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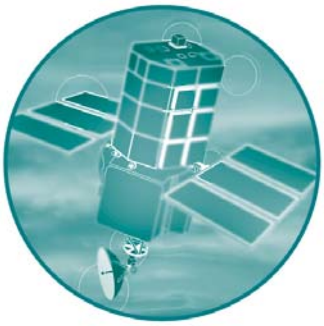

Coastal Flooding Hazard by Wave Overtopping SHADOW Phase 1

Wave Overtopping of Simple Embankments: Improved Methods

R&D Interim guidance note FD2410/GN2

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Wave Overtopping of Simple Embankments: Improved Methods

This note (intended for publication through the Defra / EA web site) summarises interim guidance developed in spring 2003 from Defra / EA research projects FD2410 and 2412 on extending empirical prediction methods for wave overtopping at sea defences. More complete guidance is being developed under FD2412 and the European "CLASH" research projects which will cover a much wider range of structures and will develop new prediction methods, including a single executable Neural Network tool to predict mean overtopping for a wide range of structure types. Final results of FD 2412 / CLASH (including the NN) will be delivered in early 2005.

In the interim, this note is intended to up-date guidance in the present EA Overtopping Manual, Besley (1999), which it is assumed the reader is familiar.

Wave overtopping prediction methods

Wave overtopping and resistance to breaching are the key response characteristics for sea defence structures, so coastal engineers need accurate methods to predict wave overtopping. Overtopping discharges vary with wall shape, crest level, water level and wave conditions. Generally design procedures are expected to calculate the crest freeboard (height of crest above water level) that would limit overtopping to below a chosen limit, see Besley (1999). Empirical models or formulae use relatively simple equations to describe wave overtopping responses (peak and mean wave overtopping volumes / discharges) in relation to defined wave and structure parameters. Empirical methods are, however, limited to a small number of simplified structure configurations. Use of such formulae out of range or for other structure types may require extrapolation, or may indeed not be valid.

Numerical models of wave overtopping can (in theory) be configured for any structure within the overall range covered, but present models are limited in the types of structure (and degrees of complexity) to which they may be applied. The use of numerical models of overtopping are discussed in a separate note.

New overtopping data on simple embankment seawalls

Studies under FD2410 and FD2412 measured new data on overtopping responses on simple slopes of 1:2, 1:10 and 1:15, extending the range of coefficients for simple sloping embankments. This note summarises early results from these new data, and indicates where methods in the EA Overtopping Manual by Besley (1999) may be improved or extended.

Under FD2410 and FD2412, 2-dimensional physical model tests on simple slopes of 1:10 and 1:15 (and 1:2) measured wave-by-wave volumes and mean overtopping discharges. These tests extended the range of structure slopes well beyond the shallowest slope (1:4) modelled by Owen (1980). This note presents conclusions from initial analysis of those 2-d test results. A limited range of tests studied 3-dimensional effects, and later analysis will provide guidelines on the effects of oblique wave attack and plan geometry (e.g. corners).

For simple sloping embankment seawalls, Box 3.1 of the EA Overtopping Manual, Besley (1999) limits the validity of Owen's (1980, 1982) method for predicting mean overtopping discharges to dimensionless freeboard values falling in the range $0.05 < R^* < 0.30$. It was uncertain whether Owen's empirical equations could be used for larger values of R^* (higher relative freeboards), when overtopping discharges are relatively low. (This may be critical for structures designed to some of the lower tolerable discharge limits.) Analysis of the recent data for 1:2 smooth slopes now indicates that this range can be extended to $0.05 < R^* < 0.50$.

Also for simple sloping embankment seawalls, coefficients for the prediction method by Owen (1980) in the Overtopping Manual are limited to slopes in the range 1:1 to 1:5. (The original tests covered 1:1. 1:2 and 1:4 slopes. Coefficients A & B for other slopes were derived by careful interpolation or extrapolation.) In the new analysis, it has been argued that values of the coefficient B for slopes shallower than 1:4 can be estimated by considering the relative run-up using the classic method by Hunt (1959), provided that coefficient A can be assumed to be constant. To do so, demands the assumption that overtopping of a structure of zero freeboard ($R_c = 0$) is relatively little influenced by the (submerged) slope. Whilst unlikely to be true in the limit, most test results to date would allow this simplification to be made.

New test data for slopes of 1:10 and 1:15 have been used to test this hypothesis, and to determine new values for B for those slopes. Measurements of mean overtopping discharges give reasonable (but not excellent) support to this simplifying hypothesis and allow values of A & B in Table 1 to be suggested from analysis of the comparative Irribarren number or surf similarity parameter.

Table 1: Suggested values of A & B for use in Owen's method for simple sloping seawalls

Slope	A	B	Comments
1:2	0.01	21.6	Good fit with test data except for low wave steepnesses, $s_{om} < 0.02$
1:4	0.01	43	Interpolated value, EA manual gives B = 41
1:6	0.01	65	Interpolated
1:8	0.01	86	Interpolated
1:10	0.01	108	Reasonable fit with test data, but tends to under-predict unless adjusted.
1:15	0.01	162	Moderate fit with test data, but generally under-predicts unless adjusted.

