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Improvements in wave overtopping analysis: the EurOtop overtopping manual and calculation tool

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ABSTRACT

This paper describes the new Wave Overtopping Manual developed in UK, the Netherlands and Germany. This new manual extends and updates the EA's Overtopping Manual (W178) edited by Besley (1999), the Netherlands TAW manual edited by Van der Meer (2002), and the German Die Küste (EAK, 2002). Considerable research on overtopping processes and prediction methods since those publications has prompted the production of an updated and extended manual combining European expertise. The new manual will cover more types of sea and shoreline defence structures, will give more details on overtopping responses, and will include a wider choice of how to calculate those responses. The manual will be supported by a Calculation Tool that guides the user through a series of steps to establish empirical overtopping predictions as described in the manual.

1. INTRODUCTION

1.1. Wave overtopping

Wave overtopping has always been of principal concern for coastal structures constructed to defend against flooding (sea defences). Similar structures may also be used to provide protection against coastal erosion (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 nearshore or detached, but most structures used for sea defence or similar function form a part of the shoreline.

Figure 1. Wave overtopping on a revetment seawall

Sloping dikes have been widely used for sea defences along the coasts of the Netherlands, Denmark, Germany, UK. 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

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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 slopes to be steeper. All such embankments need protection against direct wave erosion, often a revetment facing on the seaward side. Revetment facing may take many forms, but may commonly include closely-fitted concrete blockwork; cast in-situ concrete slabs; small rock armouring; or asphaltic materials. Embankment or dike structures are generally most common along rural frontages.

The second main type of coastal structure consists of a mound or layers of quarried rock, protected by rock or concrete armour units. 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 protection to vertical walls or revetments. Rubble mound revetments may also be used to protect embankments formed from relict sand dunes or shingle ridges. Rubble mound structures tend to be more common in areas where harder rock is available.

Figure 2. Wave overtopping on a rubble mound breakwater (courtesy Prof Leo Franco)

Along urban frontages, especially close to ports, defence structures may include vertical (or battered / steep) walls. Such walls 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 subject to intense local wave impact pressures, may overtop suddenly and severely, and will reflect much of the incident wave energy. Reflected waves can 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 generate significant hazards to such developments and their users. Residential and commercial properties along a waterfront will often be used by people who are unaware of the severity or effects of wave overtopping in storm conditions. Regulatory authorities may therefore impose onerous flood defence requirements on new developments. For instance, protection against flooding (including wave overtopping) for any developments in UK is now required to the 0.5% annual probability, equivalent to 1:200 year return. This must include effects of climate change, increasing wave heights and wave periods as well as sea level rise.

1.2. Predicting wave overtopping

A range of different methods are often available to predict overtopping of shoreline structures (usually simplified sections). Each method has strengths or weaknesses in different circumstances. In theory, an **analytical method** can be used to relate the driving process (principally wave action) and the structure to the response through equations based directly on a knowledge of the physics of the process. It is however extremely rare for the structure, waves and overtopping process to be so simple and well-controlled that an analytical method on its own can give reliable predictions. 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.

Figure 3. Wave overtopping on a battered / vertical seawall

Two other methods have been derived during the CLASH project 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 does however require some familiarity with these data. A simpler approach is to use the Neural Network tool that has been trained using the database. The **Neural Network** tool can be run automatically as a stand-alone device to predict mean overtopping discharges, 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, two alternative methods may be used, both more complicated than the methods above. A range of **numerical models** can be used to simulate the process of overtopping. They all 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.

The final method is **physical modelling** in which a scale model is tested with correctly scaled wave conditions. Such models may be built to a geometric scale typically in the range 1:10 to 1:60. Waves will be generated as random wave trains each conforming to a particular energy spectrum. For many cases, the structure cross-section may be represented by a 2-dimensional model in a wave flume. Structures with more complex plan shapes, junctions, transitions etc., should however be tested in a 3-dimensional 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 indirect responses.

1.3. Performance requirements

Most sea defence structures are constructed primarily to limit overtopping volumes. For defences that protect people living, working or enjoying themselves, designers and owners of these defences must, however, also deal with 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.

2. EMPIRICAL MODELS

Empirical methods simplify the physics of the process in equations (often dimensionless) 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 rare.

