COMMISSION OF THE EUROPEAN COMMUNITIES

FP5- EESD

CREST LEVEL ASSESSMENT OF COASTAL STRUCTURES BY FULL-SCALE MONITORING, NEURAL NETWORK PREDICTION AND HAZARD ANALYSIS ON PERMISSIBLE WAVE OVERTOPPING

CLASH

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REPORT

D38 : Report on hazard analysis

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Summary

CLASH Work Package 6

Analysis of overtopping hazards

Report CLASH WP6 D38a
November 2004

Prediction of wave overtopping discharges for seawalls / breakwaters have improved significantly over the last 25 years, but processes associated with overtopping hazards to people on or close behind seawalls are not yet well understood. Despite research advances in recent years, there remain important gaps in knowledge and disagreements over safe levels of wave overtopping and the composition and spatial extent of overtopping. Similarly, there are few data on the direct effects of overtopping flows.

This report summarises analysis developed within the EC CLASH project on the hazards arising from wave overtopping. It identifies sources of information on overtopping hazards, and discusses the basis for assessing the consequences of overtopping. The report reviews the state of guidance in Europe, describes instances of hazard, and draws potential guidance on limits to discharge, volume, velocity and depth. The report also draws supplementary data from parallel studies on overtopping and its effects.

Neither this report in particular, nor the CLASH project in general, has dealt with issues of flooding per se.

The report completes work within Work Package 6 of the research project CLASH led by University of Gent (contract EVK3-2001-0058) under the EC 5th Framework programme. In UK, HR Wallingford (HRW) are supported by DEFRA / EA (FD2410 / 2412).

This final version of deliverable D38 develops from the version discussed in the 6th CLASH workshop. Additional material from field and laboratory measurements have been used to extend and revise the initial draft. This report is accompanied by a separate report (D39) by Bouma *et al.* (2004) analysing economic consequences of overtopping.

Contents

K – Analysis of perceived overtopping hazards from field measurements at Ostia (Modimar)
L – Measurements of overtopping jets and spatial effects (Modimar / Universities of L'Aquila / Rome)
M – Safety under Wave Overtopping – How Overtopping Processes and Hazards are Viewed by the Public – paper to ICCE 2004 by W.Allsop, T Bruce, J. Pearson, L. Franco, J. Burgon & C. Ecob
N – Analysis of hazards due to wave overtopping along the German coastline – A. Kortenhaus

1. Introduction

The processes of wave overtopping of seawalls are not yet fully understood; particularly those that cause hazards to people close behind seawalls. There remain important gaps in knowledge on overtopping and post-overtopping processes, on the limits to overtopping volumes, discharges or velocities that might be accepted, despite significant improvements in recent years. To help reduce uncertainties in analysis and management of wave overtopping, the CLASH project has developed improved prediction methods for use by coastal engineers, supported by the EC under the CLASH project (contract EVK3-2001-0058), and in UK by Defra / EA under project FD2412. The overall CLASH is an extensive study by twelve partners at universities and research institutes across Europe, under the EC 5th Framework programme, see web site: http://www.clash-eu.org/.

1.1 BACKGROUND

The CLASH project ("Crest level assessment of coastal structures by full scale monitoring, neural network prediction and hazard analysis on permissible wave overtopping") led by University of Gent is intended improve analysis and design methods for coastal structures against storm surges, wave attack, flooding, and erosion. The project is supported under the EC 5th Framework programme (contract EVK3-2001-0058) to produce generic prediction methods for crest height of most coastal structures based on permissible wave overtopping supported by hazard analysis. Activities within CLASH by twelve partners at universities / research institutes across Europe are divided into ten Work Packages:

- WP 1. General methodology
- WP 2. Overtopping database
- WP 3. Full scale measurements
- WP 4. Laboratory investigations
- WP 5. Numerical modelling
- WP 6. Hazard analysis, including socio-economic impacts
- WP 7. Conclusions on scale effects and new data
- WP 8. Generic prediction method
- WP 9. Synthesis and formulation of guidelines
- WP 10. Exploitation and dissemination of the results

The CLASH project is intended to improve prediction methods, based on laboratory and field measurements and appropriate hazard analysis. A particular motivation for this research was the suggestion in the OPTICREST project, see de Rouck et al (2002), that there might be unexpected scale (and model) effects in some hydraulic modelling in which small-scale tests might under-predict overtopping at full scale. The large scale tests on vertical and steeply battered walls by the VOWS team in the large flume at Barcelona, see Pearson et al (2002), suggested that scale effects might be negligible for such impermeable and smooth structures, but further analysis suggests that some scale effects may derive from scaling of roughnesss and/or permeability and the absence of wind effects in scale models.

As part of the overall study on wave overtopping hazards, CLASH partners have therefore measured wave overtopping events at full scale at three coastal sites in Europe (WP 3). Those processes have been simulated by laboratory tests (WP 4) and compared with full scale measurements (WP 7). This report summarises the analysis of direct hazards of overtopping conducted under Work Package WP6 of CLASH, but supported by data from WP3 and WP4..

1.2 ACTIVITIES IN CLASH WORK PACKAGE 6

The overall aim of WP6 was to derive and/or refine guidance on hazards presented by overtopping. The specific objectives are to:

- Compare measured events and hindcast events with records of observed hazard in order to derive / refine limits for safety of pedestrians, car users, travellers in other vehicles;
- Derive / refine limits of overtopping for hazard to buildings and related items;
- Evaluate the risks of economic loss.

Observations of overtopping hazard have been made at selected field sites. HRW had already recorded hazard events over 4 years at Samphire Hoe by personnel who are responsible for safety of the public. These observations have been extended at Zeebrugge and Ostia by video records and direct observations during field measurements. At Zeebrugge, "instrumented" persons (dummies) have been used to give indicators of overtopping violence. Supplementary observations on breaking window glass have been made. These observations are used here to develop overall guidance.

The linked activity reported separately in D39 by Bouma *et al.* (2004) has developed / refined methods to evaluate risks of economic losses, where risk is taken as the sum of (occurrence probability x damage per event) for all relevant overtopping events. This task included Economic Assessment Approach for direct and indirect economic impacts, including economic dimensions of ecological impacts.

1.3 REPORT OUTLINE

Following this introductory section, Chapter 2 summarises the main wave overtopping processes in relation to their contribution to direct hazards. [Again it should be noted that, whilst overtopping volumes are calculated, this project does not of itself deal with overtopping induced flooding, but with the more direct effects of overtopping.]

Chapter 3 of this report presents data and analysis on overtopping hazards observed in Europe and wider. Where sufficient data are available, examples of occurrence of deaths, damage and potential hazards have been used to support the development of the guidance summarised in Chapter 4. The guidance is primarily given by limits on overtopping discharges and volumes, but is now supplemented by new data to extend guidelines used internationally over 20-30 years.

Figure 1 Run-up / overtopping on a smooth slope

2. *Wave overtopping processes and hazards*

2.1 OVERTOPPING PROCESSES

Around the coastlines of Europe and elsewhere, low-land lying areas, towns, transport infrastructure (including ports) are often protected by seawalls or related structures against flooding or erosion by waves and/or extreme surges. The hazards from direct wave and overtopping effects may arise under three general categories:

a) **Direct hazard** of injury or death to people living, working or travelling in the area defended;

b) **Damage to property, operation and / or infrastructure** in the area defended, including loss of economic (environmental or other) resource, or disruption / delay to an economic activity / process;

c) **Damage to defence** structure(s), either short-term or longer-term.

These hazards or consequences of overtopping are both site- and event-specific, see discussion in CLASH report D39 by Bouma *et al.* (2004). The hazards are driven by overtopping processes usually categorised by the direct responses:

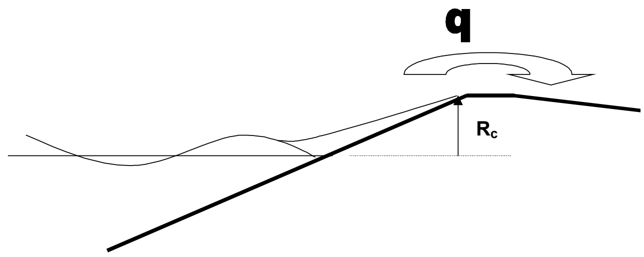

Figure 2 Wave overtopping on embankment seawall

- mean overtopping discharge, q;
- peak overtopping volumes, V_i and V_{max};
- overtopping velocities, horizontally and vertically, v_x and v_z;
- overtopping depths, d_x.

