EurOtop revisited. Part 2: Vertical Structures

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Summary

The European Manual for the Assessment of Wave Overtopping ("EurOtop") was issued free on the internet in 2007 and is now used worldwide. The manual and the accompanying Neural Network give guidance on all aspects of wave overtopping. It was the result of synthesis of existing Dutch, UK and German guidance and new research findings arising out of projects such as the EC FP7 "CLASH" project. During and since writing, the manual team has identified gaps and areas in which other improvements and clarifications could be made. This paper presents a summary of new analysis of existing data on vertical and very steep walls. A companion paper revisits sloping structures (Van der Meer *et al*., 2013).

Specifically, two themes are explored. First, a reformulation of the procedures for overtopping analysis of vertical walls is presented offering a simpler and more physically rational procedure. A distinction is made between overtopping at structures without influence of foreshore and those where the foreshore plays a role in the hydrodynamics at the structure. This leads to an improved method for higher freeboard situations. Secondly, composite vertical structures are revisited, and a procedure proposed that integrates their analysis with that of the updated procedure for plain walls.

Introduction

The following topics were treated in the EurOtop Manual (2007), but were not explored as fully as would have been desirable and are therefore key drivers for this and the companion paper:

- Wave overtopping at low and zero freeboards for sloping structures. Existing (old) theory and available data sets show lower wave overtopping than predicted by the exponential type of formulae used in the EurOtop Manual. This topic is explored in the companion paper (Van der Meer *et al.*, 2013).
- Overtopping response at very steep slopes with geometries lying between the fully-vertical and more familiar milder slopes, i.e. for slopes approximately in the range 5:1 (i.e. 5V:1H) down to 1:1. The companion paper also describes the new analysis on this topic.
- Physical explanation of the relationship between the formulae for vertical breakwaters and seawalls, for "non-impulsive" (or "pulsating") waves and for "impulsive" (breaking) wave overtopping, described by exponential and power-law formulae, respectively (EurOtop, 2007, Chapter 7). This applies also to the analysis of composite vertical breakwaters. The first section of this paper deals with this topic.
- Explanation for the difference of mean overtopping rates between the formulae of Franco *et al*. (1994) and Allsop *et al*. (1995) for vertical structures, by physically rational reasoning and reanalysis of a part of the CLASH database. This is the second focus of this paper.

Basis of the EurOtop Manual

EurOtop (2007), following from EA (1999), presents two methods for the prediction of mean overtopping discharge (q) for vertical breakwaters or seawalls depending upon whether wave breaking and resulting "impulsive" or "violent" overtopping is present. A discriminating parameter, h_* (Equation 1) is used to determine which situation applies.

$$h_* \equiv 1.3 \frac{h}{H_{m0}} \frac{2\pi h}{g T_{m-1,0}^2} \tag{1}$$

For $h_* > 0.3$, wave breaking is expected to be absent, and the familiar exponential relationship between relative (non-dimensional) discharge and relative crest freeboard (Equation 2) works well.

$$\frac{q}{\sqrt{g H_{m0}^3}} = a \exp\left(-b \frac{R_c}{H_{m0}} \right) \tag{2}$$

Early work by Franco *et al.* (1994) gave $a = 0.2$ and $b = 4.3$, while Allsop *et al.* (1995) gave $a = 0.05$ and $b = 2.78$. While the predictors are similar for lower freeboards, they diverge significantly at higher R_c/H_{m0}. This apparent problem is addressed in the new analysis later in this paper.

Overtopping at vertical and steep walls cannot however be described for all conditions simply by exponential-form equations like Equation 2. Goda's design charts (*e.g.* Goda, 2000) show quite pronounced peaks for some shallower (relative) water depths. Goda (2009) notes that local water depth on a foreshore is important. Physical model studies in the 1990s under conditions where wave breaking on the structure was present gave rise to new empirical fits of the form with a power law decrease in overtopping discharge with freeboard, rather than an exponential one. The impulsive overtopping equations (EurOtop, 2007, after Allsop *et al.*, 1995) have the form of a power law:

$$\frac{q}{h_*^2 \sqrt{g h^3}} = a \left(h_* \frac{R_c}{H_{m0}} \right)^{-b} \tag{3}$$

The coefficient a and exponent b in Equation 3 change depending on structure and wave conditions considered, taking values of a = 0.00015 and b = 3.1 for impulsive overtopping at plain vertical walls. This exponent is simply the result of fitting to data with no basis in physical reasoning. There is a strong evidence that these two, apparently distinct methods work well. The fact that discharge and freeboard are non-dimensionalised in quite different ways for impulsive and non-impulsive conditions has prevented simple comparison of the formulae. Further, these differences hamper improved physical rationalisation of the transition from one regime to the other.

Relationship between Non-impulsive and Impulsive Overtopping

In order to establish a more unified, physically rational framework of prediction tools spanning a greater breadth of structure types and wave conditions, Bruce & Van der Meer (2008) attempted to bring together the non-impulsive and impulsive formulations by fixing the exponent b in Equation 3 at a (round) value of 3. This facilitated algebraic manipulation, leading to an expression for "impulsive" overtopping that uses the familiar non-dimensionalisation of the discharge (Equation 4).

$$\Rightarrow \frac{q}{\sqrt{g H_{m0}^3}} = a \sqrt{\frac{H_{m0}}{h}} \frac{1}{s_{m-1,0}} \left(\frac{R_c}{H_{m0}} \right)^{-3} \tag{4}$$

where $s_{m-1,0}$ is the (fictitious) wave steepness with wave length based on $T_{m-1,0}$. Equation 4 indicates that the overtopping under impulsive conditions depends on a combination of breaker index H_{m0}/h and the wave steepness $s_{m-1,0}$. Using this formulation, Bruce & Van der Meer (2008) plotted prediction curves for impulsive conditions directly alongside those for non-impulsive conditions on the familiar dimensionless discharge versus dimensionless freeboard axes.