The mean overtopping discharge, q, is the main overtopping response. It is not the only measure of overtopping, but is easy to measure in a laboratory wave flume or basin (or even in the field), and most other parameters are related to the mean overtopping discharge. This discharge is generally calculated in m^3/s per m width, but in practical applications it may be quoted as litres/s per m width. Although it is given as a mean discharge, it is usually far from steady as the processes of wave overtopping are much more dynamic. For most defences, 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. Individual volumes in wave-by-wave overtopping are less easy to measure in a laboratory than mean discharges, so data on wave-by-wave volumes are rarer. Data have however been analysed for a number of idealised structures and maximum overtopping volumes can be estimated if mean discharge, q, storm duration, t, and percentage of overtopping waves, $N_{ow\%}$, are known.

As mean overtopping discharges are relatively easy to measure, many physical model tests have been performed all over the world, both for idealised structures and real applications or designs. The European CLASH project collected a database worldwide with more than 10,000 wave overtopping test results on all kind of structures. Some tests had been used directly to develop empirical prediction methods for overtopping. Such empirical methods or formulae are however only 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.

3. PC-OVERTOPPING

The computer program PC-OVERTOPPING was created using results of the TAW (2002) report “Wave run-up and wave overtopping at dikes” and is used for the 5-yearly safety assessment of all water defences in the Netherlands. The TAW Report has now been replaced by the EurOtop Overtopping Manual (2007) and the calculation program has been translated into English and is available from the EurOtop Overtopping Manual website.

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 modelled 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 modelled 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% run-up 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.

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 storm duration and mean period) for calculation of overtopping volumes.

The output gives the 2%-run-up level, the mean overtopping discharge and the percentage of overtopping waves. If the 2%-run-up level is higher than the actual dike crest, this level is calculated by extending the highest section in the cross-section. The required dike height for given mean overtopping discharges is also calculated. Finally, the output gives the number of overtopping waves in the given storm duration, together with the maximum overtopping volume, and volumes for specified overtopping percentages.

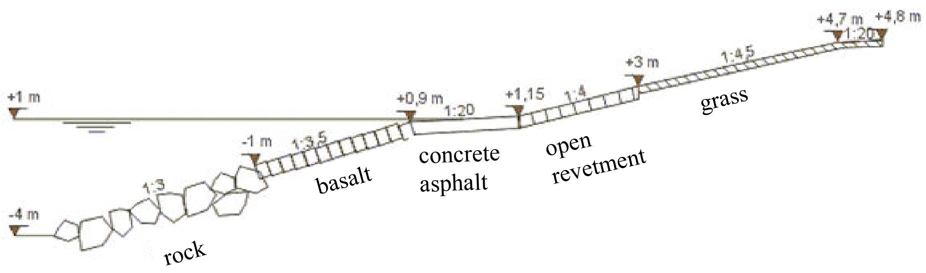

Figure 4. Example cross-section of a dike

4. NEURAL NETWORK TOOL

Neural Networks (NNs) are tools that allow meaning to be extracted from very large quantities of data. They 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 a neural network is useful where:

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

It has already been seen that overtopping cannot be predicted by a single formula, but requires a number of different formulae. A single neural network 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 CLASH project collected many test results on wave overtopping for all kinds of coastal structures and embankments.

Development of a neural network requires that all data be checked thoroughly (rubbish in = rubbish out), and that training be done by those with appropriate skills. Using a neural network as a prediction tool, however, is easy for most practical engineers! It is for this reason that the CLASH neural network was adopted as part of the EurOtop Overtopping Manual (2007).

Applying the NN requires an Excel or ASCII input file with parameters, run the programme (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 neural networks. 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 4 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 (2 parameters), two structure slopes (including vertical and wave return walls), a berm (2 parameters) and a crest configuration (3 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. Possible structure configurations are shown in Figure 5.

5. USE OF CLASH DATABASE

The EU research project CLASH generated an extensive database of overtopping from model test results submitted around the world. Each test was described 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 website.

If a user has a specific structure, they can interrogate the database to find whether a similar structures has been tested previously. It may even be possible that a similar 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.

