Wave and overtopping predictions on reservoirs and inland waterways

Tim Pullen[1], William Allsop[1] , Eunice Silva[1], Craig Goff[1] and Tracey Williamson[2],

[1] *HR Wallingford, Howbery Park, Wallingford, Oxfordshire OX10 8BA, United Kingdom*

[2] *Ove Arup and Partners*

Abstract

In their 2010 paper, Allsop et al (2010) sought to bring together the latest methodologies for predicting waves and wave overtopping for the dams and reservoirs community and modernise the extant approach. In particular, they argued that the reservoir community should move away from the out-dated wave-run up and freeboard allowance approaches and embrace better-validated approaches that limit overtopping, as used by the coastal community. This followed publication of the EurOtop overtopping manual (Pullen et al, 2007), and included many of the methods discussed there.

Since the original publication of EurOtop, there have been several improvements to the principal prediction methods and also to methods to improve predictions in cases where the crest includes a promenade and/or a wall that may include a bull-nose. These updates are especially pertinent with regard to bank full reservoirs or where there is a parapet wall at the crest of the embankment. As such, this paper provides an update to Allsop et al (2010) to incorporate the new methods in EurOtop II (van der Meer et al, 2016).

1 Introduction

For many reservoirs and inland waterways extreme meteorological conditions can drive high winds across the water generating wave action. These waves can then result in wave overtopping, and these phenomena are the primary subject of this paper.

The paper describes recent advances in the prediction of wave overtopping, drawing particularly on the recently updated EurOtop Overtopping Manual (van der Meer, 2016). It reviews the method for predicting waves (derived from ICE (1996) and BSI (1991) guidance), the new methods for low / zero freeboard and, modifiers for promenades and / or wave walls with / without bullnoses.

This paper seeks principally, to introduce the updated methods and show how they can be combined to predict discharges at reservoirs and inland waterways with decreased levels of uncertainty when compared to earlier predictions. It is by no means comprehensive and can only give an outline in the space provided. The reader is therefore advised to seek further guidance in the references given here. Moreover, EurOtop II covers many aspects of overtopping including additional methods for vertical, compound and armoured structures. This paper, however, is concerned primarily with sloping embankments, as these are both typical for reservoirs and inland waterways, and exclusively use the updated methods to be discussed here.

2 Waves on reservoirs

This paper is not primarily concerned with wave prediction, but it is important to introduce the processes of wave generation as an introduction to methods to predict wave overtopping of reservoir structures.

Even relatively mild wind action (say U10 < 10m/s) over open water will quickly cause small waves to develop. Increasing wind speeds will then drive the growth of larger waves. These will spread out over a range of directions either side of the mean wave direction, generating short-crested waves.

The size (wave height), and the length (or period), of these waves will depend on the wind speed and the length of water over which the wind can act (the fetch).

Over very long fetches in open oceans, wave heights and periods may be limited by the duration of the wind. On inland lakes or reservoirs, wave heights and periods will be limited by the fetch length (so not by duration), and by the maximum (time-averaged) wind speed. In most UK reservoirs, a steady wind of duration of no more than about 15-30 minutes will be sufficient to raise the wave condition to equilibrium.

For the prediction of wave overtopping on reservoir structures, wave conditions will be needed at a number of return periods, perhaps 10 times a year, 1:1 year, perhaps even 1:10 years for "frequent" conditions; then 1:100, 1:200, or Probable Maximum Flood (PMF). The procedures for predicting wave conditions are detailed below and will be similar at each return period, but some aspects may vary depending on the reservoir shape, and reservoir orientation with respect to the predominant wind direction. Where wave conditions are to be used to dimension armour protection on the upstream face, wave conditions should be calculated at a number of different water levels, taking account of appropriate joint probabilities of wave and water level to give the desired total return period.