Less direct responses may also be needed in assessing the effects of these processes, perhaps categorised by:

- overtopping falling distance, x_c;
- pulsating (quasi-static) or impulsive pressures, P_{qs} or P_{imp};
- post-overtopping flow depths, h, and horizontal velocities, v_x.

When considering the effects of wave action, it may be convenient to start by defining degrees of overtopping under three levels of severity and two types of load application:

Light overtopping, no impulsive effects or direct structural damage to lightly engineered structures, minor or very local flooding, damage chiefly by inundation only;

Moderate overtopping, no impulsive effects and little / no direct structural damage to engineered structures, local flooding causing some inundation damage;

Heavy overtopping requiring significant engineering to resist direct effects without damage, overtopping flows / volumes are unlikely to cause damage to a well engineered defence structure, but local and wider flooding is possible as is flood flow damage to lighter structures;

Overtopping flows with no significant "slam" effect, damage caused by velocity driven drag forces;

Impulsive overtopping with sudden and wave "slam" forces generally caused by the leading edge of an overtopping jet or bore, may lead to direct damage to property close behind and/or damage to the defence itself.

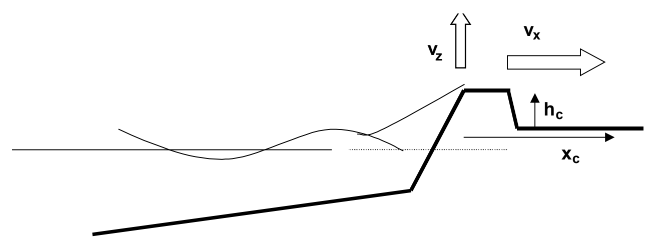

Figure 3 Wave overtopping on promenade seawall

These definitions are not of themselves sufficient to categorise overtopping effects, but they give guidance as to the main response parameters of importance. Care should be taken not to use indicators that imply some standard of protection which may therefore confuse the standard of protection required with the performance provided.

2.2 RESPONSES TO OVERTOPPING

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

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

Accept occassional hazard at acceptable probability (acceptable risk) by providing for temporary use and/or short-term evacuation with reliable warning and evacuation systems, and/or use of temporary / demountable defence systems;

Increase defence standard to reduce risk to (permanently) acceptable levels probably by enhancing the defence and / or reducing loadings.

This report, indeed most of the CLASH project, is primarily associated with this third response, although the results of this work may inform either of the first two responses.

For any structure expected to ameliorate wave overtopping, the crest level and/or the front face configuration are dimensioned to give acceptable levels of wave overtopping under specified extreme conditions or combinations of conditions (e.g. water level and waves). Setting acceptable levels of overtopping depends on the use of the defence structure itself, the land behind, national or local standards, and the economic and social basis for funding the defence. The CLASH report D39 by Bouma et al (2004) describes methods to value the hazards (and therefore the value of their avoidance). Chapter 4 suggest levels of overtopping that have been judged appropriate for various activities. Neither of these will however supecede national / local standards and administrative practice which will guide any final decision on protection standard. For instance in the UK, practice on sea defence funding is outlined by Brampton (2001) and Dltr (2001).

2.3 DEFENCE TYPES

Where the option is taken to increase defence standards, a seawall or related structure may be required, often formed as sloping embankments or dykes with revetment protection (e.g. Figure 1), or (perhaps more common in UK, France and Italy) as a steep or vertical retaining wall with promenade (e.g. Figure 4). Coastal structures may include seawalls or breakwaters formed from blockwork or mass concrete, with vertical, near vertical, or sloping faces. Under wave attack, sloping embankments tend to break waves onto the slope with overtopping being a relatively gentle process (e.g. Figures 1, 2). Steeper / vertical or compound structures (e.g. Figures 3, 4) are more likely to experience intense local wave impact pressures, may overtop severely or with greater velocities, but may also reflect much of the incident wave energy. Reflected waves cause additional wave disturbance and/or may initiate or accelerate local bed scour / erosion with consequent effects on increasing any depth-limited wave heights.

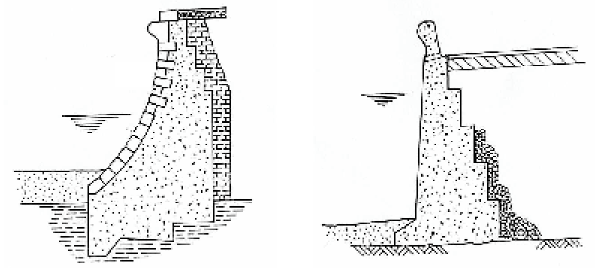

Figure 4 Example vertical / battered seawalls

Figure 5 Wave overtopping at vertical breakwater and seawall, Margate

Some sloping structures are formed by a core of quarry rock protected by layers of rock or concrete armour(e.g. Figure 6). The outer armour layers to a rubble mound should be designed to dissipate wave action without significant movement of armour units. Alternative revetment armouring (blocks or slabs) may only dissipate energy in wave breaking onto the slope and related processes. In each instance, granular under-layers / filters support the armour and separate it from the fine material in the embankment or mound. Porous sloping layers may dissipate significant wave energy in breaking and friction. Simplified rubble mounds may form rubble seawalls or give additional protection to vertical walls or revetments.

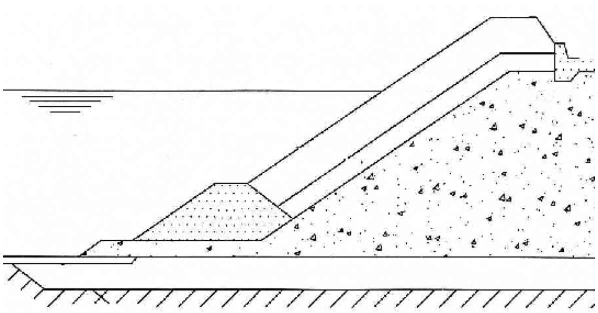

Figure 6 Example rubble mound seawall

3. Overtopping prediction methods

It is not the intention of WP6 to comment significantly on wave overtopping prediction methods which have been covered elsewhere within CLASH. It is however useful simply to summarise briefly the main methods for predicting overtopping..Three main methods: empirical, physical; and numerical, can be used to predict the overtopping responses listed earlier in 2.1:

- mean overtopping discharge, q;
- peak overtopping volumes, V_i and V_{max};
- overtopping velocities, horizontally and vertically, v_x and v_z;
- overtopping depths, d_x.

Empirical models use simplifying equations to calculate key responses, e.g. mean overtopping discharge, q, or peak overtopping volume, V_{max}, based on representative values of wave and structure parameters. Such prediction methods for different structures or conditions have been described by Owen (1980), Allsop (1994), Franco et al (1994), Besley (1999) van der Meer et al (1998) and others.

Overtopping rates predicted by empirical formulae generally include "green water" discharges and splash, since both parameters were recorded during model tests from which these prediction methods were derived.

3.1 WAVE BREAKING AND OVERTOPPING CONDITIONS

For beaches, and sloping structures, the simplest division is to separate **"plunging"** from **"surging"** conditions using the well-established surf similarity parameter (or *Iribarren number*) defined in terms of beach slope (α), and wave steepness (s_{op}, or sometimes s_{om}):

$$\xi_{op} = \tan \alpha / \sqrt{s_{op}} \qquad (1)$$

Plunging conditions occur where $\xi_{op} < 2$, and **surging** conditions are given by $\xi_{op} > 2$, see Fig. 7. On sloping structures, these definitions are commonly used in calculating armour stability for rubble mounds, see the CIRIA / CUR Rock Manual (1991), or overtopping, see van der Meer *et al* (1995, 1998).

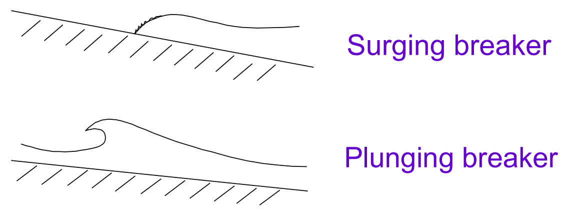

Fig. 7 Types of wave breaking on slopes

On steep walls (vertical, battered or composite), **"pulsating"** overtopping occurs when waves are relatively small in relation to the local water depth, and of lower wave steepnesses. These waves are not critically influenced by the structure toe or approach slope. Waves run up and down the wall giving rise to (fairly) smoothly-varying loads.

In contrast, **"impulsive"** breaking on steep walls occurs when waves are larger in relation to local water depths, perhaps shoaling up over the approach bathymetry or structure toe itself. Under these conditions, some waves will break violently against the wall with (short-duration) forces reaching 10-40 times greater than for "pulsating" conditions, see Allsop *et al* (1996) and McKenna (1997).

