Chapter 14

Prediction of Overtopping

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This chapter describes the processes of wave overtopping at sea defense and related coastal or shoreline structures. It introduces a range of methods to calculate mean overtopping discharges, individual and maximum overtopping volumes, and the proportion of waves overtopping a seawall. It describes the principal hazards from wave overtopping and will help engineers by suggesting limiting tolerable discharges for frequent, design, and extreme wave conditions. This chapter is supported by more detailed material in Chaps. 15 and 16 which focus on the methods to predict overtopping for rubble mound structures (with partly sloping embankments), and on vertical structures and battered walls. All of these three chapters have been based closely on the new *EurOtop* Overtopping Manual.[5]

14.1. Introduction

14.1.1. *Wave overtopping*

Wave overtopping has always been of principal concern for coastal structures constructed to defend against flooding: often termed sea defenses. Similar structures may also be used to provide protection against coastal erosion: sometimes termed coast protection. Other structures may be built to protect areas of water for ship navigation or mooring within ports, harbours, or marinas: often formed by breakwaters or moles. Within harbours, or along shorelines, reclaimed areas must be defended against both erosion and flooding. Some structures may be detached from the shoreline, often termed offshore, nearshore, or detached, but most structures used for sea defense or similar function form a part of the shoreline.

Sloping dikes have been widely used for sea defenses along the coasts of the Netherlands, Denmark, Germany, and the United Kingdom. Dikes or embankment seawalls are also used to defend low-lying areas in the Far East, including China, Korea, and Vietnam. Historically, dikes or embankment seawalls were built along many North Sea coastlines, sometimes subsuming an original sand dune line, protecting the land behind from flooding, and sometimes providing additional amenity value. Similar structures have been formed by clay materials or even from a vegetated shingle ridge, in both instances allowing the side slopes to be steeper. All such embankments need some degree of protection against direct wave erosion, often using a revetment facing on the seaward side (Fig. 14.1). Revetment facing may take many forms, but may commonly include closely-fitted concrete blockwork, cast *in situ* concrete slabs, or asphaltic materials. Embankment or dike structures are generally most common along rural frontages.

A second type of coastal structure consists of a mound or layers of quarried rock fill, protected by rock or concrete armour units (Fig. 14.2). The outer armour layer is designed to resist wave action without significant displacement of armour units. Under-layers of quarry or crushed rock support the armour and separate it from finer material in the embankment or mound. These porous and sloping layers dissipate a proportion of the incident wave energy in breaking and friction. Simplified forms of rubble mounds may be used for rubble seawalls or as protection to vertical walls or revetments. Rubble mound revetments may also be used to protect embankments

Fig. 14.1. Wave overtopping on a revetment seawall.

Fig. 14.2. Wave overtopping on a rubble mound breakwater.

formed from relict sand dunes or shingle ridges. Rubble mound structures tend to be more common in areas where harder rock is available.

Along urban frontages, especially close to ports, erosion or flooding defense structures may include vertical (or battered/steep) walls (Fig. 14.3). Such walls

Fig. 14.3. Wave overtopping on a battered/vertical seawall.

may be composed of stone or concrete blocks, mass concrete, or sheet steel piles. Typical vertical seawall structures may also act as retaining walls to material behind. Shaped and recurved wave return walls may be formed as walls in their own right, or smaller versions may be included in sloping structures. Some coastal structures are relatively impermeable to wave action. These include seawalls formed from blockwork or mass concrete, with vertical, near-vertical, or steeply sloping faces. Such structures may be liable to intense local wave impact pressures, may overtop suddenly and severely, and will reflect much of the incident wave energy. Reflected waves cause additional wave disturbance and/or may initiate or accelerate local bed scour.

It is worth noting that developments along waterfronts are highly valued with purchase or rental prices substantially above those for properties not on the waterfront. Yet, direct (or indirect) effects of wave overtopping have the potential to generate significant hazards to such developments and their users. Residential and commercial properties along a waterfront will often be used by people who may be unaware of the possibility, of the severity, or of the effects of wave overtopping in storm conditions. Regulatory authorities may therefore wish to impose onerous flood defense requirements on new developments. For instance, protection against flooding (including wave overtopping) for any new developments in the United Kingdom is now required to be 0.5% annual probability, equivalent to 1:200 year return. Exposure to overtopping of many coastal sites will, however, be influenced by climate change, probably increasing wave heights and periods as well as sea level rise.

14.1.2. *Predicting wave overtopping*

A number of different methods may be available to predict overtopping of particular structures (usually simplified sections) under given wave conditions and water levels. Each method will have strengths or weaknesses in different circumstances. In theory, an **analytical method** can be used to relate the driving process (waves) and the structure to the response through equations based directly on the knowledge of the physics of the process. It is, however, extremely rare for the structure, the waves, and the overtopping process to all be so simple and well-controlled that an analytical method on its own can give reliable predictions. Analytical methods are not therefore discussed further in this chapter.

The primary prediction methods are therefore based on **empirical methods** that relate the overtopping response to the main wave and structure parameters. These are by far the most commonly used methods to predict overtopping. Two other methods have been derived during the CLASH European project[3] based on the use of measured overtopping from model tests and field measurements. The first of these techniques uses the CLASH **database** of structures, waves, and overtopping discharges, with each test described by 31 parameters. Using the database is, however, potentially complicated, requiring some familiarity with these type of data. A simpler approach, and much more rapid, is to use the **Neural Network** tool that has been trained using the test results in the database. The Neural Network tool can be run automatically on a computer as a stand-alone device, or embedded within other simulation methods.

For situations for which empirical test data do not already exist, or where the methods above do not give reliable enough results, then two alternative methods may be used, but both are more complicated than the methods above. A range of **numerical models** can be used to simulate the process of overtopping. All such models involve some simplification of the overtopping process and are therefore limited to particular types of structure or types of wave exposure. They may, however, run sequences of waves giving overtopping (or not) on a wave-by-wave basis. Generally, numerical models require more skill and familiarity to run successfully. They will not be described in this chapter.

The final method to be mentioned is **physical modeling** in which a scale model is tested with correctly scaled wave conditions. Typically, such models may be built to a geometric scale in the range 1:10 to 1:60. Waves will be generated as random wave trains each conforming to a particular energy spectrum. The model may represent a structure cross section in a 2D model tested in a wave flume. Structures with more complex plan shapes, junctions, transitions, etc., may be tested in a 3D model in a wave basin. Physical models can be used to measure many different aspects of overtopping such as wave-by-wave volumes, overtopping velocities and depths, as well as other responses.

14.1.3. *Performance requirements*

Most sea defense structures are constructed primarily to limit overtopping volumes that might cause flooding. For defenses that protect people living, working, or

enjoying themselves, designers and owners of these defenses must, however, also deal with potential direct hazards from overtopping. This requires that the level of hazard and its probability of occurrence be assessed, allowing appropriate action plans to be devised to ameliorate risks arising from overtopping. Section 14.6 deals with tolerable wave overtopping.

14.2. Empirical Models, Including Comparison of Structures

14.2.1. *Mean overtopping discharge*

Empirical methods use a simplified representation of the physics of the process presented in (usually dimensionless) equations to relate the main response (overtopping discharge, etc.) to key wave and structure parameters. The form and coefficients of the equations are adjusted to reproduce results from physical model (or field) measurements of waves and overtopping. Empirical equations may be solved explicitly, or may occasionally require iterative methods to solve. Historically, some empirical methods have been presented graphically, although this is now very rare.

The mean overtopping discharge, q, is the main parameter in the overtopping process. It is, of course, not the only measure of overtopping, but it is relatively easy to measure in a laboratory wave flume or basin (or even in the field), and most other parameters are related in some way to this overtopping discharge. The overtopping discharge is generally calculated in m^3/s per m width, but in practical applications it may be quoted as liter/s per m width. Although it is given as a discharge, it is usually very far from a steady discharge as the actual processes of wave overtopping are much more dynamic. For most defenses, only large waves will reach the crest of the structure and will overtop, but they may do so with a lot of water in a few seconds. The individual volumes in wave-by-wave overtopping are more difficult to measure in a laboratory than the mean discharge; so data on wave-by-wave volumes are much rarer.

As mean overtopping discharges are relatively easy to measure, many physical model tests have been performed all over the world, both for idealized structures and real applications or designs. The European CLASH project[3] collected a large database worldwide with more than 10,000 wave overtopping test results on all kinds of structures (see Sec. 14.5). Some series of tests have been used to develop empirical methods for the prediction of overtopping. Such empirical methods or formulae are, however, only directly applicable to idealized structures, like smooth slopes (dikes, sloping seawalls), simple rubble mound structures or vertical structures (caissons) or walls, and may require extrapolation when applied to many existing structures.

14.2.2. *Comparing overtopping performance*

Chapters 15 and 16 will describe overtopping formulae for different kinds of structures, based on the *EurOtop* Overtopping Manual.[5] In this section, an overall view is given to compare the performance of different structure types and to give insight

into how wave overtopping behaves for different structures. Those structures considered here are: smooth sloping structures (dikes, seawalls); rubble mound structures (breakwaters, rock armored slopes); and vertical structures (caissons, sheet pile walls).

The principal prediction formula for many types of wave overtopping is

$$\frac{q}{\sqrt{gH_{m0}^3}} = a\exp(-bR_c/H_{m0}). \tag{14.1}$$

It is an exponential function with the dimensionless overtopping discharge $q/(gH_{m0}^3)^{1/2}$ and the relative crest freeboard R_c/H_{m0}. This type of equation shows a straight line on a log-linear graph, which makes it easy to compare formulae for different structures. Specific equations are given in Chaps. 15 and 16.

Two equations are considered for pulsating waves on a vertical structure. Allsop *et al.*[1] consider relatively shallow water and Franco *et al.*[6] more deep water (caissons). Vertical structures in shallow water, and often with a sloping foreshore in front, may become subject to impulsive forces, i.e., high impacts and water splashing high up into the air. Specific formulae have been developed for these kinds of situations.

For easy comparison of different structures, like smooth and rubble mound sloping structures and vertical structures for pulsating and impulsive waves, some simplifications will be assumed.

In order to simplify the smooth structure, no berm is considered ($\gamma_b = 1$), only a normal wave attack is considered ($\gamma_\beta = 1$), and the sloping seawall does not feature any wavewall on top ($\gamma_v = 1$). As the slope is smooth and impermeable, $\gamma_f = 1$. This limits the structure to a smooth and straight slope with normal wave attack. The slope angles considered for smooth slopes are $\cot\alpha = 1{-}8$, which means from very steep to very gentle. If relevant, a wave steepness of $s_{m-1,0} = 0.04$ (steep storm waves) and 0.01 (long waves due to swell or wave breaking) will be considered. Detailed definitions are given in Chap. 15.

The same equation as for smooth slopes is applicable for rubble slopes, but now with a roughness factor of $\gamma_f = 0.5$, simulating a rock armoured structure. Rubble mound structures are often steep, but rock armoured slopes may also be gentle. Therefore, slope angles with $\cot\alpha = 1.5$ and 4.0 are considered.

For vertical structures under pulsating waves, both formulae of Allsop *et al.*[1] and Franco *et al.*[6] will be compared, together with the formula for impulsive waves. Impulsive waves can only be reached with a relatively steep foreshore in front of the vertical wall. For comparison, values of the breaker ratio (wave height/water depth) of $H_{m0}/h_s = 0.5, 0.7$, and 0.9 will be used. These will be discussed further in Chap. 16.

Overtopping on smooth slopes can be compared with rubble mound slopes and with vertical structures under pulsating or impulsive conditions. First, the traditional graph is given in Fig. 14.4 with the relative freeboard R_c/H_{m0} versus the logarithmic dimensionless overtopping $q/(gH_{m0}^3)^{1/2}$.