Here, this approach is developed. We re-examine the choice of coefficient a and the strength of the influences of the steepness using existing data of the CLASH database.

Reanalysis of Vertical Walls with CLASH Data

The CLASH database contains about 10,000 tests on overtopping, see Van der Meer *et al.* (2008). A part of this database relates to tests with a vertical or battered wall. In total 15 different data sets were used from the database – those shown in Table 1 as "vertical seawall", "harbour wall", or "caisson". Also, three further datasets were used: two from the original dataset of Franco *et al.* (1994) and the dataset of vertical walls at the end of a 1:50 foreshore of Allsop *et al.* (1995). In order to describe the set-up of the dataset, information is given on the presence of a foreshore slope, the type of structure investigated and whether a berm or toe structure was present. All datasets with a foreshore had a straight foreshore with a given, fixed slope. Of the 18 vertical wall data sets, 12 had a horizontal foreshore (= "no foreshore") and 6 had a sloping foreshore. Two data sets with a slope instead of a vertical wall were included as they contained rare data with zero freeboard.

Table 1: CLASH datasets for vertical walls. The table gives the dataset number and a reference if the dataset is in the public domain.

(CLASH) dataset	Reference	Foreshore slope	Type of structure	Berm
006	Confidential	1:20	Battered 10:1	No
028	Herbert (1993)	1:10; 1:30; 1:100	Vertical seawall	No
043	Pullen *et al.* (2004)	1:30	Composite	Yes
044	Pullen *et al.* (2004)	1:30	Composite	Yes
102	Schüttrumpf and Oumeraci (2005)	No	Slope 1:4	No
106	Oumeraci *et al.* (2001)	No	Vertical seawall	No
107	Smid *et al.* (2001)	No	Vertical seawall	No
108	Smid *et al.* (2001)	No	Slope 1:1.5	No
113	Oumeraci *et al.* (2001)	No	Harbour wall	Yes
224	De Waal (1994)	1:50	Vertical seawall	No
225	De Waal (1994)	1:20	Vertical seawall	No
228	Confidential	No	Caisson	Yes
229	Confidential	No	Caisson	Yes
315	Confidential	No	Caisson	Yes
351	Confidential	No	Caisson	Yes
380	Confidential	No	Caisson	Yes
402	Confidential	No	Vertical seawall	No
502	Bruce *et al.* (2001)	1:10; 1:50	Vertical seawall	No
503	Bruce *et al.* (2001)	1:10	Battered 10:1	No
504	Bruce *et al.* (2001)	1:10	Battered 5:1	No
505	Bruce *et al.* (2001)	1:10; 1:50	Composite	Yes
507	Pearson *et al.* (2002)	1:13	Battered 10:1	No
802	Goda *et al.* (1975)	1:10; 1:30	Vertical seawall	No
914	Cornett *et al.* (1999)	No	Vertical seawall	Yes
Allsop et al. (1995)	Allsop *et al.* (1995)	1:50	Vertical seawall	No
CEPYC	Confidential	No	Caisson	Yes
ENEL CRIS	Confidential	No	Caisson	Yes

All datasets were then plotted individually, with four prediction curves for comparison: Franco *et al.* (1994), Allsop *et al.* (1995), a steep smooth slope (Equation 2) and one specific curve for impulsive waves (Equation 4 with a = 0.000192 with h/H_{m0} = 0.9 and $s_{m-1,0}$ = 0.03). Examples are shown in Figures 1 and 2. Figure 1 shows dataset 802 of Goda *et al.* (1975) and shows clearly the increased overtopping for seawalls at the end of a foreshore slope, as all data points are along or above the curve for impulsive wave attack, with few points around the Allsop or Franco curves. Figure 2 shows CLASH dataset 914 of Cornett *et al.* (1999) with tests on a vertical wall with deep water without a foreshore and with a small and deep berm. The overtopping is now significantly less than in Figure 1 and is grouped well around the line of Franco *et al.* (1994).

Individual analysis of all datasets led to the clear conclusion that there is a distinct difference between overtopping responses of vertical structures with and without a sloping foreshore, with a sloping foreshore always giving larger overtopping. On the basis of this, the datasets were split into two groups and each group was then analysed separately.

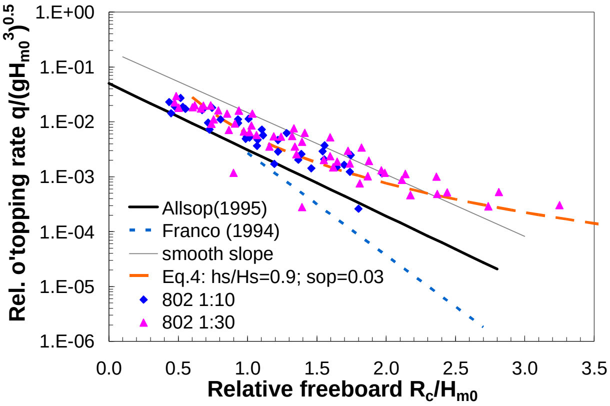

Figure 1. Overtopping results of CLASH dataset 802, a sloping foreshore with a seawall (Goda *et al*., 1975).

