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3. Tolerable discharges

3.1 Introduction

Most sea defence structures are constructed primarily to limit overtopping volumes that might cause flooding. Over a storm or tide, the overtopping volumes that can be tolerated will be site specific as the volume of water that can be permitted 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 is being developed within Floodsite (FLOODSITE), but flooding volumes and flows, per se, are not distinguished further in this chapter.

For sea defences that protect people living, working or enjoying themselves, designers and owners of these defences 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.

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 often helpful to analyse direct wave and overtopping effects, and their consequences under four 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;
d) Low depth flooding (inconvenient but not dangerous).

The character of overtopping flows or jets, and the hazards they cause, also depend upon the geometry of the structure and of the immediate hinterland behind the seawall crest, and the form of overtopping. For instance, rising ground behind the seawall may permit visibility of incoming waves, and will slow overtopping flows. Conversely, a defence that is elevated significantly above the land defended may obscure visibility of incoming waves, 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.

3.1.1 Wave overtopping processes and hazards

Hazards driven by overtopping 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 or promenade, 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 defence) may be used to assess the 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}.

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

a) Move human activities away from the area subject to overtopping and/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 expected to ameliorate wave overtopping, the crest level and/or the front face configuration will be dimensioned to give acceptable levels of wave overtopping under specified extreme conditions or combined conditions (e.g. water level and waves). Setting acceptable levels of overtopping depends on:

- the 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. 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. 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.

Additional hazards that are not dealt with here are those that arise from wave reflections, often associated with steep faced defences. Reflected waves increase wave disturbance, which may cause hazards to navigating or moored vessels; may increase waves along neighbouring frontages, and/or may initiate or accelerate local bed erosion thus increasing depth-limited wave heights (see section 2.4).

3.1.2 Types of overtopping

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 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 also be generated by wind acting directly on wave crests, particularly when reflected waves interact with incoming waves to give severe local 'clapotii'. 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 modelled. 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/m might increase by up to 4 times under strong winds, especially where much of the overtopping is as spray. Discharges between q = 1 to 0.1 l/s/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.

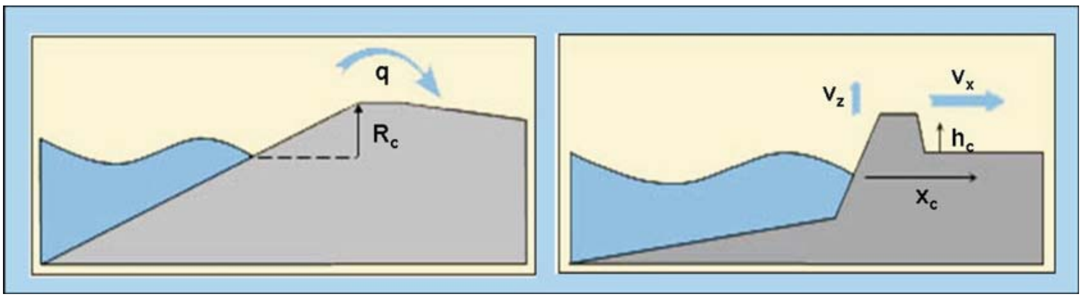

Fig. 3.1: Overtopping on embankment and promenade seawalls

3.1.3 Return periods

Return periods at which overtopping hazards are analysed, and against which a defence 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 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 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 3.1.

In practice, some of these return periods may be regarded as 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 sea defences in the UK however offer levels of protection far lower than these.

Table 3.1: Hazard Type

Hazard type and reason	Design life	Level of Protection[(1)]
	(years)	(years)
Temporary or short term measures	1–20	5–50
Majority of coast protection or sea defence walls	30–70	50–100
Flood defences protecting large areas at risk	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*

It is well known that the Netherlands is low-lying with two-thirds of the country below storm surge level. Levels of protection were increased after the flood in 1953 where almost 2000 people drowned. Large rural areas have a level of protection of 10,000 years, less densely populated areas a level of 4,000 years and protection for high river discharge (without threat of storm surge) of 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. This design life increases for very special structures with high capital costs, like the Eastern Scheldt storm surge barrier, Thames barrier, or the Maeslandtkering in 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 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.

3.2 Tolerable mean discharges

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 analysis in Japan of overtopping perceived by port engineers to be safe (GODA et al. [1975], FUKUDA et al. [1974]). 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 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 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 overtopping discharge by a factor of *x*, so the effective overtopping discharge at *x* (over a range of 5–25 m), $q_{effective}$ is given by:

$$q_{effective} = q_{seawall}/x. \tag{3.1}$$

The overtopping limits suggested in Table 3.2 to Table 3.5 therefore 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 3.2 show a logical sequence, with allowable discharges reducing steadily as the recipient's ability or willingness to anticipate or receive the hazard reduces.

