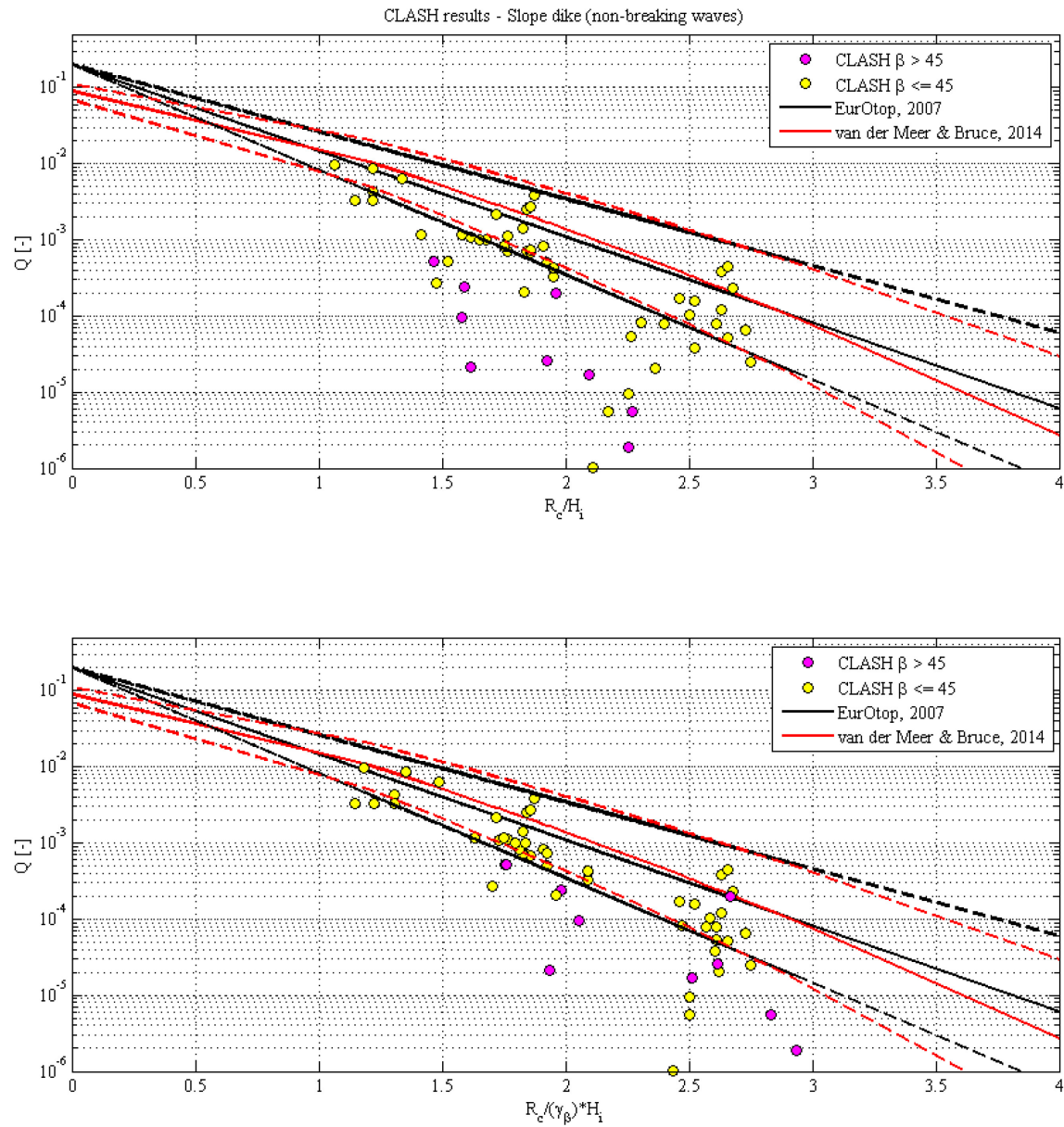
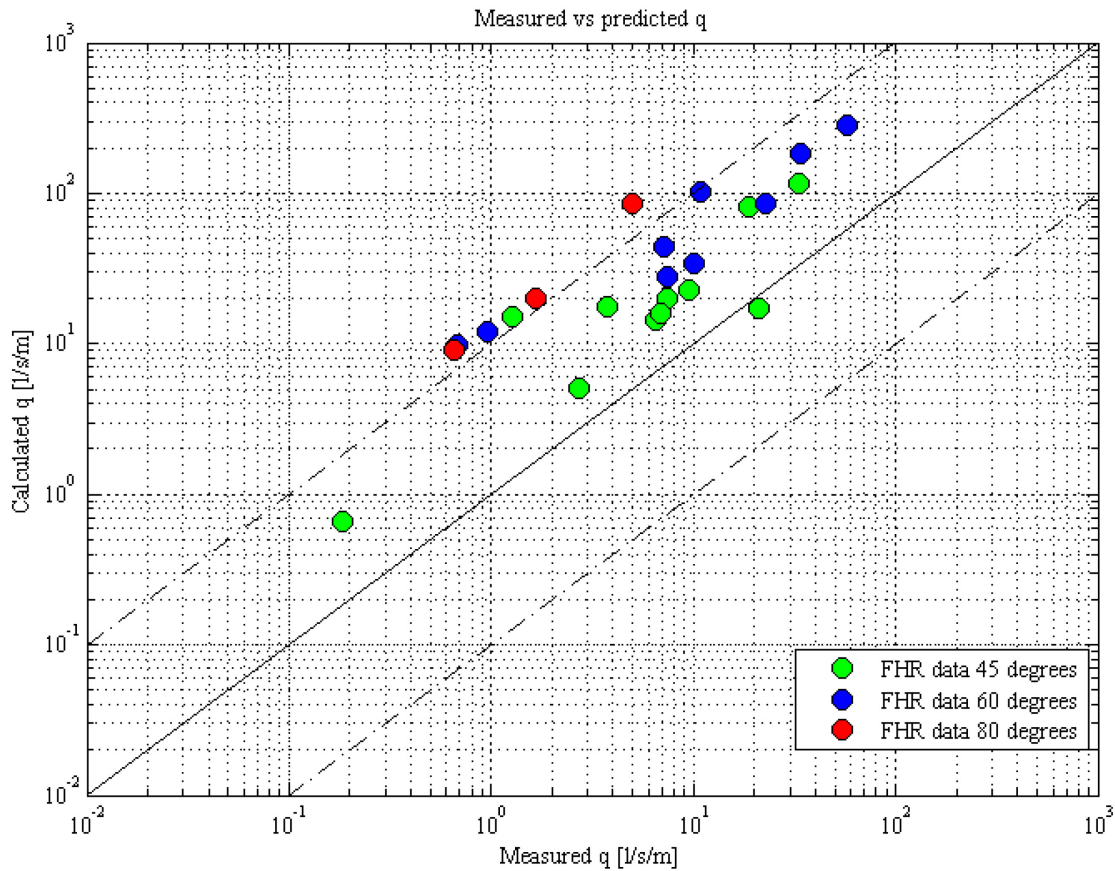




Figure 12. Dike: predicted (EurOtop, 2007) vs measured overtopping discharges



The effect of the obliqueness on the overtopping discharge has been evaluated calculating the reduction coefficient of each case starting from eq. (29) as follows:

$$\gamma_{\beta} = -2.6 \frac{R_c}{H_{m0} \gamma_{prom_v}} \frac{1}{\ln\left(\frac{q}{0.2 \sqrt{g H_{m0}^3}}\right)} \quad (32)$$

The calculation has been performed both for the FHR data and for the selected CLASH data. Three different dataset have been selected from CLASH (for only non-breaking wave conditions):

- Dataset 030 (Owen, 1980): 1:2 slope with 1:20 foreshore
- Dataset 220 (van der Meer and De Waal, 1990): 1:2.5 slope with 1:1000 foreshore
- dataset 222 (van der Meer and De Waal, 1990): it includes data for 1:2.5 and 1:4 slope with 1:1000 foreshore.

Figure 13 shows the variation of the reduction coefficient with the wave angle. The CLASH data are labelled in Figure 13 as red triangles whose size is proportional to the slope (e.g. 1:2 bigger size than 1:4). Several proposed formulations have been analysed to calculate the reduction coefficient as function of the wave angle. The formulas predictions are plotted in Figure 13. The confidence interval for the FHR data is also plotted.

Table 9 summarizes the analysed approaches. The review is based on [Bornschein et al. \(2014\)](#), [EurOtop \(2007\)](#) and [Goda \(2009\)](#). Analysing the prediction results of each formula, it can be concluded that:

- All the formulations, except Goda (2009) and Kortenhaus et al. (2006), are quite similar up to 30°, but they diverge significantly for larger wave angles;

- Goda (2009) and Kortenhaus et al. (2006) seems to match with the CLASH data for angles smaller than 30°;
- EurOtop (2007) represents an upper limit of the formulas for large wave angles. Generally, the formula seems to overestimate the reduction coefficient that means to underestimate the reduction of wave overtopping discharge. It is reasonable since the tests conducted at Flanders Hydraulics Research used long-crested waves, whereas the EurOtop formula is based mainly on short-crested waves (for sea dikes the reduction for short-crested waves is smoothed by the wave energy directional spreading);
- For long-crested waves, Kortenhaus et al. (2006) prediction matches the FHR data. In particular, calculating the average value of  $\gamma_\beta$  for 45°, 60° and 80° based on FHR results we obtained respectively 0.72, 0.54, 0.44 whereas the values calculated with Kortenhaus et al. (2006) are 0.66, 0.54 and 0.39;
- FlowDike (2012) for 1:3 slope presents a trend that also may describe the FHR results ( $\gamma_\beta$  for 45°, 60° and 80° equal respectively to 0.70, 0.54, 0.41);
- de Waal & van der Meer (1992) overestimates the results for wave angles larger than 50°, assuming a constant value for the reduction coefficient;
- Goda (2009) represents a sort of upper limit of the FHR and CLASH data for angles larger than 50° ( $\gamma_\beta$  for 60° and 80° equal respectively to 0.62 and 0.58), therefore it can be proposed as safe approach. The overestimation is smaller for the wave overtopping reduction for very oblique waves in the case of Goda formula than the previous listed formulas.