It is important to note that the simplified approach above disguises the fact that plunging and surging waves behave differently on slopes, and might therefore be expected to give different overtopping responses. Analysis of overtopping on the very shallow slopes tested (1:10 and 1:15) suggests that processes involved in overtopping on long shallow slopes are rather different to those that occur on steep slopes. The new coefficients suggested in Table 1 are therefore not recommended for detailed design, but may be appropriate for initial estimates.

For very long slopes, it is not immediately obvious where the approach beach ends, and the defence structure starts, indeed this distinction may be entirely artificial. Some uncertainty therefore attaches as to the definition of “incident wave height” to be used in any prediction method, and this is increased for steep beach slopes (steeper than 1:30) or shallow structure slopes (shallower than 1:6).

Use of the overtopping prediction method above for slopes of 1:10 and 1:15 was improved for the test cases when the “incident wave height” was corrected to a “shoaled, pre-breaking” wave height. Simple linear shoaling was applied to the incident wave height up to, but not beyond, the point of breaking. This “adjusted” wave height was then used in calculations of Q* and R* using Owen’s method and coefficients in Table 1 above. To determine this adjustment, it is assumed that waves need to travel up to 80% of the local wavelength before they complete the breaking process, see Allsop et al (2002). If the horizontal distance from the toe of the structure to the SWL on the structure slope is greater than 0.8L, then the incident wave height should be adjusted by an appropriate shoaling coefficient up to that position before R* is calculated.

If the toe of the sea defence structure is not simply defined, or the slope of the “structure“ is very similar to that of the approach beach / sea bed, then a distance 0.8 L out from the SWL should defined as giving the effective “toe”. The incident wave condition should then be recalculated at this “toe” by shoaling the waves in from the depth / location where waves are defined.

An alternative method that might be used for overtopping of shallow slopes has been developed in the Netherlands, outlined initially by van der Meer & Janssen (1995) and now implemented by the Technical Committee on Water Defences (TAW, 2001). This approach is discussed briefly in Annex 1 to this note.

Conditions of low N_{ow}

During analysis of overtopping measurements for slopes of 1:2, 1:10 and 1:15, it was noted that conditions of low N_{ow} showed some unexpected features. When the recent data are plotted in Owen’s dimensionless R* and Q*, the graphs suggest that waves with low N_{ow} are subject to some different processes than those with high N_{ow}. Generally, waves with N_{ow} > 5% behave in accordance with Owen’s method as described in the Overtopping Manual. Some conditions with low overtopping, typically N_{ow} < 3%, show lower values of Q* for the same range of R*. The same behaviour patterns are seen with calculations run with a numerical model, suggesting that these observations are not simply an experimental phenomenon. The recent test data suggest that Owen’s method may therefore over-predict N_{ow} for low wave steepnesses, $s_p < 0.02$, and higher breaker parameter, $\xi_p > 3.5$. These conditions may be better predicted by using van der Meer’s surging wave formula, as shown in Annex 1.

For conditions in the category Low Q - Low N_{ow} (see below), overtopping is not likely to be hazardous to structures or give significant flooding. Moderate levels of uncertainty are probably acceptable, unless the predictions are being made to address a public safety issue, in which case simple empirical calculations are not appropriate and hydraulic modelling may be needed. Future research should specifically address uncertainties in predicting low values of N_{ow}, as recent measurements are well-correlated with predicted peak volumes, if N_{ow} itself can be predicted more precisely.

Comments in gaps / weaknesses in prediction methods

During analysis of overtopping measurements for slopes of 1:2, 1:10 and 1:15, some unexpected features were observed, and are summarised in Table 2. The classifications in Table 2, supported by the following definitions, categorise overtopping into four basic groups, determined by whether the discharge (Q) and the proportion overtopping waves (N_{ow}) are high / moderate or comparatively low. These descriptions involve considerable simplification of complex natural phenomenon, so should be treated with caution, but they assist in identifying areas of confidence and of concern in present knowledge.

Table 2: Overtopping hazards

	Higher frequency N_{OW} > 2%	Low frequency of overtopping N_{OW} < 2%
High Q	Use Besley (1999)	This type of overtopping may be dangerous and is not well predicted by current empirical methods. Specific studies may be required.
Low Q	Use Besley (1999)	This overtopping approaches the lower thresholds of measurement / prediction, unlikely to cause any significant hazard.

High Q - High N_{ow}: Severe overtopping as might be experienced by a rural seawall during severe storms. Overtopping discharges are predicted by methods in the Overtopping Manual within expected uncertainties. Whilst flooding and property damage might occur, these conditions are inherently rare, are probably the least hazardous situation for public safety if proper precautions are observed, and are relatively straightforward to predict with present knowledge.