2.1 Wave fetches

The first steps in making any simple prediction of waves on a given water area is to calculate the fetch lengths / areas over which the wind can act, and to predict extreme wind speeds and directions. The classical approach is to draw out fetch lengths at given direction increments across the reservoir from the point of interest (usually the centre of the dam) out across the open water area to the opposite shore. For reservoirs that are relatively circular (as opposed to elongated), fetch rays will probably be drawn at 30° increments as that best corresponds to a realistic resolution for predicted wind directions. For elongated reservoirs, where waves at the dam originate over a narrow range of fetches, or those where shoreline features require a smaller increment, the closer spacing of fetch rays may be used, perhaps down to 7.5°, see Figure 1. For reservoirs with long embankments, or others with wave-sensitive structures at more than one position around the reservoir, fetches may be derived for a number of different prediction points.

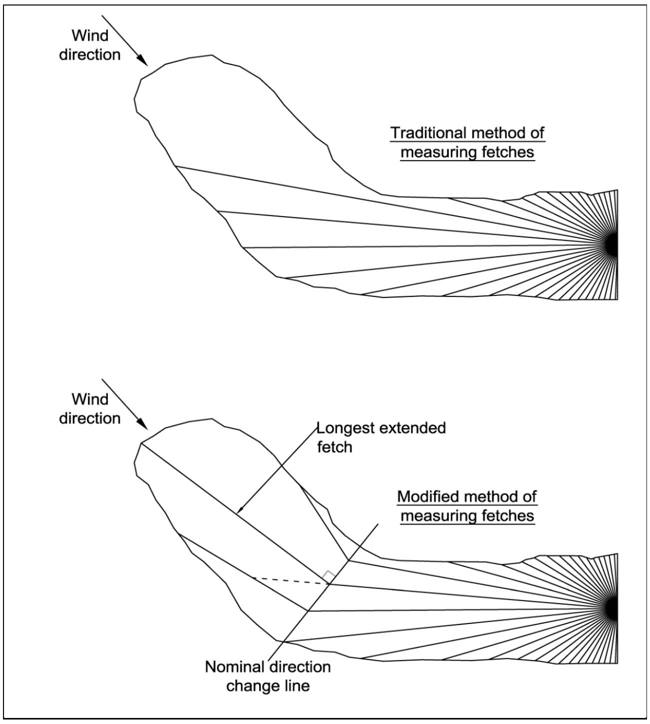


Figure 1: Example fetch rays on a "banana" shaped reservoir

2.2 Wind speeds

Wind data from a reliable source are required to predict extreme wind and wave conditions. Historically, data on wind speeds / directions have often been obtained from the Meteorological Office for the most suitable weather station, and these data have been used for extrapolation to longer return periods. Predicted extreme wind speeds made using data from "standard" sites will be far more robust if scaled by measurements at the reservoir site over (say) 2 years.

For initial work and/or for studies on less important structures, methods are available however to predict a design wind speed starting with a basic wind speed, generally as suggested by ICE (1996) and BSI (1991). For simple predictions, the design wind speeds, U, can be derived from generic guidance as in Figure 2, with the "basic wind speeds" adjusted by factors for altitude (S_a); direction (S_d); seasonality (S_s); and probability / return period (S_p). Appropriate values for these coefficients have been suggested previously by Yarde et al, 1996, or may be given by appropriate wind codes Eurocode 1, 1991. An "over-water speed-up" factor may be applied to compensate for the reduced roughness of the water surface relative to agricultural land. These factors will be used to give representative wind speeds at each of the wind directions of interest, and at each return period of concern.