In most cases the steep smooth slope gives the largest overtopping. Steep means $\cot\alpha < 2$, but also a little gentler if long waves (less steepness) are considered. Under these conditions, waves surge up the steep slope. For gentler slopes, waves break as

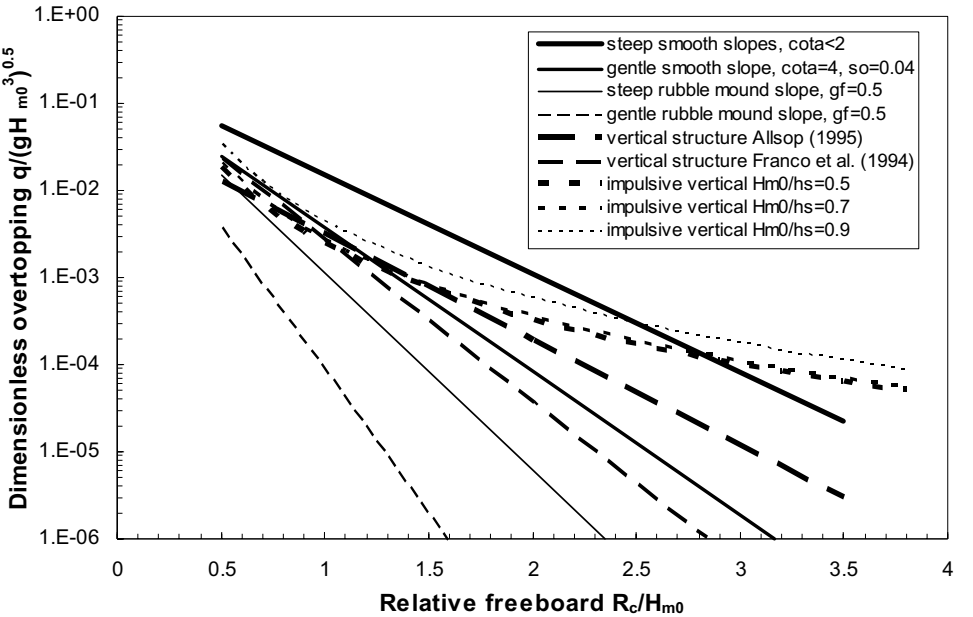

Fig. 14.4. Comparison of wave overtopping formulae for various structure types.

plunging waves and this reduces wave overtopping. The gentle slope with $\cot\alpha = 4$ gives much lower overtopping than the steep smooth slopes. Both slope angle and wave period have influence on overtopping for gentle slopes.

The high roughness and permeability of a rubble mound can reduce overtopping substantially (Fig. 14.4). A roughness factor of $\gamma_f = 0.5$ was used here although $\gamma_f = 0.4$ (two layers of rock on a permeable under layer) would reduce the overtopping further. A gentle rubble mound slope with $\cot\alpha = 4$ gives very low overtopping.

Vertical structures under pulsating waves[1,6] give lower overtopping than steep smooth slopes, but more than a rough rubble mound slope. The impulsive conditions give a different trend. First of all, the influence of the relative water depth is fairly small as all curves with different H_{m0}/h_s are quite close. For low vertical structures ($R_c/H_{m0} < 1.5$), there is hardly any difference between pulsating and impulsive conditions. The large difference is present for higher vertical structures and certainly for the very high structures. With impulsive conditions, water can be thrown high into the air, which means that overtopping occurs even for very high structures. The vertical distance that the discharge travels is more or less independent of the actual height of the structure. For $R_c/H_{m0} > 3$ the curves are almost horizontal.

Another way of comparing the effectiveness of structure types is to show the influence of slope angle on wave overtopping, as in Fig. 14.5. A vertical structure means $\cot\alpha = 0$. Battered walls have $0 < \cot\alpha < 1$. Steep slopes are generally described by $1 \leq \cot\alpha \leq 3$. Gentle slopes have roughly $\cot\alpha \geq 2$ or 3. Overtopping

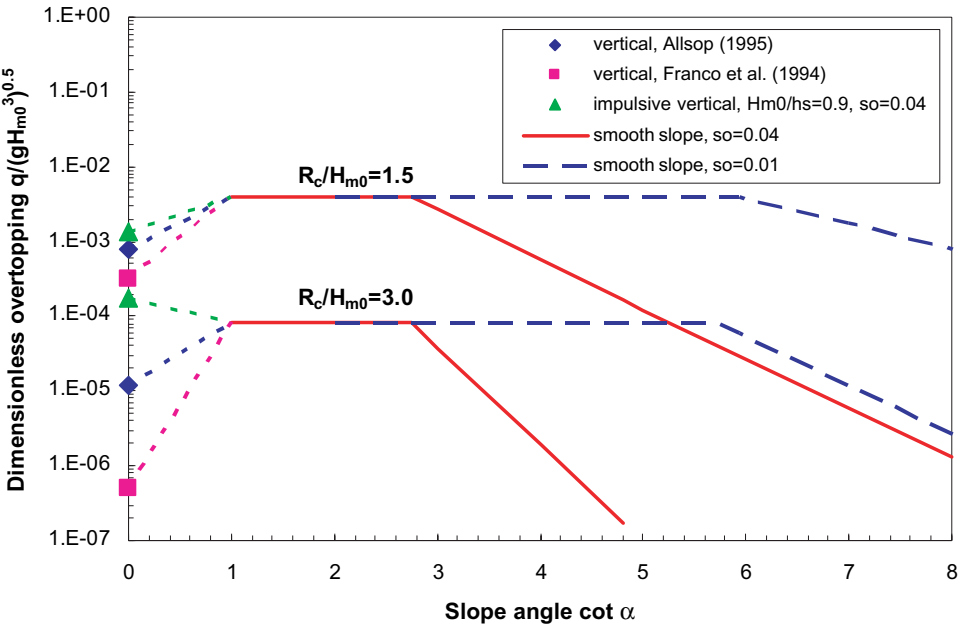

Fig. 14.5. Comparison of wave overtopping as a function of slope angle.

prediction curves for two relative freeboards: $R_c/H_{m0} = 1.5$ and 3.0 are shown in Fig. 14.5.

Of course, similar conclusions can be drawn as for the previous comparison. Steep slopes give the largest overtopping, which reduces for gentler slopes, for a given wave condition and water level. Vertical slopes give less overtopping than steep smooth slopes, except for a high vertical structure under impulsive conditions. This graph gives also the method to calculate for a battered wall: interpolate between a vertical wall and a slope 1:1 with cotα as the parameter to interpolate.

Details of all equations used here are described in more detail in Chaps. 15 and 16 (sloping smooth structures, rubble mound structures, and vertical structures).

14.2.3. *Overtopping volumes and* $V_{\max}$

Wave overtopping is a dynamic and irregular process and the mean overtopping discharge, q, does not cover this aspect. But by knowing the storm duration, t, and the number of overtopping waves in that period, N_{ow}, it is possible to give some description of this irregular and dynamic overtopping, if the overtopping discharge, q, is known. Each overtopping wave gives a certain volume of water, V, and these can be described by a distribution.

The two-parameter Weibull distribution can be fitted to many distributions. This equation has a shape parameter, b, and a scale parameter, a. The shape parameter gives a lot of information on the type of distribution. Figure 14.6 gives an overall view of some well-known distributions. The horizontal axis gives the probability of

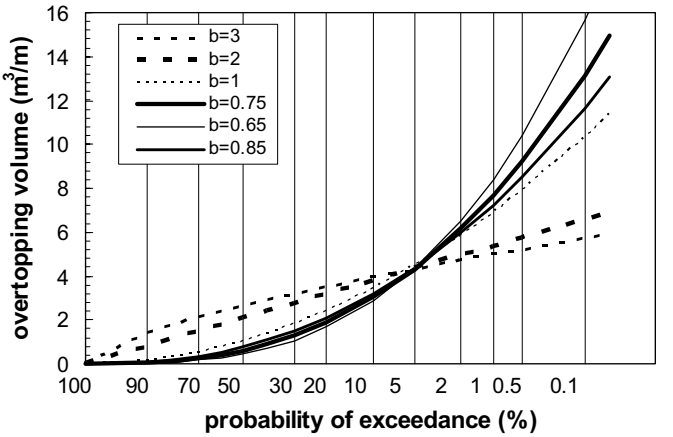

Fig. 14.6. Various distributions on a Rayleigh scale graph. A straight line ($b = 2$) is a Rayleigh distribution.

exceedance and has been plotted according to the Rayleigh distribution. It is known that wave heights in deep water generally conform to a Rayleigh distribution; so, responses governed by deep water wave conditions will plot on or close to a straight line, whilst shallow water effects will show deviations from the Rayleigh distribution.

When waves enter shallow water and the highest waves break, wave heights more closely match a Weibull distribution with $b > 2$. An example with $b = 3$ is shown in Fig. 14.6, and this indicates that there are more large waves of similar height. The exponential distribution (often found for extreme wave climates) has $b = 1$, and shows that extremes become larger compared to most of the data. Such an exponential distribution would give a straight line in a log-linear graph.

The distribution of overtopping volumes for all kinds of structures has average values even smaller than $b = 1$. Such a distribution is even steeper than an exponential distribution. It means that the wave overtopping process can be described by many fairly small overtopping volumes and a few very large volumes. The *EA-manual*[2] gives values for b and a, based on limited data sets. The b-values are mostly within the range $0.6 < b < 0.9$. For comparison, curves with $b = 0.65$ and $b = 0.85$ are given in Fig. 14.6. The curves are very similar, except that the extremes differ a little. It is for this reason that an *average* value of $b = 0.75$ was chosen for smooth slopes and not different values for various subsets. The same average value has been used for rubble mound structures, which makes smooth and rubble mound structures easily comparable. The exceedance probability, P_V, of an overtopping volume per wave is then similar to

$$P_V = P\left(\underline{V} \leq V\right) = 1 - \exp\left[-\left(\frac{V}{a}\right)^{0.75}\right], \tag{14.2}$$

with

$$a = 0.84 \cdot T_m \cdot \frac{q}{P_{ov}} = 0.84 \cdot T_m \cdot q \cdot N_w/N_{ow} = 0.84 \cdot q \cdot t/N_{ow}. \tag{14.3}$$

Equation (14.3) shows that the scale parameter a depends not only on the overtopping discharge, q, but also on the mean period, T_m, and the probability of overtopping, N_{ow}/N_w, or (similarly) on storm duration, t, and the actual number of overtopping waves N_w.

The maximum overtopping during a certain event is fairly uncertain as most maxima, but depends on the duration of the event. In a 6-h period, individual peak volume is likely to be larger than that in 15 min. The maximum overtopping volume by a single wave during an event depends on the actual number of overtopping waves, N_{ow}, and can be calculated by

$$V_{\max} = a \cdot [\ln(N_{ow})]^{4/3}. \tag{14.4}$$

A comparison can be made between the mean overtopping discharge, q, and the maximum overtopping volume in the largest wave. Note that the mean overtopping is given in l/s per m width and that the maximum overtopping volume is given in liters per m width.

Again, three example structures are considered: a smooth 1:4 slope; a rubble slope at 1:1.5; and a vertical wall, and again specific equations are given in Chaps. 15 and 16. The storm duration has been assumed as 2 h (the peak of the tide), and a fixed wave steepness of $s_{m-1,0} = 0.04$ has been considered. Figure 14.7 gives the $q - V_{\max}$ lines for the three structures and for relatively small waves of $H_{m0} = 1$ m and for fairly large waves of $H_{m0} = 2.5$ m.

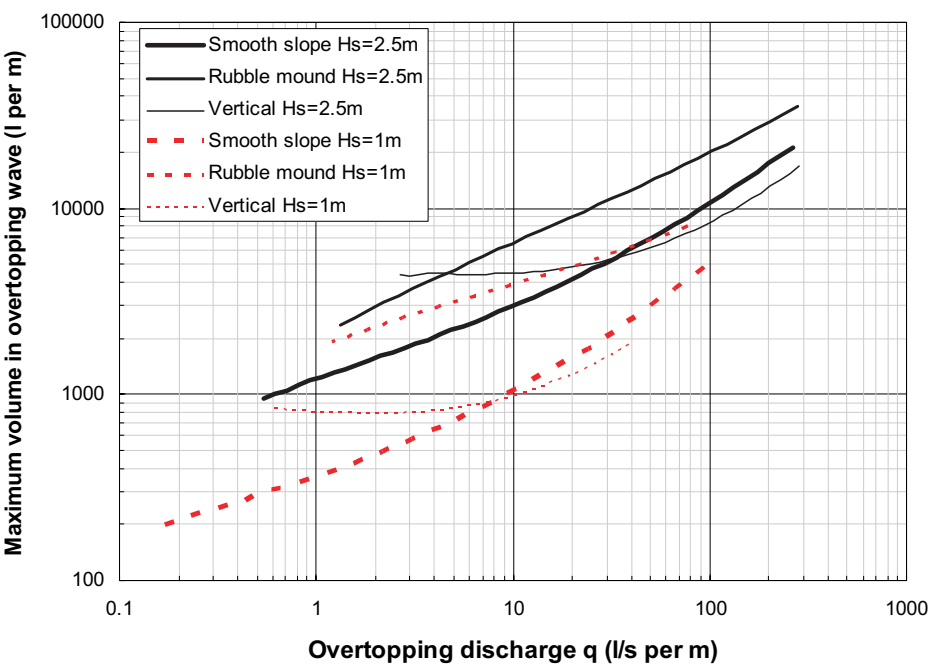

Fig. 14.7. Relationship between mean discharge and maximum overtopping volume in one wave for smooth, rubble mound, and vertical structures for wave heights of 1 m and 2.5 m.

A few conclusions can be drawn from Fig. 14.7. First of all, the ratio $q/V_{\max}$ is about 1000 for small q (roughly around 1 l/s per m), and about 100 for large q (roughly around 100 l/s per m). So, the maximum volume (in liters per m width) in the largest wave is about 100–1000 times larger than the mean overtopping discharge (in l/s per m).