For steep / vertical walls, the onset of impulsive breaking is given primarily by the slope and/or width of the approach slope or toe berm, and by the incident wave length. Methods to distinguish between breaking / response types for wave forces have been developed within the PROVERBS project, see Oumeraci *et al* (2001) or Allsop & Kortenhaus (2001). A different approach was developed for overtopping by Besley *et al* (1998) using a dimensionless depth, h^*, based on local depth, h, and incident wave conditions:

$$h_* = \frac{h}{H_s}\left(\frac{2\pi h}{gT_m^{\,2}}\right) \qquad (2)$$

Analysis by Allsop *et al* (1995) reported by Besley *et al* (1998) suggest that pulsating conditions predominate at the wall when $h_* > 0.3$, and impulsive conditions occur when $h_* \leq 0.3$.

Another helpful distinction describes the physical form of overtopping. Overtopping when waves break onto or over the seawall generally generates "**green water overtopping**" where the overtopping volume is relatively continuous. For waves that break seaward of the face of the structure, or where the seawall is high in relation to the wave height, overtopping may be as a stream of fine droplets. This "**spray overtopping**" can be carried over the wall under their own momentum, or may be driven by onshore wind. Spray overtopping may also be generated directly by wind acting on wave crests, most noticeable when waves reflected from steep walls interact with incoming waves to give severe local 'clapotii'. Effects of wind on spray overtopping are seldom modelled, largely due to inherent difficulties in scaling wind effects in laboratory tests, but also because the importance of wind effects have not yet been established. De Waal *et al* (1992, 1996) suggested that onshore winds might have relatively little effect on green water overtopping, but that wind might increase overtopping of vertical walls by up to a factor of three for discharges under q = 1 l/s.m where much of the overtopping may take the form of spray. Pullen *et al* (2004) report experiments to measure the influence of wind on overtopping distributions for vertical walls. Generic advice is developed elsewhere within CLASH.

3.2 OVERTOPPING PREDICTION METHODS

The simplest and most robust method to predict wave overtopping is by empirical equations that relate overtopping discharges to seawall crest level, wall configuration and roughness, sea bed slope or toe berm size, local water depth and wave conditions. Such design methods are generally configured to calculate the crest freeboard (R_c) required to give an acceptable mean discharge (q). Empirical models or formulae use relatively simple equations to describe mean overtopping

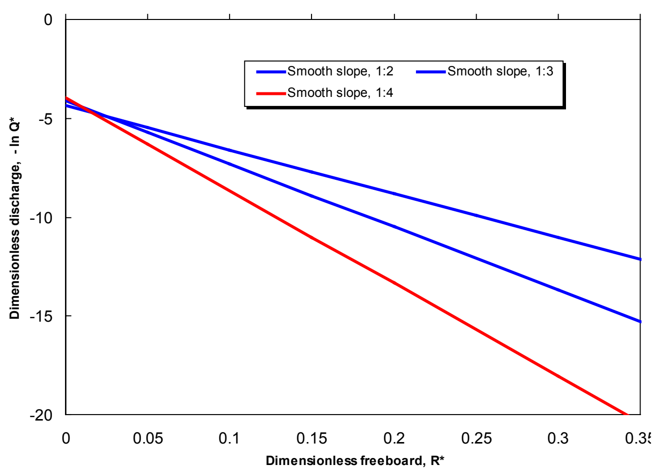

Fig. 8 Example monotonic prediction method for simple slopes, 1:2 to 1:4, after Owen (1980)

discharges, q, in relation to defined wave and structure parameters. As with any empirical method, these may be limited to relatively simple structure configurations. Use out of range, or for other structure types, may require uncertain and insecure extrapolation of the equations or coefficients.

3.2.1 Overtopping on slopes

Rural seawalls on the coasts of Denmark, Germany, Netherlands and UK are often of simple trapezoidal section, formed by sandy and weaker clays requiring slopes of 1:4 – 1:8. Use of stiff clays in UK allows relatively steep slopes of 1:2 - 1:4. Overtopping of these steeper slopes was related to freeboard R_c, and wave parameters H_s, T_m by Owen (1980, 1982). Owen defined dimensionless discharge and freeboard parameters $Q*$ and $R*$:

$$Q* = q / (gT_mH_s) \tag{3}$$

$$R* = R_c / T_m \sqrt{(gH_s)} \tag{4}$$

Owen's prediction equation was of exponential form (see Fig. 8) with roughness coefficient, r, and empirical coefficients A and B for each slope given in the Environment Agency overtopping manual by Besley (1999).:

$$Q* = A \exp (-B R*/r) \tag{5}$$

The validity of Equation (5) has been expanded to $0.05 < R* < 0.3$. The form of Owen's equation is simple and monotonic. For embankments with small relative freeboards and/or large wave heights, predicted overtopping discharges converge, when the slope angle no longer has much influence in controlling overtopping, the slope is said to be "drowned out". Over the normal range of freeboards, the characteristics for slopes of 1:1, 1:1.15 and 1:2 are similar, but overtopping reduces significantly for slopes shallower than 1:2. Increasing wave height or period increases overtopping discharges, as does reducing the freeboard, either by raising the crest or lowering the water level. Owen's method was developed for smooth slopes, but the roughness coefficient, r, allowed it to be extended to rough and even armoured slopes.

Alternative prediction methods for smooth and armoured slopes have been developed since 1980 for sea dikes by de Waal & van der Meer (1992), van der Meer & Janssen (1995) and van der Meer *et al* (1998). Their formulae distinguish between plunging and surging conditions on the structure slope as defined by the surf similarity parameter, ξ_{op}, and use different definitions of dimensionless discharge for breaking waves, Q_b, or dimensionless freeboard, R_b:

$$Q_b = \frac{q}{\sqrt{gH_s^3}} \cdot \sqrt{\frac{s_{op}}{\tan\alpha}} \tag{6}$$

$$R_b = \frac{R_c}{H_s} \cdot \frac{\sqrt{s_{op}}}{\tan\alpha} \cdot \frac{1}{\gamma_b \cdot \gamma_h \cdot \gamma_f \cdot \gamma_\beta} \tag{7}$$

where γ_b, γ_h, γ_f, and γ_β are reduction factors for berm width, shallow depth, roughness and wave obliquity.

In van der Meer *et al*'s approach, overtopping for **plunging** conditions, $\xi_{op} < 2$, is calculated from:

$$Q_b = 0.06 \exp(-4.7 R_b) \qquad (8)$$

Similar relationships are available for **surging** conditions when $\xi_{op} > 2$, using different parameters, Q_n = dimensionless discharge for surging waves, and R_n = dimensionless freeboard:

$$Q_n = q / \sqrt{(gH_s^3)} \qquad (9)$$

$$R_n = \frac{R_c}{H_s} \cdot \frac{1}{\gamma_b \cdot \gamma_h \cdot \gamma_f \cdot \gamma_\beta} \qquad (10)$$

where the prediction equation for overtopping under **surging** conditions is given:

$$Q_n = 0.2 \exp(-2.3 R_n) \qquad (11)$$

3.2.2 Overtopping on vertical walls

The development of formulae for vertical walls followed a similar path towards single or monotonic formulae. Graphical methods by Goda *et al* (1975), see also Herbert & Owen (1995), showed that there could be two rather different processes, rather than a single monotonic process, but no formulae were developed to describe the overtopping predictions of those graphs, and Goda's results were limited to relatively low wave steepnesses s_{op} < 0.036, which excludes most storm conditions in the North Sea or Mediterranean.

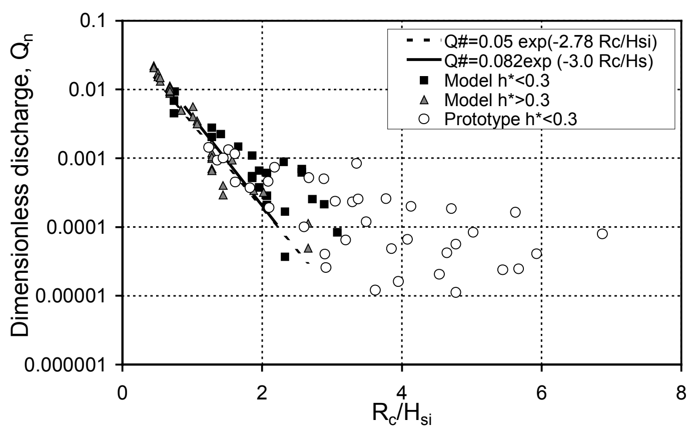

Fig. 9 Model and prototype overtopping discharges against equation of Franco *et al* (1994).

