DIRECT HAZARDS FROM WAVE OVERTOPPING – THE FORGOTTEN ASPECT OF COASTAL FLOOD RISK ASSESSMENT?

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ABSTRACT

Any assessment of flood risk for a coastal site should include assessment of direct hazards caused by wave overtopping as well as flood extent and volumes. The direct hazards are likely to be critical for users and property on or immediately behind the defence line. This paper summarises recent advances in methods to predict wave overtopping; summarises present guidance on levels of direct hazards caused by wave overtopping; and discusses key areas where present knowledge and guidance are insufficient.

Recent work in UK, the Netherlands and Germany has derived substantially improved and extended guidance on the prediction of wave overtopping in the new EurOtop Manual http://www.overtopping-manual.com/. The EurOtop Manual has drawn together results of national and European research projects to formulate the best prediction methods and advice available. Those tools are based on results of previous field and laboratory studies, most of which require simplifying the structure being analysed. The paper discusses the manual, but will also highlight areas for future improvement, both in prediction tools for overtopping, and its consequences, and hence guidance on tolerable limits.

Increasing emphasis on mitigating flood risks, on adaptation of defences, and on improving resilience of infrastructure all require better guidance for effective management of coastal defences. Reducing flood risks requires methods to warn or safeguard individuals against overtopping hazards, particularly as studies under CLASH (see: http://www.clash-eu.org/) suggest that on 4-8 people are killed each year in UK through direct effects of waves on seawalls and similar structures. It is difficult to find records of overtopping damage to buildings or transport operations, but site specific model tests by HRW suggest that post-overtopping wave loads can substantially exceed load levels (wind etc.) against which typical building facades are designed. Stoppages to trains on the line at Dawlish by wave overtopping confirm that these hazards are significant, sometimes under relatively frequent conditions. So future analysis, predictions and guidance will require improvements on:

a) What overtopping can be tolerated by grass or promenade protection?
b) Safe velocities / depths of overtopping for pedestrians or vehicles?
c) Violent (impulsive) overtopping tolerable by pedestrians or vehicles?
d) How do the effects of overtopping alter with distance and/or wind speed?
e) Loadings on buildings / defences subject to wave splash or spray overtopping?
f) How can impulsive loadings from wave splash or spray be scaled to full scale?

The paper will illustrate the new methods to predict overtopping from the EurOtop overtopping manual (see: http://www.overtopping-manual.com/manual.html), and will describe the tools available through the EurOtop calculation tool web-site, http://www.overtopping-manual.com/calculation_tool.html.

The paper will describe current guidance on overtopping hazards; and will highlight the key gaps in current knowledge, suggesting possible ways to overcome these gaps and thus to reduce present uncertainties.

1. INTRODUCTION

Wave overtopping has always been of principal concern for coastal structures constructed to defend against flooding: often termed sea defences. Similar structures may also be used to provide protection against coastal erosion: sometimes termed coast protection. Other structures may be built to protect ship navigation or mooring within ports, harbours or marinas formed by breakwaters. 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 defence or similar function form a part of the shoreline.

Figure 1: Wave overtopping on a revetment seawall

Sloping dikes or embankment seawalls are widely used along the coasts of the Netherlands, Denmark, Germany, UK, and in China, Korea and Vietnam. Embankment seawalls may subsume an original sand dune line, with the main embankment core formed by sand overlain by clay. Similar structures around UK have been formed by clay allowing side slopes to be steeper. All such embankments need protection against direct wave erosion, often using a revetment facing on the seaward side, commonly closely-fitted concrete blockwork, cast in-situ concrete slabs, or asphaltic materials. Embankment or dike structures are most common along rural frontages.

Figure 2: Wave overtopping on a battered / vertical seawall

A second coastal structure is formed by a mound or layers of quarried rock fill, protected by rock or concrete armour units. The outer armour layer must resist wave action without significant armour displacement. Under-layers of quarry or crushed rock support the armour and separate it from finer material in the mound. Porous materials dissipate part of the incident wave energy in breaking and turbulence. Simple rubble mounds may be used as seawalls or protection to walls or revetments. Rubble mound structures tend to be more common in areas where harder rock is available.