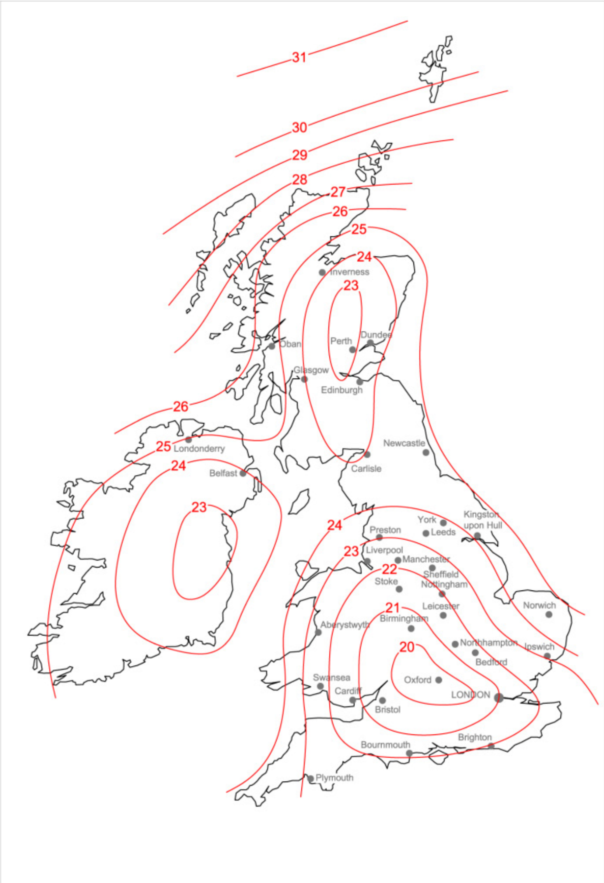


Figure 2: Basic (1:50 year) wind speed as given by ICE

2.3 Empirical wave predictions

Most wave prediction methods have been derived for ocean or coastal water conditions, but may later have been adapted for limited fetch lengths. In the 1980s, HR Wallingford analysed winds and waves measured on example reservoirs concluding that SMB / Saville and Donelan / JONSWAP methods were most appropriate for wave height prediction in reservoirs (Owen and Steele, 1998 and Owen 1987). The simple prediction method recommended by ICE (1996) used a simplified Donelan / JONSWAP method to predict significant wave heights (H_s) and peak periods (T_p) from fetch distances (F) and design wind speed (U). A more detailed analysis showed that wave heights could be underpredicted, but Owen & Steele suggested that a partial safety factor of γ_U = 1.05 on wind speeds would cover these uncertainties in design use.

This method is applied for each wind direction sector, usually at 30° increments. For UK reservoirs with the long axis facing W or SW, the maximum wave condition will probably be given by the predominant wind direction, but for other alignments, it may not be obvious whether the direction of longest fetch, or the direction of strongest wind speed, will give the greatest wave heights, so appropriate sensitivity analysis is recommended.

2.4 Calculation of wave condition

Taking into account each of the adjustments above, the wind speed (U) required to predict wave conditions should be calculated by multiplying the 1:50 year hourly wind speed derived from Figure 2 by each of the wind speed factors:

$$U = U_{50} S_a S_d S_S S_p$$

Wind generated waves on any open area of water contain a range of heights and periods. Such (random or irregular) waves may most usefully be described by the significant wave height, H_s (average of the highest one third of wave heights) and a mean wave period, T_m. The recommended method to estimate wave heights is the simplified Donelan / JONSWAP method for which:

$$H_s = 0.00178\, U\, (F/g)^{0.5}$$

where H_s is the significant wave height in metres, U is the required wind speed in m/s, and F is the fetch in metres. It should also be noted, that in coastal / ocean engineering there are at least three different definitions of and notations for significant wave height, including H_s, $H_{1/3}$ and H_{m0}. In inland reservoirs, however, these definitions are effectively the same, so no distinction is made within this paper, although alternative versions may be used in, for example, the EurOtop manual.

For wave action on UK reservoirs, the peak wave period (T_p) can be estimated from:

$$T_p = 0.0712\, F^{0.3}\, U^{0.4}$$

and the mean wave period ($T_{m-1,0}$) by $T_{m-1,0} = T_p / 1.1$. For reference, a plot of the simple relationship between fetch length, wind speed and H_s is presented in

Figure 3. The more complete method is described by Herbert et al (1995).

As a precaution, a known weakness of the simplified wave prediction methods when used over short fetches for very strong wind speeds (as would be expected for long return periods and elevated reservoirs) is to predict wave periods that are too short. This can be seen in wave steepness greater than $s_m = H_s/L_m > 0.077$, which would exceed the practical physical limit for wind waves. Again, the simplest solution has generally been to predict longer wave periods, although it is likely that more correct results would be given by wave generation / transformation models able to limit wave heights taking account of limiting wave steepnesses.