Table 9 - Dike: existing formulae for  $\gamma_\beta$  (literature review)

| Formula                                                                                                                                                                                | Reference                                                | Wave conditions                        |
|----------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------|----------------------------------------------------------|----------------------------------------|
| $\gamma_\beta=1.0$ $0^\circ \leq \beta < 10^\circ$<br>$\gamma_\beta=\cos^2(\beta-10^\circ)$ $10^\circ \leq \beta \leq 50^\circ$<br>$\gamma_\beta=0.6$ $50^\circ < \beta \leq 80^\circ$ | de Waal & Van der Meer (1992),<br>Nørgaard et al. (2013) | Long-crested                           |
| $\gamma_\beta=0.10+0.90\cos\beta$ (1:3 slope dike)<br>$\gamma_\beta=0.35+0.65\cos\beta$ (1:6 slope dike)                                                                               | Oumeraci et al. (2002)                                   | Short and long-crested                 |
| $\gamma_\beta=1.0-0.0076\beta$                                                                                                                                                         | Kortenhaus et al. (2006)                                 | Long-crested                           |
| $\gamma_\beta=0.39+0.61\cos^2\beta$ (1:3 slope dike)<br>$\gamma_\beta=0.51+0.49\cos^2\beta$ (1:6 slope dike)                                                                           | FlowDike (2012)                                          | Long-crested                           |
| $\gamma_\beta=1.0-0.0033 \beta $                                                                                                                                                       | EurOtop (2007)                                           | Short-crested                          |
| $\gamma_\beta=1.0-0.0096\beta+0.000054\beta^2$                                                                                                                                         | Goda (2009)                                              | Short and long-crested<br>(CLASH data) |
| $\gamma_\beta=0.95-0.0035\beta$                                                                                                                                                        | Nørgaard et al. (2013)                                   | Short-crested                          |

Figure 13 - Dike: variation of reduction coefficient with wave angle; comparison with existing formulae

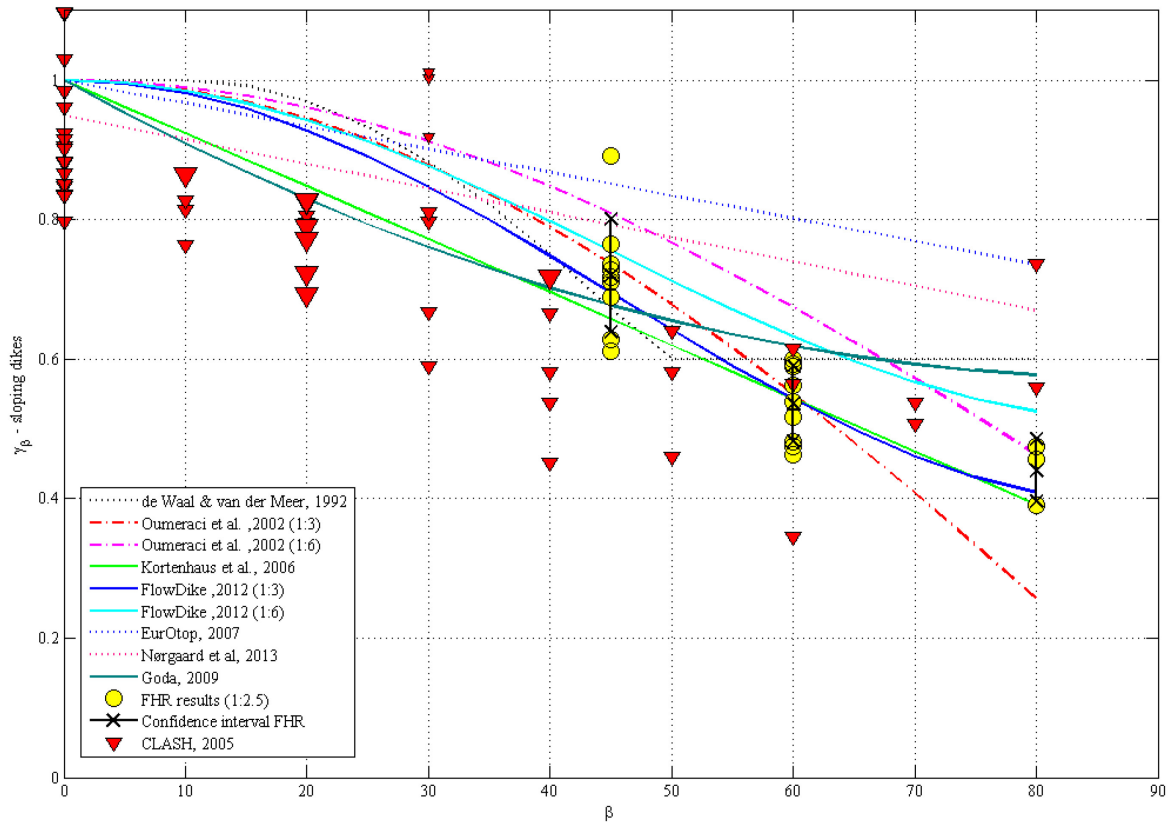
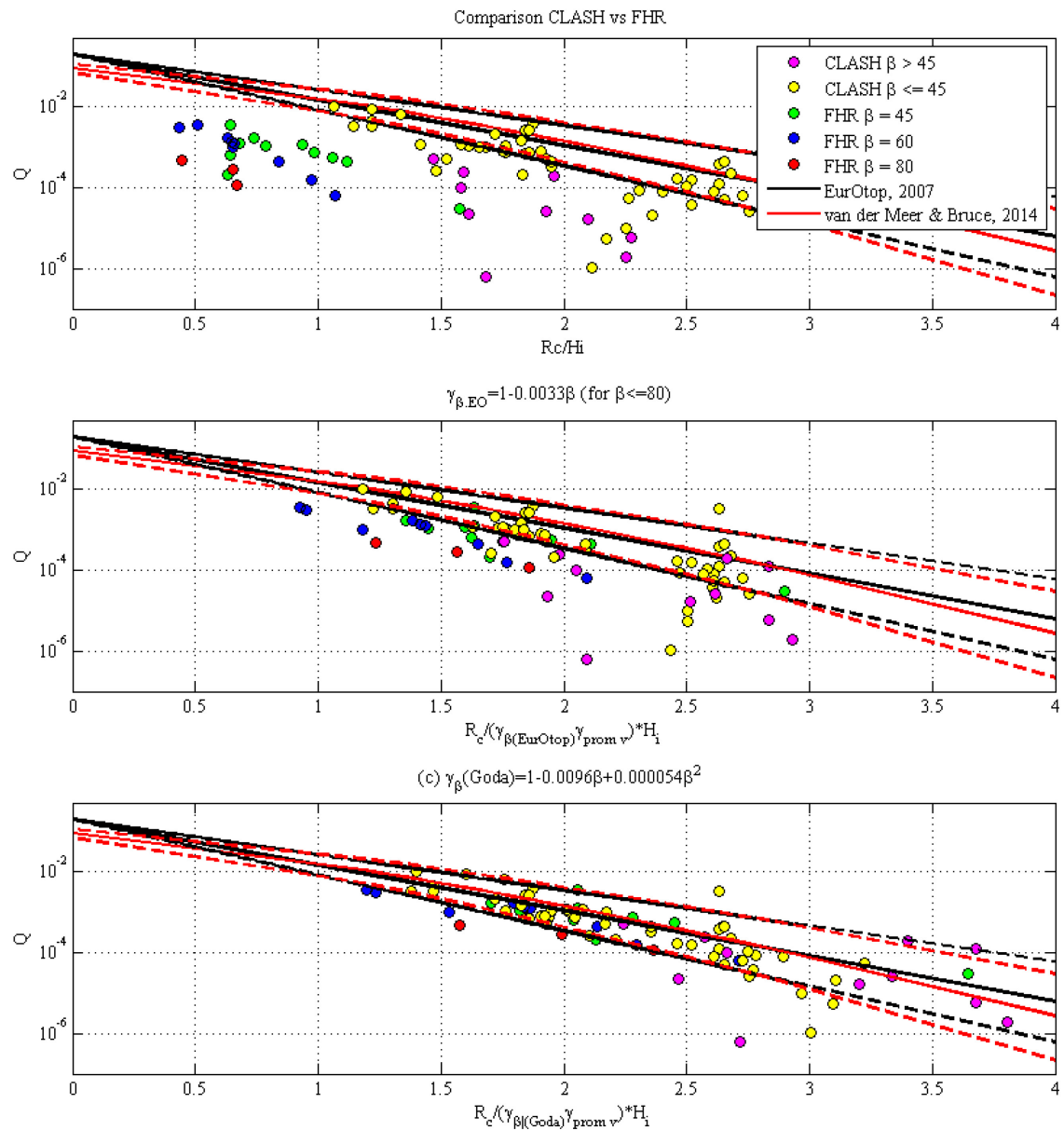


Figure 14 shows the FHR data, the CLASH data and the EurOtop predictions. Three different plots are depicted:

- the values of  $Q$  are plotted against the non-dimensional freeboard  $R_c/H_i$ ;
- the values of  $Q$  are plotted against the non-dimensional freeboard  $R_c/H_i \gamma_{\beta(EurOtop)} \gamma_{prom\_v}$ , where  $\gamma_{\beta(EurOtop)}$  is the correction coefficient calculated using the EurOtop (2007) formula and  $\gamma_{prom\_v}$  is the reduction coefficient calculated by means of Van Doorslaer et al. (2015);
- the values of  $Q$  are plotted against the non-dimensional freeboard  $R_c/H_i \gamma_{\beta(Goda)} \gamma_{prom\_v}$ , where  $\gamma_{\beta(Goda)}$  is the correction coefficient calculated using the Goda (2009) formula.

It can be noticed that the use of Goda (2009) formula is improving the wave overtopping prediction in case of oblique wave attack with respect to the EurOtop (2007) formula.

Figure 14 - FHR and CLASH data vs formula predictions



### 3.4 Vertical quay

A vertical quay wall without foreshore has been tested during the experimental campaign. Only CLASH data with the very gentle foreshore have been used for comparison. Most of the tests consist of non impulsive conditions.

Five wave angles (i.e. obliqueness) have been tested: 0°, 45°, 60°, 70° and 80°. The angle is calculated between the propagation direction and the normal to the edge of the structure. Only one formula from literature has been considered:

- Eq. 7.3 from [EurOtop \(2007\)](#)

The formula is expressed as follows:

**EurOtop (2007), Eq. 7.3**

$$\frac{q}{\sqrt{gH_{m0}^3}} = 0.04 \exp\left(-2.6 \frac{R_c}{H_{m0}\gamma_\beta}\right) \quad (33)$$

where:

- $q$  is the overtopping discharge per meter of width of the structure [ $\text{m}^3/\text{s}/\text{m}$ ]
- $H_{m0}$  is the significant incident wave height, measured at the toe of the structure [m]
- $R_c$  is the crest freeboard [m]
- $\gamma_\beta$  is the reduction coefficient that considers the effect of obliqueness [-]

Eq. (33) corresponds to the formula proposed in EurOtop (2007) for only non-impulsive conditions. As already argued by [Goda \(2009\)](#), the set of formulations in the EurOtop manual presumes that different regimes of non-breaking, impulsive breaking and broken waves may produce differences in the overtopping. Although the wave loadings on vertical walls by individual waves are certainly affected by the wave regime, there are doubts that the overtopping is affected in the same way. In fact, the overtopping discharge is a mean value where many non-breaking, breaking and broken waves can contribute in the same wave train. Therefore, as proposed by [Goda \(2009\)](#), only the structure of eq. (33) is assumed for both non-impulsive and impulsive wave conditions.