Secondly, the lines for a 1 m wave height are lower than for a larger 2.5 m wave height, which means that for lower wave heights, *but similar mean discharge, q*, the maximum overtopping volume is also smaller. For example, a vertical structure with a mean discharge of 10 l/s per m gives a maximum volume of 1000 l/m for a 1 m wave height and a volume of 4000 l/m for a 2.5 m wave height.

Finally, the three different structures give different relationships, depending on the equations to calculate q and the equations to calculate the number of overtopping waves.

14.2.4. *Wave transmission by wave overtopping*

For structures like nearshore breakwaters, large overtopping can be allowed as this overtopping simply falls into the water behind the structure causing new waves behind the structure. This is termed wave transmission and is most easily defined by the wave transmission coefficient $K_t = H_{m0,t}/H_{m0,i}$, with $H_{m0,t}$ = transmitted significant wave height, and $H_{m0,i}$ = incident significant wave height. The limits of wave transmission are $K_t = 0$ (no transmission) and 1 (no reduction in wave height). If a structure has its crest above water, the transmission coefficient will never be larger than about $K_t = 0.4$–0.5.

Wave transmission was studied in the European DELOS project,[4] and the following prediction formulae were derived for smooth sloping structures:

$$K_t = \left[-0.3 \cdot \frac{R_c}{H_{m0,i}} + 0.75 \cdot (1 - \exp(-0.5 \cdot \xi_{op}))\right] \cdot (\cos\beta)^{2/3}, \tag{14.5}$$

with a minimum $K_t = 0.075$ and maximum $K_t = 0.8$; and limitations $1 < \xi_{op} < 3$; $0° \leq \beta \leq 70$, and $1 < B/H_{m0,i} < 4$, where β is the angle of wave attack and B is the crest width (and not berm width).

The transmission coefficient K_t is shown in Fig. 14.8 as a function of the relative freeboard R_c/H_{m0} and for a smooth structure with slope angle $\cot\alpha = 4$ (a gentle smooth low-crested structure). Three wave steepnesses have been used: $s_{op} = 0.01$ (long waves), 0.03 and 0.05 (short wind waves). Again, normal wave attack has been assumed. Wave transmission decreases for increasing crest height, but longer waves give more transmission. Wave overtopping can be calculated for the same structure and wave conditions (see Chap. 15 and Fig. 14.9).

Wave overtopping and transmission can be related if Figs. 14.8 and 14.9 are combined, and Fig. 14.10 shows this relationship. For convenience, the graphs are not dimensionless, but for a wave height of $H_{m0,i} = 3$ m.

A small transmitted wave height of $H_{m0,t} = 0.1$ m is found if overtopping exceeds $q = 30-50$ l/s per m. In order to reach a transmitted wave height of $H_{m0,t} = 1$ m ($K_t \approx 0.33$), the overtopping discharge should exceed $q = 500-2500$ l/s per m or

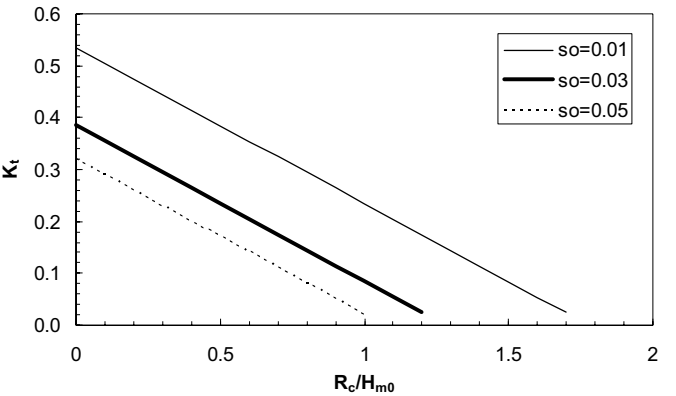

Fig. 14.8. Wave transmission for a gentle smooth structure of 1:4 and for different wave steepneses.

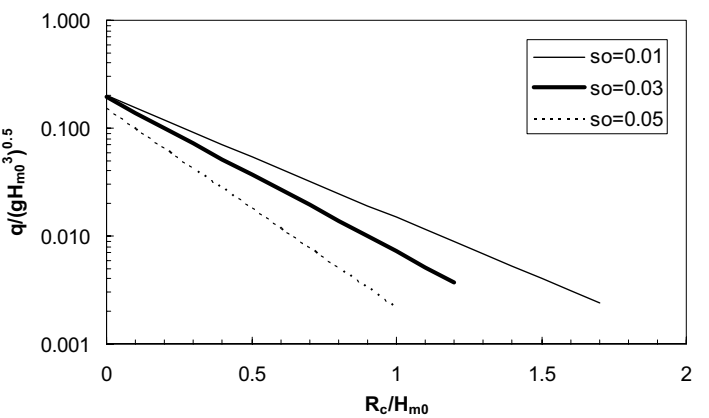

Fig. 14.9. Wave overtopping for a gentle smooth structure of 1:4 and for different wave steepnesses.

0.5–2.5 m^3/s per m. One may conclude that any significant wave transmission is always associated with (very) large wave overtopping.

Wave transmission for rubble mound structures has also been investigated in the European DELOS project,[4] and the following prediction formulae were derived for wave transmission:

$$K_t = -0.4R_c/H_{m0} + 0.64B/H_{m0} - 0.31(1 - \exp(-0.5\xi_{op})) \quad \text{for } 0.075 \leq K_t \leq 0.8. \tag{14.6}$$

Wave overtopping for a simple trapezoidal rubble mound can be calculated by methods presented in Chap. 15. A typical rubble mound structure has been used as an example, with $\cot\alpha = 1.5$, armour rock of 6–10 ton ($D_{n50} = 1.5$ m), and crest

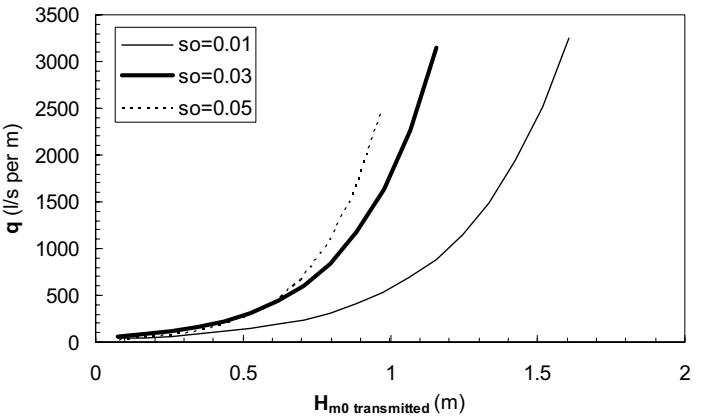

Fig. 14.10. Wave transmission versus wave overtopping for a smooth 1:4 slope and a wave height of $H_{m0} = 3$ m.

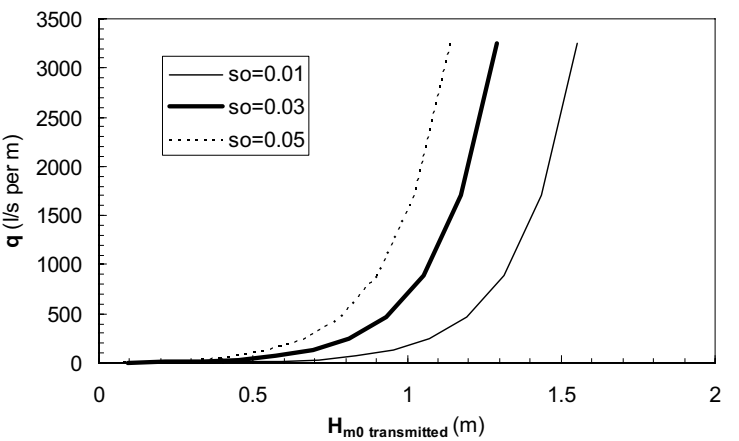

Fig. 14.11. Wave transmission versus wave overtopping discharge for a rubble mound structure, $\cot\alpha = 1.5$, 6–10 ton rock, $B = 4.5$ m, and $H_{m0} = 3$ m.

width of $B_w = 4.5$ m ($3D_{n50}$). A wave height of $H_{m0,i} = 3$ m has been assumed with the following wave steepness: $s_{m-1,0} = 0.01$ (long waves), 0.03 and 0.05 (short wind waves). In the calculations, the crest height has been changed to calculate wave transmission as well as wave overtopping.

These are compared in Fig. 14.11 which shows that longer waves ($s_{m-1,0} = 0.01$) give more wave transmission, even for similar overtopping discharge. The reason is probably the effect of wave action penetrating through the permeable upper layers, easier for long waves, thus contributing to the waves behind the structure.

So, for permeable mounds, there may still be significant wave transmission through the structure even without substantial overtopping discharge. In this

example, transmitted wave heights between $H_{m0,t} = 0.5$ m and 1 m are found for overtopping discharges smaller than $q = 100{-}200$ l/s per m.

A simple equation for wave transmission at vertical structures has been given by Goda,[7] although again more complete methods are given in Chap. 16:

$$K_t = 0.45 - 0.3R_c/H_{m0} \quad \text{for } 0 < R_c/H_{m0} < 1.25. \tag{14.7}$$

For vertical walls, transmission is primarily governed by the relative crest height with little or no effect of wave period or steepness. A simple vertical structure has been used as an example with $H_{m0,i} = 3$ m. Overtopping and wave transmission are compared in Fig. 14.12, where in the calculations the crest height has been changed to calculate wave transmission as well as wave overtopping.

For comparison, the same rubble mound structure has been used as the example in Fig. 14.11, with $\cot\alpha = 1.5$, 6–10 ton rock ($D_{n50} = 1.5$ m) as armour, a crest width of 4.5 m ($3D_{n50}$), and a wave steepness $s_{op} = 0.03$. The curve for a smooth structure (Fig. 14.10) and for $s_{op} = 0.03$ has been given too in Fig. 14.12.

A rubble mound structure gives more wave transmission than a smooth structure for similar overtopping discharge, but a vertical structure can give even more transmission. The reason may be that overtopping water over the crest of a vertical breakwater always falls in a mass from a distance into the water, rather than flowing relatively smoothly over or through a sloping structure.

One may conclude that even without considerable wave overtopping discharge at the crest of a vertical structure, there still might be significant wave transmission. In this example of a vertical structure, transmitted wave heights between 0.5 m and 1 m are found for overtopping discharges smaller than 100–200 l/s per m.

Finally, an example is shown of both wave overtopping and wave transmission on a rubble mound breakwater in Fig. 14.13.

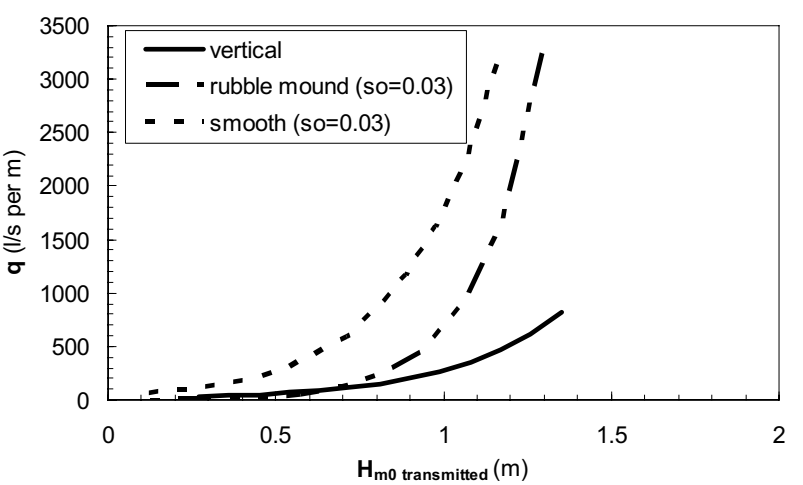

Fig. 14.12. Comparison of wave overtopping and transmission for a vertical, rubble mound, and smooth structure.

Fig. 14.13. Wave overtopping and transmission at breakwater IJmuiden, the Netherlands.

14.3. PC-OVERTOPPING

The computer program PC-OVERTOPPING was created using the results of the TAW[8] Report "Wave runup and wave overtopping at dikes" and is used for the five-yearly safety assessment of all water defenses in the Netherlands. The TAW[8] Report has now been replaced by Chap. 5 (dikes and embankments) in the *EurOtop* Overtopping Manual[5] and extended for rubble mound and vertical structures in Chaps. 6 and 7 of that manual. PC-OVERTOPPING has been translated into English, and is available from the *EurOtop* Overtopping Manual[5] web site.

The program was based on *dike-type structures*. The structure should be sloping, although a small vertical wall on top of the dike may be included. Some effects of roughness and/or permeability can be included, but not a crest with permeable and rough rock or armour units. In such a case, the structure should be modeled up to the transition to the crest, and other formulae should be used to take into account the effect of the crest.