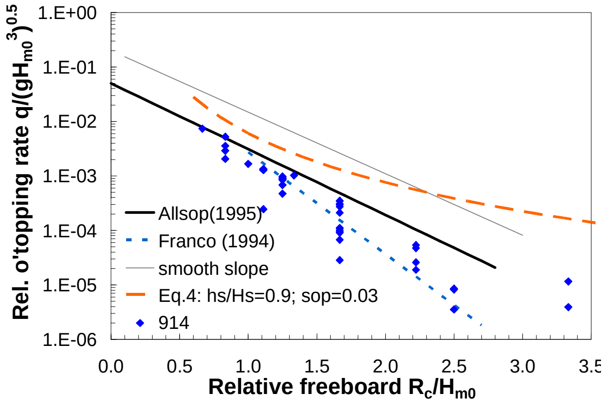

Figure 2. Overtopping results of CLASH dataset 914, a vertical wall at deep water (Cornett *et al*., 1999).

Vertical structures without foreshore

Figure 3 shows all results for tests without a sloping foreshore. Tests of dataset 113 with R_c = 0 have been shifted artificially a little to the right in order to distinguish them from dataset 107 with R_c = 0. For the lower freeboards / larger overtopping rates, the scatter is small. The scatter becomes larger for R_c/H_{m0} > 1. It turns out that Franco *et al.* (1994) describes these smaller overtopping discharges very well. Figure 3 shows that Franco *et al.* (1994) will over-predict overtopping for lower freeboards. However, the other line, Allsop *et al*. (1995), covers this area well. It means that both formulae are valid for vertical structures without a sloping foreshore, but each with their own range of application.

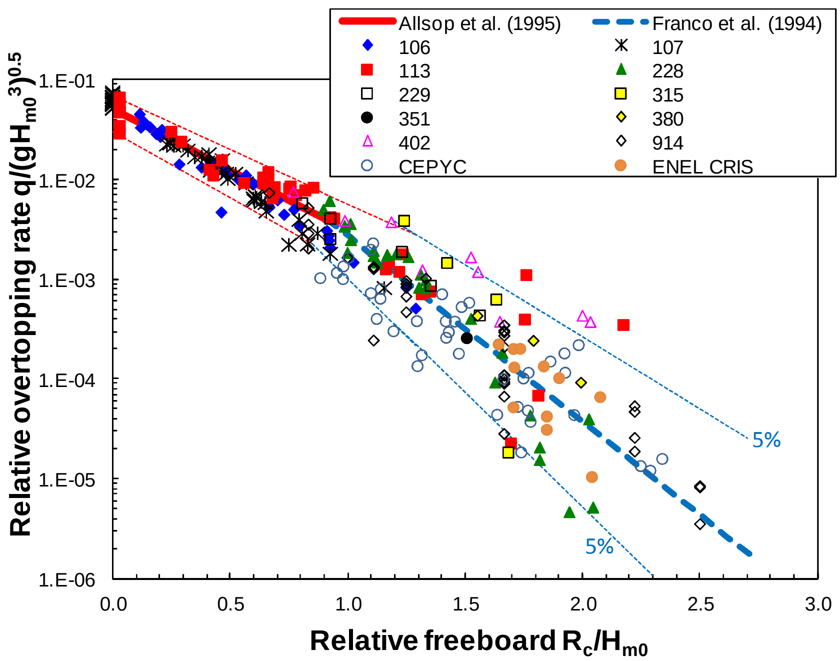

Figure 3. Vertical structures on relatively deep water, no sloping foreshore.

Structures in Figure 3 can be described as caissons, vertical flood walls in harbours, and gates of locks in flood situations. They may have a berm type structure relatively deep below water, which does not affect overtopping. The description of wave overtopping is then given by:

For R_c/H_{m0} < 0.91: Allsop *et al.* (1995):

$$\frac{q}{\sqrt{gH_{m0}^3}} = 0.05\exp\left(-2.78\frac{R_c}{H_{m0}}\right) \quad (5)$$

For R_c/H_{m0} > 0.91: Franco *et al.* (1994):

$$\frac{q}{\sqrt{gH_{m0}^3}} = 0.2\exp\left(-4.3\frac{R_c}{H_{m0}}\right) \qquad (6)$$

The reliability of Equation 5 is given by σ(2.78) = 0.17; that of Equation 6 by σ(4.3) = 0.6.

Vertical seawalls on sloping foreshore

All available CLASH datasets for vertical walls with a foreshore were cases with a simple, single-gradient foreshore slope. For all, the wave height was taken at the location of the vertical wall. First $h^2/(H_{m0}\,L_{m-1,0})$ = 0.23 was used as a discriminator between deflecting or non-impulsive and impulsive wave conditions. Note that this is approximately equivalent to h_* = 0.3, due to the factor 1.3 in Equation 1. Data with $h^2/(H_{m0}L_{m-1,0})$ < 0.23 are plotted in Figure 4. Note that the non-dimensionalisation of *q* used for the y-axis of Figure 4 is a generalised form of that arising from the manipulation by Bruce & Van der Meer (2008).