Table 3.2: Limits for overtopping for pedestrians

Hazard type and reason	**Mean discharge**	**Max volume**[(1)]
	q (l/s/m)	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 $v_c \approx 10$ 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.*

A further precautionary limit of $q = 0.03$ l/s/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/m in Table 3.2 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 3.3 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 3.3 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 3.3: Limits for overtopping for vehicles

Hazard type and reason	Mean discharge	Max volume
	q (l/s/m)	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 3.4 for damage to buildings, equipment or vessels behind defences.

Table 3.4: Limits for overtopping for property behind the defence

Hazard type and reason	Mean discharge	Max volume
	q (l/s/m)	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(1)	1,000–10,000
Building structure elements	1(2)	~
Damage to equipment set back 5–10 m	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.*

A set of limits for defence structures in Table 3.5 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, see Fig. 3.1: Embankment seawalls or sea dikes with the defence structure elevated above the defended area, so overtopping flows can pass over the crest and down the rear face; or promenade defences 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 where the nature of the flow may be more impulsive.

Table 3.5: 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

Wave overtopping tests were performed in early 2007 on a real dike in the Netherlands. The dike had a 1:3 inner slope of fairly good clay (sand content smaller than 30 %) with a grass cover. The wave overtopping simulator (see Section 3.3.3) was used to test the erosion resistance of this inner slope. Tests were performed simulating a 6 hour storm for every overtopping condition at a constant mean overtopping discharge. These conditions started with 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. 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/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 Fig. 3.2) 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.

Fig. 3.2: Wave overtopping test on bare clay; result after 6 hours with 10 l/s per m width

One remark, however, should be made on the strength of the inner slopes of dikes by wave overtopping. Erosion of the slope is one of the possible failure mechanisms. The other one, which happened often in the past, is a slip failure of the slope. Slip failures may directly lead to a breach, but such slip failures 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 fifty years have been based on 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.

3.3 Tolerable maximum volumes and velocities

3.3.1 Overtopping volumes

Guidance on suggested limits for maximum individual overtopping volumes have been given in Table 3.2 to Table 3.5 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.

3.3.2 Overtopping velocities

Few data are available on overtopping velocities and their contribution to hazards. For simply sloping embankments Chapter 5 gives guidance on overtopping flow velocities at crest and inner slope as well as on flow depths. Velocities of 5–8 m/s are possible for the maximum overtopping waves during overtopping discharges of about 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 $v_x < 2.5$ to 5 m/s. Also refer to Section 5.5.

Upward velocities (v_z) for vertical and battered walls under impulsive and pulsating conditions have been related to the inshore wave celerity, see Chapter 7. 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 $v_z/c_i \approx 3 - 7$.

3.3.3 Overtopping loads and overtopping simulator

Post-overtopping wave loads have seldom been measured on defence structures, buildings behind sea defences, 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 (site specific) model study 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 rock armoured slope. Pulsating wave pressures were measured on the secondary wall against the effective overtopping discharge arriving at the secondary wall, plotted here in Fig. 3.3. This was deduced by applying Equation 3.1 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 5kPa.

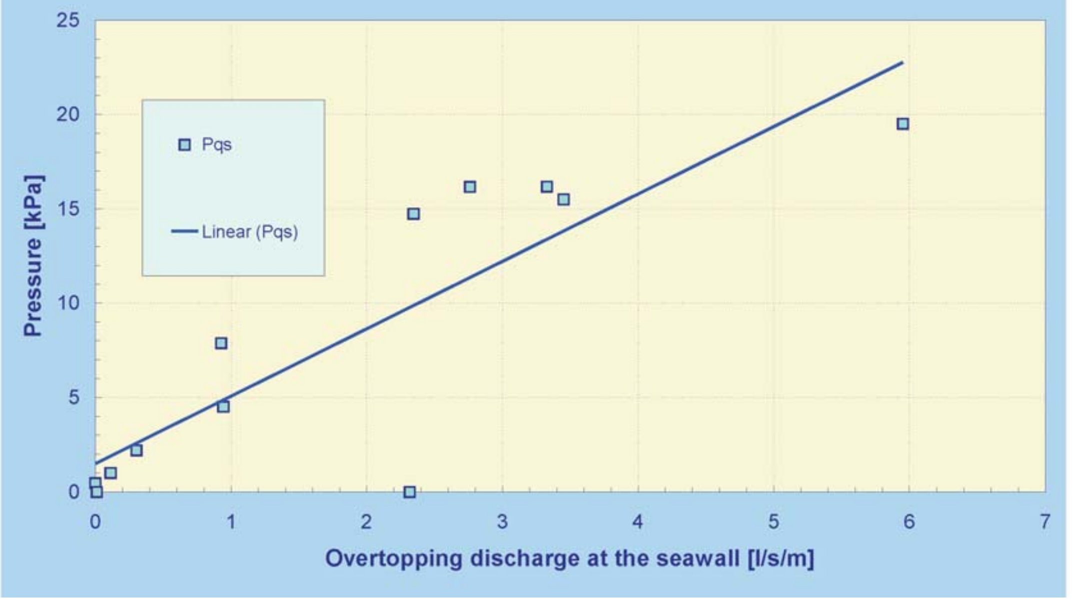

Fig. 3.3: Example wave forces on a secondary wall

During 2007, a new wave overtopping simulator was developed to test the erosion resistance of crest and inner slope of a dike, starting from the idea that:

- knowledge on wave breaking on slopes and overtopping discharges is sufficient (Chapter 5);
- knowledge on the pattern of overtopping volumes, distributions, velocities and flow depth of overtopping water on the crest, is sufficient as well (Chapter 5);
- only the overtopping part of the waves need to be simulated;
- tests can be performed in-situ on each specific dike, which is much cheaper than testing in a large wave flume.

The simulator was developed and designed within the ComCoast, see Fig. 3.4. Results of the calibration phase with a 1 m wide prototype were described by VAN DER MEER (2006).

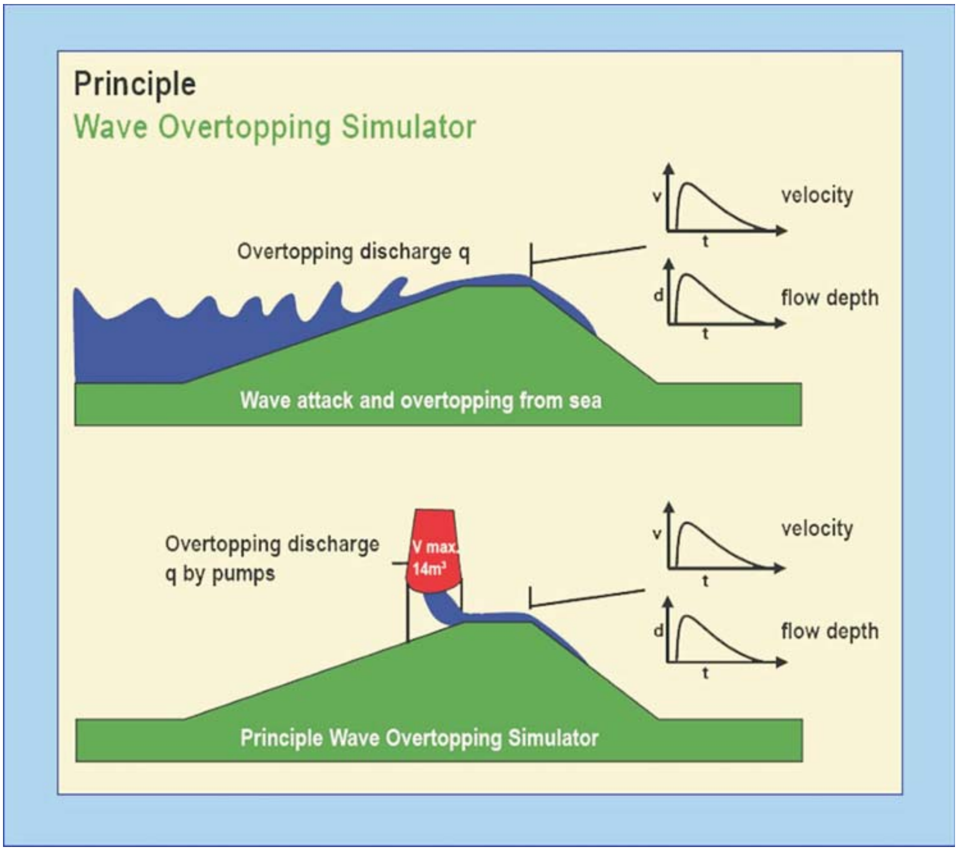

Fig. 3.4: Principle of the wave overtopping simulator

The simulator consists of a mobile box (adjustable in height) to store water. The maximum capacity is 3.5 m^3 per m width (14 m^3 for the final, 4 m wide, simulator see Fig. 3.5). This box is filled continuously with a predefined discharge q and emptied at specific times through a butterfly valve in such a way that it simulates the overtopping tongue of a wave at the crest and inner slope of a dike. As soon as the box contains the required volume, V, the valve is opened and the water is released on a transition section that leads to the crest of the dike. The discharge is released such that flow velocity, turbulence and thickness of the water

Fig. 3.5: The wave overtopping simulator discharging a large overtopping volume on the inner slope of a dike

tongue at the crest corresponds with the characteristics that can be expected (see Chapter 5). The calibration (VAN DER MEER, 2006) showed that it is possible to simulate the required velocities and flow depths for a wide range of overtopping rates, significantly exceeding present standards.

3.4 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.