PC-OVERTOPPING was set up so that almost *every sloping structure* can be modeled by an unlimited number of sections. Each section is given by x–y coordinates, and each section can have its own roughness factor. The program calculates most relevant overtopping parameters (except flow velocities and flow depths), such as:

- 2% runup level;
- mean overtopping discharge;
- percentage of overtopping waves;
- overtopping volumes per wave (maximum and for every percentage defined by the user);
- required crest height for given mean overtopping discharges (defined by the user).

The main uses of PC-OVERTOPPING are:

- modeling of overtopping on any sloping structure, including different roughnesses along the slope;
- calculation of most overtopping parameters, not only the mean discharge.

A disadvantage of PC-OVERTOPPING is that it does not calculate overtopping for vertical structures or for rubble structures with rough/permeable crest.

An example illustrates some of its capabilities. An example dike cross section with the design water level at +1 m CD is shown in Fig. 14.14. Different materials are used on the slope: rock, basalt, concrete asphalt, open concrete system, and grass on the upper part of the structure. The structure has been schematized in Fig. 14.15 by x–y coordinates and selection of the material of the top layer. The program selects the right roughness factor.

The input parameters are wave height, wave period (either spectral period $T_{m-1,0}$ or peak period T_p), wave obliquity, water level (with respect to the same level as used for the structure geometry), and finally, number of waves (derived from the storm duration and mean period) for the calculation of overtopping volumes, etc. Fig. 14.16 gives the input file.

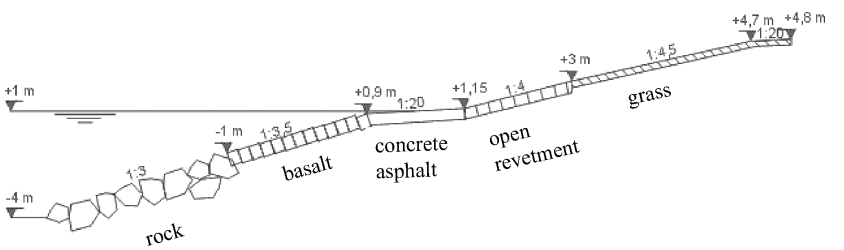

Fig. 14.14. Example cross section of a dike.

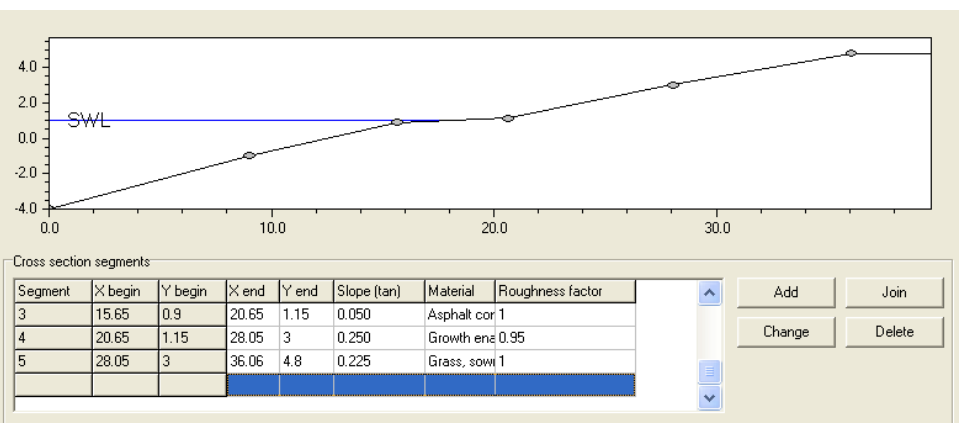

Fig. 14.15. Input of geometry by x–y coordinates and choice of top material.

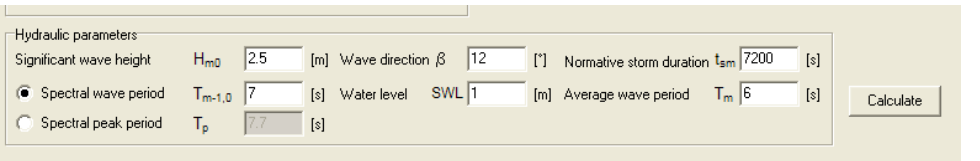

Fig. 14.16. Input file.

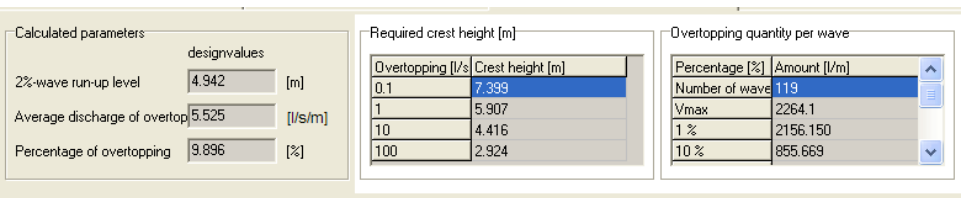

Fig. 14.17. Output of PC-OVERTOPPING.

The output is given in three columns (Fig. 14.17). The left column gives the 2% runup level, the mean overtopping discharge, and the percentage of overtopping waves. If the 2% runup level is higher than the actual dike crest, this level is calculated by extending the highest section in the cross section. The middle column gives the required dike height for the given mean overtopping discharges. Again, the highest section is extended if required. Finally, the right-hand column gives the number of overtopping waves in the given storm duration, together with the maximum overtopping volume, and volumes for specified overtopping percentages, given as a percentage of the total number of overtopping waves.

PC-OVERTOPPING also provides a check on whether the results for the 2% runup level and mean overtopping discharge fall within the measured ranges. Results on which the formulae were based are shown in the runup or overtopping graphs (see Figs. 14.18 and 14.19). These show the measured runup or overtopping, including effects of reductions due to roughness, berms, etc. The curve gives the maximum for smooth straight slopes with normal wave attack. The program then plots the calculated point in these graphs (the point within the circle).

14.4. Neural Network Tools

Artificial neural networks are tools that allow meaning to be extracted from very large quantities of data. Neural networks (NN) are organized in the form of layers, within which there are one or more processing elements called "neurons." The first layer is the input layer, and the number of neurons in this layer is equal to the number of input parameters. The last layer is the output layer and the number of neurons in this layer is equal to the number of output parameters to be predicted. The layers between the input and output layers are the hidden layers, consisting of a number of neurons to be defined in configuring the NN. Neurons in each layer receive

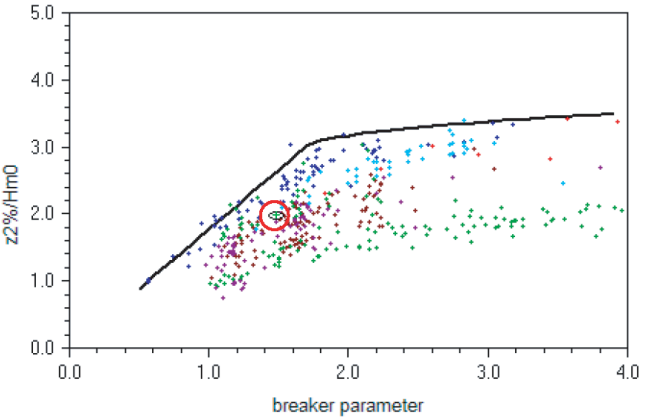

Fig. 14.18. Check on 2% runup level.

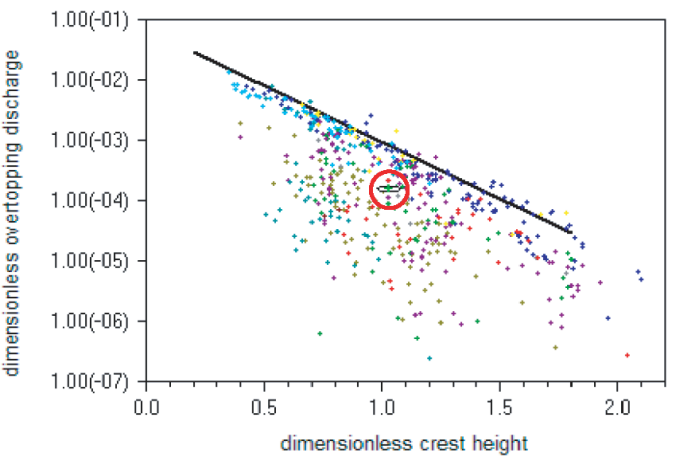

Fig. 14.19. Check on mean overtopping discharge.

information from the preceding layer, carry out a standard operation, and produce an output. Each connection has a weight factor assigned from the calibration of the NN. The input of a neuron consists of a weighted sum of the outputs of the preceding layer. This procedure is followed for each neuron; the output neuron generates the final prediction of the NN.

Artificial NNs have applications in many fields including coastal engineering where examples have been applied to predicting armour stability, forces on walls, wave transmission, and wave overtopping. The development of an artificial NN is useful, where:

- the process to be described is complicated with many parameters involved,
- there is a large amount of data.

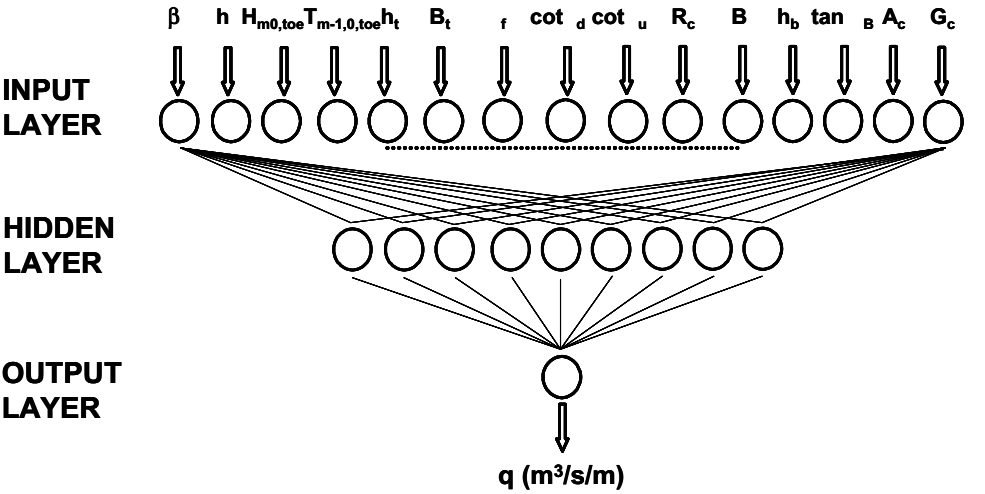

Fig. 14.20. Configuration of the NN for wave overtopping.

It has already been seen that overtopping cannot be predicted by a single formula, but requires a number of different formulae. A single NN can, however, cover the full range of structures, provided that sufficient data are available to "train" the NN. If too few data are available, predictions in the less-well populated regions will be unreliable, particularly where the prediction is trying to extrapolate out of range. Providentially, international cooperation supported by the European CLASH research project[3] collected many test results on wave overtopping for all kinds of coastal structures and embankments.

Within the CLASH project, two NNs were developed, one within the main project, and one alongside a PhD project.[9] In both cases, the NN configuration was based on Fig. 14.20, where the input layer has 15 input parameters (β, h, $H_{\mathrm{m0}toe}$, $T_{\mathrm{m-1,0}toe}, h_t, B_t$, γ_f, $\cot\alpha_d$, $\cot\alpha_u$, R_c, B, h_b, $\tan\alpha_b$, A_c, G_c) and one output parameter in the output layer (i.e., mean overtopping discharge, q). CLASH was focused on a three-layered NN, where a configuration with one single hidden layer was chosen.

Development of an artificial NN requires that all data be checked thoroughly (rubbish in = rubbish out), and that training be done by those with appropriate skills. Using NN as a prediction tool, however, is easy for most practical engineers! It is for this reason that the CLASH NN was adopted as part of the *EurOtop* Overtopping Manual.[5]

Applying NN requires an Excel or ASCII input file with parameters, run the program (push a button), and get a result file with mean overtopping discharge(s). It is therefore as easy as using a formula programmed in Excel and does not need knowledge about NNs. The advantages of the neural network are:

- it works for almost every structure configuration;
- it is easy to calculate trends instead of just one calculation with one answer.

The input exists of 10 structural parameters and four hydraulic parameters. The hydraulic parameters are wave height, wave period, and wave angle and water depth just in front of the structure. The structural parameters describe almost every possible structure configuration by a toe (two parameters), two structure slopes (including vertical and wave return walls), a berm (two parameters), and a crest configuration (three parameters). The tenth structural parameter is the roughness factor for the structure (γ_f) and describes the *average roughness of the whole structure*. Although guidance is given, estimation is not easy if the structure has different roughness on various parts of the structure. An overall view of possible structure configurations is shown in Fig. 14.21. It clearly shows that the NN is able to cope with most structure types.