For simple vertical breakwaters in deeper water, Franco *et al* (1994) developed a single empirical formula based on equation (11) using relative freeboard, R_c/H_s, reduction factors for specific front face geometries, γ_S, and dimensionless discharge, $Q\# = Q_n$:

$Q_n = 0.2 \exp((-4.3/\gamma_S)(R_c/H_s))$ valid for $0.03 < R_c/H_s < 3.2$ (12)

Returning to intermediate and shallower water, Allsop *et al* (1995) refined by Besley *et al* (1998) demonstrated that overtopping processes at vertical and composite walls are strongly influenced by the form of incident wave breaking, not just by values of H_s and T_p alone. When waves are small compared to depth, waves at vertical or composite walls are reflected. If the waves at the wall are large relative to depth, then they may break directly onto the structure, leading to significantly more abrupt overtopping.

These observations, together with the development of the "wave impact parameter map" in PROVERBS, see Allsop *et al* (1996), led to development of the wave breaking parameter, h^*, Equation (2). Use of this to separate pulsating or impulsive breaking is illustrated in Fig. 9 where un-separated data from model tests in UK (Herbert, 1996, Besley *et al*, 1998) and the Netherlands (de Waal *et al*, 1996) are plotted against versions of Franco's equation. Much of the data for low values of R_c/H_s fit Equation (12), but data at higher values of R_c/H_s fall very much higher than predicted by that method. For **pulsating** conditions ($h_* > 0.3$), Besley *et al* (1998) developed a modified version of Franco's equation, now plotted in Fig. 10:

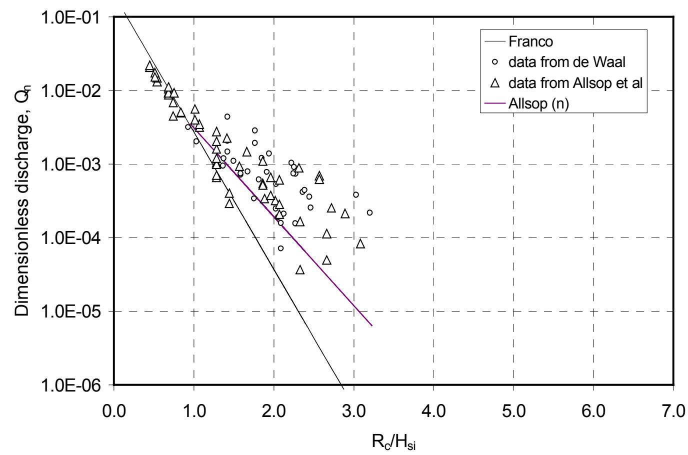

Fig. 10 Overtopping for pulsating conditions ($h_* > 0.3$).

$$Q_n = 0.05 \exp(-2.78\, R_c/H_s) \qquad \text{valid over } 0.03 < R_c/H_s < 3.2: \qquad (13)$$

For **impulsive** conditions given by $h_* \leq 0.3$ and therefore excluding all pulsating conditions, Besley *et al* (1998) used model test data from MCS and other projects to derive a new equation for impulsive overtopping with new dimensionless discharge, Q_h, and freeboard parameters, R_h. The new equation included h_* to give:

$$Q_h = 1.37 \times 10^{-4} R_h^{-3.24} \qquad \text{valid over } 0.05 < R_h < 1.0 \qquad (14)$$

where:

$$Q_h = q / (gh^3)^{0.5} / h_*^2 \qquad (15)$$

$$R_h = (R_c / H_s)\, h_* \qquad (16)$$

These equations were originally derived using small-scale model test data, but were later tested against full-scale data from Herbert's (1996) field measurements with relatively good agreement.

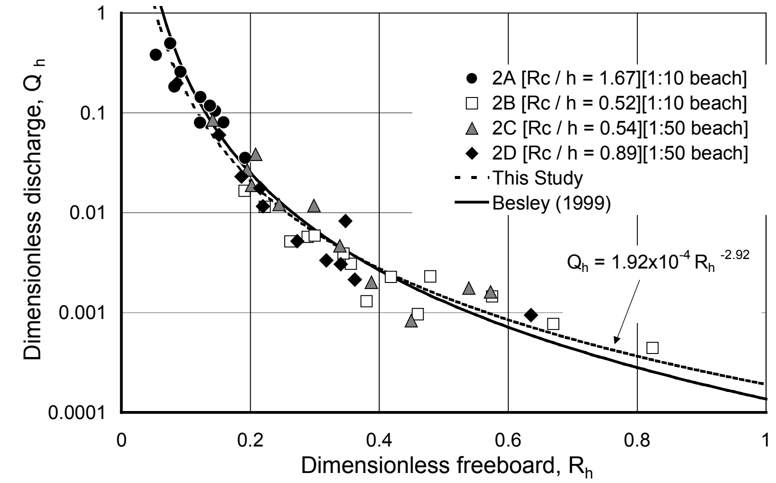

Fig. 11 Overtopping from VOWS tests compared with Besley (1999) and revised prediction equation.

Measurements at small scale from the VOWS tests at Edinburgh were compared by Bruce *et al* (2001) with Equation (14), see Fig. 11. In general, agreement between these data and the prediction is remarkably good, particularly given the wide range of dimensionless freeboards covered. There is a tendency for divergence from the original line of Equation 15, so a slightly revised prediction line is suggested:

$$Q_h = 1.92 \times 10^{-4} R_h^{-2.92} \qquad \text{valid over } 0.05 < R_h < 1.0 \qquad (17)$$

Within the VOWS study, tests for vertical walls were repeated for near-vertical walls with 10:1 and 5:1 batter commonly found for older UK seawalls and breakwaters, as reviewed by Allsop & Bray (1994). A 1:10 approach slope was used, representative of shingle or steeper sand beaches. Measurements of Q_h for 10:1 and 5:1 walls indicate discharges slightly in excess of those predicted by Besley *et al* (1998), by factors up to 3 – 4, over a wide range of dimensionless freeboards.

For conditions tested by Bruce *et al* (2001), the 10:1 and 5:1 battered walls exhibit similar overtopping characteristics. Initial analysis suggested that an amplification factor based on the predicted mean dimensionless discharge for the vertical case could be applied. For the 10:1 battered wall, the average increase factor on discharge is 1.3, and for the 5:1 battered wall, the factor is 1.4. Alternatively, revised equations fitted to these data are given in Equations 18 and 19, (valid over $0.05 < R_h < 1.0$):

$$Q_h = 1.89 \times 10^{-4} R_h^{-3.15} \quad \text{for impulsive conditions on 10:1 battered walls} \quad (18)$$

$$Q_h = 2.81 \times 10^{-4} R_h^{-3.09} \quad \text{for impulsive conditions on 5:1 battered walls} \quad (19)$$

3.2.3 Overtopping on composite walls

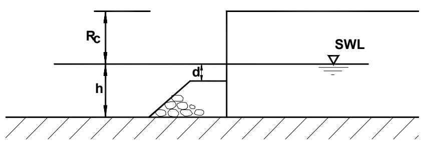

Fig. 12 Definitions of depths for toe berms.

Studies within the PROVERBS project on vertical breakwaters (Oumeraci *et al,* 2001) have illustrated how a relatively small toe berm can change wave breaking characteristics, thus substantially altering the type and magnitude of wave loadings. Besley (1999) notes that many vertical seawall walls may be fronted by rock mounds with the intention of protecting the toe of the wall from scour. The toe configuration can vary considerably, see Fig. 12, potentially modifying the overtopping behaviour of the structure. Three types of mound can be identified

i) Small toe mounds which have an insignificant effect on the waves approaching the wall – here the toe may be ignored and calculations proceed as for simple vertical (or battered) walls.

ii) Moderate mounds, which significantly affect wave breaking conditions, but are still below water level. Here a modified approach is required.

iii) Emergent mounds in which the crest of the armour protrudes above still water level. Prediction methods for these structures may be adapted from those for crown walls on a rubble mound, but are not discussed further here.

For overtopping of composite seawalls, Besley *et al* (1998) defined a modified breaking parameter d_* based on h_*:

$$d_* = \frac{d}{H_s}\left(\frac{2\pi h}{gT^2}\right) \quad (20)$$

When $d_* > 0.3$, the mound was classified as small and overtopping can be predicted by the standard method given previously for **pulsating** conditions, Equation 14.