Figure 3: Wave overtopping on a rubble mound breakwater (courtesy Prof. Leo Franco)

Urban frontages and ports may use vertical (or battered / steep) walls to defend against wave action. Such walls may use stone or concrete blocks, mass concrete, or sheet steel piles, and may also act as retaining walls to material behind. Shaped and recurved wave return walls may be formed as walls in their own right. Most such seawalls are impermeable to wave action. They may be liable to intense wave impact pressures, may overtop severely, and will reflect much of the incident wave energy, which may cause additional wave disturbance and/or initiate or accelerate local bed scour.

Developments along waterfronts are highly valued with prices substantially above those for properties further inland. 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 defence requirements on new developments. For instance, protection against flooding (including wave overtopping) for any new developments in UK is now required for 100 year life at the 0.5% annual probability, equivalent to 1:200 year return. Exposure to overtopping of many such sites will be influenced by climate change, increasing wave heights / periods as well as sea level rise.

Overtopping of particular structures (usually simplified sections) under given wave conditions and water levels may be predicted by a number of different methods, each strengths or weaknesses in different circumstances. **Analytical methods** can be used to predict structure responses through equations based on the physics of the process. It is rare however for the structure, the waves and the overtopping process to all be so 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 response to wave and structure parameters using data to derive appropriate empirical equations. Example empirical methods (although quite sophisticated) are used in the computer program PC-OVERTOPPING, created using results in the TAW (2002) report and used for 5-yearly safety assessments of water defences in the Netherlands. PC-OVERTOPPING has been translated into English and is available from the EurOtop website, http://www.overtopping-manual.com/manual.html

Two other methods were derived during the CLASH project based on the use of measured overtopping from model tests and field measurements, and are available through the EurOtop calculation tool. The first uses the **CLASH database** of overtopping tests, each described by 31 parameters. Using the database does however requiring some familiarity with these data. A simpler and more rapid approach is to use a **Neural Network** tool trained using the database. The Neural Network tool can be run as a stand-alone device, or embedded within other simulation methods.

For cases where empirical data do not already exist, or where other methods do not give reliable results, then two alternative (but more complicated) methods may be used. **Numerical models** can simulate the process of overtopping. All such models involve some simplification of the overtopping process although the most complex CFD tools can simulate all processes of relevance, if at a cost of computer time and code complexity. Simpler models will be run for sequences of waves (say 1000 waves) giving overtopping on a wave-by-wave basis. The final method here is **physical modelling** in which a scale model is tested with scaled wave conditions, typically at scale of 1:10 to 1:60. Waves will be generated as random wave trains each to a given wave spectrum. Two-dimensional model will represent structure cross-sections in a wave flume. Structures with more complex plan shapes, junctions, transitions etc., may be tested in a 3 dimensional model in a wave basin. Physical models can measure many aspects of overtopping including wave-by-wave volumes, overtopping velocities and depths, and post-overtopping loads.

2. SOURCES AND TYPES OF OVERTOPPING HAZARDS

Most sea defence structures are constructed primarily to limit overtopping that might cause flooding. Over a particular storm or tide, overtopping volumes that can be tolerated will be site specific, depending 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 has been developed within the Floodsite project (see: http://www.floodsite.net/html/project_overview.htm), but flood volumes, per se, are not discussed further in this paper. Instead, advice here focuses on direct hazards from wave overtopping.

Figure 4: Defended area below seawall and foreshore level

For defences that protect people more directlyagainst overtopping hazards, owners must assess the level of hazard and its probability, allowing appropriate plans to ameliorate overtopping risks. 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 the UK through wave action, chiefly on seawalls and similar structures (although this rose to 11 in 2005). To understand and counteract these hazards, it is often helpful to analyse direct wave and overtopping effects, and their consequences under three general 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.