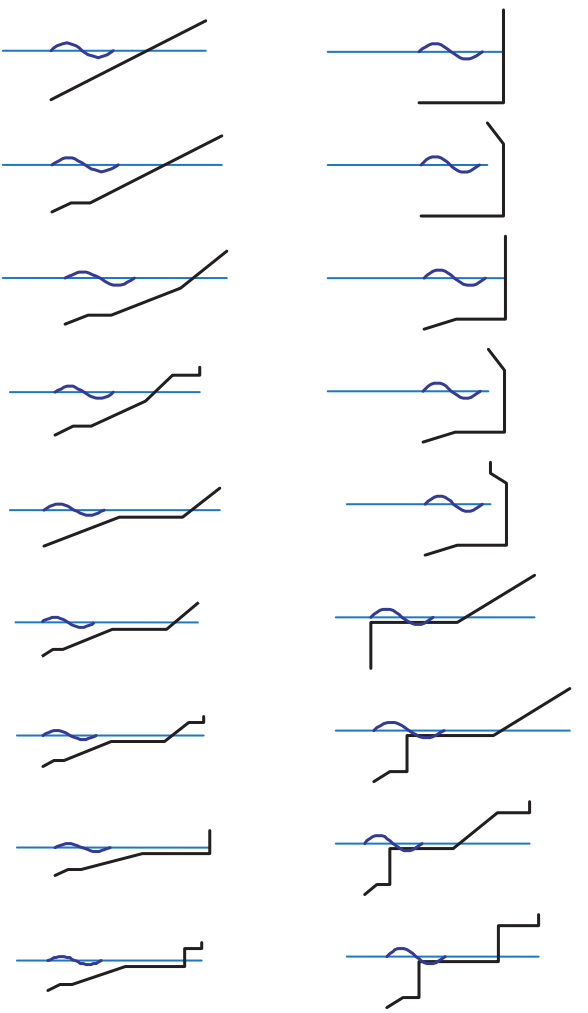

Fig. 14.21. Overall view of possible structure configurations for the NN.

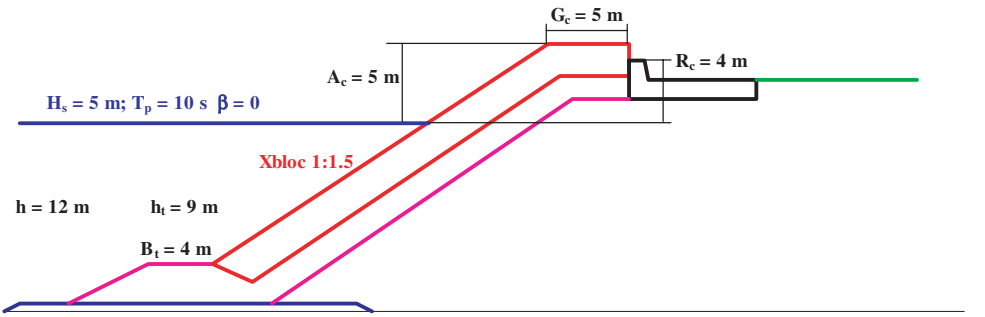

Fig. 14.22. Example cross section with parameters for application of NN.

Very often one is not only interested in one calculation, but in more. As the input file has no limitations in the number of rows (= number of calculations), it is easy to incrementally increase one or more parameters and to find a trend for a certain (design) measure. As an example for the calculation of a trend with the NN tool, an example cross section of a rubble mound embankment with a wave wall has been chosen (Fig. 14.22).

If, for example, the cross section in Fig. 14.22 experiences too much overtopping, the following measures could be considered:

- increasing the crest;
- applying a berm;
- widening the crest;
- increasing only the crest wall.

An example input file with the first six calculations is shown in Table 14.1, where incremental increase of the crest will show the effect of raising the crest on the amount of wave overtopping. Calculations will give an output file with the mean overtopping discharge q (m^3/s per m width) and with confidence limits. Table 14.2 shows as an example the output related to the input in Table 14.1.

Assembling the input file for this example took 1 h and resulted in 1400 rows or calculations. The calculation of the NN took less than 10 s. The results were copied into the Excel input file, and the resulting graph was plotted within Excel, which took another hour. Figure 14.23 gives the final result, where the four trends are

Table 14.1. Example input file for NN with first six calculations.

β	h	H_{m0}	$T_{m-1,0}$	h_t	B_t	γ_f	$\cot\alpha_d$	$\cot\alpha_u$	R_c	B	h_b	$\tan\alpha_B$	A_c	G_c
0	12	5	9.1	9	4	0.49	1.5	1.5	4	0	0	0	5	5
0	12	5	9.1	9	4	0.49	1.5	1.5	4.05	0	0	0	5.05	5
0	12	5	9.1	9	4	0.49	1.5	1.5	4.1	0	0	0	5.1	5
0	12	5	9.1	9	4	0.49	1.5	1.5	4.15	0	0	0	5.15	5
0	12	5	9.1	9	4	0.49	1.5	1.5	4.2	0	0	0	5.2	5
0	12	5	9.1	9	4	0.49	1.5	1.5	4.25	0	0	0	5.25	5

Table 14.2. Output file of NN with confidence limits.

q (m^3/s per m)	2.50%	5.00%	25.00%	50.00%	75.00%	95.00%	97.50%
$5.90E{-}02$	$2.45E{-}02$	$2.77E{-}02$	$4.15E{-}02$	$5.91E{-}02$	$8.35E{-}02$	0.1299	0.1591
$5.64E{-}02$	$2.35E{-}02$	$2.64E{-}02$	$3.99E{-}02$	$5.58E{-}02$	$7.91E{-}02$	0.1246	0.1516
$5.40E{-}02$	$2.26E{-}02$	$2.49E{-}02$	$3.82E{-}02$	$5.33E{-}02$	$7.52E{-}02$	0.119	0.1448
$5.16E{-}02$	$2.19E{-}02$	$2.39E{-}02$	$3.69E{-}02$	$5.08E{-}02$	$7.17E{-}02$	0.1133	0.1383
$4.94E{-}02$	$2.07E{-}02$	$2.27E{-}02$	$3.55E{-}02$	$4.85E{-}02$	$6.89E{-}02$	0.1079	0.1324
$4.73E{-}02$	$1.99E{-}02$	$2.18E{-}02$	$3.38E{-}02$	$4.62E{-}02$	$6.60E{-}02$	0.1033	0.1265

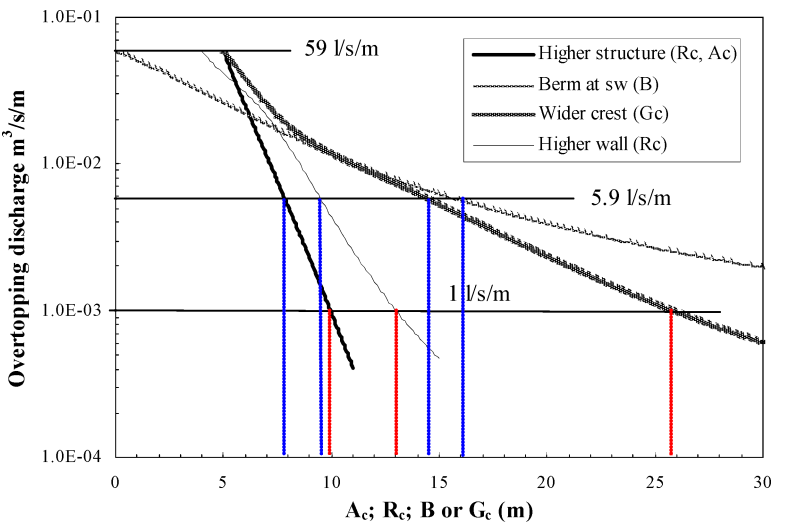

Fig. 14.23. Results of a trend calculation.

shown. The base situation had an overtopping discharge of 59 l/s per m. The graph clearly shows what measures are required to reduce the overtopping by, for example, a factor 10 (to 5.9 l/s per m) or to only 1 l/s per m. It also shows that increasing the structure height is most effective, followed by increasing only the crest wall.

Within the CLASH research project, two different NNs exist. One is the official NN developed by Delft Hydraulics in the main part of the CLASH project. It runs as executable and can be downloaded from the CLASH web site or the *EurOtop* Overtopping Manual web site.[5]

The other NN has also been developed within CLASH, but as part of a PhD thesis at Gent University.[9] The network was developed in MatLab® and so it can only be run if the user has a licence for MatLab®. The advantage of this NN is that it first decides whether there will be overtopping or not (classifier). If there is no overtopping, it gives $q = 0$. If there is overtopping, it will quantify the overtopping with a similar network as the CLASH network (quantifier). This use of both "qualifier" and "quantifier" NNs is certainly an advantage over the single-stage CLASH

NN. The CLASH network was only trained with overtopping data (tests with "no overtopping" were not considered) and, therefore, this network always gives a prediction of overtopping, even in the range where no overtopping should be expected.

14.5. Use of CLASH Database

The EU research project CLASH generated an extensive database of overtopping tests from data submitted around the world. Each test was described not only by 31 parameters as hydraulic and structural parameters, but also parameters describing the reliability and complexity of the test and structure. The database includes more than 10,000 tests and was set up as an Excel database. At its simplest, the database is no more than a matrix with 31 columns and more than 10,000 rows. It can be downloaded from the CLASH or Manual web site.

If a user has a specific structure, there is a possibility to look into the database and find more or less similar structures with measured overtopping discharges. It may even be possible that the structure has already been tested with the right wave conditions! Finding the right tests can be done by using filters in the Excel database. Every test of such a selection can then be studied thoroughly. One example will be described here in depth.

Suppose one is interested in the improvement of a *vertical wall with a large wave return wall*. The wave conditions are $H_{m0\ \mathrm{toe}} = 3$ m, the wave steepness $s_{m-1,0} = 0.04$ ($T_{m-1,0} = 6.9$ s), and the wave attack is perpendicular to the structure. The design water depth $h = 10$ m and the wave return wall starts 1 m above design water level and has a height and width of 2 m (the angle is 45° seaward). This gives a crest freeboard $R_c = 3$ m, equal to the wave height. Have tests been performed which are close to this specific structure and given wave conditions?

The first filter selects data with a vertical down slope, i.e., $\cot\alpha_d = 0$. The second filter could select data with a wave return wall overhanging more than about 30° seaward. This means $\cot\alpha_u < -0.57$. In the first instance every large wave return wall can be considered, say, at least $0.5H_{m0}$ wide. This gives the third filter, selecting data with $-\cot\alpha_{u^*}\ (A_c + h_b)/H_{m0} \geq 0.5$. With these three filters, the database gives 212 tests from four independent test series.

These data are summarized in Fig. 14.24 with the expression of Franco *et al.*[6] for a vertical wall. There are 22 tests which gave no measurable overtopping. These results are represented in the figure with a value of $q/(gH^3_{m0\ \mathrm{toe}})^{1/2} = 10^{-7}$. The majority of the data are situated below the simple prediction curve for a vertical wall, indicating that a wave return wall is efficient, but the data are too scattered to be decisive.

The next step in the filtering process could be that only wave return walls overhanging more than 45° seaward are selected. This means $\cot\alpha_u < -1$. The water depth is relatively large for the considered case, and shallow water conditions are excluded if $h/H_{m0\ \mathrm{toe}} > 3$. Figure 14.25 shows this further filtering process. The number of data has been reduced to 78 tests from only two independent series. In total, 12 tests result in no measureable overtopping. The data show the trend that the overtopping discharges are on average about 10 times smaller than for a vertical

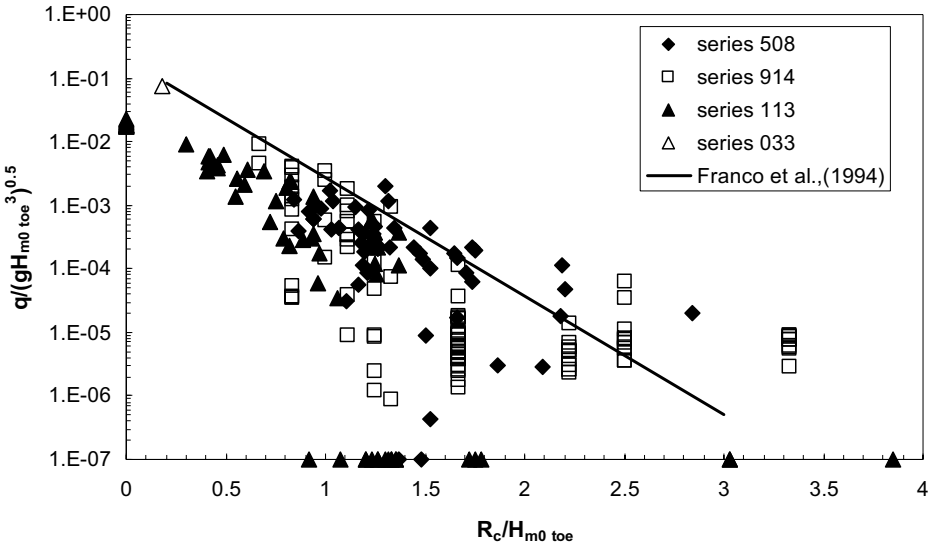

Fig. 14.24. Overtopping for large wave return walls — first selection.