For larger mounds when $d_* \leq 0.3$, Besley (1999) recommends a modified version of the **impulsive** prediction method, accounting for the presence of the mound by use of d and $d*$, (valid over $0.05 < R_d < 1.0$):

$$Q_d = 4.63 \text{ x } 10^{-4} R_d^{-2.79} \quad (21)$$

$$Q_d = q / (gh^3)^{0.5} / d_*^2 \quad (22)$$

$$R_d = (R_c / H_s)\, d_* \quad (23)$$

Results from the VOWS tests generally supported the use of this approach as a conservative prediction, but Bruce *et al* (2001) and Allsop et al (2005) suggested that the prediction line of Equation (21) might lie towards the upper bound of the data rather than representing any central estimate. Considering some of the originally outliers, it appeared that the limit for **impulsive** conditions on composite structures might be better set at $d_* \leq 0.2$ (rather than $d_* \leq 0.3$), provided that this is only applied for conditions where $h_* \leq 0.3$. This limit for the onset of impact conditions is lower than recommended by Besley (1999). Measurements limited by $d_* \leq 0.2$ give the revised prediction:

$$Q_d = 5.88 \text{ x } 10^{-4} R_d^{-2.61} \qquad (h_* \leq 0.3 \text{ and } d_* \leq 0.2) \quad (24)$$

3.2.4 Overtopping of broken waves

Many seawalls are constructed at or towards the top of a beach such that breaking waves never reach the seawall, at least not during frequent events where overtopping is of primary importance. For these conditions, particularly for typical shallow beach slopes, $m < 1/30$, design wave conditions may be given by waves which start breaking many metres seaward of the wall, indeed perhaps kilometres seaward. **Broken** waves are inherently much less likely to re-form to give a plunging breaker, so less likely to give **impulsive** conditions at the wall.

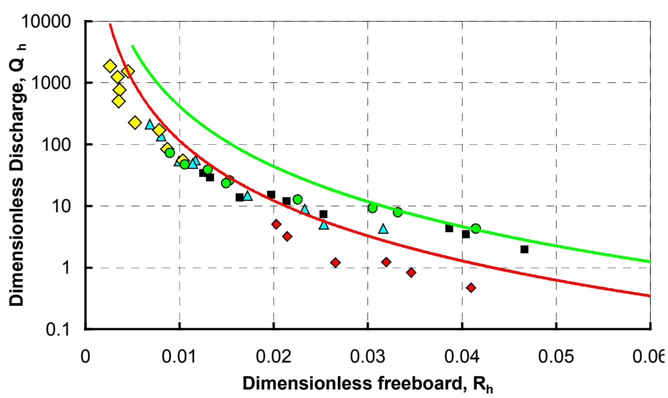

Fig. 13 Overtopping for broken waves on vertical walls, submerged toe.

In the region where the water depth at the toe is positive, $h > 0$, and **broken** waves predominate (i.e. when dimensionless freeboard $R_h <\approx 0.03$), tentative guidance is suggested by Bruce *et al* (2003) based on a modification and extrapolation of Besley's method, Equation 15. The modified equation (25) is plotted as the lower line in Fig.13:

$Q_{h(broken)} = 0.27 \times 10^{-4} R_h^{-3.24}$ (for $R_h < 0.03$) (25)

For conditions falling in the range $0.03 < R_h < 0.05$, Bruce *et al*'s data suggest that it will probably be safe to extrapolate Besley's method (Equation 15) slightly outside of its recommended range, shown in the upper line in Fig. 13.

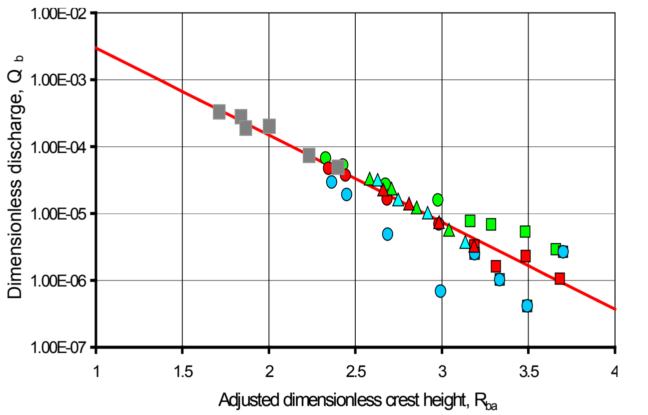

Fig. 14 Overtopping for broken waves on vertical walls, emergent toe.

For configurations where the toe of the wall is above water, $h < 0$, Bruce *et al* (2003) suggest an adaptation of the prediction equation for plunging waves by van der Meer & Janssen (1995) using the sea bed slope of $\tan \alpha$ in evaluating Q_b defined in Equation (6), and R_b defined in Equation (7):

$$Q_b = 0.06 \exp(-4.7 R_{b(broken)}) \qquad (1.0 < R_{b(broken)} < 4.0) \qquad (26)$$

$$R_{b(broken)} \equiv R_b \, s_{op}^{-0.17} \qquad (27)$$

Results of this analysis are compared in Figure 14 with predictions for sloping structures by van der Meer & Janssen (1995). Despite the differences between the structure in this study and those examined by van der Meer & Janssen, the overtopping characteristics are broadly similar. Equation (26) above is used to adjust the prediction of van der Meer & Janssen (1995) in Figure 15.

3.3 OVERTOPPING VELOCITIES

The importance of the form of wave breaking onto vertical / battered walls is illustrated by measurements of overtopping velocities (peak vertical speeds) by Pearson *et al* (2002) and Bruce *et al* (2002) at small and large scales. Video records were analysed of the largest 20 individual overtopping events (in $N_z = 1000$ waves). The upward velocity (u_z) of the leading edge of the water was estimated from frame-by-frame analysis, and u_z was non-dimensionalised by the inshore wave celerity c_i, given by $c_i = (gh)^{1/2}$. Relative velocities, u_z/c_i, are plotted in Fig 15 against the wave breaking parameter, h_*.

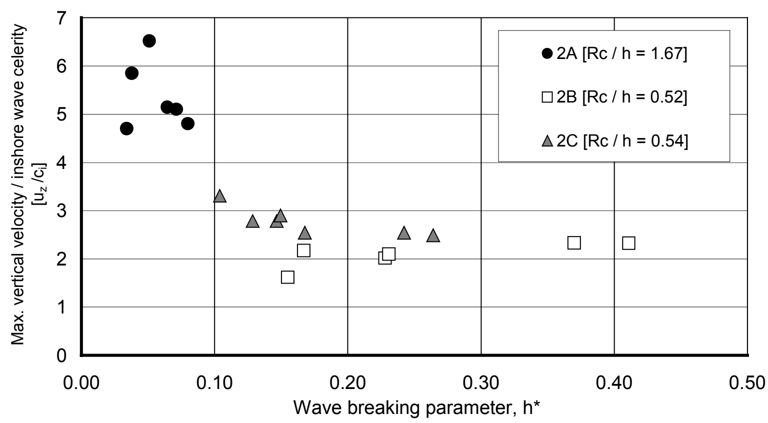

Figure 15 Overtopping velocities for vertical walls.

It is noted in Fig. 15 that the non-dimensional velocity is roughly constant at $u_z/c_i \approx 2.5$ for $h_* > 0.2$, but velocities increase significantly when $h_* \leq 0.2$ reaching $u_z/c_i \approx 3 - 7$. In this context, it is useful to note that Richardson *et al* (2002) measured crest velocities of around $u_z/c_i \approx 2$ for 1:2 slopes under plunging conditions.

4. Evidence of overtopping hazards

4.1 WAVE OVERTOPPING PROCESSES

Overtopping which occurs when waves run up the face of the seawall or breakwater reach and pass over the crest of the wall, is often termed 'green water' overtopping, see exampleas earlier in Figures 1 and 2. A different form of overtopping occurs when waves break seaward of the defence structure or on its seaward face, producing significant volumes of spray, see Figures 5 and 16. These droplets may be carried over the wall either under their own momentum or driven by an onshore wind, known as 'spray' overtopping. Spray may also be generated by wind acting directly on wave crests approaching the wall, particularly noticeable when reflected waves from steep walls interact with incoming waves to give severe local 'clapotii'. Without the influence of strong onshore wind, this spray probably does not contribute significantly to overtopping volumes, but may cause some direct hazards. The overtopping in Figure 16 would certainly surprise a less-aware pedestrian, and could cause them to loose their footing and fall. The overtopping in Figure 17 is almost certainly severe enough to knock over even an aware person.

Figure 16 Overtopping at vertical harbour wall, Chania, Crete.

Figure 17 Overtopping at Stone Pier, Margate

Light spray may contribute little to direct hazard except reducing visibility and extending the spatial extent of salt spray effects. An exception is the effect of spray in reducing visibility on coastal highways where the sudden loss of visibility may cause significant driving hazard, see the example in Japan National Highway 336 discussed in Annex A after Kimura *et al.* (2000).