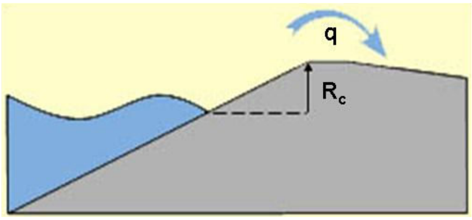
The character of overtopping flows / jets, and their hazards, also depend upon the geometries of the structure, the hinterland behind the seawall, and the form of overtopping. Rising ground behind the seawall may allow people to see incoming waves, and the slope will slow overtopping flows. Conversely, a defence that is elevated above the land will obscure visibility of incoming waves, see

Figure 4, and post-overtopping flows may increase in speed rather than reduce. It is not therefore possible to give unambiguous limits to tolerable overtopping for all conditions. Some guidance is, however, offered here on mean discharges and maximum overtopping volumes for a range of circumstances or uses, and on inundation flows and depths. Overtopping hazards can be linked to a number of simple direct flow parameters (see Figure 5):

- mean overtopping discharge, q;
- individual and maximum overtopping volumes, V_i and V_{max};
- crest overtopping velocities, horizontally and vertically, v_{xc} and v_{zc} or v_{xp} and v_{zp};
- overtopping flow depth, on crest or promenade, d_{xc} or d_{xp}.

Less direct responses / further back from the defence may be used to assess effects of overtopping, perhaps categorised 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}.



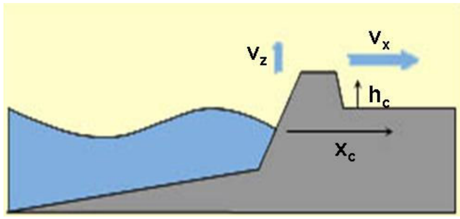


Figure 5: Overtopping on embankment and promenade seawalls

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 the area subject to overtopping or flooding hazard, 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 required to reduce overtopping, the crest level and/or 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 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 forms of wave attack, waves tend to break before or onto sloping embankments with the overtopping process being relatively gentle, Figure 1. A few combinations of water level and wave condition may however cause “impulsive” breaking where overtopping flows are sudden and violent. Conversely, steeper, vertical or compound structures are more likely to experience occasional but intense local breaking, perhaps overtopping violently and with greater velocities, Figure 3. The form of breaking will therefore influence the distribution of overtopping volumes and their velocities, both of which will impact on the hazards that they cause.

There is little guidance on the effects of spray, and some of it is contradictory and/or confusing. Under still conditions or light onshore winds, overtopping spray will seldom contribute significantly to overtopping volumes, but might cause local hazards. Spray will extend salt damage to vegetation and deterioration of buildings. Spray on coastal highways (particularly when intermittent) can cause sudden loss of visibility, leading drivers to veer suddenly. Limited research studies suggest that onshore winds on large green water overtopping have little effect, but that overtopping below q = 1 l/s/m might increase by up to 4 times under strong winds, especially where much of the overtopping is as spray. Discharges between q = 0.1 to 1.0 l/s/m are however already greater than some limits suggested for pedestrians or vehicles, suggesting that wind effects may influence overtopping at and near acceptable limits for these hazards.

Return periods against which a defence might be designed, are generally set by national guidelines, or by owner requirements. Acceptable risk levels (probability x consequence) also depend on the balance between risk and benefits, and the level of overall exposure. Heavily trafficked areas might 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. National guidelines generally recommend that new developments be designed for low risk, e.g. low probability in UK may be 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 UK defences however offer levels of protection far lower than these. In the Netherlands, 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 1:4,000 years and protection for high river discharge (without threat of storm surge) is given to 1:1,250 years.

3. TOLERABLE OVERTOPPING

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 overtopping volumes. Suggested limits quoted previously were derived from analysis in Japan of overtopping perceived by port engineers to be safe, later extended by guidance from Iceland for ddamage to port equipment or cargo. Significantly different limits are suggested for embankments 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. There are no generic data on overtopping velocities apart from that derived for vertical walls by Bruce et al (2002) included in guidance by Allsop et al (2005) and Schüttrumpf and Van Gent (2003) on crests and inner slopes of embankments.