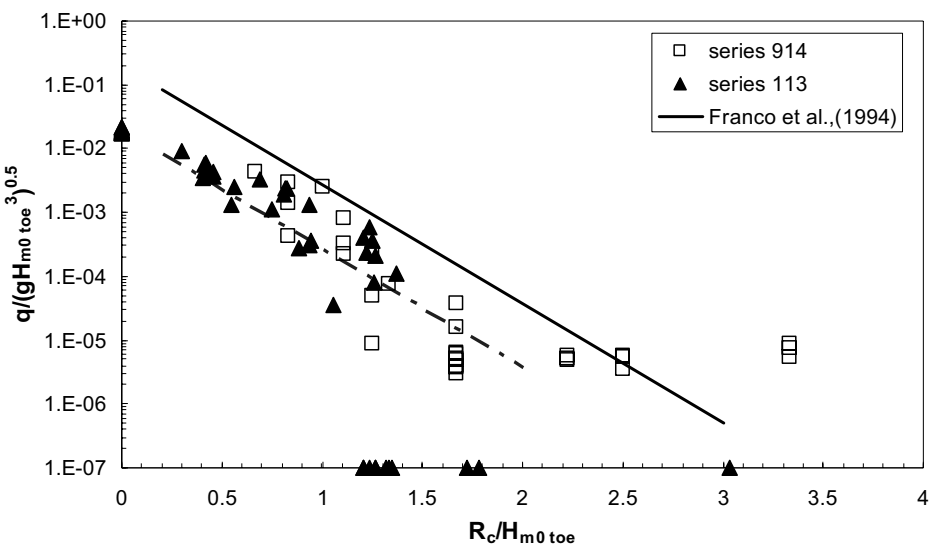

Fig. 14.25. Overtopping for large wave return walls — second selection with more criteria.

wall, given by the dashed line. But for $R_c/H_{m0\ \mathrm{toe}} > 1$, there are quite some tests without any overtopping.

As still quite some data are remaining in Fig. 14.25, it is possible to narrow the search area even further. With a wave steepness of $s_{m-1,0} = 0.04$ in the considered case, the wave steepness range can be limited to $0.03 < s_{m-1,0} < 0.05$. The width

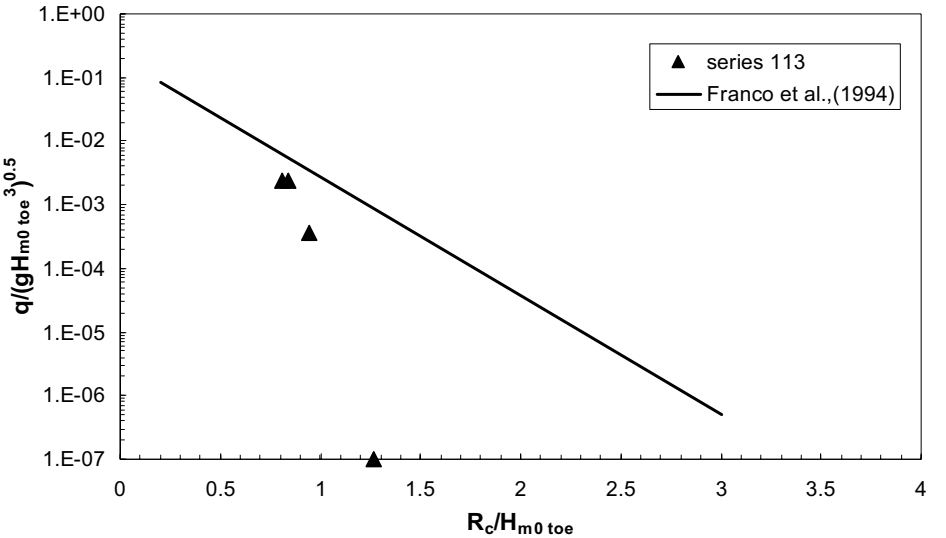

Fig. 14.26. Overtopping for wave return wall with $s_o = 0.04$, seaward angle of 45°, a width of 2 m and a crest height of $R_c = 3$ m. For $H_{m0\ \mathrm{toe}} = 3$ m, the overtopping can be estimated from $R_c/H_{m0\ \mathrm{toe}} = 1$.

of the wave return wall of 2 m, with the wave height of $H_{m0} = 3$ m, gives a relative width of 0.67. The range can be limited to $0.5 < -\cot\alpha_{u^*}(A_c + h_b)/H_{m0} < 0.75$. Finally, the transition from vertical to wave return wall is 1 m above the design water level, giving $h_b/H_{m0\ \mathrm{toe}} = -0.33$. The range can be set at $-0.5 < h_b/H_{m0\ \mathrm{toe}} < -0.2$.

The final selection obtained after filtering is given in Fig. 14.26. Only four tests remain from one test series, one of which gave no measurable overtopping. The data now give a clear picture. For a relative freeboard lower than about $R_c/H_{m0\ \mathrm{toe}} = 0.7$, the overtopping will not be much different from the overtopping at a vertical wall. The wave return wall, however, becomes very efficient for large freeboards and effectively prevents any measurable overtopping for $R_c/H_{m0\ \mathrm{toe}} > 1.2$. For the structure considered with $R_c/H_{m0\ \mathrm{toe}} = 1$, the wave overtopping will be 20–40 times less than that for a vertical wall and will probably amount to $q = 0.5 - 2$ l/s per m width. In this particular case, it was possible to find four tests in the database with very close similarities to the considered structure and wave conditions.

14.6. Tolerable Discharges

14.6.1. *Hazards from overtopping*

Most sea defense structures are constructed primarily to limit overtopping that might cause flooding. Over a particular storm or tide, the overtopping volumes that can be tolerated will be site-specific as the overall volume of water that can be accepted will depend on the size and use of the receiving area, extent, and magnitude

of drainage ditches, damage versus inundation curves, and return period. Guidance on modeling inundation flows is being developed within the Floodsite project (see: http://www.floodsite.net/html/project_overview.htm), but flood volumes, *per se*, are not distinguished further in this chapter. Instead, advice here focuses on direct hazards from wave overtopping.

For sea defenses that protect people living, working, or enjoying themselves, designers and owners of these defenses must deal with potential direct hazards from overtopping. This requires that the level of hazard and its probability of occurrence be assessed, allowing appropriate action plans to be devised to ameliorate risks arising from overtopping.

The main hazards on or close to sea defense structures are of death, injury, property damage, or disruption from direct wave impact or by drowning. On average, approximately 2–5 people are killed each year in each of the United Kingdom and Italy through wave action, chiefly on seawalls and similar structures (although this rose to 11 in the United Kingdom during 2005). It is often helpful to analyze direct wave and overtopping effects, and their consequences under three general categories:

- direct hazard of injury or death to people immediately behind the defense;
- damage to property, operation, and/or infrastructure in the area defended, including loss of economic, environmental, or other resource, or disruption to an economic activity or process;
- damage to defense structure(s), either short-term or longer-term, with the possibility of breaching and flooding.

The character of overtopping flows or jets, and the hazards they cause, also depend upon the geometries of the structure, the hinterland behind the seawall, and the form of overtopping. Rising ground behind the seawall may allow people at potential risk to see incoming waves, and the slope will slow overtopping flows. Conversely, a defense that is elevated significantly above the land defended will obscure visibility of incoming waves (Fig. 14.27), and post-overtopping flows may increase in speed rather than decreasing. Hazards caused by overtopping therefore depend upon both the local topography and structures as well as on the direct overtopping characteristics.

It is not possible to give unambiguous or precise limits to tolerable overtopping for all conditions. Some guidance is, however, offered here on tolerable mean discharges and maximum overtopping volumes for a range of circumstances or uses, and on inundation flows and depths. These limits may be adopted or modified depending upon the circumstances and uses of the site.

14.6.1.1. *Wave overtopping processes and hazards*

Overtopping hazards can be linked to a number of simple direct flow parameters (Fig. 14.28):

- mean overtopping discharge, q;
- individual and maximum overtopping volumes, V_i and $V_{\max}$;

Fig. 14.27. Defended area below seawall and foreshore (saltmarsh) level.

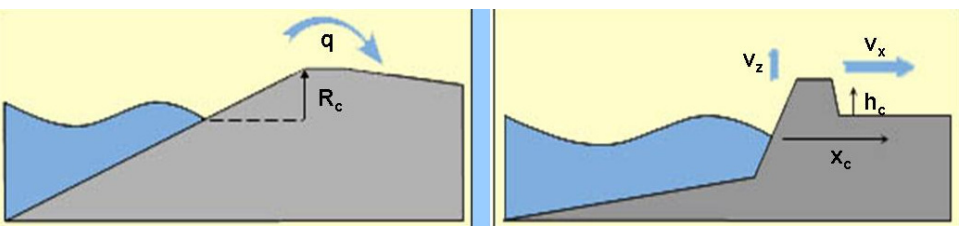

Fig. 14.28. Overtopping on embankment and promenade seawalls.

- overtopping velocities over the crest, horizontally and vertically, v_{xc} and v_{zc} or v_{xp} and v_{zp};
- overtopping flow depth, again measured on crest or promenade, d_{xc} or d_{xp}.

Less direct responses (or similar responses, but farther back from the defense) may be used to assess the effects of overtopping, perhaps categorized by:

- overtopping falling distances, x_c;
- post-overtopping wave pressures (pulsating or impulsive), p_{qs} or p_{imp};
- post-overtopping flow depths, d_{xc} or d_{xp}; and horizontal velocities, v_{xc} or v_{xp}.

The main response to direct overtopping hazards has most commonly been the construction of new defenses, but should now always consider three options, in increasing order of intervention:

- move human activities away from the area subject to overtopping or flooding hazard, thus modifying the land-use category and/or habitat status;

- accept hazard at a given probability (acceptable risk) by providing for temporary use and/or short-term evacuation with reliable forecast, warning and evacuation systems, and/or use of temporary/demountable defense systems;
- increase defense standard to reduce risk to (permanently) acceptable levels probably by enhancing the defense and/or reducing loadings.

For any structure expected to reduce overtopping, the crest level and/or the front-face configuration will be dimensioned to give acceptable levels of overtopping under specified extreme conditions or combined conditions (e.g., water level and waves). Setting acceptable levels of overtopping depends on:

- use of the defense structure itself;
- use of the land behind;
- national and/or local standards and administrative practice;
- economic and social basis for funding the defense.

Under most forms of wave attack, waves tend to break before or onto sloping embankments with the overtopping process being relatively gentle (see Fig. 14.1). Relatively few water levels and wave conditions may cause "impulsive" breaking where the overtopping flows are sudden and violent. Conversely, steeper, vertical, or compound structures are more likely to experience intense local impulsive breaking, and may overtop violently and with greater velocities (see Fig. 14.3). The form of breaking will therefore influence the distribution of overtopping volumes and their velocities, both of which will have impact on the hazards that they cause.

Additional hazards that are not dealt with here are those that arise from wave reflections, often associated with steep-faced defenses. Reflected waves increase wave disturbance, which may cause hazards to navigating or moored vessels; may increase waves along neighboring frontages, and/or may initiate or accelerate local bed erosion thus increasing depth-limited wave heights.

14.6.1.2. *Form of overtopping hazard*

Wave overtopping which runs up the face of the seawall and over the crest in (relatively) complete sheets of water is often termed "green water." In contrast, "white water" or spray overtopping tends to occur when waves break seaward of the defense structure or break onto its seaward face, producing noncontinuous overtopping, and/or significant volumes of spray. Overtopping spray may be carried over the wall either under its own momentum, or assisted and/or driven by an onshore wind. Additional spray may also be generated by wind acting directly on wave crests, particularly when reflected waves interact with incoming waves to give severe local "clapoti." This type of spray is not classed as overtopping nor is it predicted by the methods described in this manual.

Without a strong onshore wind, spray will seldom contribute significantly to overtopping volumes, but may cause local hazards. Light spray may reduce visibility for driving, important on coastal highways, and will extend the spatial extent of salt spray effects such as damage to crops/vegetation, or deterioration of buildings. The effect of spray in reducing visibility on coastal highways (particularly when

intermittent) can cause sudden loss of visibility in turn leading drivers to veer suddenly.

Effects of wind and generation of spray have not often been modeled. Some research studies have suggested that effects of onshore winds on large green water overtopping are small, but that overtopping under $q = 1$ l/s per m might increase up to four times under strong winds, especially where much of the overtopping is as spray. Discharges between $q = 1$ and 0.1 l/s per m are, however, already greater than some discharge limits suggested for pedestrians or vehicles, suggesting that wind effects may influence overtopping at and near acceptable limits for these hazards. This is discussed further in Sec. 14.7.