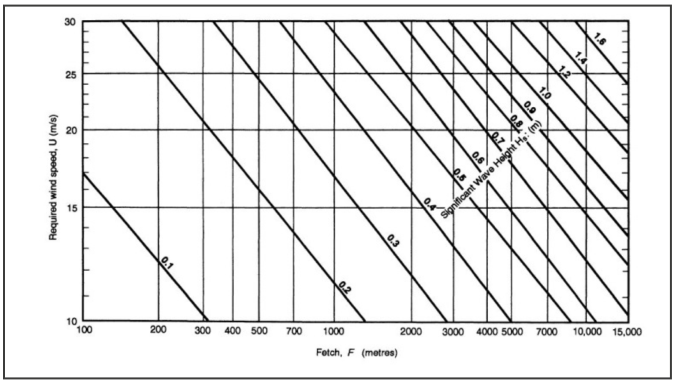


Figure 3: Simplified relationship between fetch length, wind speed and significant wave height

3 Wave overtopping

Overtopping prediction methods for a wide range of coastal and shoreline structures were substantially improved in 2007 by publication of the EurOtop Wave Overtopping Manual (Pullen et al, 2007). Since EurOtop (2007), new information was established on wave overtopping over very steep slopes up to vertical, on better formulae up to zero relative freeboard, and on a better understanding of wave overtopping over vertical structures. These updates were published in the second edition of EurOtop (van der Meer et al, 2016), and the relevant advances applicable to reservoirs are reviewed here.

3.1 Reservoir dam structures

For the purposes of overtopping, reservoir dams may be simplistically divided into: vertical (or steep) structures (comprising concrete and masonry dams); and sloping structures. The latter category includes earth embankments with (relatively) smooth faces and rough-faced rock armoured mounds / embankments. Some dams may be formed by more than one type of structure, see Figure 4.

Figure 4: Example reservoir structure comprising both sloping and vertical sections.

Dam structures with relatively smooth slopes are most common on embankments formed of sand and clay. The upstream face wave protection is commonly formed by concrete blocks or slabs, stone pitching, or by soil cement or asphaltic concrete, whilst grass is the normal surface to the downstream face. Occasionally, geotextiles or concrete blocks are used to reinforce the slope. Embankment dams may also be formed as rubble mounds armoured by rock armour or rip-rap, see Figure 5.

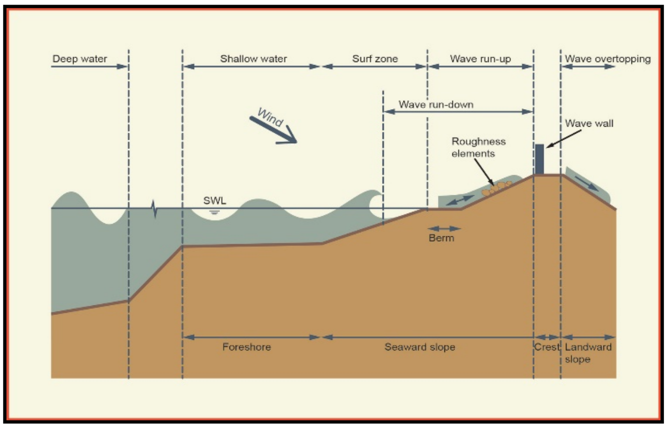
Figure 5: Rock armoured (asphaltic grouted rip-rap) embankment dam (Megget).

Some embankment structures may include a small wave wall at the crest, serving either to terminate the front face armouring, provide an edge to any crest roadway, and / or to reduce wave overtopping. Re-curved or bull-nosed wave walls may be very efficient, often reducing wave overtopping by 5 times or more, relative to a smooth slope to the same crest level.

3.2 Wave overtopping prediction

Prediction of overtopping for most shoreline defences has been advanced by development of empirical prediction methods for random waves on a range of idealised structure types. Of these methods, those for sloping dikes or embankments, see Figure 6, are certainly the most studied and probably the most well calibrated.

Figure 6: Example of a complex compound embankment structure.