Tests on effects of overtopping on people suggest that mean overtopping discharges alone may not give reliable indicators of safety for some circumstances. Maximum individual volumes might be better indicators of hazard than average discharges. Volumes of largest events vary significantly with wave condition and structure type, even for a given mean discharge. There remain two difficulties in specifying safety levels with reference to volumes rather than 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.

Table 1. 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.

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 defence line. As a rule of thumb, the hazard effect of overtopping at a distance x metres back from the seawall crest will be to reduce the effect of the overtopping by a factor of x, so the effective overtopping discharge at x (over a range of 5 - 25m), $q_{effective}$ is given by:

$$q_{effective} = q_{seawall} / x \qquad (1)$$

The overtopping limits suggested in Tables 1 to 4 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 1 show a logical sequence, with allowable discharges reducing steadily as the recipient's ability or willingness to anticipate or receive the hazard reduces.

A precautionary limit of q = 0.03 l/s/m might apply for conditions where pedestrians have no clear view of incoming waves; may be easily upset or frightened; are not dressed to get wet; may be on a narrow walkway; or are 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 1 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 2 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 2 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 2. 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 (not necessarily reducing discharges). Guidance derived from the CLASH research project and previous work suggests limits in Table 3 for damage to buildings, equipment or vessels behind defences.

Table 3. 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)]	~
Sinking small boats set 5-10m from wall. Damage to equipment set back 5-10m	0.4[(1)]	~

[(1)] *Note: These limits probably relate to overtopping defined at the defence.*
[(2)] *Note: These limits relate to overtopping defined at the building.*

A set of limits for structures in Table 4 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:

a) embankment seawall or dike, elevated above the defended area, so overtopping flows pass over the crest and down the rear face;

b) 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 the overtopping flows can fall from the defence crest, and where the nature of the flow may be more impulsive. The limits in Table 4 are precautionary and are generally based on old data.

Table 4. 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

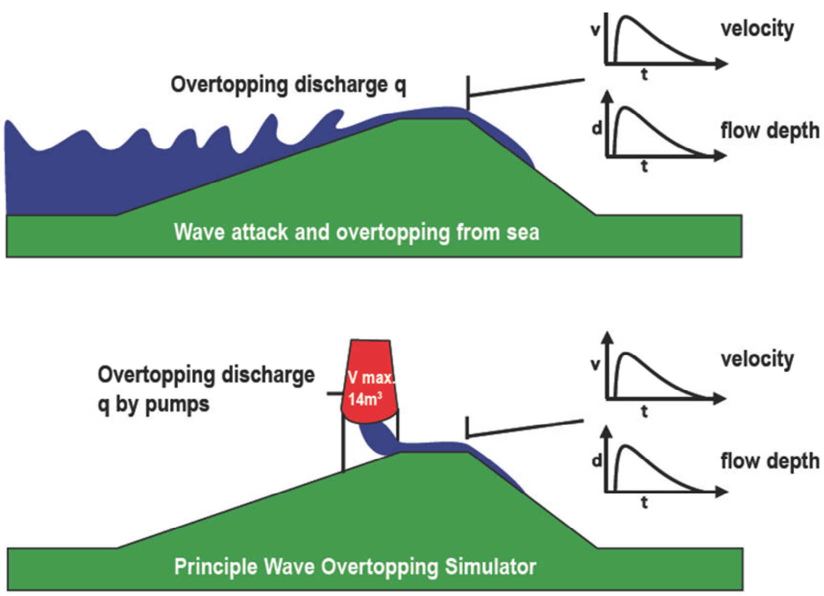


Figure 6: Principle of the wave overtopping simulator

In order to clarify erosion resistance of grass protection under wave overtopping, tests have been performed using the overtopping simulator (Figures 6 and 7) in 2007 and 2008 on a four seadikes in the Netherlands. For the 2007 tests, 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 hour storm for each overtopping condition. These started with overtopping equivalent to a mean discharge of 0.1 l/s/m and increased to 1; 10; 20; 30 and finally even 50 l/s/m. After all these simulated storms the slope was still in good condition and showed little erosion.