Low Q - High N_{ow}: Less severe condition than High Q - High N_{ow}, and one that is more likely to occur where the relative freeboard has been increased. Many of the tolerable discharge rates for public safety and minimum property damage fall within this category. These conditions are generally well predicted by methods in the Overtopping Manual.

Low Q - Low N_{ow}: Overtopping in these conditions is often limited to light spray. These conditions are predicted less well by empirical methods, including those in the EA Overtopping Manual, as few test data are available. These conditions however generally cause little concern as mean overtopping discharges are likely to fall below critical thresholds. At some structure types, typically steep / vertical walls, these conditions may however generate comparatively large peak volumes or overtopping velocities of potential concern to safety of local traffic (people and vehicles). New data will be needed to improve prediction methods at low / no overtopping.

Moderate Q - Low N_{ow}: These are potentially the most dangerous conditions for public safety as methods to predict occurrence and magnitude of rare events are weakly validated. Analysis of recent measurements confirm that methods to calculate peak overtopping volumes in the Overtopping Manual, are not accurate below $N_{ow} < 2\%$. (The manual recommends use only down to $N_{ow} \approx 5\%$) Recent tests on 1:2 and other slopes show several instances with $N_{ow} \approx 0.2\%$, but relatively high peak volumes. These events are perhaps relatively rare, and probably pose no significant problem for flooding, but such conditions may cause significant hazards to unsuspecting members of the public who consider themselves to be safe. Research (some current under FD2412 / CLASH) is required to improve prediction methods for these situations.

Summary of new guidance

The research results discussed in this note have widened the range of application of methods in the Overtopping Manual. The range of dimensionless freeboards has been expanded to $0.05 < R^* < 0.50$.

New values of the empirical coefficients A & B (and a method to adjust for very shallow slopes, have been derived for a wide range of slope angles, given in Table 1 above.

Conditions have been highlighted where present knowledge is limited, but where overtopping may cause danger under conditions that might previously have been assumed to be benign.

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Annex 1: Wave overtopping predictions by van der Meer & Janssen

A method for estimating storm wave run-up and overtopping on sea dikes was developed for the Netherlands by van der Meer & Janssen (1995). The method has general applicability, is based on extensive laboratory testing, and claims a well-tested methodology for overtopping discharge calculations. This method, however, distinguishes between breaking and non-breaking wave conditions, as identified by the surf similarity (ξ_{op}) or breaker parameter, where

$$\xi_{op} = \frac{\tan\alpha}{\sqrt{s_{op}}} \quad (1)$$

ξ_{op} = breaker parameter
α = shoreline slope angle (generally the structure slope)
s_{op} = wave steepness = $2\pi / gT_p^2$
g = acceleration due to gravity
H_s = significant wave height near toe of the slope
T_p = peak period of the wave spectrum

When ξ_{op}<2 waves are considered to be breaking on the slope, and the overtopping rate for breaking waves is calculated from an empirical relationship between the dimensionless overtopping rate

$$Q_b = \frac{q}{\sqrt{gH_s^3}} \cdot \sqrt{\frac{s_{op}}{\tan\alpha}} \quad (2)$$

and the dimensionless crest height

$$R_b = \frac{R_c}{H_s} \cdot \frac{\sqrt{s_{op}}}{\tan\alpha} \cdot \frac{1}{\gamma_b \cdot \gamma_h \cdot \gamma_f \cdot \gamma_\beta} \quad (3)$$

where;

Q_b = dimensionless overtopping rate for breaking waves
q = average overtopping rate
R_b = dimensionless crest of structure (breaking wave formulation)
R_c = structure crest height above still-water line
γ_b = reduction factor for influence of a berm
γ_h = reduction factor for influence of shallow foreshore
γ_f = reduction factor for influence of roughness
γ_β = reduction factor for influence of angle of wave attack

The main overtopping prediction equation recommended by van der Meer & Janssen for breaking waves (plunging) is;

$$Q_b = 0.06 \cdot e^{-4.7 \cdot R_b} \quad (4)$$

For the non-breaking case the following formulation is used:

$$Q_n = \frac{q}{\sqrt{gH_s^3}} \tag{5}$$

$$R_n = \frac{R_c}{H_s} \cdot \frac{1}{\gamma_b \cdot \gamma_h \cdot \gamma_f \cdot \gamma_\beta} \tag{6}$$

where;

Q_n = dimensionless overtopping rate for non-breaking waves
R_n = dimensionless crest of beach profile with non-breaking waves

and the recommended prediction equation for non-breaking waves is:

$$Q_n = 0.2 \cdot e^{-2.3 \cdot R_n} \tag{7}$$

There are several constraints on the previous relationships given by van der Meer & Janssen, based on the range of conditions tested and the nature of the laboratory testing. The product of all γ reduction factors is limited to a minimum value of 0.5. The accepted range of application of the breaking wave relationship is for $0.3 < R_b < 2$, and for the non-breaking method, use R_w when $\xi_{op} > 2$.

The product of the four reduction factors (γ) can generally be assumed to take a value of unity for simple impermeable slopes, independent of slope angle.