The main equations for wave overtopping over sloping structures in EurOtop (2016) have been changed compared to EurOtop (2007). The improvement is specially in the area for very low freeboards including a zero freeboard (crest level equal to the water level). For sloping structures where the freeboard is at least half the wave height $R_c/H_{m0} > 0.5$, the differences between EurOtop (2007) and EurOtop (2016) formulae is quite small as shown in Figure 7. The main difference, and the one that is particularly important for our present discussion, is for conditions where $R_c/H_{m0} < 0.5$. The EurOtop (2007) method was not valid below $R_c/H_{m0} < 0.5$, but if extrapolated to $R_c/H_{m0} = 0$, it can be seen that it would overpredict by around a factor of five.

The principal formula used for wave overtopping is:

$$\frac{q}{\sqrt{gH_{m0}^3}} = a\exp\left[-\left(b\frac{R_c}{H_{m0}}\right)^c\right] \qquad \text{for } R_c \geq 0$$

This is a Weibull-shaped function with the dimensionless overtopping discharge $q/(gH_{m0}^3)^{½}$ and the relative crest freeboard R_c/H_{m0}. The EurOtop (2007) formulae for sloping structures used for the exponent c in the equation with a value of $c = 1$. The main differences in the updated method are the values of the coefficients a and b and the different value of c, and it can be seen in Figure 7, that with a value of $c = 1.3$ the line is curved on a log-linear graph giving a lower prediction at $R_c/H_{m0} = 0$. A more detailed discussion is given in Chapter 4 of the updated EurOtop (2016) manual.

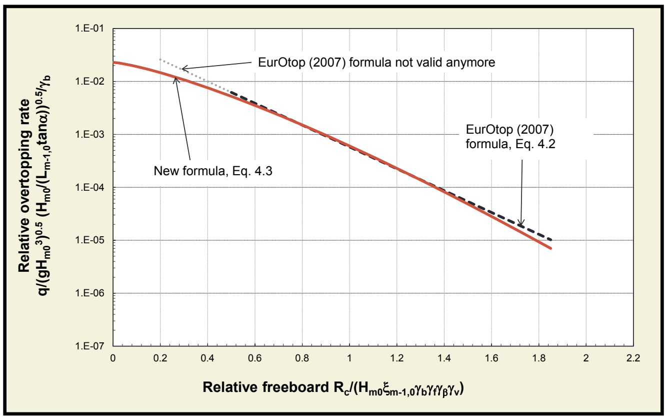


Figure 7: Comparison of EurOtop (2007) formula with the EurOtop (2016) formulae

3.3 Overtopping at dikes and embankments

The wave overtopping discharge can be described by two formulae, for a design or assessment approach, one for breaking waves on the slope and another for non-breaking waves.

Breaking

$$\frac{q}{\sqrt{g\cdot H_{m0}^3}} = \frac{0.026}{\sqrt{\tan\alpha}}\gamma_b\cdot\xi_{m-1,0}\cdot\exp\left[-\left(2.5\frac{R_c}{\xi_{m-1,0}\cdot H_{m0}\cdot\gamma_b\cdot\gamma_f\cdot\gamma_\beta\cdot\gamma_v}\right)^{1.3}\right]$$

Non-breaking

$$\frac{q}{\sqrt{g\cdot H_{m0}^3}} = 0.1035\cdot\exp\left[-\left(1.35\frac{R_c}{H_{m0}\cdot\gamma_f\cdot\gamma_\beta\cdot\gamma^*}\right)^{1.3}\right]$$

These equations may include a number of influence factors: γ_b is the influence factor for a berm, γ_f is the influence factor for roughness elements on a slope (see Table 1), γ_β is the influence factor for oblique wave attack. There is insufficient space here to describe each of these in any detail, so the reader is advised to refer to the discussions in the updated EurOtop (2016). In general though, when there are no influencing factors a value of 1.0 is used, and when the product of the γ factors is less than 1.0, the wave overtopping discharge will decrease.

The remaining factor is γ_v, which is the influence factor for a wall at the end of a slope. The main change compared to EurOtop (2007), is that an influence factor γ^* has been added for non-breaking waves for a wave wall on a slope or promenade. This influence factor γ^* is a combined factor dealing for different arrangements of wave walls and promenades, and to which we now turn our attention.