14.6.1.3. *Return periods*

Return periods at which overtopping hazards are analyzed, and against which a defense might be designed, may be set by national regulation or guidelines. As with any area of risk management, different levels of hazard are likely to be tolerated at inverse levels of probability or return period. The risk levels (probability × consequence) that can be tolerated will depend on local circumstances, local and national guidelines, the balance between risk and benefits, and the level of overall exposure. Heavily trafficked areas might therefore be designed to experience lower levels of hazard applied to more people than lightly used areas, or perhaps the same hazard level at longer return periods. Guidance on example return periods used in evaluating levels of protection suggest example protection levels versus return periods as shown in Table 14.3.

In practice, some of the return periods in Table 14.3 may be too short. National guidelines have recommended lower risk, e.g., a low probability of flooding in the United Kingdom is now taken as <0.1% probability (1:1000 year return), and the medium probability of sea flooding as between 0.5% and 0.1% (1:200 to 1:1000 year return). Many existing defenses, however, offer levels of protection far lower than these.

In the Netherlands, where two-thirds of the country lies below the storm surge level, protection was substantially improved after the flood in 1953 where almost 2000 people drowned. Standards of protection for large rural areas are currently 1:10,000 years, less densely populated areas at 1:4000 years, and protection for high river discharge (without threat of storm surge) is given to 1:1250 years.

Table 14.3. Hazard types.

Hazard type and reason	Design life (years)	Level of protection[a] (years)
Temporary or short-term measures	1–20	5–50
Majority of coast protection or sea defenses	30–70	50–100
Flood defenses protecting large areas	50–100	100–10,000
Special structure, high capital cost	200	Up to 10,000
Nuclear power stations, etc.	—	10,000

[a]Total probability return period.

The design life for flood defenses like dikes which are fairly easy to upgrade, is taken in the Netherlands as 50 years. In urban areas, where it is more difficult to upgrade a flood defense, the design life is taken as 100 years. This design life increases for very special structures with high capital costs, like the Eastern Scheldt storm surge barrier, Thames barrier, or the Maeslandtkering at the entrance to Rotterdam. A design life of around 200 years is then usual.

Variations from simple "acceptable risk" approach may be required for publicly funded defenses based on benefit — cost assessments, or where public aversion to hazards causing death requires greater efforts to ameliorate the risk, either by reducing the probability of the hazard or by reducing its consequence.

14.6.2. *Tolerable mean discharges and overtopping simulator*

Guidance on overtopping discharges that can cause damage to seawalls, buildings, or infrastructure, or danger to pedestrians and vehicles have been related to mean overtopping discharges or (less often) to peak volumes. Guidance quoted previously were derived initially from the analysis in Japan of overtopping perceived by port engineers to be safe. Further guidance from Iceland suggests that equipment or cargo might be damaged for $q \geq 0.4$ l/s per m. Significantly, different limits are discussed for embankment seawalls with back slopes, or for promenade seawalls without back slopes. Some guidance distinguishes between pedestrians or vehicles, and between slow and faster speeds for vehicles.

Tests on the effects of overtopping on people suggest that information on mean discharges alone may not give reliable indicators of safety for some circumstances, and that maximum individual volumes may be better indicators of hazard than average discharges. The volume (and velocity) of the largest overtopping event can vary significantly with wave condition and structure type, even for a given mean discharge. There remain, however, two difficulties in specifying safety levels with reference to maximum volumes rather than to mean discharges. Methods to predict maximum volumes are available for fewer structure types, and are less well-validated. Secondly, data relating individual maximum overtopping volumes to hazard levels are still very rare.

In most instances, the discharge (or volumes) discussed here are those at the point of interest, e.g., at the roadway or footpath or building. It is noted that the hazardous effect of overtopping waters reduces with the distance away from the defense line. As a rule of thumb, the hazard effect of an overtopping discharge at a point x m back from the seawall crest will be to reduce the overtopping discharge by a factor of x; and so the effective overtopping discharge at x (over a range of 5–25 m), $q_{\text{effective}}$ is given by

$$q_{\text{effective}} = q_{\text{seawall}}/x. \tag{14.8}$$

The overtopping limits suggested in Tables 14.4–14.7 derive from a generally precautionary principle informed by previous guidance and by observations and measurements made by the CLASH partners and other researchers. Limits for pedestrians in Table 14.4 show a logical sequence, with allowable discharges reducing

Table 14.4. Limits for overtopping for pedestrians.

Hazard type and reason	Mean discharge q (l/s per m)	Max volume[a] $V_{\max}$ (l/m)
Trained staff, well shod and protected, expecting to get wet, overtopping flows at lower levels only, no falling jet, low danger of fall from walkway.	1–10	500 at low level
Aware pedestrian, clear view of the sea, not easily upset or frightened, able to tolerate getting wet, wider walkway.[b]	0.1	20–50 at high level or velocity

[a]These limits relate to overtopping velocities well below vc $\leq$10 m/s. Lower volumes may be required if the overtopping process is violent and/or overtopping velocities are higher.
[b]Not all of these conditions are required, nor should failure of one condition on its own require the use of a more severe limit.

Table 14.5. Limits for overtopping for vehicles.

Hazard type and reason	Mean discharge q (l/s per m)	Max volume[a] $V_{\max}$ (l/m)
Driving at low speed, overtopping by pulsating flows at low flow depths, no falling jets, vehicle not immersed	10–50[a]	100–1000
Driving at moderate or high speed, impulsive overtopping giving falling or high velocity jets	0.01–0.05[b]	5–50[b] at high level or velocity

[a]These limits probably relate to overtopping defined at highway.
[b]These limits relate to overtopping defined at the defense, but assumes the highway to be immediately behind the defense.

steadily as the recipient's ability or willingness to anticipate or receive the hazard reduces.

A further precautionary limit of $q = 0.03$ l/s per m might apply for unusual conditions where pedestrians have no clear view of incoming waves; may be easily upset or frightened or are not dressed to get wet; may be on a narrow walkway or in close proximity to a trip or fall hazard. Research studies have, however, shown that this limit is only applicable for the conditions identified, and should NOT be used as the general limit for which $q = 0.1$ l/s per m in Table 14.4 is appropriate.

For vehicles, the suggested limits are rather more widely spaced as two very different situations are considered. The higher overtopping limit in Table 14.5 applies where wave overtopping generates pulsating flows at roadway level, akin to driving through slowly varying fluvial flow across the road. The lower overtopping limit in Table 14.5 is, however, derived from considering more impulsive flows, overtopping at some height above the roadway, with overtopping volumes being projected at

speed and with some suddenness. These lower limits are however based on few site data or tests, and may therefore be relatively pessimistic.

Rather fewer data are available on the effects of overtopping on structures, buildings, and property. Site-specific studies suggest that pressures on buildings by overtopping flows will vary significantly with the form of wave overtopping, and with the use of sea defense elements intended to disrupt overtopping momentum (not necessarily reducing discharges). Guidance derived from the CLASH research project and previous work suggests limits in Table 14.6 for damage to buildings, equipment, or vessels behind defenses.

A set of limits for structures in Table 14.7 has been derived from early work by Goda[7] and others in Japan. These give a first indication of the need for specific protection to resist heavy overtopping flows. It is assumed that any structure close to the sea will already be detailed to resist the erosive power of heavy rainfall and/or spray. Two situations are considered:

- embankment seawall or dike, elevated above the defended area, and so overtopping flows pass over the crest and down the rear face;
- promenade defense in which overtopping flows remain on or behind the seawall crest before returning seaward.

Table 14.6. Limits for overtopping for property behind the defense.

Hazard type and reason	Mean discharge q (l/s per m)	Max volume[a] $V_{\max}$ (l/m)
Significant damage or sinking of larger yachts	50	5,000–50,000
Sinking small boats set 5–10 m from wall.		
Damage to larger yachts	10[a]	1,000–10,000
Building structure elements	1[b]	∼
Damage to equipment set back 5–10 m	0.4[a]	∼

[a]These limits relate to overtopping defined at the defense.
[b]This limit relates to the effective overtopping defined at the building.

Table 14.7. Limits for overtopping for damage to the defense crest or rear slope.

Hazard type and reason	Mean discharge q (l/s per m)
Embankment seawalls/sea dikes	
No damage if crest and rear slope are well protected	50–200
No damage to crest and rear face of grass covered embankment of clay	1–10
No damage to crest and rear face of embankment if not protected	0.1
Promenade or revetment seawalls	
Damage to paved or armoured promenade behind seawall	200
Damage to grassed or lightly protected promenade or reclamation cover	50

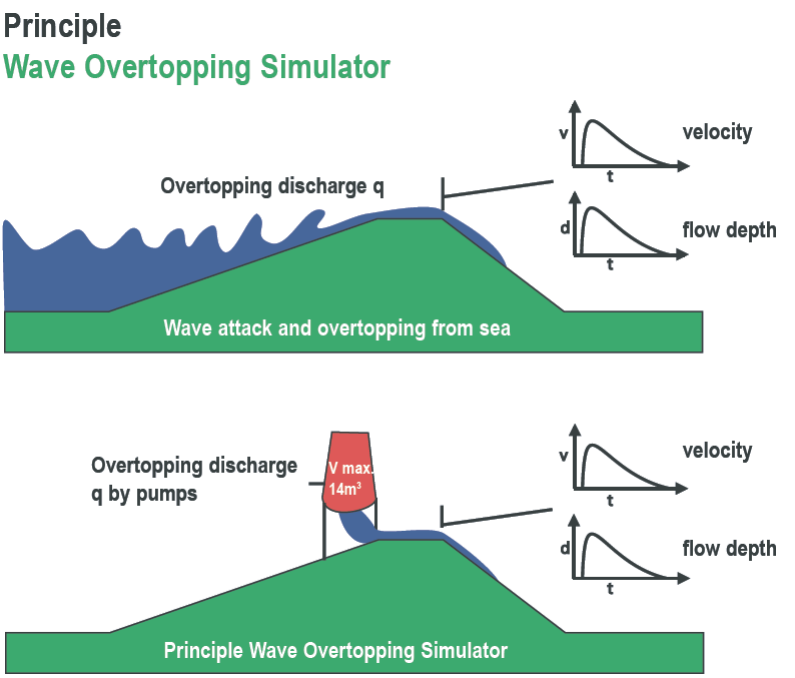

Fig. 14.29. Principle of the wave overtopping simulator.

The limits for the latter category cannot be applied where the overtopping flows can fall from the defense crest where the nature of the flow may be more impulsive. The limits in Table 14.7 are precautionary and are generally based on old data.

In order to clarify the erosion resistance of grass protection under wave overtopping, tests were performed using the overtopping simulator (Figs. 14.29 and 14.30) in 2007 on a sea dike in the Netherlands. The grass dike had a 1:3 inner slope of fairly good clay, sand content smaller than 30%. The overtopping simulator was used to test the erosion resistance of this inner slope for a simulated 6 h storm for each overtopping condition. These started with overtopping equivalent to a mean discharge of 0.1 l/s per m and increased to 1, 10, 20, 30, and finally even 50 l/s per m. After all these simulated storms, the slope was still in good condition and showed little erosion. The erosion resistance of this dike was very high.

Another test was performed on bare clay by removing the grass sod over the full inner slope to a depth of 0.2 m. Overtopping conditions of 0.1 l/s per m, 1, 5, and finally, 10 l/s per m were performed, again for 6 h each. Erosion damage started for the first condition (two erosion holes) and increased during the other overtopping conditions. After 6 h at a mean discharge of 10 l/s per m (see Figs. 14.30 and 14.31), there were two large erosion holes, about 1 m deep, 1 m wide, and 4 m long. This situation was considered as "not too far from initial breaching." The overall conclusion of this first overtopping test on a real dike is that clay with grass can be highly erosion-resistant. Even without grass the good quality clay also survived extensive overtopping. The conclusions may not yet be generalized to all dikes as

Fig. 14.30. Overtopping simulator discharging a large volume on the inner slope of a dike.

Fig. 14.31. Wave overtopping test on bare clay; result after 6 h with 10 l/s per m width.

clay quality and type of grass cover still may play a role and, therefore, more testing is required to come to general conclusions.

One remark, however, should be made on the strength of the inner slopes of dikes by wave overtopping. Direct erosion of the slope is one possible failure mechanism. A major failure mechanism, especially in the past, was slip failure of the (rear) slope. Slip failures may directly lead to a breach, and such slip failures often occur

mainly for steep inner slopes like 1:1.5 or 1:2. For this reason, most dike designs in the Netherlands in the past 50 years have used a 1:3 inner slope, where it is unlikely that slip failures will occur due to overtopping. This mechanism might, however, occur for steep inner slopes, and so it should be taken into account in safety analysis.