Effects of wind and generation of spray are seldom modelled. Tests by de Waal *et al.* (1992, 1996) suggest that onshore winds have relatively little effect on large green water events, but may increase discharges under Q_{bar} = 1 l/s.m where much of the overtopping may take the form of spray. Such discharges are however already substantially greater than discharge limits suggested for pedestrians or vehicles, see Table 4.1. Studies by Ward et al (1994, 1996) consider wind effects on waves, the run-up process and overtopping at laboratory scale, but do not lead to firm scaling conclusions. Substantial advances have been made on this issue within CLASH, see separate discussions in field mesurements and laboratory tests (WP3 and WP4).

4.2 WAVE OVERTOPPING DISCHARGES / VOLUMES

In assessments of flooding by wave overtopping, most analysis has evaluated flood volumes / areas using the total overtopping volume. This aspect is not the subject of this project which is primarily focussed on the direct and local effects of wave overtopping limits. Most descriptions of overtopping have been in terms of mean overtopping discharges derived from total overtopping volumes collected over 250 to 1000 T_m. The mean discharge is then expressed as flow rate per metre run of seawall, typically $m^3/s.m$.

Limits to identify onset of damage to seawalls, buildings or infrastructure, or danger to pedestrians and vehicles have been defined relative to mean discharges or peak volumes. Guidelines were derived by Owen (1980) from work in Japan by Goda (1975) and Fukuda *et al.* (1974) and are summarised in Table 3.1 below. Significantly different limits were given for embankment seawalls (with back slopes) and promenade seawalls (without back slopes), and for pedestrians or vehicles.

It has been argued (see e.g. Besley, 1999) that use of mean overtopping discharges only in assessment of safety levels is questionable. It was regarded as probable that the maximum individual volume was of much greater significance than the average discharge to hazards. Franco *et al.* (1994) and Besley (1999) and have shown that, for a given level of mean discharge, the volume of the largest overtopping event can vary significantly with wave condition and structural type. There are however two difficulties in specifying safety levels with reference to peak volumes and not to mean discharges. Firstly, methods to predict peak volumes are significantly less well-validated than for mean discharge.

Secondly, the data relating individual overtopping events to hazard levels have been rare. Franco et al (1994) used model tests and experiments on volunteers to demonstrate that danger levels to people or vehicles from an individual overtopping event could be related to its volume. A volume was defined as "safe" if it created a less than 10% chance of a person falling over. An event was defined as "very dangerous" if it gave greater than 90% chance of a person falling over. It is felt that this higher limit represents an unacceptable risk to pedestrians and that the tolerable discharge should be closer to the lower 10% limit.

In many instances, people / vehicles can be excluded from the hazardous area, see discussion on limits in Chapter 4 below, but overtopping can still give problems to buildings or related structures, or to the defence structure itself.

Franco et al (1994) suggested that a "safe" limit for an individual overtopping volume for people operating behind a vertical wall was $v_{max} = 0.1m^3/m$, whilst for a horizontally composite structure it was $v_{max} = 0.75m^3/m$. It should however be noted that Franco et al (1994) also noted that a volume as low as $v_{max} = 0.05m^3/m$ could unbalance an individual when striking their upper body without warning. This latter figure was determined from experiments at full scale on volunteers rather than from model tests and may therefore give use a more realistic estimate of tolerable events. Even so, it must be noted that the experimenters were still anticipating being hit by overtopping water. They may therefore have been able to tolerate rather more severe conditions than might be reasonable for workers or the public who are hit by (usually cold) sea water without apparent warning. Franco's advice for an able-bodied pedestrian falling over at less than 10% probability (low overtopping hazard) was $v_{max} \leq 0.1$ m^3/m, but at 90% probability (high overtopping hazard) $v_{max} \leq 0.7$ m^3/m..

Franco et al (1994) also noted that the "safe" limit would vary with structural type. They found that a given volume overtopping a vertical structure was more dangerous than the same volume overtopping a horizontally composite structure. Two effects will be important here, particularly for personal safety. Different velocities will influence the danger caused by any particular overtopping volume, and the elevation at which a person is hit will alter the degree of danger. These effects will be influenced by the form of wave breaking onto the structure, and by the geometry of the structure's crest detail, in particular the height of any parapet wall, if present.

Figure 18 Overtopping flows on a sloping revetment

Smith et al (1994) reported on full scale tests conducted on dykes or embankments. An observer on the crest of the dyke judged safe overtopping limits for personnel carrying out inspection and repair work. Smith et al (1994) concluded that work on the dyke was unsafe when the mean discharge exceeded q = 0.01 m^3/s.m. Examination of Smith et al's data suggest that this probably corresponded to v_{max} = 1 to 2 m^3/m. This is considerably higher than the limits determined by Franco et al (1994) for work behind a tall crown wall, but does match their observation that safe limit of v_{max} varies with structural type and therefore the different way in which the water strikes the individual. In tests reported by Smith et al (1994) most of the overtopping discharge acted on the observer's legs only. It must again also be borne in mind that the safety limits for trained personnel working on a structure and anticipating overtopping are higher than those for other users.

Information on prototype safety was also derived by Herbert (1996) who monitored overtopping behind a vertical seawall. During installation and operation of the measurement equipment, Herbert observed that personnel could work safely on the crest of the wall during mean discharges up to q = 0.1 l/s.m. Individual overtopping volumes were not measured, but the analysis methods described by Besley (1999) can be used to estimate peak volumes, given the mean discharge and incident wave conditions. These calculations give a limiting volume of approximately v_{max} = 0.04m^3/m for the sea state which caused q = 0.1l/s.m. This is in close agreement with Franco et al's estimate of v_{max} = 0.05m^3/m to cause someone to lose their balance.

Herbert (1996) also used field data to note that overtopping became dangerous to vehicles when the mean discharge exceeded q = 0.2 l/s.m. Using the process above, this corresponds to v_{max} = 0.06m^3/m, suggesting that v_{max} = 0.05m^3/m should be applied as a safe upper limit for pedestrians and for vehicles driven at any speed.

At the start of the CLASH project, existing limits reviewed above were summarised as in Table 3.1 below.

Table 3.1 Initial Guidance on Tolerable Mean Overtopping Discharges (m^3/s.m)

Embankment Seawalls :-					
No damage			q	<	0.002
Damage if crest not protected	0.002	<	q	<	0.02
Damage if back slope not protected	0.02	<	q	<	0.05
Damage even if fully protected			q	>	0.05
Promenade Seawalls :-					
No damage			q	<	0.05
Damage if promenade not paved	0.05	<	q	<	0.2
Damage even if promenade is paved			q	>	0.2
Buildings :-					
No damage			q	<	$1x10^{-6}$
Minor damage to fittings etc	$1x10^{-6}$	<	q	<	$3x10^{-5}$
Structural damage			q	>	$3x10^{-5}$
Vehicles :-					
Safe at moderate / higher speeds			q	<	$1\ x10^{-6}$
Unsafe at moderate / higher speeds	$1\ x10^{-6}$	<	q	<	$2\ x10^{-5}$
Dangerous			q	>	$2\ x10^{-5}$
Pedestrians :-					
Wet, but not unsafe			q	<	$3\ x10^{-6}$
Uncomfortable, but not unsafe	$3\ x10^{-6}$	<	q	<	$3\ x10^{-5}$
Dangerous			q	>	$3\ x10^{-5}$

4.3 NEW EVIDENCE ON PERSONNEL HAZARDS

Every year, people drown after being swept from breakwaters, seawalls and rocky coasts. Example incidents for the UK gleaned from a single source for 1999-2002 are summarised in Appendix C and for Italy between 1983 and 2002 in Appendix D. To the individual, the waves responsible for such incidents may appear to be sudden and surprising, so it is probable that the people concerned had relatively little idea of the hazard to which they exposed themselves. It is however likely that many of these events could be predicted by informed analysts using some weather / wave forecasting and the results of recent research.

Figure 18 Moderate overtopping at Samphire Hoe, note warning flag

An early example of a custom-built overtopping warning system is described by Gouldby *et al.* (1999) for the low-lying reclamation at Samphire Hoe near Dover. This artificial reclamation was formed by chalk spoil from the excavations of the Channel Tunnel retained by a vertical sheet pile wall. The broad promenade is widely used as a leisure resource, but is subject to wave overtopping during storms, see Figure 18. Careful management of access was therefore important to ensure visitor safety. A warning system was therefore developed in which overtopping above agreed thresholds were predicted by output from an appropriate numerical wave model. Wave conditions were correlated with incidents of known overtopping hazard, categorised as low, moderate or high, see Figure 19. These warning levels were then communicated by the use of warning flags, see Figure 18, and ultimately by closing access to the seawall.