Table 1. Surface roughness factors for typical embankment revetments – Table 5.2 of EurOtop (2016)[0]

Reference type	γ_f
Concrete	1.0
Asphalt	1.0
Closed concrete blocks	1.0
Grass	1.0
Basalt, basalton	0.90
Placed revetment blocks (Haringman, Fixtone)	0.90

3.4 Wave walls and promenades

The two main changes from Allsop et al (2010) and EurOtop (2007) that are of particular concern here, are the improved prediction of the overtopping as R_c/H_{m0} approaches zero, and the new reduction factors contained within γ^* for wave walls and promenades. Again, a comprehensive appreciation can only be gained by reference to EurOtop (2016), but we can introduce here an overview of the methods and how they might be applied.

Essentially, there are five different arrangements of wave walls with / without a bullnose and / or promenades that can be considered. These are:

A wave wall at the top of the slope with no bullnose;

A wave wall at the top of the slope with a bullnose;

A promenade (G_c) with a wide slopping crest and no wave wall;

A wave wall at the rear of the promenade with no bullnose;

A wave wall at the rear of the promenade with a bullnose.

Though not exhaustively, these features are illustrated in the following figures. A gently sloping promenade with a plain wave wall is shown in Figure 8. A wave wall with a bullnose at the top of a slope is shown in Figure 9; note the rear promenade here is shown as horizontal to distinguish it from the sloping one in Figure 8. Finally, details of the bullnose and the λ function (used below) are shown in Figure 10.

Figure 8: Showing a wave wall with no bullnose at rear of a promenade

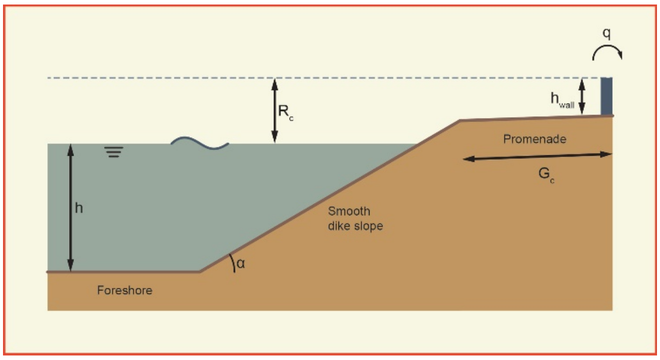


Figure 9: Showing a wave wall with bullnose at the top of a slope

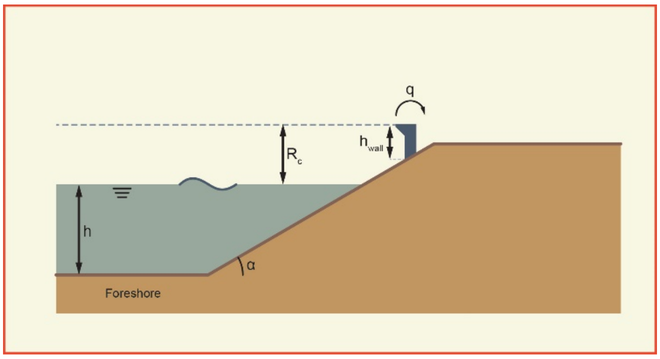
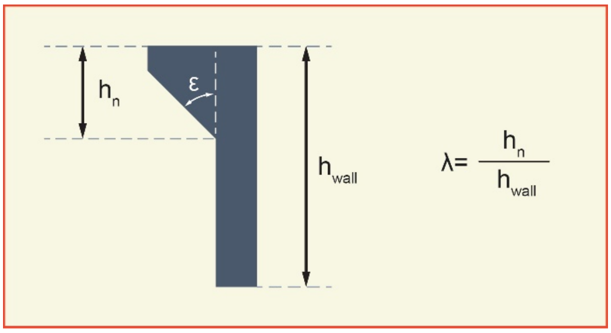


Figure 10: Detail of a bullnose



3.4.1 *Wave wall at the top of the slope with no bullnose*

For the first of the five cases, the discharge factor ($\gamma^* = \gamma_v$) is based primarily on the wave wall height (h_{wall}) with respect to that the freeboard (R_c), where γ_v can be found from the following:

$$\gamma_v = exp\left(-0.56\frac{h_{wall}}{R_c}\right)$$

This is valid over the range $h_{wall}/R_c = 0.08 - 1.00$.