The coefficient  $\gamma_\beta$  is expressed in EurOtop (eq. 7.16) as:

$$\begin{aligned} \gamma_\beta &= 1 - 0.0062|\beta| & \text{for: } 0^\circ \leq \beta \leq 45^\circ \\ \gamma_\beta &= 0.72 & \text{for: } \beta > 45^\circ \end{aligned} \quad (34)$$

The selected CLASH data are plotted in Figure 15. The predictions made based on EurOtop (2007) are included. The CLASH data have been divided between wave attacks with wave angle smaller than 45° (magenta triangles in Figure 15) and with wave angle equal to 45° (orange triangles in Figure 15). No data with angles larger than 45° have been found in CLASH for similar layouts. The non-dimensional wave overtopping results ( $Q=q/(gH_{m0}^3)^{0.5}$ ) are plotted against both the non-dimensional freeboard  $R_c/H_i$  and the equivalent non-dimensional freeboard  $R_c/H_i\gamma_\beta$  where the reduction coefficient has been preliminarily calculated using eq. (34).  $H_i$  stands for the incident significant wave height  $H_{m0}$ .

Figure 15 - CLASH data vs EurOtop (2007)

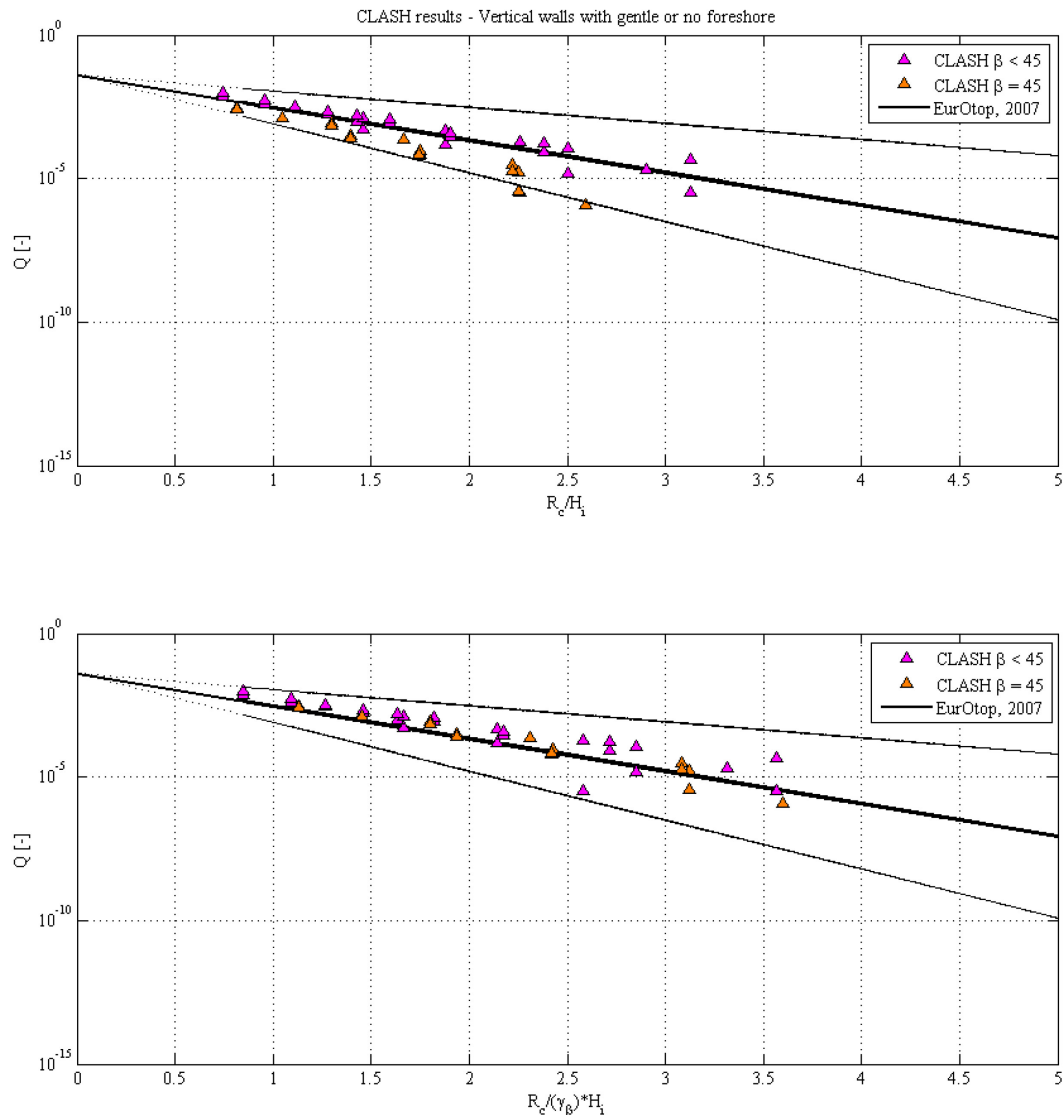


Figure 16 shows the results of the FHR tests in a graph with the measured discharges plotted against the predicted ones, expressed in l/s/m (prototype scale). The plotted data include cases without a crest berm (distance of the wall from the edge of the quay,  $d_w$ , equal to 0 m) and with a crest berm ( $d_w$  larger than 0 m). The dash-dot lines indicate a prediction of 10 times larger and smaller with respect to the central line (ratio predicted/measured equal to 1:1). It can be noticed that the formula overestimates the overtopping discharge for the 70° and 80°, meanwhile the 0°, 45° and 60° results are in acceptable agreement or within the above mentioned range.

The effect of the obliqueness on the overtopping discharge has been evaluated calculating the reduction coefficient of each case starting from equation (33) as follows:

$$\gamma_\beta = -2.6 \frac{R_c}{H_{m0}} \frac{1}{\ln\left(\frac{q}{0.04 \sqrt{g H_{m0}^3}}\right)} \quad (35)$$

The calculation has been performed both for FHR data and for the selected CLASH data. Figure 17 shows the variation of the reduction coefficient with the wave angle. The proposed formulations from literature have been analysed to calculate the reduction coefficient as function of the wave angle. The formulae predictions are also plotted in Figure 17.

The review is based on [Franco and Franco \(1999\)](#), [EurOtop \(2007\)](#) and [Goda \(2009\)](#). The formulations are summarized in Table 11. [Kerpen and Schlurmann \(2012\)](#) carried out tests for wave overtopping over vertical walls and sea dikes with vertical walls, with wave angles between 0° and 60°. However the formula proposed by the authors to consider the effect of the obliqueness has coefficients calibrated on the specific layouts that were tested. For this reason the formula by [Kerpen and Schlurmann \(2012\)](#) is not reported here.

The results are quite scattered as noticeable from Figure 17. Similar scatter can be noticed in [Goda \(2009\)](#). Nevertheless, a certain trend is recognized. As for the sloping dike, the differences due to short-crested waves can lie in the range of scattering of the same formulations.

Analysing the prediction results of each formulation, it can be concluded that:

- The Franco and Franco (1999) formula for long-crested waves presents a constant value for all wave angles larger than 37°. However the results of the present analysis show completely different trend: the larger the wave angle, the smaller the wave overtopping. The formula underestimates the reduction (high values for  $\gamma_\beta$ ), thus it leads to an overestimation of the overtopping discharge.
- Franco and Franco (1999) for short-crested waves seems to be the closest one to the FHR results, even though the tests conducted at Flanders Hydraulics Research are only with long-crested waves. However, as previously remarked, the differences due to the short-crestedness lie within the scattering of the formula. In general Franco and Franco (1999) stated that the directional spreading might allow reducing the freeboard with 30% in respect to cases with only long-crested waves.
- EurOtop (2007) presents a constant value for all wave angles larger than 45°. However the results of the present analysis show a different trend: the overtopping volumes are inversely proportional with respect to the wave angle. As for Franco and Franco (1999), the formula underestimates the reduction (high values for  $\gamma_\beta$ ), thus it leads to an overestimation of the overtopping discharge.
- The Goda (2009) formula seems to underestimate the reduction coefficient for a wave angle equal to 60°, while is overestimating them for larger wave angles; however the uncertainties on the 60° case might affect the results as previously stated.
- The differences in the measured overtopping volumes between the cases 45° and 60° are less significant than those between cases 60° and 70° or 80°. There is no clear explanation at the moment and further investigation would be needed. The FHR results for the 60° case show an average reduction coefficient of 0.75, in between Franco and Franco (1999) and EurOtop (2007). But these two approaches clearly underestimate the reduction of wave overtopping for wave angles equal to 70° and 80°.

Concluding, there is no formula, among those proposed, that fit well the results of the present experimental campaign. However, it is preferable to use the formula proposed by Goda (2009) for large angles due to three main reasons:

- a) the Goda (2009) correction coefficient represents an upper limit (safe approach) for the present cases with very oblique waves, although not excessively too high as EurOtop (2007);
- b) Goda (2009) expression for  $\gamma_\beta$  is applicable up to 80°, meanwhile EurOtop (2007) presents a constant value for wave angles larger than 45°.

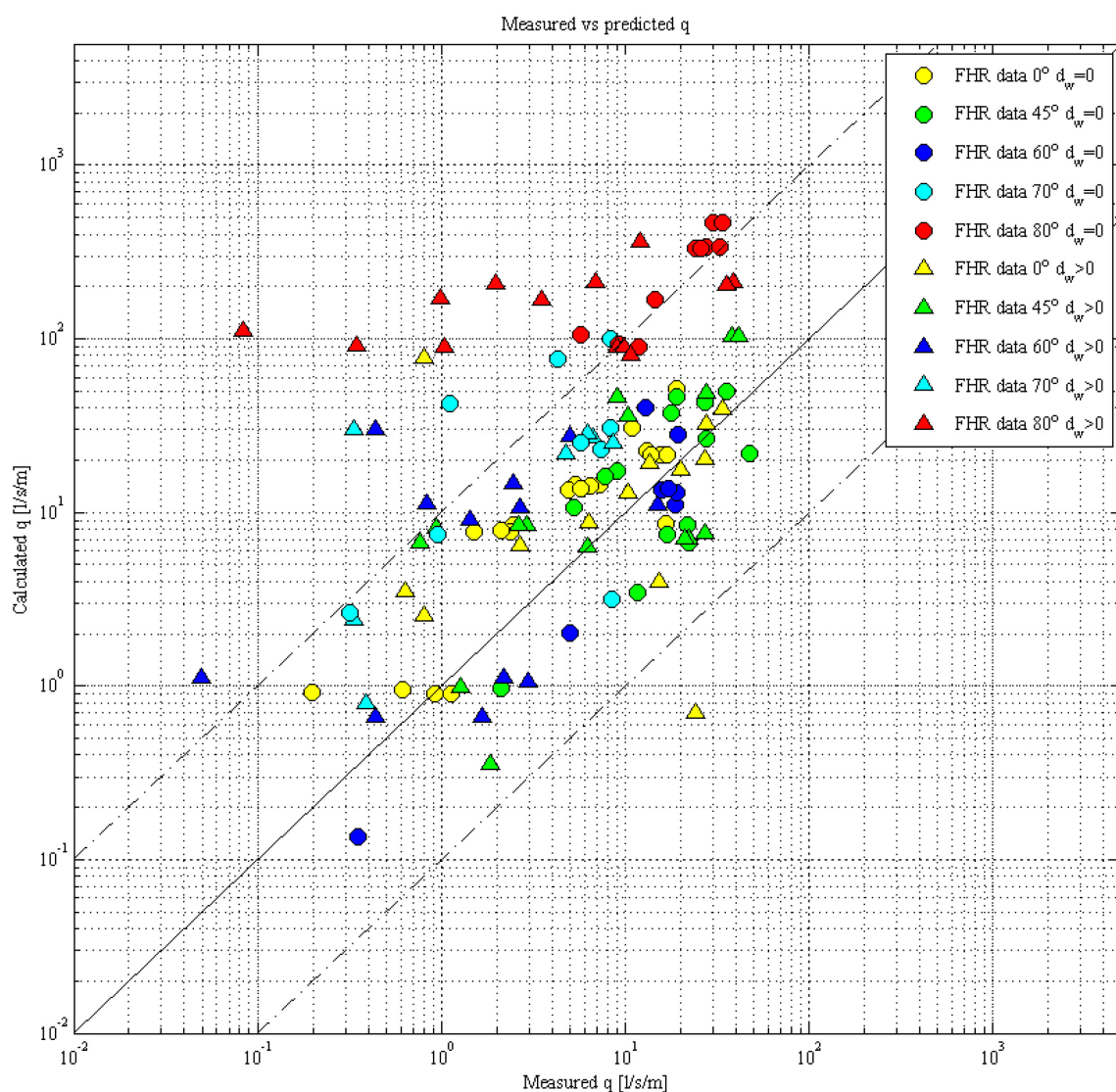
Based on the FHR data new values for the reduction coefficient are presented in Table 10. It is suggested to use them for similar conditions as those of the FHR test campaign (see section 3.2).