14.6.3. *Tolerable maximum volumes and velocities*

14.6.3.1. *Overtopping volumes and velocities*

Guidance on suggested limits for maximum individual overtopping volumes have been given in Tables 14.4–14.7 where data are available. Research studies with volunteers at full scale or field observations suggest that danger to people or vehicles might be related to peak overtopping volumes, with "safe" limits for people covering:

$V_{\max} = 1000 - 2000$ l/m for trained and safety-equipped staff in pulsating flows on a wide-crested dike;
$V_{\max} = 750$ l/m for untrained people in pulsating flows along a promenade;
$V_{\max} = 100$ l/m for overtopping at a vertical wall;
$V_{\max} = 50$ l/m where overtopping could unbalance an individual by striking their upper body without warning.

Few data are available on overtopping velocities and their contribution to hazards. Chapter 15 gives guidance on overtopping flow velocities and depths at crest and inner slope for simple sloping embankments as well. Velocities of 5–8 m/s are possible for maximum overtopping waves during overtopping discharges of 10–30 l/s per m width. Studies of hazards under steady flows suggest that limits on horizontal velocities for people and vehicles will probably need to be set below $v_x < 2.5$–5 m/s.

On vertical and battered walls, upward projected velocities (v_z) have been related to inshore wave celerity (see Chap. 16). Relative velocities, v_z/c_i, have been found to be roughly constant at $v_z/c_i \approx 2.5$ for pulsating and slightly impulsive conditions, but increase significantly for impulsive conditions, reaching $v_z/c_i \approx 3$–7.

14.6.3.2. *Overtopping loads*

Post-overtopping wave loads have seldom been measured on defense structures, buildings behind sea defenses, or on people; so little generic guidance is available. If loadings from overtopping flows could be important, they should be quantified by interpretation of appropriate field data or by site-specific model studies.

An example model study during the CLASH research project indicates how important these effects might be. A simple 1 m high vertical secondary wall was set in a horizontal promenade about 7 m back from the primary seawall, itself a concrete recurve fronted by a steep beach and short rock armour slope. Pulsating wave pressures were measured on the secondary wall against the effective overtopping discharge arriving at the secondary wall. (This discharge was deduced by applying Eq. (14.7) to the overtopping measured at the primary wall, 7 m in front.) Whilst

strongly site-specific, these results suggest that quite low discharges (0.1–1.0 l/s per m) may lead to loadings up to 5 kPa.

14.6.4. *Effects of debris and sediment in overtopping flows*

There are virtually no data on the effect of debris on hazards caused by overtopping, although anecdotal comments suggest that damage can be substantially increased for a given overtopping discharge or volume if "hard" objects such as rocks, shingle, or timber are included in overtopping. It is known that impact damage can be particularly noticeable for seawalls and promenades where shingle may form the "debris" in heavy or frequent overtopping flows.

14.7. Model Effects, Scale Effects, and Uncertainties

14.7.1. *Model and scale effects*

This section deals with the types of model and scale effects that result from the use of hydraulic models to quantify wave overtopping. Firstly, scale and model effects are defined. Secondly, a methodology based on the current knowledge is introduced on how to account for these effects.

Model or laboratory effects originate from the incorrect reproduction of the geometry or materials of the prototype structure, or of the waves and currents, or due to the boundary conditions of a wave flume (sidewalls, wave paddle, etc.). Modeling techniques have developed significantly, but model effects may still influence test results. One noticeable feature is that very few model tests include wind, whilst wind effects may be significant for some forms of overtopping, particularly splash or spray.

Scale effects result from incorrect (or distorted) reproduction of a prototype water-structure interaction in the model. The ideal model requires that both Froude's and Reynolds' laws are met simultaneously. This is not possible without changing the test fluid; so, scale effects cannot be avoided when performing scaled model tests. They can, however, be minimized for the main processes, and/or corrections can be applied where the distortion is understood.

Gravity, pressure, and inertial forces are the relevant forces for wave motion; so, physical models of seawalls/breakwaters are scaled according to Froude's law. Viscosity forces are governed by Reynolds' law, elasticity by Cauchy's law, and surface tension forces by Weber's law, and these forces are generally neglected for most models. Distortions or errors resulting from ignoring these forces are called scale effects, and are generally unquantified.

Measurement effects result from distortions to the process by the use of measurement equipment and/or data sampling methods. These distortions may significantly influence the comparison of results between prototype and model, or between two models. It is therefore essential to quantify the effects and the uncertainty related to the different techniques available.

Whilst these definitions are reasonably clear, in reality, it is sometimes difficult to assign all the causes of differences between model and prototype data. During CLASH, the major contributions to model effects were found to be wind since this is seldom included in hydraulic models. Additional differences were also found and were ascribed to the model effects in addition to those by the wind.

Examining repeatability of example tests on armoured slopes showed that wave parameters (H_{m0}, T_p, $T_{m-1,0}$) have a coefficient of variation $CoV \sim 3\%$. Differences between overtopping measured in two wave flumes were $CoV \sim 13\%$ and $CoV \sim 10\%$. Different time windows for wave analysis and different types of wave generation methods had little or no influence on the estimated wave parameters ($CoV \sim 3\%$). The number of waves in a test influences overtopping, and the use of 200 waves compared to 1000 waves shows differences in the mean overtopping rates up to 20%. The position of the overtopping tray at the side of the flume also showed differences in the overtopping rates ($CoV \sim 20\%$) from the results where the tray was located at the center of the crest. This could be because of the different arrangements of the armour units in front of the overtopping tray or due to the influence of the sidewalls of the flume.

Scale effects have been investigated by various authors, and this has led to some generic rules that should be observed for physical model studies. Generally, water depths in the model should be much larger than $h = 2.0$ cm, wave periods larger than $T = 0.35$ s, and wave heights larger than $H_s = 5.0$ cm to avoid the effects of surface tension. For rubble mound structures, the Reynolds number for the stability of the armour layer should exceed $Re = 3 \times 10^4$; for overtopping of coastal dikes $Re > 1 \times 10^3$; and stone sizes in underlayers and core of rubble mounds should be scaled according to the velocities in the core rather than the stone dimensions, especially for small models. This leads to the use of larger material in the core than that demanded by simple Froude scaling. Critical limits for the influence of viscosity and surface tension are given in Table 14.8.

From the observations in the prototype and scaled models, a methodology was derived to account for these differences without specifically defining the contribution from the model, scale, or measurement effects. These recommendations are given

Table 14.8. Scale effects and critical limits.

Process	Relevant forces	Similitude law	Critical limits
Wave propagation	Gravity force	Fr_W,	$Re_W > Re_{W,\mathrm{crit}} = 1 \times 10^4$
	Friction forces	Re_W,	$T > 0.35$ s; $h > 2.0$ cm
	Surface tension	We	
Wave breaking	Gravity force	Fr_W,	$Re_W > Re_{W,\mathrm{crit}} = 1 \times 10^4$
	Friction forces	Re_W,	$T > 0.35$ s; $h > 2.0$ cm
	Surface tension	We	
Wave runup	Gravity force	Fr_A, Fr_q	$Re_q > Re_{q,\mathrm{crit}} = 10^3$
	Friction forces	Re_q,	$We > We_{\mathrm{crit}} = 10$
	Surface tension	We	
Wave overtopping	Gravity force	Fr_A, Fr_q,	$Re_q > Re_{q,\mathrm{crit}} = 10^3$
	Friction forces	Re_q,	$We > We_{\mathrm{crit}} = 10$
	Surface tension	We	

in Chaps. 15 and 16 for dikes, for rubble mound slopes, and for vertical walls, respectively.

14.7.2. *Uncertainties in predictions*

Sections 14.2–14.5 have proposed various models or ways to predict wave overtopping of coastal structures. These models will now be discussed with regard to their uncertainties.

14.7.2.1. *Empirical models*

Unless the empirical method has been configured to give an "upper bound" estimate, model uncertainty can usually be described by using a mean factor of 1.0 and a Gaussian distribution around the mean prediction. The standard deviation is derived by comparing model data and the prediction. This has two implications for design. Probabilistic design values for all empirical models used in this manual describe the mean approach for all underlying data points. This means that, for normally distributed variables, about 50% of the data points exceed the prediction by the model, and 50% are below the predicted values. This value should be used if probabilistic design methods are used.

Deterministic design values may be given as the mean value plus one standard deviation, which in general gives a safer approach, and takes into account that model uncertainty for wave overtopping is always significant.

14.7.2.2. *Neural network*

When running the NN, the user is provided with overtopping rates based on the CLASH database and the NN prediction. Together with these results the user will also obtain the uncertainties of the prediction through the 5% and 95% confidence intervals. Assuming a normal distribution will allow the standard deviation of overtopping to be estimated from those confidence intervals, and (if required) the whole Gaussian distribution.

14.7.2.3. *CLASH database*

The CLASH database was described earlier. It provides a large data set of model data on wave overtopping of coastal structures. As these are model data, it is noted that corrections for model/scale effects discussed in Sec. 14.7.1 above have not been applied to the database. The user will therefore need to decide to apply any scale/model correction procedure whenever these data are used for prototype predictions.

With respect to uncertainties, all model results show variations in measured overtopping. Most of these variations will result from the measurement and model effects as discussed earlier, and will therefore apply to the database.

14.8. Guidance on Use of Methods

The *EurOtop* Overtopping Manual[5] is accompanied by an overall Calculation Tool, which includes the following elements:

- **Empirical Calculator** programmed with the main empirical overtopping equations in this chapter and the next two (limited to those that can be described explicitly, that is without iteration).
- **PC-Overtopping**, which codes all the prediction methods for mean overtopping discharge and other parameters, for (generally shallow sloped) sea dikes, see Sec. 14.3.
- **Neural Network** tool developed in the CLASH research project to calculate mean overtopping for many types of structures (see Sec. 14.4).
- **CLASH database**, a listing of input parameters and mean overtopping discharge from each of approximately 10,000 physical model tests on both idealized (research) test structures, and site-specific designs. These data can be sifted to identify test results that may apply for configurations close to the reader's (see Sec. 14.5).

None of these methods give the universally "best" results, and indeed there may still be a need for site-specific model tests for some defenses. The most reliable method to be used will depend on the type and complexity of the structures, and the closeness with which it conforms to simplifying assumptions used in the previous model testing (on which all of the methods above are inherently based).

In selecting which method to use, or which set of results to prefer when using more than one method, the user will need to take account of the origins of each method. It may also be important in some circumstances to use an alternative method to give a check on a particular set of calculations. To assist these judgments, a set of simple rules of thumb are given here, but as ever, these should not be treated as universal truths.

- For **simple vertical, composite, or battered walls** which conform closely to the idealizations in Chap. 16 on vertical walls, the results of the Empirical Calculator are likely to be more reliable than the other methods as test data for these structure types do not feature strongly in the Database or NN, and PC-OVERTOPPING is not applicable.
- For **simple sloped dikes** with a single roughness, many test data have been used to develop the formulae in the Empirical Calculator; so, this may be the most reliable, and simplest to use/check. For dikes with multiple slopes or roughness, PC-OVERTOPPING is likely to be the most reliable, and easiest to use, although independent checking may be more complicated. The Database or NN methods may become more reliable where the structure starts to include further elements.
- For **armoured slopes and mounds**, open mound structures that most closely conform to the simplifying models may best be described by the formulae in the Empirical Calculator. Structures of lower permeability may be modeled using PC-OVERTOPPING. Mounds and slopes with crown walls may be best represented by the application of the Database or NN methods.

- For **unusual or complex structures with multiple elements**, mean overtopping discharge may be most reliably predicted by PC-OVERTOPPING (if applicable) or by the Database or NN methods.
- For structures that require use of the NN method, it is possible that the use of many data for other configurations to develop a single NN method may introduce some averaging. It may therefore be appropriate to check in the database to see whether there are already test data close to the configuration being considered. This procedure may require some familiarity with manipulating these data.

In almost all instances, the use of any of these methods will involve some degree of simplification of the true situation. The further the structure or design (analysis) conditions depart from the idealized configurations tested to generate the methods/tools discussed, the wider will be the uncertainties. Where the importance of the assets being defended is high, and/or the uncertainties in using these methods are large, then the design solution may require use of site-specific physical model tests.

14.9. Conclusions and Outlook

It is clear that increased attention to flood risk reduction, and to wave overtopping in particular, have increased interest and research in this area. The *EurOtop* Overtopping Manual[5] is therefore not expected to be the "last word" on the subject; indeed even whilst preparing the first version of the manual, the author team expected that there will be later revisions. The reader of this handbook is therefore advised to check whether an improved version of the *EurOtop* Overtopping Manual has been released. Beyond that manual, it is probable that there will be significant improvements in numerical modeling, although it should be acknowledged that improved numerical models will require substantial measurement data to validate them before their results can be relied upon in detailed analysis or design.

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