Figure 19 Categorisation of overtopping hazards at Samphire Hoe, low, moderate and high

Use of this system is analysed in Appendix B and by Allsop *et al.* (2003). Examples of the occurrence of percieved hazards are categorised, and mean overtopping discharges were calculated for each "hazard" event. These were used by Allsop *et al.* (2003) to support the continuing use of $q \leq 0.03$ l/s.m as a safe limit for (unaware) pedestrians when subject to impulsive jets.

The general approach to reducing risks described by Gouldby *et al.* (1999) is however only possible where an owner / operator has the means and resources to obtain advance forecasts of hazards, and then to operate such an exclusion system. Elsewhere it is generally only possible to issue warnings.

Figure 20 Public watching / dodging overtopping at Oostende

5. Perceptions of overtopping

It is appreciated by engineers and coastal managers that seawalls reduce wave overtopping, but it requires a sophisticated understanding to be aware that seawalls do not always stop, but simply reduce overtopping. Under storm action, waves still overtop seawalls, sometimes frequently and perhaps violently. These processes may excite considerable public interest, see the example in Fig. 20 at Oostende where tourists gather during storms.

Fig. 21a Beach, seawall and promenade at San Sebastian, Spain

Fig. 21b Artificial beach, breakwaters and resort at Lanzarote

The key problem identified during the PPA project is that most messages to tell the public about the seaside and coastal activities (particularly those marketing a vision) present only the "sunny" view of coastal processes. There is no motivation for the developer / architect / advertiser to show "stormy" or winter views where hazards might be more easily perceived. This imbalance is compounded by tools that communicate messages of hazard well to engineers and scientists, but do not carry the same message to members of the public.

Fig. 22a Yacht harbour of Salivoli (Tuscany) during storm in November 2001

Fig. 22b West Harbour, Hartlepool, under 1:50 year storm, physical model

Examples of this problem are illustrated in Figures 21 and 22. The first of these show example of coastal structures as experienced by most members of the public. The sun is shining, the waves are small. There are no obvious hazards. Contrasting views of substantially greater hazard are shown in Figure 22 showing severe waves at two small harbours. The first photo shows waves of H_s = 3-3.5m at the Italian harbour of Salivoli (Tuscany) in November 2001. The second shows waves equivalent to H_s = 4m at the harbour of Hartlepool, UK, as modelled at a scale of about 1:40. All coastal engineers will be able to perceive equivalent levels of hazard to either situation, experienced as she / he is in scaling the process to full scale. The problem identified by the non-engineer members of the PPA project is that members of the public cannot easily make the same mental jump. To them, there is no obvious hazard from waves of 50-100mm height! It was clear, therefore, that any graphic or photograph seeking to explain wave / coastal / overtopping processes would have to take account of this perception "blind-spot".

5.1 CHANGING PUBLIC PERCEPTIONS

Changes to public behaviour will partially be driven by changes to direct management practices at coastal sites, but will also require improvements in awareness of potential hazards, and some understanding of the key drivers. This will require changes on a number of fronts: increasing general awarenesss of sea / coastal processes; greater awareness of hazards posed by wave overtopping and related processes; and use of site specific warnings.

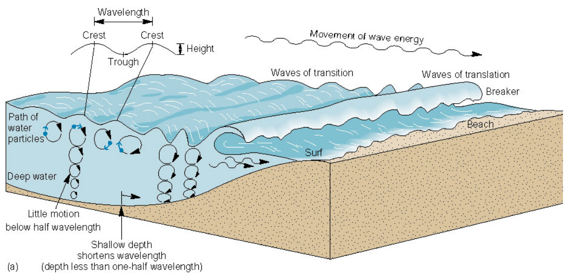

Fig. 23 Example of clear graphic showing coastal wave process, but note that there is no scale

At the most general level, work is needed by coastal engineers in general to engage with the public media to explain coastal engineering processes in general. Most such work is most obviously focussed on teaching, where each learning increment builds on previous understanding. The example in Fig. 23 shows wave processess in cartoon fashion, but does not need to be correct in terms of scale.

Fig. 24 Extracts from video of overtopping incident at Giant's Causeway, 16 August 2002

A major danger in producing simplifying explanations are the consequences of media tendencies to sensationalise the issue, submerging reality in hyperbole. Use of the term “freak waves” for any large wave (however predictable by modelling of wave statistics or processes of wave-wave interactions) is the prime example of such distortions. The use of such “tabloid” expressions debases the public view of the probability of encountering large waves. A particular area of weakness is the widespread lack of understanding of shoaling of swell waves, likely to give inshore waves many times greater than offshore where waves of low steepness (say $s_{op} < 0.5\%$) shoal up over steep slopes. Given that this is exactly the process by which surfing waves are generated, it is perhaps surprising that so few professionals and publi appreciate the process which was probably the prime cause of the incident at Giant’s Causeway shown in Fig. 24.

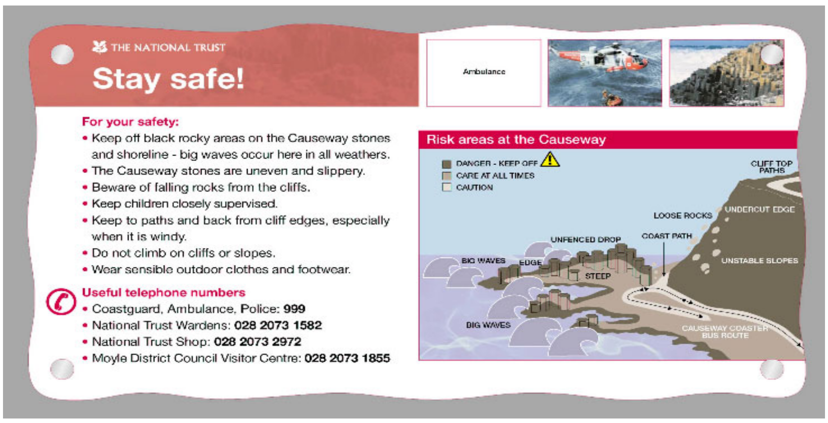

Fig. 25 Information board, Giant’s Causeway

In this incident on 16 August 2002 at Giant's Causeway, 8 children and a "responsible" adult were swept into the sea by a "freak" wave, see Figure 24. All were rescued, but this incident highlights typical misperceptions of risk in such situations, and lack of serious attention to warnings.

Further evidence of (mis-)perceptions of the danger of overtopping are provided by contrasting the judgements of quite high allowable thresholds made by "students" and "experts" viewing video of overtopping at Ostia in Appendix K with the rather lower thresholds given earlier in Table 3.1.

With climate change bringing increased storminess, there will be more locations where these hazard will increase. The public are aware of climate change, but will not make the link to overtopping hazards unless better informed. This is aggravated by media references to "freak waves" that are in truth entirely predictable by an informed person, and media concentration on tsunamis and other "televisual" hazards of very low probability.

Fig. 26a Example warning notice, tidal threat

Fig. 26b Example notice, overtopping threat

5.2 AWARENESS OF COASTAL PROCESSES

The most immediate action of any owner or responsible authority aware of a potential hazard is to ensure that the public are made aware of the hazard. The general issue of hazards on coastal structures has been discussed by Halcrow (1997) and Heald (2002) who show examples of poor signage. Better examples of warnings from National Trust sites are shown in Figures 25 and 26.

A more complete approach to raising awareness is illustrated in Figure 25 where the full range of hazards at Giant's Causeway are identified. It may be noted that the sign in Fig. 25 specifically identifies the inherent danger of large waves on the more exposed end of the Causeway.

Some tools that can be used to train coastal engineers, scientists, and perhaps managers, may not be so useful in informing the public. Example cartoons developed by HRW and the PPA project for the UK Environment Agency are shown in Appendix M to illustrate the development of overtopping and possible damage under extreme storms.

6. *Post overtopping velocities and loads*

6.1 OVERTOPPING VELOCITIES

Until recently, few data have been available on overtopping velocities. Pearson *et al* (2002) and Bruce *et al* (2002) have presented measurements at small and large scales of upward velocities (u_z) form vertical / battered walls under impulsive and pulsating conditions. They related the measured upward velocity u_z to the inshore wave celerity given by $c_i = (gh)^{1/2}$. Relative velocities, u_z/c_i, were plotted against the wave breaking parameter, h_*., see Figure 15 in Chapter 3. Non-dimensional velocities were roughly constant at $u_z/c_i \approx 2.5$ for pulsating and slightly impulsive conditions $h_* > 0.2$, but overtopping velocities increase significantly for impulsive conditions when $h_* \leq 0.2$ reaching $u_z/c_i \approx 3$ - 7.