Table 10 - Reduction coefficient  $\gamma_\beta$  for the effect of obliqueness on wave overtopping, based on new FHR data

| wave angle $\beta$ | $\gamma_\beta$ |
|--------------------|----------------|
| 45°                | 0.76           |
| 60°                | 0.75           |
| 70°                | 0.44           |
| 80°                | 0.28           |

Figure 16 - Quay wall: predicted (EurOtop, 2007) vs. measured overtopping discharges.



The circles indicate the cases without crest berm ( $d_w=0$ ),  
the triangles indicate the cases where a crest berm is present ( $d_w>0$ ).



Table 11 - Vertical quay: existing formulae for  $\gamma_\beta$  (literature review)

| Formula                                                                                                                  | Reference                | Wave conditions                     |
|--------------------------------------------------------------------------------------------------------------------------|--------------------------|-------------------------------------|
| $\gamma_\beta = 0.83$ $0^\circ \leq \beta \leq 20^\circ$<br>$\gamma_\beta = 0.83\cos(20^\circ-\beta)$ $\beta > 20^\circ$ | Franco and Franco (1999) | Short-crested                       |
| $\gamma_\beta = \cos\beta$ $0^\circ \leq \beta \leq 37^\circ$<br>$\gamma_\beta = 0.79$ $\beta > 37^\circ$                | Franco and Franco (1999) | Long-crested                        |
| $\gamma_\beta = 1.0-0.0062 \beta $ $0^\circ \leq \beta \leq 45^\circ$<br>$\gamma_\beta = 0.72$ $\beta > 45^\circ$        | EurOtop (2007)           | Short-crested                       |
| $\gamma_\beta = 1.0-0.0096\beta+0.000054\beta^2$                                                                         | Goda (2009)              | Short and long-crested (CLASH data) |

Figure 17 - Quay wall: variation of reduction coefficient with wave angle; comparison with existing formulae

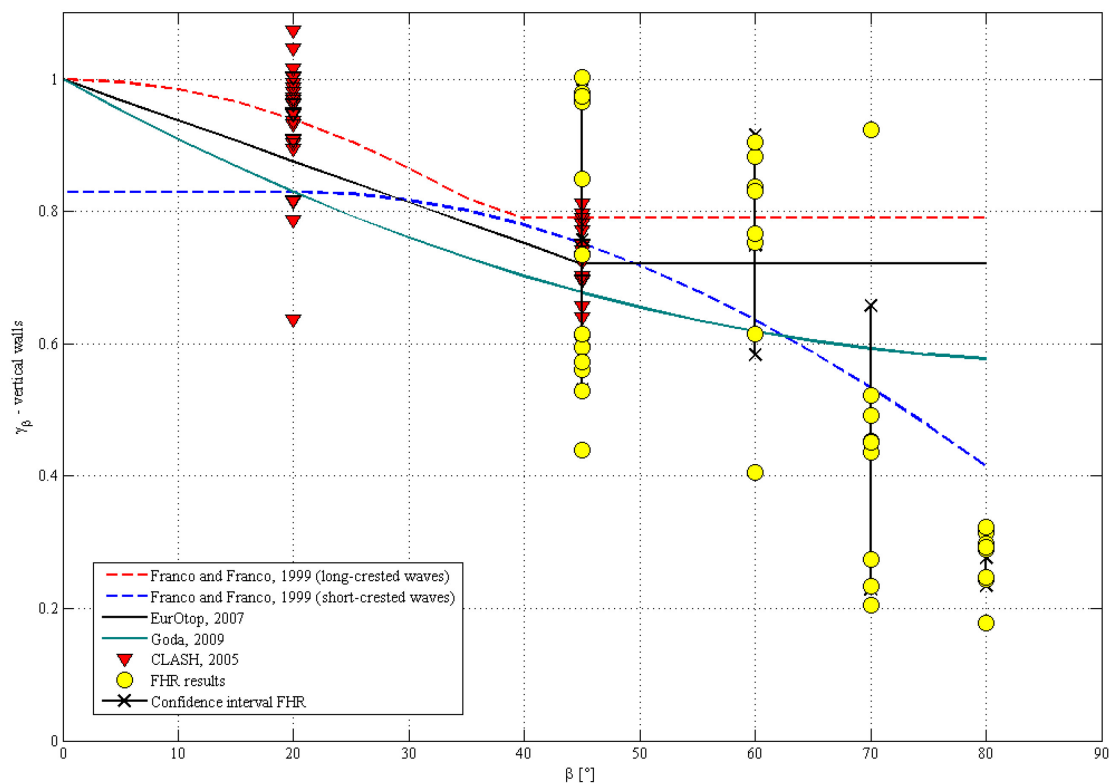
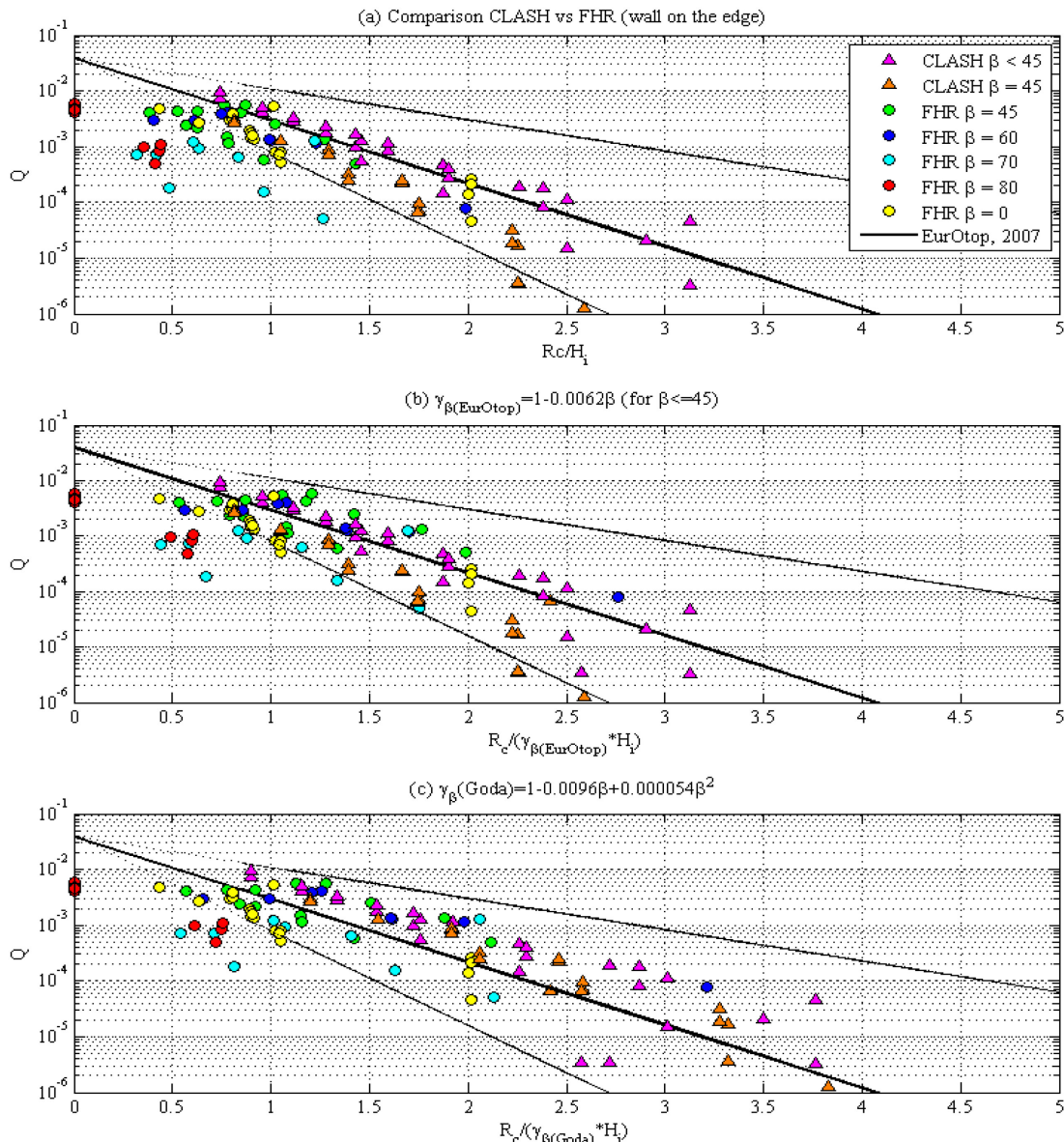


Figure 18 shows the FHR data, the CLASH data and the EurOtop predictions in term of non-dimensional discharge  $Q=q/(g \cdot H_{m0}^3)^{0.5}$ . Only the FHR cases with the wall on the edge of the quay are plotted in order to avoid misjudgements due to the effects of the width of the crest berm. Three different plots are depicted in Figure 18:

- the values of  $Q$  are plotted against the non-dimensional freeboard  $R_c/H_i$ ;
- the values of  $Q$  are plotted against the non-dimensional freeboard  $R_c/H_i \gamma_{\beta(EurOtop)}$ , where  $\gamma_{\beta(EurOtop)}$  is the correction coefficient calculated using the EurOtop (2007) formula;
- the values of  $Q$  are plotted against the non-dimensional freeboard  $R_c/H_i \gamma_{\beta(Goda)}$ , where  $\gamma_{\beta(Goda)}$  is the correction coefficient calculated using the Goda (2009) formula;

It can be noticed that the use of Goda (2009) formula is improving the wave overtopping prediction in case of oblique wave attack with respect to the EurOtop (2007) formula. In most of the cases, especially for very oblique angles, still the EurOtop formula seems to overestimate the overtopping; meanwhile, using Goda (2009) correction factors, the results are spread around the formula prediction and only few of them are still overestimated by the formula.