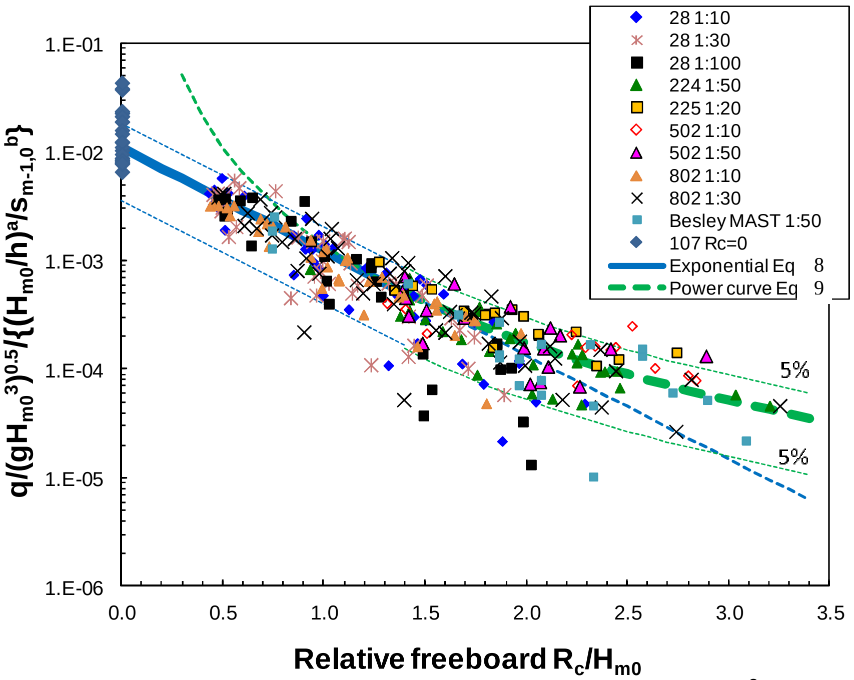

Figure 4. All data of seawalls on sloping foreshore for impulsive waves ($h^2/(H_{m0}L_{m-1,0})$ < 0.23) and with optimum values of a = 0.5 and b=-0.5. The fine dashed lines indicate 5% under and upper exceedance limits. The upper-left dotted line represents the extension of the power law (Equation 9) to freeboards below the appropriate range, while the dotted line in the extreme lower-right shows the extension of the exponential (Equation 8) to freeboards higher than the appropriate range.

An optimum was sought for the best value of $h^2/(H_{m0}L_{m-1,0})$ as discriminator as well as the best parameter group on the vertical axis in Figure 4, by changing the exponents *a* and *b* in the expression in Equation 7:

$$\text{impulsive / non-impulsive discriminator} = \frac{q}{\sqrt{gH_{m0}^3}} / \left(\frac{H_{m0}}{h}\right)^a s_{m-1,0}^b \qquad (7)$$

In Equation 4, the values are *a* = 0.5 and *b* = -1.0. This gives quite a large influence of the wave period on wave overtopping, where there is little or no such influence observed on steep slopes and at vertical walls in deep water. By analysing the exponents *a* and *b* in Equation 7, the conclusion was drawn that *a* = 0.5 and *b* = -0.5 were good values, showing least scatter and a little less influence of the wave steepness. This gives on the vertical axis the parameter group $\{q/(gH_{m0}^3)^{0.5}\}$ / $\{H_{m0}/(h\, s_{m-1,0})^{0.5}\}$. Analysis confirmed that $h^2/(H_{m0}L_{m-1,0})$ = 0.23 was indeed the optimum value to discriminate between non-impulsive and impulsive waves, validating the earlier value of h_* = 0.3.

The results for non-impulsive waves ($h^2/(H_{m0}L_{m-1,0}) > 0.23$) on vertical seawalls at the end of a sloping foreshore show that Allsop *et al.* (1995) describes the wave overtopping for these kinds of structures and for given wave conditions very well.

The remaining data for impulsive wave attack are given in Figure 4. Dataset 107 for deep water and zero freeboard were also given for comparison as no data were available for zero freeboard. There is quite some scatter below the average trend of the data and almost all of that data belong to dataset 28. But the other data give a trend of a straight line starting from zero freeboard to rather large relative freeboards (R_c/H_{m0} ~ 1.5 – 2) and then becomes a more horizontal trend for very large freeboards. Actually, such a more or less horizontal line goes on even beyond relative freeboards of R_c/H_{m0} = 3 – 5. In this region, a power curve like Equation 4 fits well (Figure 4). From that point of view, there is no reason to abandon these kind of formulae, which originate from Allsop *et al.* (1995). It is however clear that a power function cannot give the trend for small or zero freeboards as it will use the vertical axis as an asymptote. This is also clearly shown in Figure 4 with the dashed line. It is for this reason that it was decided to keep the power function for larger freeboards and to introduce the common exponential function for zero and low freeboards. The formulae are described by:

$$\frac{q}{\sqrt{gH_{m0}^3}} = 0.011\left(\frac{H_{m0}}{h\,s_{m-1,0}}\right)^{0.5} \exp\left(-2.2\frac{R_c}{H_{m0}}\right) \quad \text{for } R_c/H_{m0} < 1.35 \text{ and} \qquad (8)$$

$$\frac{q}{\sqrt{gH_{m0}^3}} = 0.0014\left(\frac{H_{m0}}{h\,s_{m-1,0}}\right)^{0.5} \left(\frac{R_c}{H_{m0}}\right)^{-3} \quad \text{for } R_c/H_{m0} > 1.35 \qquad (9)$$

The reliability of Equation 8 is given by σ(0.011) = 0.0045 and that of Equation 9 by σ(0.0014) = 0.0006.

Wave overtopping at vertical walls is thus now given by Equations 5 and 6 (relatively deep water, no sloping foreshore); Equation 5 (sloping foreshore, non-impulsive waves) and Equations 8 and 9 (sloping foreshore with impulsive waves).

Composite Vertical Structures

For impulsive conditions at composite vertical structures, EurOtop (2007) gives:

$$\frac{q}{d_*^2\sqrt{gd^3}} = 4.1\times10^{-4}\left(d_*\frac{R_c}{H_{m0}}\right)^{-2.9} \qquad (10)$$

where

$$d_* \equiv 1.3\frac{d}{H_{m0}}\frac{2\pi h}{gT_{m-1,0}^2} \qquad (11)$$

and d is the water depth above the berm. In the same way that the h_* parameter (Equation 3) is used in the EurOtop (2007) method as a discriminator between impulsive and non-impulsive conditions at a plain vertical wall, so d_* discriminates between two sets of formulae for composite vertical structures.