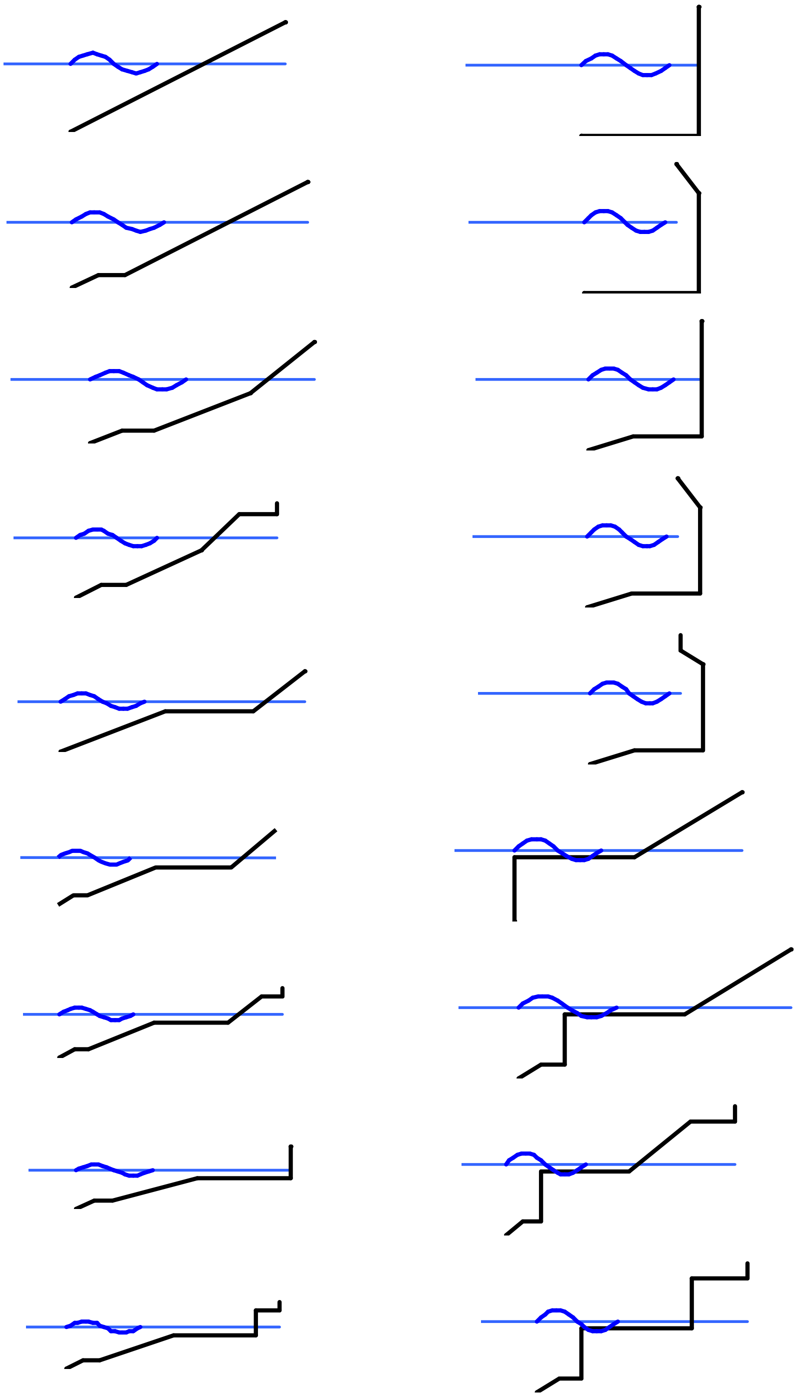

Figure 5. Possible structure configurations for the neural network

6. TOLERABLE DISCHARGES

6.1. Hazards from overtopping

Most sea defence structures are constructed primarily to limit overtopping that might cause flooding. Over a particular storm or tide, overtopping 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 modelling inundation flows (perhaps by wave overtopping) 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 here. Instead, advice in this section focuses on direct hazards from wave overtopping.

For sea defences that protect people living, working or enjoying themselves, designers and owners of these defences 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 defence 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 UK and Italy through wave action, chiefly on seawalls and similar structures (although this rose to 11 in UK during 2005). It is helpful to analyse direct wave and overtopping effects, and their consequences under three categories:

a) Direct hazard of injury or death to people immediately behind the defence;
b) 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;
c) Damage to defence structure(s), either short-term or longer-term, with the possibility of breaching and flooding.

Overtopping flows and their hazards also depend upon the geometries of the structure, the hinterland behind the seawall, and the form of overtopping. Rising ground behind the seawall will allow people at potential risk to see incoming waves, and the rising slope will slow overtopping flows. Conversely, a defence that is elevated above the land defended will obscure visibility of incoming waves, see Figure 6, and post overtopping flows may increase in speed rather than reduce. 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 on the circumstances and uses of the site.