Another of the 2007 tests 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/m; 1; 5 and finally 10 l/s/m were performed, again for 6 hours each. Erosion damage started for the first condition (two erosion holes) and increased during the other overtopping conditions. After 6 hours at a mean discharge of 10 l/s/m (see Figures 7 and 8) 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 clay quality and type of grass cover still may play a role and, therefore, more testing is required to come to general conclusions.

Figure 7: Overtopping simulator discharging a large volume on the inner slope of a dike

Figure 8: Wave overtopping test on bare clay; result after 6 hours with 10 l/s per m width

In 2008 the wave overtopping simulator was enlarged to simulate mean discharges up to 75 l/s per m. In total 9 different dike sections were tested, including dikes with a sand core and clay layer, less erosion resistent clay (larger sand content), steeper inner slope (1:2.5), bad coverage of grass, initial damages on the inner slope by farmer work, many mole holes, and finally overtopping resistant solutions with open asphalt concrete and a newly developed system called elastocoast (small gravel glued together by two-component glue).

Results are still being analysed, but two main observations are:

- No dike section failed or showed significant erosion damage for mean overtopping discharges of 30 l/s per m or less. Only one section failed for 50 l/s per m. Some sections failed at 75 l/s per m and some sections did not show significant damage after full testing.
- In 6 out of 9 sections significant damage occurred at the horizontal inner toe of the dike. It appears that the transition from a slope to a horizontal section is a weak point with regard to substantial wave overtopping.

It should be noted direct erosion of the slope is only one possible failure mechanism. A major failure mechanism on steep inner faces (typically 1:1.5 and 1:2) in the past was slip failure of the (rear) slope. Such slip failures may lead directly to a breach. For this reason most dike designs in the Netherlands in the past fifty 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, so should be taken into account in safety analysis.

4. TOLERABLE OVERTOPPING VOLUMES / VELOCITIES

Guidance on suggested limits for maximum individual overtopping volumes have been given in Tables 1-4 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. The EurOtop manual gives guidance on overtopping flow velocities and depths for embankments. Example data suggest that 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 $v_x < 2.5$ m/s. On vertical and battered walls, upward projected velocities (v_z) have been related to inshore wave celerity, see Bruce et al (2002). 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 $v_z/c_i \approx 3 – 7$.

5. OVERTOPPING LOADS AND EFFECTS

Post-overtopping wave loads have seldom been measured on defence structures, buildings behind sea defences, or on people, so little generic guidance is available. Where loadings from overtopping flows can 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 project indicated 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. Whilst strongly site specific, these results suggest that quite low discharges (0.1-1.0 l/s/m) might lead to loadings up to 5 kPa. Since that research, a site specific study at HRW measured wave loads on a building approximately 20m back from a vertical wall, albeit on in which 3-d features caused impulsive breaking. Post-overtopping (quasi-static) wave loads on the building (at 1/250 exceedance level) reached $p_{q\text{-}s1/250} \approx 2$ kPa for the 1:1000 year conditions, but impulsive loads could reach $p_{imp1/250} \approx 20$ kPa.

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.

6. CONCLUSIONS AND OUTLOOK

It is clear that increased attention to flood risk reduction, and to wave overtopping in particular, have increased interest in this area. The EurOtop Manual has consolidated advice on predicting wave overtopping. It is hoped that these improvements will be maintained by future revisions / extensions of the manual. There will also be improvements in numerical modelling, although improved models will require substantial measurement data to validate them before their results can be relied upon in detailed analysis or design. The major areas for future work are however in understanding and quantifying post-overtopping effects, particularly the influence of flows, jets and spray on hazards to defence assets, and to people and buildings

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