As for vertical walls, it is not straightforwardly possible to analyse in a generic way the differences between impulsive and non-impulsive forms, and even harder to get a sense of the physical transition.

Bruce and Van der Meer (2008) set the exponent in Equation 10 to –3 and then rearranged the relation to:

$$\frac{q}{\sqrt{gH_{m0}^3}} = b\left(\frac{d}{h}\right)^{0.5}\left(\frac{H_{m0}}{h}\right)^{0.5}\frac{1}{\left(s_{m-1,0}\right)^a}\left(\frac{R_c}{H_{m0}}\right)^{-3} \qquad (12)$$

with a = 1 and b = 0.00041. Note that there is a typographical error in Bruce and Van der Meer (2008). Equation 12 is the corrected version of Equation 12 in that paper. The vertical wall re-analysis of the preceding section found that the influence of steepness was better represented by $s_{m-1,0}^{0.5,}$ *i.e.* by setting a = 0.5 in Equation 12. The similarity of the physical situation suggests this adjustment be included for the composite structures too, giving a tentative prediction equation (Equation 13).

$$\frac{q}{\sqrt{gH_{m0}^3}} = b\left(\frac{d}{h}\right)^{0.5}\left(\frac{H_{m0}}{h\,s_{m-1,0}}\right)^{0.5}\left(\frac{R_c}{H_{m0}}\right)^{-3} \qquad (13)$$

The apparently separate formulations for plain and composite vertical structures have thus been reduced to a single set, with the difference between Equations 9 and 13 being the value of b and a factor of $(d / h)^{0.5}$ which becomes unity for plain vertical walls with zero berm height ($h = d$).

Before an enhanced prediction scheme can be proposed, a number of further issues required exploration, based upon the CLASH database data. Composite structures were identified by vertical upper slopes ($\cot \alpha_u = 0$), and by the presence of a toe or mound, *i.e.* where the water depth at the toe or berm is less than that offshore.

The constant multiplier (b) in Equation 13 is not the same as that for plain vertical walls (Equation 9), so for no berm (d/h=1), these do not converge as they should. Comparing new Equations 4 (for plain vertical) and 14 (composite), it is apparent that the two predictors coincide at a value of $d/h \approx 0.6$, suggesting that the mound's influence should cease for $d > 0.6\,h$, which seems physically sensible.

Does **the value of the discriminating parameter** $d_* = 0.3$ for transition between impulsive and non-impulsive regimes remain optimal when applied to the wider CLASH dataset? EurOtop (2007) gives the discriminator $d_* < 0.3$ for impulsive conditions. This criterion was applied to all CLASH database data for composite structures, with separate plots being made for conditions predicted to be impulsive or non-impulsive. The data identified as impulsive was well-predicted. For data predicted to be in the non-impulsive regime, it was apparent, however that there was a group of data at higher freeboards which was significantly under-predicted. The under-predicted data (Figure 5) belongs to set 505 for which there was a 1:10 foreshore present. Before considering this influence however, the $d_* = 0.3$ cross-over was tested. Moving from $d_* = 0.3$ to 0.85 as the critical value, the performance of the scheme improves. Mean error goes from 3.15 to 0.87, and geometric error (measuring the standard deviation of the scatter about the mean of the logarithm of the data) from 0.47 to 0.38, indicating average success in the range $\times/\div$ 2.4, improved from $\times/\div$ 3.0 with the previous, lower value of d_*.

Data that remains significantly over-predicted includes many data from sets 228 and 914, neither of which had foreshores. Does the presence or absence of a **foreshore influence** overtopping at composite structures as was shown for vertical structures in the earlier analysis? Adoption of adjusted forms of the new vertical wall procedures was then explored. In addition to the advantage of consistency of approach, such a switch would also bring the physically sensible behaviour at lowest freeboards to the analysis of overtopping of composite walls. The new vertical wall prediction scheme was then applied, adjusted to composite structures by application of a correction factor of $1.3 \times (d/h)^{0.5}$ for all $d/h < 0.6$. The multiplier of 1.3 allows composite and vertical formulae to coincide at $d/h = 0.6$ (Equations 14 and 15):

$$\frac{q}{\sqrt{gH_{m0}^3}} = 1.3\left(\frac{d}{h}\right)^{0.5} \times 0.011\left(\frac{H_{m0}}{h}\right)^{0.5}\left(\frac{1}{s_{m-1,0}}\right)^{0.5} \exp\left(-2.2\frac{R_c}{H_{m0}}\right) \quad \text{for } R_c/H_{m0} < 1.35 \qquad (14)$$

$$\frac{q}{\sqrt{gH_{m0}^3}} = 1.3\left(\frac{d}{h}\right)^{0.5} \times 0.0014\left(\frac{H_{m0}}{h}\right)^{0.5}\left(\frac{1}{s_{m-1,0}}\right)^{0.5}\left(\frac{R_c}{H_{m0}}\right)^{-3} \quad \text{for } R_c/H_{m0} \geq 1.35 \qquad (15)$$

The composite wall data, excluding those with zero freeboard, are plotted with Equations 14 and 15 in Figure 5. The geometric error is 0.39. As noted above, the exponent of d/h was set at 0.5 – an influence of $(d/h)^{0.5}$ – on the basis of the algebraic manipulation of the EurOtop formulation, after Allsop *et al.*'s (1995) equation. Exploring alternative exponents demonstrated that the 0.5 value is optimal.