3.4.2 *Wave wall at the top of the slope with a bullnose*

Where the wave wall includes a bullnose, the discharge factor $\gamma^* = \gamma_v \cdot \gamma_{bn}$ is used. There is an additional modification that is required for very long period waves ($\gamma_{s0,bn}$), but since this is not relevant to reservoirs it is not covered here. The values of γ_{bn}, depends on the angle ε and the position λ of the bullnose as shown in Figure 10 and is calculated based on the value of h_{wall}/R_c as follows:

For $h_{wall}/R_c \geq 0.25$,

$$\gamma_{bn} = 1.8\gamma_\varepsilon\gamma_\lambda$$

where:

$$\gamma_\varepsilon = 1.53\cdot 10^{-4}\varepsilon^2 - 1.63\cdot 10^{-2}\varepsilon + 1 \qquad \text{if } 15° \leq \varepsilon \leq 50$$

$$\gamma_\varepsilon = 0.56 \qquad \text{if } \varepsilon > 50$$

$$\gamma_\lambda = 0.75 - 0.20\lambda \qquad \text{if } 0.125 \leq \lambda \leq 0.6$$

and for $h_{wall}/R_c < 0.25$

$$\gamma_{bn} = 1.8\gamma_\varepsilon\gamma_\lambda - 0.53$$

where:

$$\gamma_\varepsilon = 1 - 0.003\varepsilon \qquad \text{if } 15 \leq \varepsilon \leq 60$$

$$\gamma_\lambda = 1 - 0.144\lambda \qquad \text{if } 0.1 \leq \lambda \leq 1$$

3.4.3 *Promenade with a wide slopping crest*

A wide crest can operate as a method to reduce overtopping provided that it slopes back towards the reservoir. For those situations, a reduction factor based on the width of the promenade (G_c) and the deep-water wavelength ($L_{m-1,0} = gT^2_{m-1,0}/2\pi$) can be used. For this case the reduction factor due to the promenade is $\gamma^* = \gamma_{prom}$, and is found as follows:

$$\gamma_{prom} = 1 - 0.47\frac{G_c}{L_{m-1,0}}$$

with the range of application for $G_c/L_{m-1,0} = 0.05 - 0.5$.

3.4.4 *Wave wall at the rear of a promenade with no bullnose*

For situations where there is a plain wave wall at the rear of a promenade, the reduction factor becomes $\gamma^* = \gamma_{prom\_v}$. This combines two previous factors with a modifier as follows:

$$\gamma_{prom\_v} = 0.87\gamma_{prom}\gamma_v$$

with the range of application given by $G_c/L_{m-1,0} = 0.05 - 0.4$ and $h_{wall}/R_c = 0.07 - 0.80$.

3.4.5 *Wave wall at the rear of a promenade with a bullnose*

For situations where a wave wall with bullnose is at the rear of a promenade, the influence factor $\gamma^* = \gamma_{prom\_v\_bn}$ is used. As with the method above, this combines two previous factors with a modifier as follows:

$$\gamma_{prom\_v\_bn} = 1.19\gamma_{prom\_v}\,\gamma_{bn}$$

Note that in this case, the modifier is greater than 1.0, and this reflects the fact that overtopping is likely to arrive at the wall as a bore wave, in which case the effectiveness of the bullnose feature is reduced when compared to a position at the top of a slope. The range of application is given by $G_c/L_{m-1,0} = 0.04 - 0.4$; $h_{wall}/R_c = 0.17 - 0.80$, $\varepsilon = 30°, 45°$; and $\lambda = 0.25 - 0.38$.

4 Concluding remarks

The paper by Allsop et al 2010, was directed at the Dams and Reservoirs community to update guidance and methodologies to be used for assessing wave overtopping discharges and the consequences of overtopping. Since that time, the fundamental method for predicting discharges has been updated, and methods have become available to assess the potential reductions in discharges for different geometrical arrangements of wave walls and promenades. Each of the updates discussed here are covered in more detail in EurOtop (2016), and the purpose of this paper has been to introduce the methods that can be found there for the assessment of overtopping at typical reservoirs and inland waterways.

5 Acknowledgements

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