Figure 18 - CLASH and FHR (wall on the edge of the quay) data vs EurOtop (2007) predictions



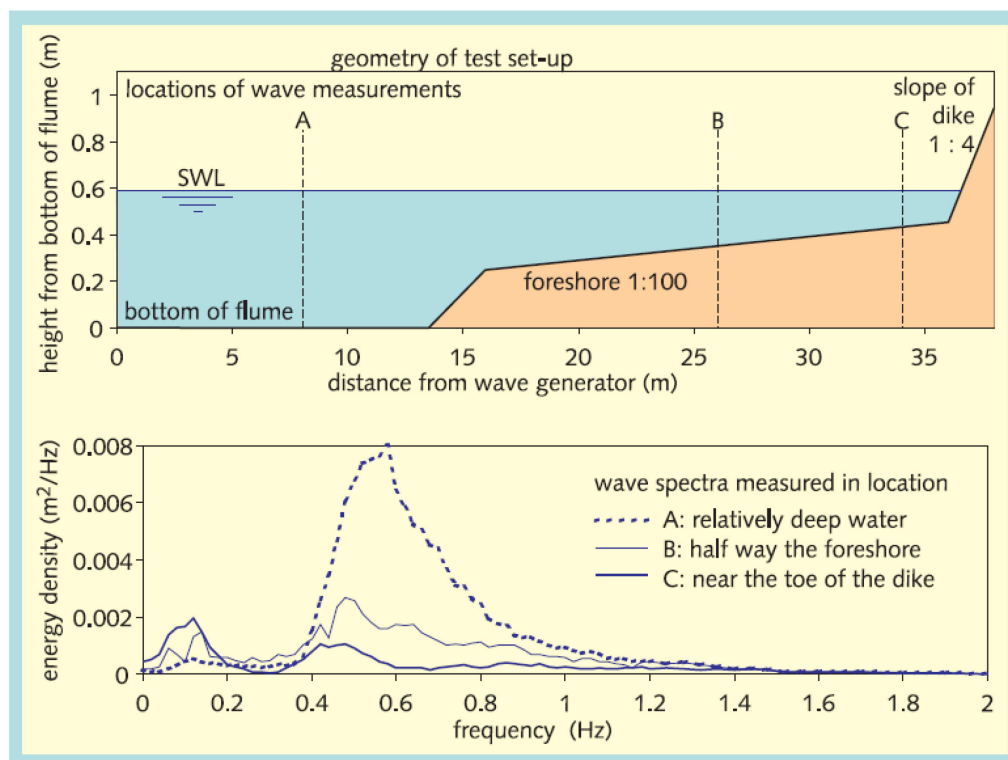
## 4 Wave overtopping in “very shallow water” conditions

### 4.1 Literature review

For overtopping calculations, it is useful to use empirical formulae (EurOtop, 2007; Goda, 2009; Van der Meer and Bruce, 2013; Mase et al., 2013). Those formulae give a reasonable overtopping discharge for typical dike configuration cases based on a large number of physical model tests. Not only for deep water conditions, EurOtop (2007) also includes a formula for shallow foreshore cases (Van Gent, 1999).

Van Gent’s equation, as reported in the EurOtop (2007), has been expected to be used for the estimation of the overtopping discharge on the dikes at the Flemish coast, since it has a shallow foreshore in front of the coastal dikes. Scale model tests have been conducted at Flanders Hydraulics Research to measure the mean overtopping discharge for sea dikes in shallow and very shallow foreshore conditions. The results have been compared with Van Gent’s prediction formula: the measured overtopping for most of the cases did not fit the overtopping estimation from Van Gent’s equation very well. One of the reasons can be explained by the calculation conditions. For the overtopping calculation in extreme wave conditions such as the 1000 year storm and the +8 m super storm defined in the Coastal safety master plan (Afdeling Kust, 2011), the water depth at the toe of the dike is very shallow. The spectral wave period in these conditions is much larger than the ones in the calculation of Van Gent’s equation. It means that the conditions at the toe of the dike are much shallower than the condition in Van Gent. A sketch of the geometrical set-up as used in Van Gent (1999) is depicted in Figure 19 and also described in Van der Meer (2002).

Figure 19 - sketch of the geometrical set-up as in Van Gent’s tests



(Source: Van der Meer, 2002)

The results from [Van Gent \(1999\)](#) are also included into the CLASH database (2005). [Van Gent \(1999\)](#) performed tests from relatively deep water to shallow water in the following conditions:

$$\frac{H_{m0-DEEP}}{d_{toe}} \leq 1.50$$

with  $H_{m0-DEEP}$  the spectral wave height calculated in deep water and  $d_{toe}$  the water depth at the toe of the dike. Later on the differences with the Flemish coast conditions will be shown.

The other reason of uncertainties in the former use of Van Gent's equation depends on the uncertainty of the calculation of the incident wave height. In some cases, the incident wave height is calculated by incident and reflection analysis by [Mansard and Funke \(1980\)](#) from three wave gauges located at the very shallow zone. As also stated in [Van Gent \(1999\)](#), it is not suitable for the calculation of incident wave height because this analysis is based on linear wave theory, while non-linear effects are dominating in the very shallow foreshore case.

[Mase et al. \(2013\)](#) propose new formulations for wave run-up and overtopping discharge in very shallow water that use the deep-water wave characteristics, differently from Van Gent and EurOtop. The reason of using the deep-water conditions is just because they are easier to measure in the laboratory than the wave characteristics at the toe of the structure. The main drawback of this approach is that first the run-up is calculated by the new introduced formula and then that result is used in the new overtopping formula. In case of errors in the run-up calculations, these errors are also brought in the overtopping calculations, increasing the uncertainty of the results. It would be better to define an overtopping formula only depending on measured parameters and not calculated ones.

[Van der Meer and Bruce \(2013\)](#) 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.

Based on the discussion above, it is evident that there is a lack of knowledge and it is necessary to investigate thoroughly the overtopping law for very shallow foreshores. The approach presented here is based on the use of existing physical models.

## 4.2 Experimental campaigns

In this study, around 80 tests have been conducted in the wave flume at Flanders Hydraulics Research. In addition, physical model results from other experimental campaigns have been used (ref. project no. 00\_025 and 00\_142). All tests are characterized by a 1:35 foreshore slope and the 1:25 model scale. Figure 20 shows a sketch of the experimental set-up as in the Wenduine experimental campaign (ref. project no. 00\_025). The set-up of the 00\_142 project (also described in [Chen et al., 2015](#)) is depicted in Figure 21. The same set-up of Figure 21 (b) and (c) has been used in the new experimental tests conducted at FHR (ref. project no. 13\_116): the mean overtopping discharge has been measured over two different dike slopes, namely 1:3 and 1:6.

From here on, in all the following figures, we will refer to each test campaign as follows:

- *FHR 1:3 dike*: results from project no. 13\_116 for the sea dike with 1:3 slope.
- *FHR 1:6 dike*: results from project no. 13\_116 for the sea dike with 1:6 slope.
- *Chen (1:3 dike)*: results from project no. 00\_142 for the sea dike with 1:3 slope.
- *Wenduine tests*: results from project no. 00\_025 (all of them characterized by 1:2 sloping dike).

In order to compare with Van Gent (1999), the investigated hydraulic boundary conditions of the three mentioned experimental campaigns can be summarized as follows:

$$1.00 \leq \left| \frac{H_{m0-DEEP}}{d_{toe}} \right| \leq 50.00$$

The range is totally different from the Van Gent's one. 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 transformation and in the long wave generation process.

Quoting [Van der Meer \(2002\)](#), *"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...."*

It is questionable which slope value should be used in the Van Gent equation. In fact, that equation is function of the surf similarity parameter, that contains the tangent of the slope angle. But in cases where the water is very shallow at the toe of the dike, the 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 (further details in §4.3).

This modification in the input for [Van Gent \(1999\)](#) is discussed in the next section as well as the applicability of [Van Gent \(1999\)](#) to the data from FHR experiments.

Figure 20 - Experimental set-up of the Wenduine experiments (project no. 00\_025)

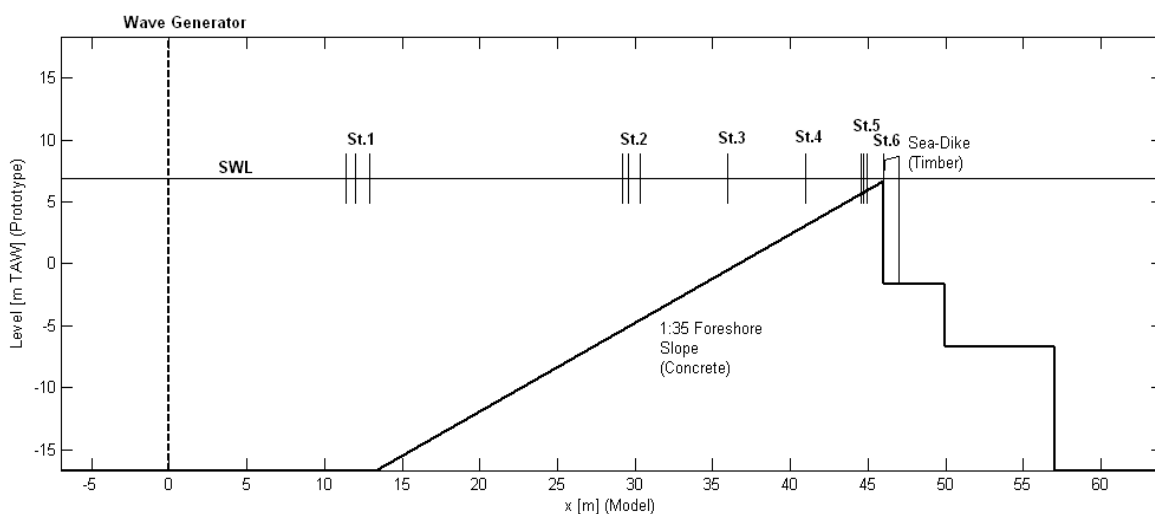
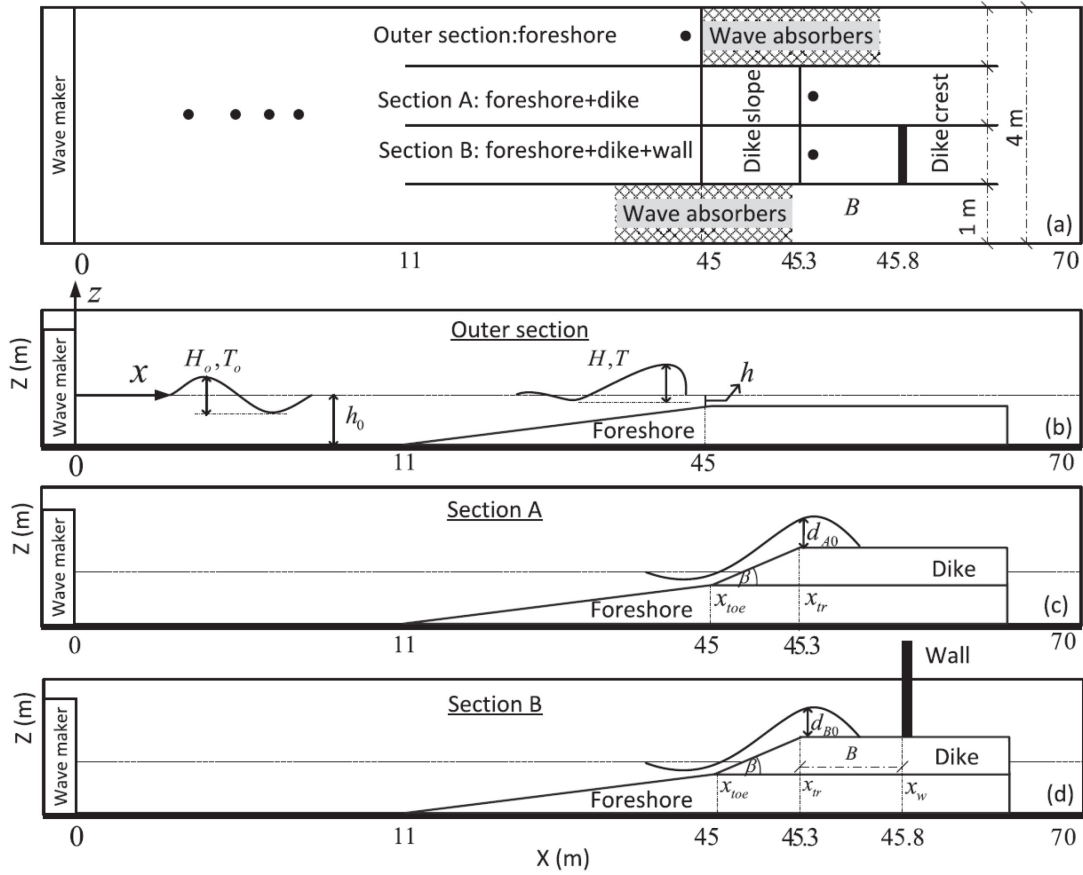