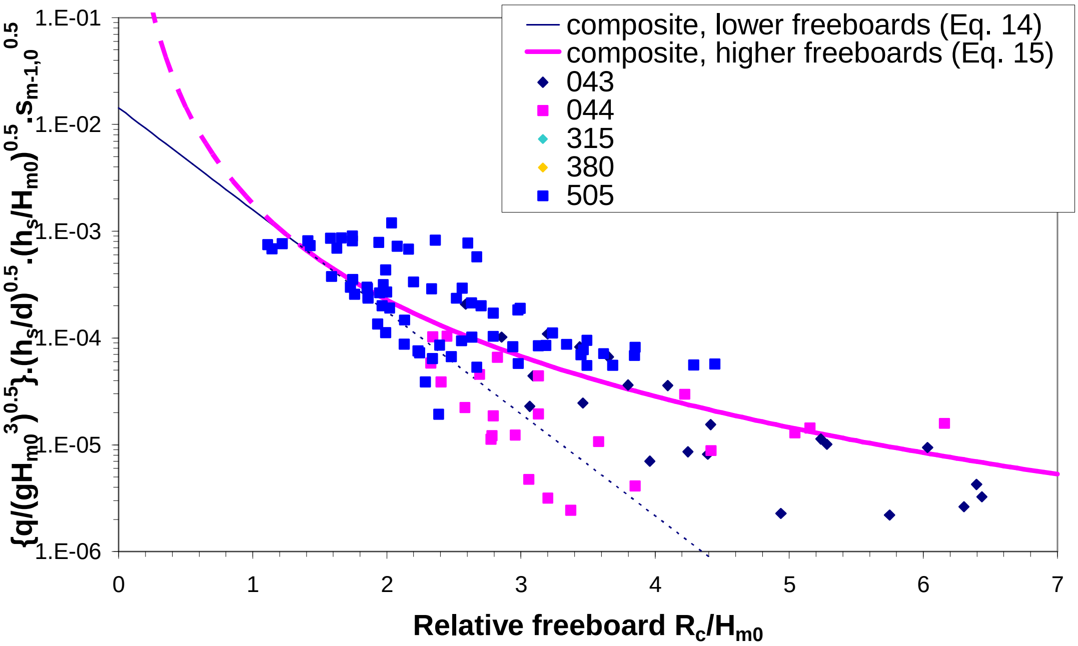

Figure 5. Comparison of all CLASH database tests for composite structures with foreshore with new scheme for composite structures, based upon new vertical wall approach, adjusted d_* cut-off; applied for all d / h_s < 0.6.

The influence of the berm is indicated in Figure 6. From this, it can be seen that under conditions established as “impulsive”, the berm’s influence is to reduce overtopping discharges. The scale of the influence is not that great, however – of the same orders of magnitude as the influence of wave steepness and relative depth on impulsive overtopping at plain vertical walls.

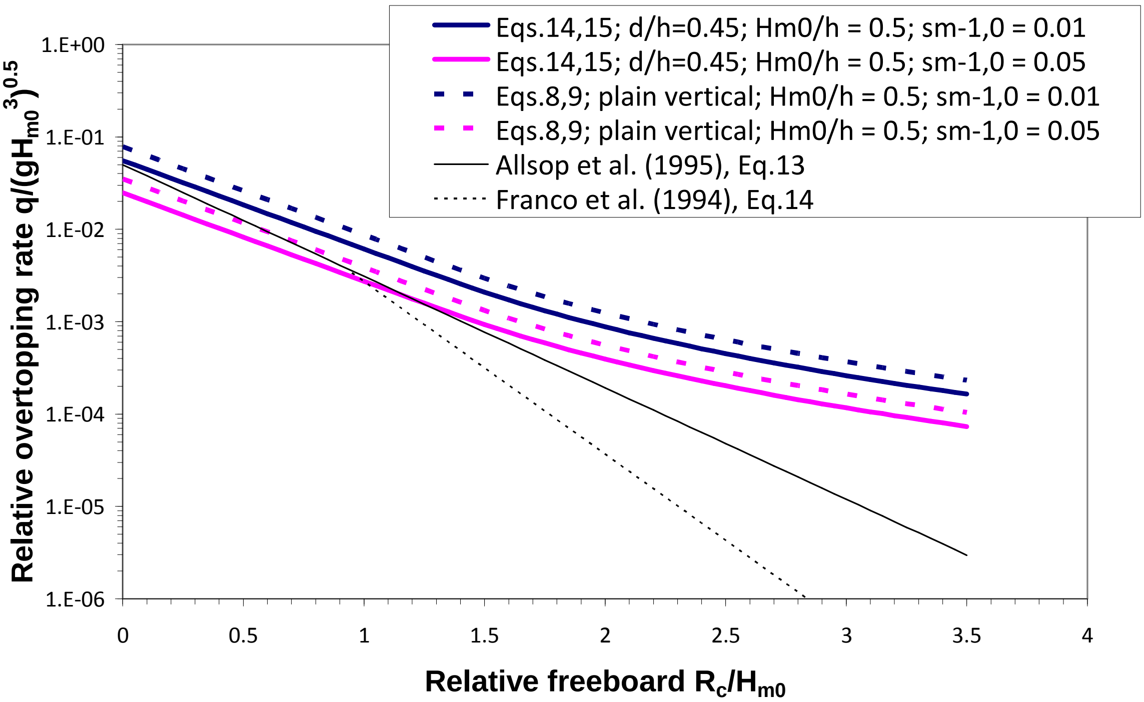

Figure 6. Comparison of overtopping at composite and plain vertical structures, for non-impulsive/deflecting waves as well as for impulsive waves .