Figure 6. Defended area below seawall and foreshore (saltmarsh) level

Wave overtopping processes and hazards

Overtopping hazards can be linked to a number of simple direct flow parameters:

- mean overtopping discharge, q;
- individual and maximum overtopping volumes, V_i and V_{max};
- overtopping velocities over the crest, horizontally and vertically,;
- overtopping flow depth, again measured on crest or promenade.

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

a) Move human activities away from areas subject to overtopping or flooding hazards, thus modifying the land-use category and/or habitat status;
b) 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 defence systems;
c) Increase defence standard to reduce risk to (permanently) acceptable levels probably by enhancing the defence and / or reducing loadings.

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

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

Under most wave attack, waves tend to break before or onto sloping embankments with the process being relatively gentle, see Figure 1. Relatively few water levels and wave conditions 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 example in Figure 3. The form of breaking will therefore influence the distribution of overtopping volumes and their velocities, both of which impact on the hazards.

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 termed ‘green-water’. In contrast, ‘white-water’ or spray overtopping tends to occur when waves break seaward of the defence structure or break onto its seaward face, producing non-continuous 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 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 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 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 driver’s visibility (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 modelled. Some research studies have suggested that effects of onshore winds on green-water overtopping are small, but overtopping under $q = 1$ l/s/m might increase by up to 4 times under strong winds, especially where much of the overtopping is spray. Discharges between $q = 1$ to 0.1 l/s/m are already greater than some discharge limits for pedestrians or vehicles, suggesting that wind effects may influence overtopping at and near acceptable limits for these hazards.

Table 1. Hazard type

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

[1] *Note:* *Total probability return period*

Return periods

Return periods at which overtopping hazards are analysed, and against which defences may be designed, are often 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 x 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 at longer return periods. Guidance on example return periods used in evaluating levels of protection are shown in Table 1.

In practice, some of the return periods in Table 1 may be too short. National guidelines have recommended lower risk, e.g. a low probability of flooding in UK is now taken as <0.1% probability

(1:1000 year return) and medium probability of sea flooding as between 0.5% and 0.1% (1:200 to 1:1000 year return). Many existing defences however offer levels of protection far lower than these.

In the Netherlands, where two-thirds of the country lies below storm surge level, protection was substantially improved after the flood in 1953 where almost 2000 people drowned. Standards of protection for large areas are currently 1:10,000 years, less densely populated areas at 1:4,000 years and protection for high river discharge (without threat of storm surge) is given to 1:1,250 years.

The design life for flood defences 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 defence, the design life is taken as 100 years, and this is now used in the UK for residential properties, 60 years for commercial developments. Design life increases for special structures with high capital costs, like the barriers to the Eastern Scheldt, River Thames, or the entrance to Rotterdam, perhaps around 200 years.

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

6.2. Tolerable mean discharges

Guidance on overtopping that can cause damage to seawalls, buildings or infrastructure, or danger to pedestrians and vehicles, has been related to mean overtopping discharges or (less often) to peak volumes. Limits quoted previously were derived initially from 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/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 fast speeds for vehicles.

Historic tests on effects of overtopping on people suggested that information on mean discharges alone will 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 distance away from the defence line. As a rule of thumb, the hazard effect of an overtopping discharge at a point x metres back from the seawall crest will be to reduce the effect of overtopping at the seawall, $q_{seawall}$, by a factor of x, so that the effective overtopping discharge at set-back distance x (over a range of 5 - 25m), $q_{effective}$ is given by:

$$q_{effective} = q_{seawall} / x \tag{1}$$

Table 2. Limits for overtopping for pedestrians

Hazard type and reason	Mean discharge q (l/s/m)	Max volume[1] 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[2].	0.1	20 – 50 at high level or velocity

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

Overtopping limits suggested in Tables 2 to 5 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 2 show a logical sequence, with allowable discharges reducing 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/m may apply for 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/m in Table 2 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 3 applies where wave overtopping generates pulsating flows at roadway level, akin to driving through slowly varying fluvial flows. The lower overtopping limit in Table 3 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.

Table 3. Limits for overtopping for vehicles

Hazard type and reason	Mean discharge q (l/s/m)	Max volume(1) V_{max} (l/m)
Driving at low speed, overtopping by pulsating flows at low flow depths, no falling jets, vehicle not immersed	10 – 50(1)	100 – 1,000
Driving at moderate or high speed, impulsive overtopping giving falling or high velocity jets	0.01 – 0.05(2)	5 – 50(2) at high level or velocity

(1) Note: These limits probably relate to overtopping defined at highway.
(2) Note: These limits relate to overtopping defined at the defence, but assumes the highway to be immediately behind the defence.