Figure 21 - Experimental set-up of the 00\_142 project (Chen et al., 2015)



### 4.3 Use of Van Gent for very shallow waters

#### 4.3.1 Van Gent (1999) classical approach: $\tan\alpha$ equal to the dike slope

The formulae proposed by [Van Gent \(1999\)](#) and also reported in [Van der Meer \(2002\)](#) and [EurOtop \(2007\)](#) are expressed by the following equations, based on the probabilistic and deterministic approach respectively:

$$\frac{q}{\sqrt{gH_{m0}^3}} = 10^c \exp\left(-\frac{R_c}{H_{m0}\gamma_f\gamma_\beta(0.33+0.022\xi_{m-1,0})}\right) \quad (36)$$

$$\frac{q}{\sqrt{gH_{m0}^3}} = 0.21 \exp\left(-\frac{R_c}{H_{m0}\gamma_f\gamma_\beta(0.33+0.022\xi_{m-1,0})}\right) \quad (37)$$

where

$$\xi_{m-1,0} = \frac{\tan(\alpha)}{\sqrt{2\pi H_{m0}/gT_{m-1,0}^2}} \quad (38)$$



is the surf similarity parameter. The angle  $\alpha$  is usually referring to the dike slope angle with respect to the horizontal. The formula has been applied to the FHR model tests. In equation 36 the parameter  $c$  is a normally distributed stochastic function with a mean of  $-0.92$  and a standard deviation  $\sigma$  of  $0.24$ . Therefore the 5% upper and under exceedance limits can be calculated as  $(-0.92) \pm 1.64\sigma$ .

The value of  $10^{-0.92}$  is replaced in equation 37 by  $0.21$ , that represents a sort of upper limit for the prediction of the mean overtopping discharge. Equation 37 will be used in the Methodology Book as recommended approach for shallow water conditions (recommended = deterministic, average = probabilistic).

The CLASH database (Verhaeghe et al., 2008) has been investigated in order to extract data from different experimental campaigns that could be representative of overtopping in shallow water conditions. The aim is to compare these data with the results from FHR and the prediction using Van Gent's equation. The data extracted from CLASH finally correspond to the following experimental campaigns (labelled as in CLASH database):

- DS-042:  $\cot\alpha=[2; 4]$   $m=[10; 20; 50]$ ;
- DS-109:  $\cot\alpha=6$   $m=29$ ;
- DS-221:  $\cot\alpha=[3; 4]$   $m=100$ ;
- DS-226 (indicated in the figures as *Van Gent, 1999*):  $\cot\alpha=[2.5; 4]$   $m=[100; 250]$ ;
- DS-227:  $\cot\alpha=[3; 4; 6]$   $m=100$ ;
- DS-956:  $\cot\alpha=[2.72]$   $m=[48; 49; 50]$ ;

with  $\cot\alpha$  slope of the sea dikes expressed as the cotangent of the angle between the dike surface and the horizontal and  $m$  the foreshore slope (again expressed as cotangent).

The results are plotted in Figure 22 and Figure 23. The dash blue line indicates the recommended equation (37). It can be preliminary noticed that *Van Gent (1999)* overestimates the measured overtopping discharges especially for the tests of project no. 13\_116 (indicated "FHR 1:3 dike" and "FHR 1:6 dike" in the figures) and for the tests of the project no. 00\_025 (Wenduine tests).

Uncertainties exist on the measured incident wave height and spectral wave period at the toe of the dike for some tests of projects 00\_025 and 13\_116. In the Wenduine case there was no direct measurement of the wave conditions at the toe in absence of the dike, but only wave reflection analysis was performed and only at a distance from the toe equal to 5 times the wave height at  $-5m$  TAW. For project no. 13\_116, the uncertainties were related to the accuracy of the wave gauges in some cases. For all the aforementioned reasons, SWASH models have been used to get more accurate boundary conditions at the toe of the dike. SWASH has been used as described as follows:

1. SWASH has been validated against cases from projects no. 00\_025, 00\_142 and 13\_116 of which physical model results were judged as accurate;
2. the dike has been removed later on in SWASH and a sponge layer has been introduced to damp the wave reflection at the end of the domain;
3. SWASH has been run and the wave height and period have been calculated in absence of the dike.
4. The results have been compared with the measured ones (when possible).
5. The hydraulic boundary conditions so-calculated at the location that corresponds to the toe of the dike have been used for the overtopping calculation.

The differences of the prediction for the different experimental data can be explained looking at Figure 24 and Figure 25 where the ratio between the predicted non-dimensional overtopping and the measured overtopping is shown in function of different quantities. The predicted value is calculated using equation 36 (i.e. probabilistic = average approach). The range of relative water depth with respect to the wave length (Figure 25) is totally different with *Van Gent (1999)*; furthermore the test cases from Wenduine and 13\_116 present the largest differences of the ratio  $H_{m0-DEEP}/d_{toe}$  with Van Gent's data (Figure 24). This is a further indication that the mechanisms of wave transformation and breaking are not the same among all the data sets and must be investigated in detail (out of scope of this project).

Figure 22 - Non-dimensional overtopping discharge: measurements vs Van Gent's prediction

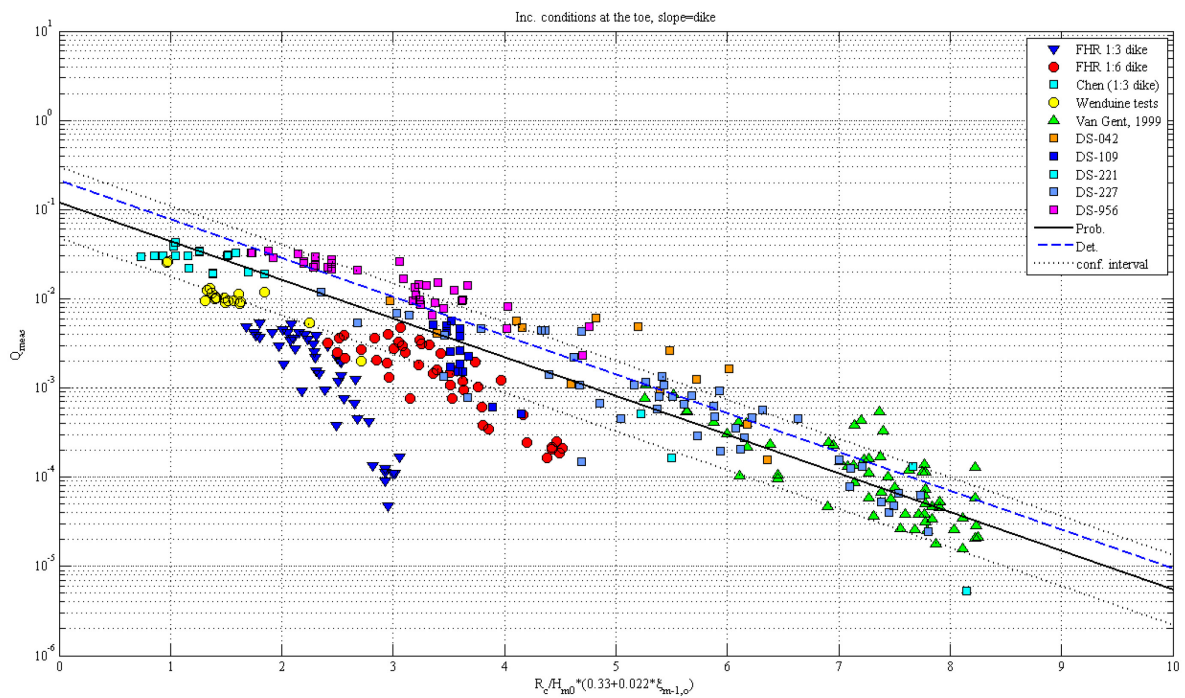
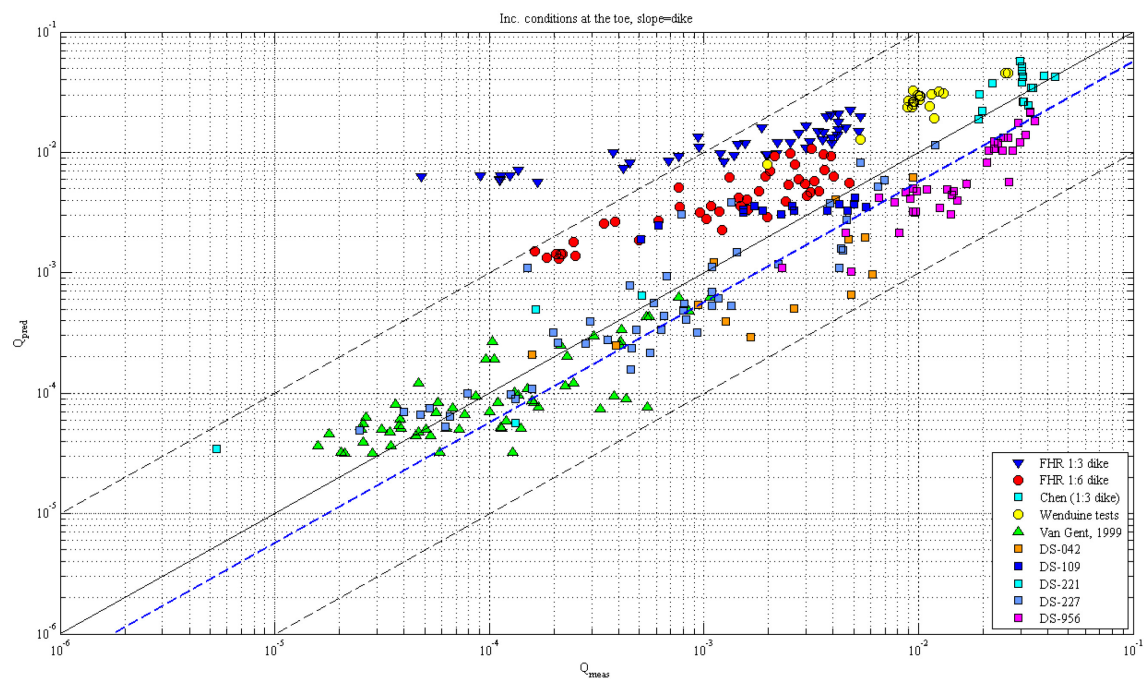
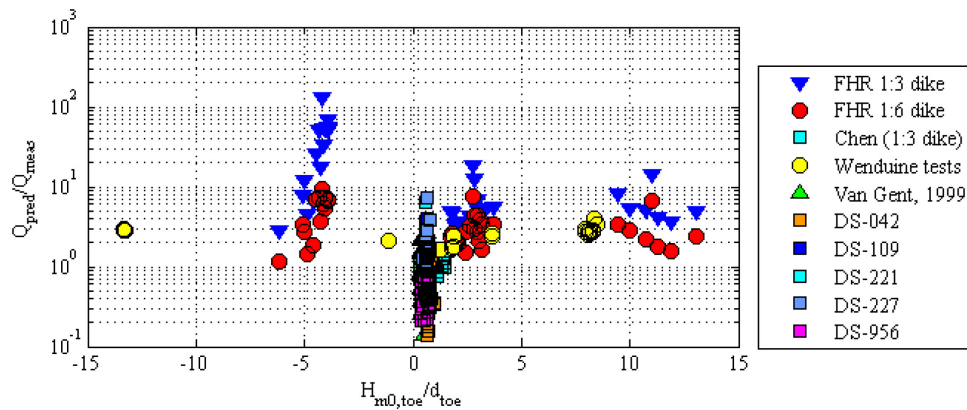
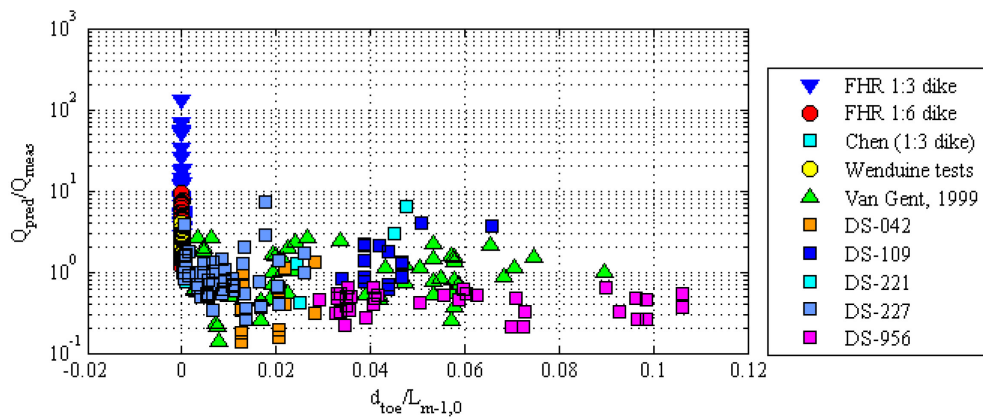