The scheme for composite structures is thus now aligned with the improved vertical scheme giving physically rational behaviour at lowest freeboards (which was not the case for the previous, power-law-only scheme). In summary, overtopping at composite structures may be considered according to the right side of the decision chart (Figure 7). In cases where the mound is small ($d/h > 0.6$), the structure is treated as vertical. For $d/h \geq 0.6$, in the absence of a foreshore and possible breaking, the structure is again treated as plain vertical. In the case of possible breaking, however, the overtopping is predicted according to the method for plain walls, but with a factor of $1.3 \times (d/h)^{0.5}$ included.

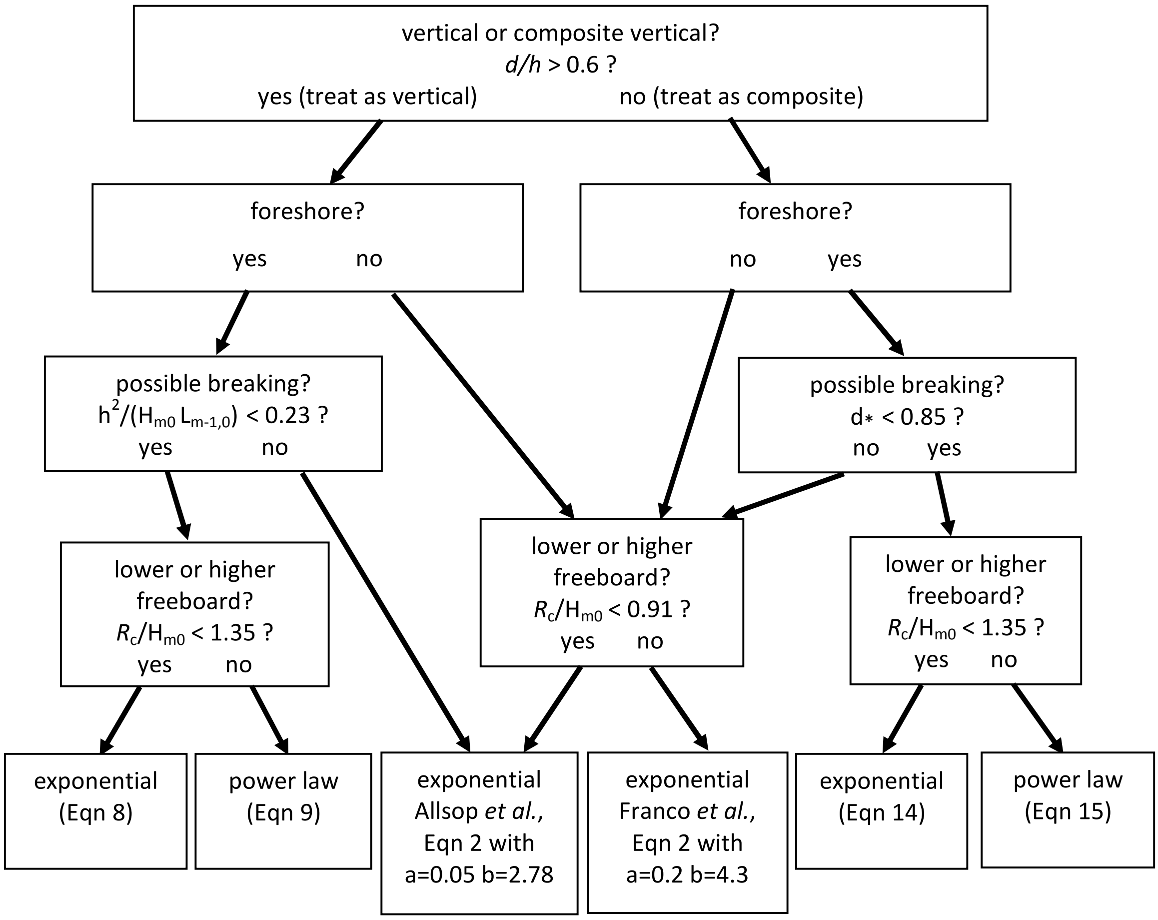

Figure 7. Decision chart showing new schemes; vertical to left; composite to right.

Conclusions

Vertical structures should be divided in structures at relatively deep water without a sloping foreshore and seawalls at the end of such a sloping foreshore. The first type of structure describes caissons, flood walls in harbours and gates in locks at high water. The Franco *et al.* (1994) formula, Equation 6, is valid in this situation for larger freeboard, where the Allsop *et al.* (1995) formula, Equation 5, is valid from zero freeboard until the crossing with the Franco formula. The discriminator $h^2/(H_{m0}L_{m-1,0})$ divides wave overtopping at a seawall on a sloping foreshore in non-impulsive or deflecting waves with $h^2/(H_{m0}L_{m-1,0}) > 0.23$, where the Allsop *et al.* (1995) gives the predicting formulae. For smaller values of the discriminator impulsive waves give larger wave overtopping and sometimes even significant wave overtopping for large freeboards, see Figure 9. Equations 8 and 9 describe overtopping under impulsive wave conditions. Guidance how to apply the correct formulae is given in Figure 7.

Overtopping at composite structures can be analysed according to a close analogue of this new scheme for plain vertical structures. Adjustments are applied for berms higher than $d/h_s < 0.6$. The adjustment is simply a factor of 1.3 $(d/h_s)^{0.5}$. A “decision chart” summary of the proposed, unified scheme for plain vertical and composite structures is presented (Figure 7).