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 defence elements intended to disrupt overtopping momentum. Guidance derived from CLASH and previous work suggests limits in Table 4 for damage to buildings, equipment or vessels behind defences.

Table 4. Limits for overtopping for property behind the defence

Hazard type and reason	Mean discharge q (l/s/m)	Max volume(1) V_{max} (l/m)
Significant damage or sinking of larger yachts	50	5,000 – 50,000
Sinking small boats set 5-10m from wall. Damage to larger yachts	10(1)	1,000 – 10,000
Building structure elements	1(2)	~
Damage to equipment set back 5-10m	0.4(1)	~

(1) Note: *These limits relate to overtopping defined at the defence.*
(2) Note: *This limit relates to the effective overtopping defined at the building.*

Limits for defence structures in Table 5 have been derived from early work by Goda 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, so overtopping flows pass over the crest and down the rear face;
- promenade defence in which overtopping flows remain on or behind the seawall crest before returning seaward.

The limits for the latter category cannot be applied where overtopping flows can fall from the defence crest onto the promenade or rear face where the nature of the flow may be more impulsive.

Table 5. Limits for overtopping for damage to the defence crest or rear slope

Hazard type and reason	Mean discharge q (l/s/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

6.3. Tolerable maximum volumes and velocities

Overtopping volumes and velocities

Guidance on suggested limits for maximum individual overtopping volumes have been given in Tables 2-5 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 to 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. For simple sloping embankments, velocities of 5-8 m/s are possible for maximum overtopping waves during overtopping of 10-30 l/s per m width. Studies of hazards under steady flows suggest that limits on horizontal velocities (v_x) for people and vehicles will probably need to be set below v_x < 2-5 m/s.

On vertical and battered walls, upward projected velocities (v_z) have been related to inshore wave celerity. 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 $v_z/c_i \approx 3 – 10$.

Post-overtopping loads

Post-overtopping wave loads have seldom been measured on defence structures, buildings behind sea defences, or on people, so little generic guidance is available. There are however anecdotal evidence that such loads can be severe, so if important to the structure design, they should be quantified by interpretation of appropriate field data or by site-specific model studies.

An example model study during the CLASH 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 Equation 1 to 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/m) may lead to loadings up to)or perhaps in excess of) 5 kPa.

7. GUIDANCE AND CONCLUSIONS

The new EurOtop Overtopping Manual (2007) extends and updates the EA's Overtopping Manual (W178) edited by Besley (1999), the Netherlands TAW manual edited by Van der Meer (2002), and the German Die Küste (EAK, 2002). Considerable research on overtopping processes and prediction methods since those publications has prompted the development of an updated and extended manual combining European expertise in understanding and predicting wave overtopping. The new manual will cover more types of sea and shoreline defence structures, will give more details on overtopping responses, and will include a wider choice of how to calculate those responses.

The manual is accompanied by an overall Calculation Tool, which includes:

- **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 Section 3.
- **Neural Network** tool developed in the CLASH research project to calculate mean overtopping for many types of structures, see Section 4.
- **CLASH database**, a listing of input parameters and mean overtopping discharge from each of approximately 10,000 physical model tests on both idealised (research) test structures, and site specific designs.

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

In selecting which method, or which 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 judgements, a set of simple rules of thumb are given here, but should not be treated as universal truths.

- For **simple vertical, composite, or battered walls** which conform closely to the idealisations for vertical walls, 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 Neural Network, 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 Neural Network methods may become more reliable where the structure starts to include further elements.
- **Armoured slopes and mounds** that most closely conform to the simplifying models may best be described by formulae in the Empirical Calculator. Structures of lower permeability may be modelled using PC-OVERTOPPING. Mounds and slopes with crown walls may be best represented by application of the Database or Neural Network 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 Neural Network methods.
- For structures that require use of the Neural Network method, it is possible that the use of many data for other configurations to develop a single Neural Network 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 that the structure or design (analysis) conditions depart from the idealised 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.

Increased attention to flood risk reduction, and to wave overtopping in particular, have increased interest and research in this area. The EurOtop Manual 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 future revisions. The reader is therefore advised to check whether an improved version of the EurOtop Manual has been released. Beyond that manual, it is probable that there will be significant improvements in numerical modelling, 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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