Figure 23 - Measured vs predicted (eq. 36) non-dimensional overtopping discharge



the dash blue line indicates the recommended (= deterministic) approach.



Figure 24 - Error in the prediction of non-dimensional overtopping discharge in function of  $H_{m0-DEEP}/d_{toe}$ Figure 25 - Error in the prediction of non-dimensional overtopping discharge in function of  $d_{toe}/L_{m-1,0}$ 

#### 4.3.2 Use of the foreshore slope instead of the dike slope

In the previous section the results of the application of [Van Gent \(1999\)](#) were shown. The slope of the sea dike for each case was used to calculate the surf similarity parameter in the formula. The comparison between predicted mean overtopping discharges and results from four different experimental campaigns (including Van Gent's one) underlined the inaccuracy of the prediction for cases in very shallow water.

For all the reasons mentioned above, the use of the foreshore slope into the Van Gent's equation has been investigated. The results are plotted in Figure 26 and Figure 27.

It is clear that the use of the foreshore slope instead of the dike slope leads to an underestimation of the mean overtopping discharge with respect to the measured values. It can be argued that the foreshore is significantly influencing the results for very shallow waters ( $H_{m0-DEEP}/d_{toe} \gg 3$ ) whereas for lower values the waves still "feel" the dike more than the foreshore. Speaking in terms of wave breaking, in cases with very shallow waters, the foreshore is the main responsible of the wave breaking process, otherwise both foreshore and dike can play an important role.

For all the above mentioned reasons, the use of an average slope will be proposed and investigated in the next section. The procedure to calculate the average slope is reported both in [Van der Meer \(2002\)](#) and [Mase et al. \(2013\)](#).

Figure 26 - Non-dimensional overtopping discharge: measurements vs Van Gent's prediction using the foreshore slope into the equation.

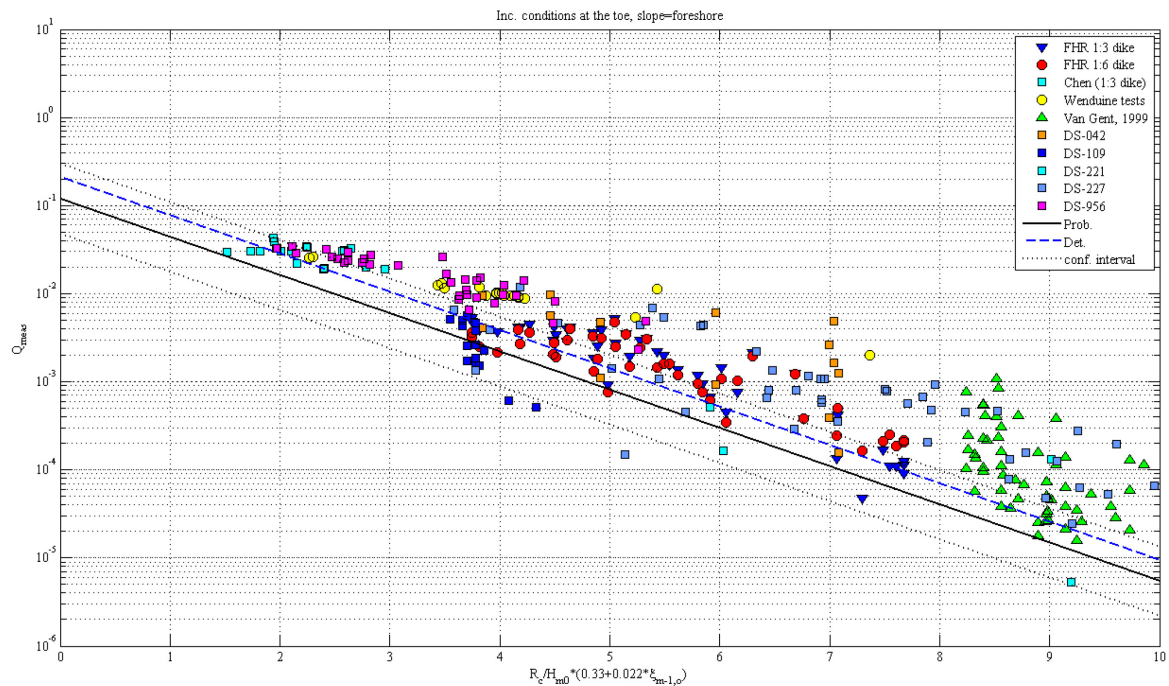
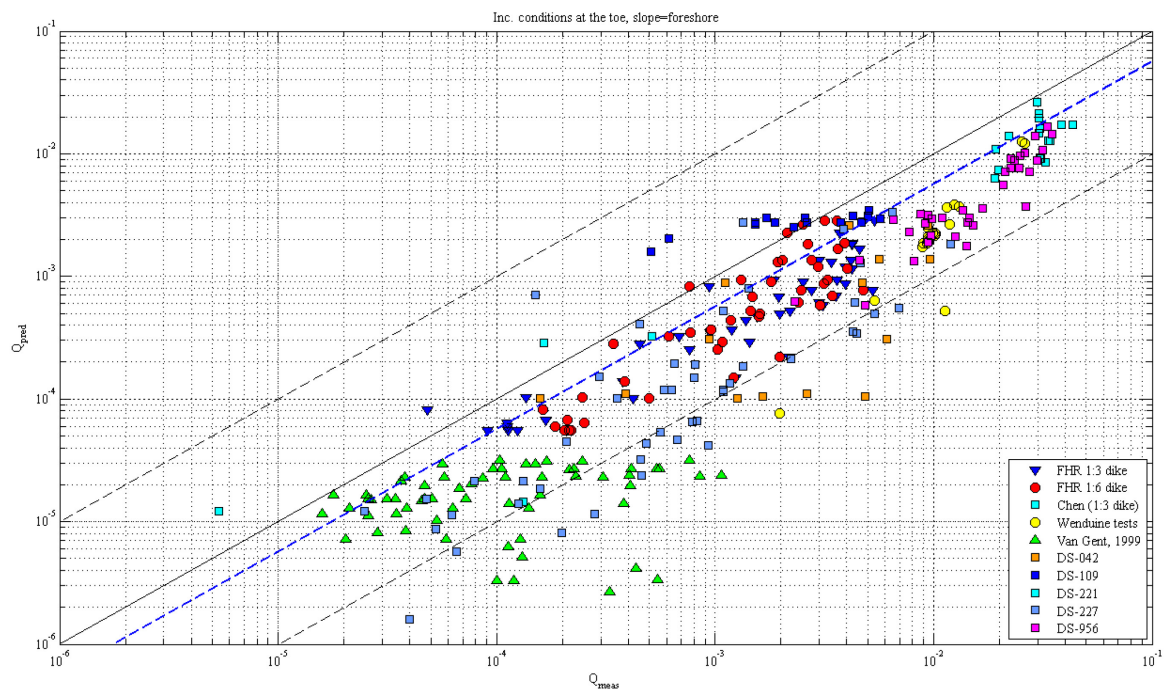


Figure 27 - Measured vs predicted non-dimensional overtopping discharge using the foreshore slope into the equation.