It is recommended that an update of EurOtop (2007) be considered, to include the new insights and design formulae found in this paper for practical use.

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References
Allsop, N.W.H., Besley, P. and Madurini, –. (1995). Overtopping performance of vertical walls and composite breakwaters, seawalls and low reflection alternatives. Final report of Monolithic Coastal Structures (MCS) project, University of Hannover

Bruce, T. and van der Meer, J.W. (2008), Integrated prediction tools for wave overtopping at vertical structures, Proc. 31st Int. Conf. on Coastal Eng., pp3110-3119, World Scientific (ASCE)

Bruce, T., Allsop, N.W.H. and Pearson, J. (2001). Violent overtopping of seawalls – extended prediction methods, Proc. Breakwaters, Coastal Structures and Coastlines, pp.245–256, Thomas Telford

Cornett, A., Li, Y. and Budvietas, A. (1999). Wave overtopping at chamfered and overhanging vertical structures. Proc. Int. Workshop on Natural Disasters by Storm Waves and Their Reproduction in Experimental Basins, Kyoto, Japan.

De Waal, J.P. (1994). Wave overtopping of vertical structures. Influence of wave breaking and wind. Delft Hydraulics, report on physical model tests and desk study, H1635.

EA/Besley, P (1999). Overtopping of seawalls – design and assessment manual. R & D Technical Report W 178, Environment Agency, Bristol, ISBN 1 85705 069 X

EurOtop (2007). European Manual for the Assessment of Wave Overtopping. Eds. Pullen, T. Allsop, N.W.H. Bruce, T., Kortenhaus, A., Schüttrumpf, H. and van der Meer, J.W. Available from www.overtopping-manual.com

Franco, L., de Gerloni, M. and van der Meer, J.W. (1994). Wave overtopping on vertical and composite breakwaters. Proc. 24th Int. Conf. on Coastal Engineering, pp.1030−1045 (ASCE)

Goda, Y. (2000). Random seas and design of maritime structures (2nd edition). World Scientific Publishing, Singapore, ISBN 981-02-3256-X

Goda, Y. (2009). Derivation of unified wave overtopping formulas for seawalls with smooth, impermeable surfaces based on selected CLASH datasets. Coastal Engineering 56, pp. 385–399

Goda, Y., Kishira, Y. and Kamiyama, Y. (1975). Laboratory investigation on the overtopping rates of seawalls by irregular waves. Ports and Harbour Research Institute, 14, 4, pp.3–44, Yokosuka, Japan

Herbert D.M. (1993). Wave overtopping of vertical walls. Hydraulics Research Wallingford, Report R316.

Oumeraci, H., Kortenhaus, A. and Haupt, R. (2001). Investigations on reductions of wave overtopping over vertical walls by parapets (Untersuchung zur Abminderung des Wellenüberlaufs bei senkrechten Wänden durch Wellenabweiser), Bericht Nr.865, TU Braunschweig (in German)

Pearson, J., Bruce, T., Allsop, N.W.H. and Gironella, X. (2002). Violent wave overtopping - measurements at large and small scale, Proc. 28th Int. Conf. Coastal Engineering, pp.2227–2238 (ASCE)

Pullen, T., Allsop, N.W.H., Bruce, T., Pearson, J. and Geeraerts, J. (2004). Violent wave overtopping at Samphire Hoe: field and laboratory measurements. Proc. 29th Int. Conf. Coastal Engineering. pp.4379–4390 (ASCE) World Scientific, Singapore, ISBN 981-256-298-2

Schüttrumpf, H. and Oumeraci, H. (2005). Layer thicknesses and velocities of wave overtopping flow at seadikes. Coastal Engineering, 52, 6, pp.473–495.

Smid, R.,Schüttrumpf, H. and Möller, J. (2001).). Investigations on determining the mean wave overtopping ratio at a vertical wall and a 1:1.5 sloped seawall with and without freeboard (Untersuchungen zur Ermittlung der mittleren Wellenüberlaufrate an einer senkrechten Wand und einer 1:1,5 geneigten Böschung für Versuche mit und ohne Freibord), Leichtweiβ Institute for Hydraulics, TU Braunschweig, Germany (in German)

Steendam, G.J., van der Meer, J.W.,Verhaeghe, H., Besley, P., Franco, L. and van Gent, M.R.A. (2004). The international database on wave overtopping. Proc. 29th Int. Conf. Coastal Engineering, pp. 4301–4313 (ASCE). World Scientific, Singapore, ISBN 981-256-298-2

TAW (1985). Leidraad voor het ontwerpen van rivierdijken. Deel 1 - Bovenrivierengebied. "Guideline for design of river dikes. Part 1 - Upper area". Technical Advisory Committee on Water Defences (in Dutch)

TAW (1989). Leidraad voor het ontwerpen van rivierdijken. Deel 2 - Benedenrivierengebied. "Guideline for design of river dikes. Part 2 - Lower area". Technical Advisory Committee on Water Defences (in Dutch)

Van der Meer, J.W., Verhaeghe, H. and Steendam , G.J.(2008). The new wave overtopping database for coastal structures. Coastal Engineering, 56, pp.108–120

Victor, L. (2012) Optimaization of the Hydrodynamic Performance of Overtopping Wave Energy Converters: Experimental Study of Optimal Geometry and Probability Distribution of Overtopping Waves. PhD-thesis, Ghent University. ISBN 978-90-8578-478-4.

Victor, L., van der Meer, J.W. and Troch, P. (2012). Probability distribution of individual wave overtopping volumes for smooth impermeable steep slopes with low crest freeboards. Coastal Engineering, 64, pp.87–101.