For simply sloping embankments, such as shown in Figure 18, Richardson *et al* (2002) measured crest velocities of around $u_z/c_i \approx 2$ behind a 1:2 slope under plunging conditions. Simulations for 1:1-1:5 slopes discussed in Appendix H showed overtopping bore velocities in the range u = 2-5 m/s.

Figure 27 Observations of overtopping velocities and secondary wall loads

Further data on overtopping velocities have been presented by Romestang in Appendix F. Analysis of video of overtopping velocities in the Samphire Hoe 3-d model gave peak velocities of u_z = 1-9m/s, corresponding to $u_z/c_i \approx 0.2 - 1.2$, much lower than found by the VOWS tests. Analysis of video recordings from the Carlyon 3-d model, see Figure 27, gave horizontal overtopping velocities behind the recurve seawall of u_x = 3.5 to 5.5m/s.

These levels of velocity may be put into context by findings from UK studies on flood risks to people, see Ramsbottom *et al.* (2004) who present hazard classification tables based on flow depths and velocities. The suggested limits from Table 3.4 of Ramsbottom *et al.* (2004) are re-represented here as Figure 28. As these velocity / depth limits were originally derived for relatively steady flows, it would be wise to take a precautionary view of these limits in the derivation of any suggested limits. The middle threshold in Figure 28 suggests that flow velocities above $u_z \geq$ 2.5m/s will be difficult to resist for depths greater than d > 0.5m, and $u_z \geq$ 5m/s will be difficult to resist for depths greater than d > 0.25m.

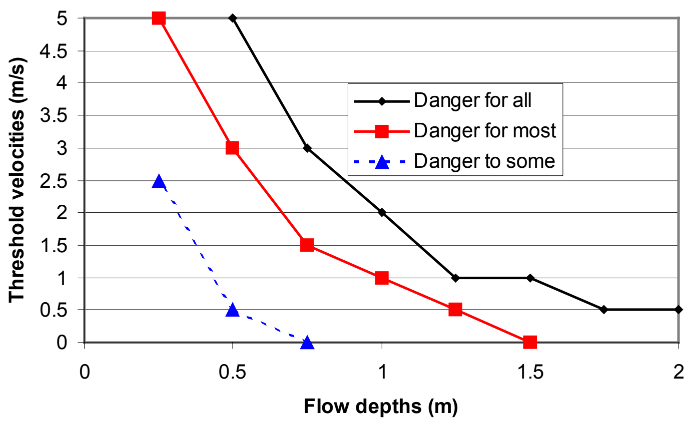

Figure 28 Suggested velocity / depth limits from Ramsbottom *et al.* (2004)

6.2 POST OVERTOPPING WAVE LOADS ON STRUCTURES

Wave loads have seldom been measured on defence structures, buildings behind sea defences, or on people. Under CLASH, post overtopping loads on person-sized dummies and a length of pipeline have been measured at full scale at Zeebrugge, and at small scale at LWI and HRW.

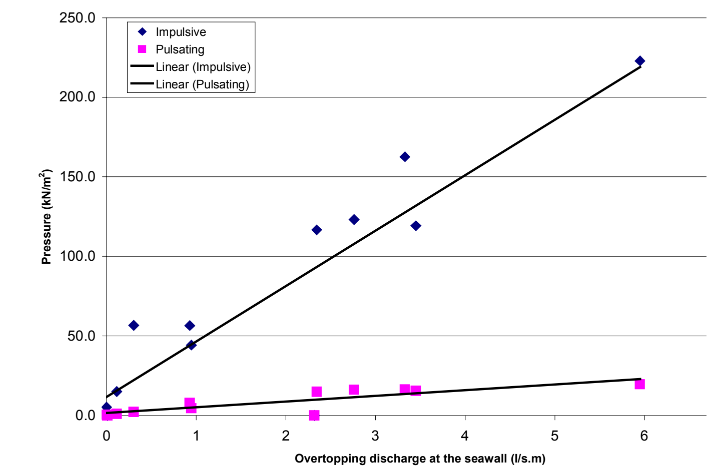

Figure 29 Wave loads (impulsive and pulsating) on secondary wall from Appendix F

For test conditions described by Romestang in Appendix F, wave pressures measured on the 1m high secondary wall set 7m back from the primary (recurve) wall are shown in Figure 21 plotted against mean discharges measured just behind the primary seawall.

The impulsive pressures (examples are shown in Appendix F) were approximately 11 x greater than the quasi-static loads. Extrapolating the trend lines in Figure 22 down to an overtopping condition of q=0.03 l/s.m suggest that the quasi-static pressures might reduce to $p_{q-s} \approx 2$ kN/m^2, but that impulsive pressures might not fall below $p_{imp} \approx 20$ kN/m^2 . These may be put into context when noting that few buildings are designed for horizontal wind loads above $p_{av} \approx 0.5$ kN/m^2.

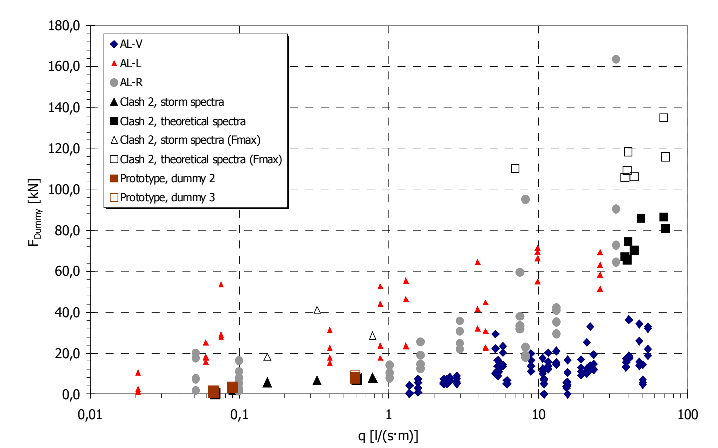

Figure 23 Wave loads on person dummies (from tests at LWI, see Appendix J)

Measurements on the person dummies are also discussed in Appendix J, and a summary graph of results is shown here in Figure 30.

These measurements suggest that wave loads on a person increase rapidly for increasing overtopping discharges. Advice quoted by Kleidon in Appendix J cites work by Endoh et al as giving force limits on individuals of up to F_h = 140 kN. Given other data collected for this and related studies, this force limit appear much too high. e

7. *Guidance on wave overtopping limits*

This section discusses the present state of knowledge on tolerable wave overtopping. It includes gudance derived from the CLASH field and laboratory work, and builds on previous guidance, see Fukuda *et al.* (1975), Owen (1980), Besley (1999) and Allsop *et al.* (2003). A number of limits are suggested in Table 5.1 for mean overtopping discharge or peak overtopping volume. These limits derive from a generally precautionary principle informed by previous guidance and by the various observations and measurements made by the CLASH partners and research colleagues. The main evidence for changing or extending previous advice was summarised in Chapter 4.

Table 5.1 Suggested limits for overtopping mean discharges or peak volumes

Hazard type / reason	Mean discharge, q	Peak volume, V_{max}	Comments or other limits
Pedestrians			
Unaware pedestrian, no clear view of the sea, relatively easily upset or frightened, narrow walkway or close proximity to edge	0.03 l/s.m	2-5 l/m at high level or velocity	
Aware pedestrian, clear view of the sea, not easily upset or frightened, able to tolerate getting wet, wider walkway.	0.1 l/s.m	20-50 l/m at high level or velocity	
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 l/s.m	500 l/m at low level,	$d.u^2 < 1\text{-}5\ m^3/s^2.m$ TB – velocity limit from Cox?
Vehicles			
Driving at moderate or high speed, impulsive overtopping giving falling or high velocity jets	0.01-0.05 l/s.m	5 l/m at high level or velocity	
Driving at low speed, overtopping by pulsating flows at low levels only, no falling jets	10-50 l/s.m	1 m^3/m	
Property			
Damage to windows / cladding / fittings set back 5-10m			
Structural elements set back 5-10m			
Sinking small boats set 5-10m from wall. Damage to larger yachts	q = 10 l/s.m	1 - 10 m^3/m	Volumes depend on vessel position etc., form of overtopping flow and wave transmission
Significant damage or sinking of larger yachts	q = 50 l/s.m	5 - 50 m^3/m	

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