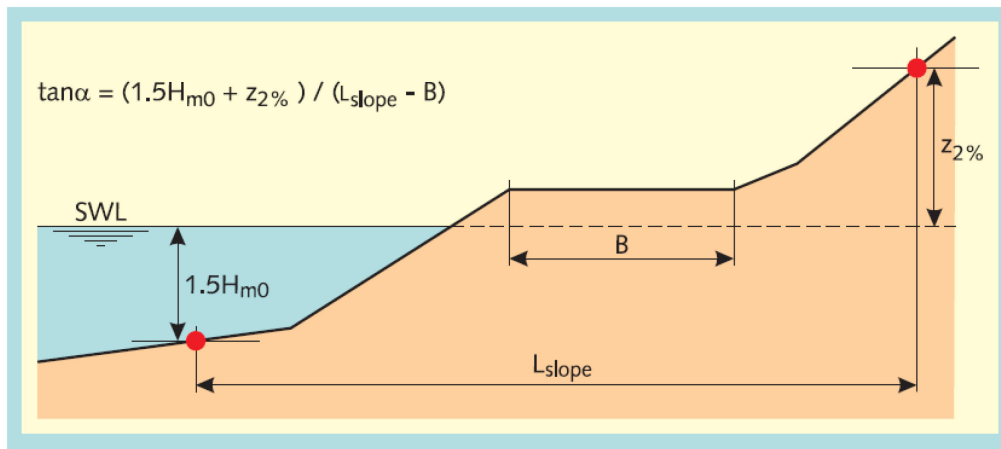
#### 4.3.3 Use of an “average” slope

An average slope is defined as the average one in the zone between **SWL-1.5H<sub>m0</sub>** and **SWL+Z<sub>2%</sub>**, where SWL is the still water level, H<sub>m0</sub> is the significant wave height at the toe of the structure and Z<sub>2%</sub> is the wave run-up exceeded by 2% of the incident waves. The average slope is calculated as follows:

$$\tan\alpha = \frac{(1.5H_{m0} + Z_{2\%})}{L_{\text{slope}} - B} \quad (39)$$

where the quantity L<sub>slope</sub> is the horizontal length of the zone between **SWL-1.5H<sub>m0</sub>** and **SWL+Z<sub>2%</sub>** and B is the width of a possible intermediate berm. In cases where only the foreshore and the dike slope are present the berm width B is equal to 0 (see Figure 28).

Figure 28 - Establishing a representative slope angle made by a slope of different slope gradients, excluding any berm



(Van der Meer, 2002).

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

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

$$Z_{2\%} = \left(4.0 - \frac{1.5}{\sqrt{\xi_{m-1,0}}}\right) \gamma_b H_{m0} \quad (40)$$

Eq. (40) corresponds to the calculation of the wave run-up height following a probabilistic (= average) approach meanwhile in the Methodology Book the deterministic (= recommended) approach will be proposed: if the deterministic formula is used to calculate the wave run-up, than the coefficients 1.65, 4.0 and 1.5 have to be replaced by 1.75, 4.3 and 1.63 respectively.

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 (40) requires an iterative process that is stopped when the result for wave run-up is starting to converge.

- The first run-up value can be estimated as 1.5\*H<sub>m0</sub>.
- With this first run-up value, the new slope is calculated using equation (39).
- This slope is reintroduced in equation (38) to calculate the new surf similarity parameter.

- Then equation (40) is used for the 2<sup>nd</sup> run-up calculation.
- 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%.
- The final calculated slope will be used as the average slope.

This iterative process has been applied to all test cases of the experimental.

It is pretty clear that the foreshore is not taking part in the calculation when the water depth at the toe of the dike is larger than  $1.5 \cdot H_{m0}$ . This is the case for most of the tests from Van Gent (1999). But as the water depth becomes smaller and smaller, the share of the foreshore in  $L_{slope}$  is larger and larger. In fact,  $L_{slope}$  can be calculated as:

$$L_{slope} = \frac{(Z_{2\%} + d)}{\tan \alpha} + m \cdot (1.5H_{m0} - d_{toe}) \quad (41)$$

where  $m$  is the cotangent of the foreshore angle (measured with respect to the horizontal) and  $d_{toe}$  is the water depth at the toe of the dike. In this way, the average slope can be calculated also in cases with a water level below the toe level, assuming  $d_{toe} < 0$ . The influence of the foreshore will be consequently more important than in cases with a wet toe.

The results of using the average slope to calculate the mean overtopping discharge are shown in Figure 29 and Figure 30. The accuracy of the prediction is immediately noticeable, being most of the results within the confidence interval of the probabilistic formula. Furthermore the use of the deterministic (i.e. recommended) formula seems to match with the average of most of the cases with largest overtopping discharge. Few cases show low accuracy, but this is mainly depending on uncertainties on the measured wave height and wave period at the toe of the dike. All the cases are now in between 0.1 and 10 times the average predicted value (see Figure 30).

**The results demonstrate that the use of the average slope is giving the best prediction. Moreover, the wave conditions have to be estimated only at the toe of the dike, both for still water level below and above the toe level of the dike. 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 [Van Gent \(1999\)](#).**

Figure 29 - Non-dimensional overtopping discharge: measurements vs Van Gent's prediction using the average slope into the equation.

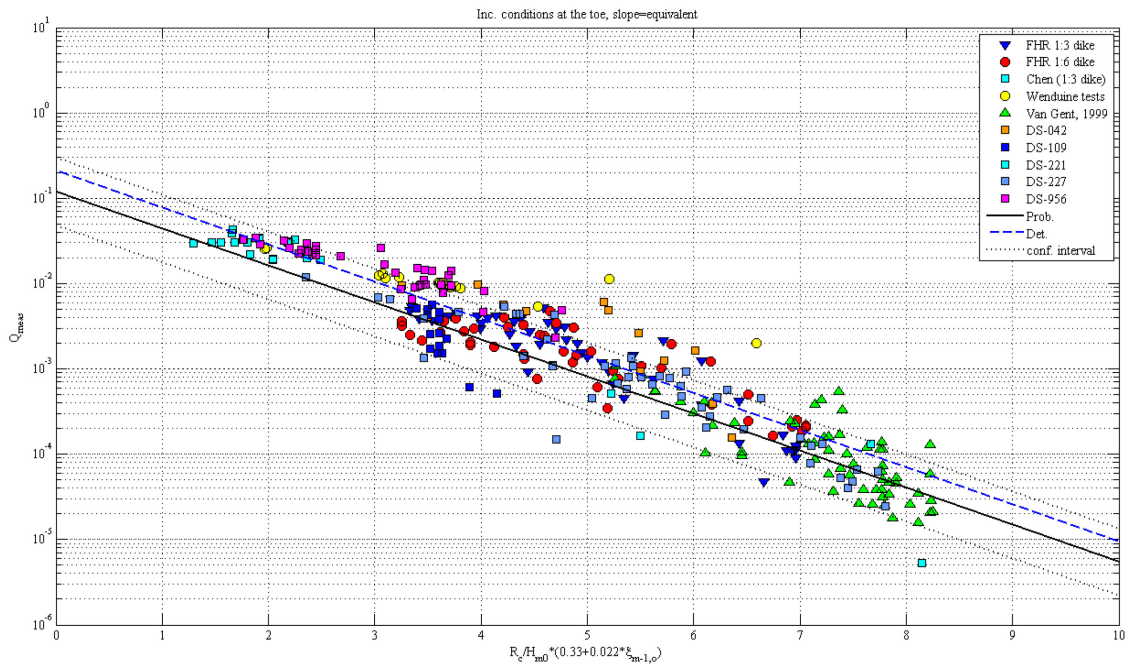
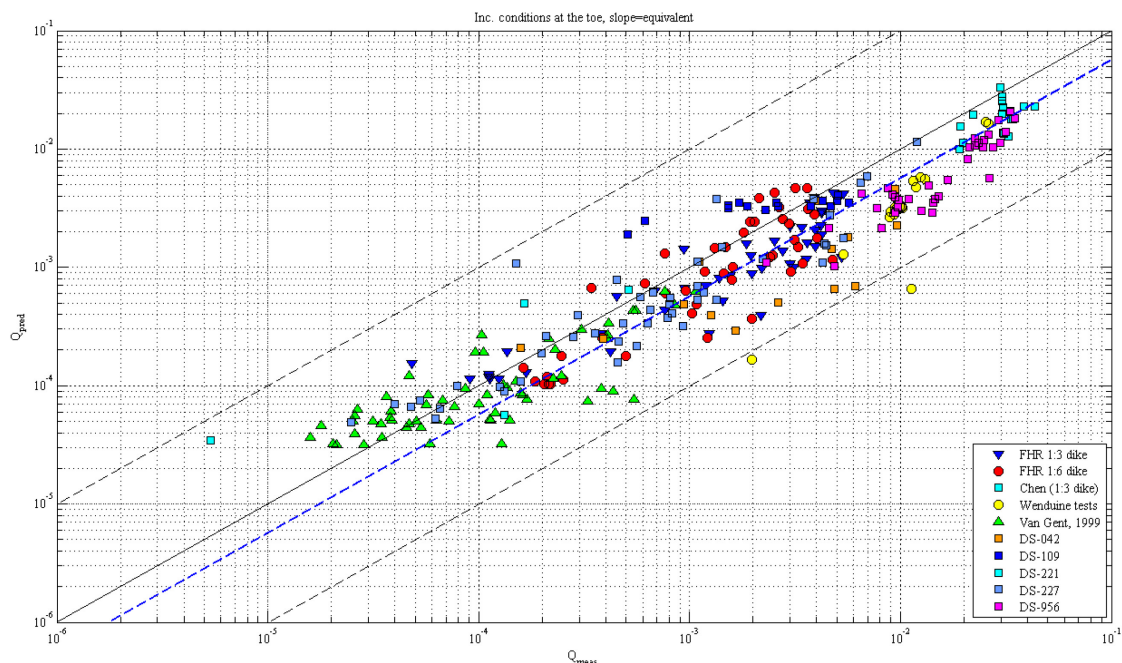


Figure 30 - Measured vs predicted non-dimensional overtopping discharge using the average slope into the equation.



It has to be noticed that in case of emergent toe (toe in dry conditions) or very low water depth at the toe, only broken waves like bores reach the toe of the structure. In those specific cases there are no 'normal' waves anymore at the toe position. Still the concept of spectral wave height and period are used: the spectral analysis is conducted on the time series of the water surface elevation as measured at the toe, although is not anymore a time series of sea waves.

# Conclusions

The present report discusses different topics related to the overtopping assessment of sea dikes and quay walls with geometrical-hydraulic conditions characteristic for the Flemish coast. The analysis aims to help upgrading the methodologies used for the last safety assessment as defined in IMDC (2008).

Five main topics have been analysed in detail:

- 1) **dikes with complex geometrical layout:** [EurOtop \(2007\)](#) + [Van Doorslaer et al. \(2015\)](#) is proposed as the new approach for sea dikes with non-standard geometries and non-breaking waves.
- 2) **vertical quays with storm return walls:** [EurOtop \(2007\)](#) + [Trouw et al. \(2012\)](#) can be used as preliminary evaluation of the mean wave overtopping discharge.
- 3) **effects of high wave obliqueness** (*finally not included in the next version of the Methodology Book*): [Goda \(2009\)](#) seems to be a reliable approach to estimate the wave overtopping reduction with a certain safety level;
- 4) influence of **very shallow foreshore** on wave overtopping estimation: [Van Gent \(1999\)](#) can be still used but it is recommended to use the average slope instead of the dike slope to calculate the surf similarity parameter when the water depth at the toe is smaller than 1.5 times the incident spectral wave height,  $H_{m0}$ , measured at the toe of the dike.
- 5) Numerical and physical modelling are recommended, in case the formulae are out of range of applicability or the geometrical layout cannot be schematized as one of those studied and described in this report.



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