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Coastal Flooding Hazard by Wave Overtopping SHADOW
Phase 1

Use of Numerical Models of Wave Overtopping:
a Summary of Current Understanding

R&D Interim guidance note FD2410/GN1

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Use of Numerical Models of Wave Overtopping: a Summary of Current Understanding

This note (intended for publication through the Defra / EA web site) summarises interim guidance developed in spring 2003 from research projects FD2410 and 2412 on the use of numerical models to predict wave overtopping at sea defences. More complete guidance is being developed under FD2412, and final results will be reported in 2004 / 2005. It is expected that the user of this note will already be familiar with the EA Overtopping Manual by Besley (1999). A separate note suggest improvements to some of the empirical methods within that manual.

Numerical modelling of wave overtopping

For sea defence structures, wave overtopping (usually given by the mean overtopping discharge) may be predicted by empirical or numerical models. The mean overtopping discharge varies 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 discharges in relation to defined wave and structure parameters. Empirical equations and coefficients are, however, limited to a relatively small number of simplified structure configurations. Their use out of range, or for other structure types, may require extrapolation, or may indeed not be valid.

Numerical models of wave overtopping may be less restricted, in that any validated numerical model can (in theory) be configured for any structure within the overall range covered. Numerical models based on non-linear shallow water wave equations are potentially very efficient, so wave trains of (for example) 1000 random waves can be simulated rapidly. Such models have become increasingly attractive for scheme design and for flood forecasting. If these models are to be used with confidence, then clear guidance will be required for end users on: what processes the model will reproduce; what wave conditions are valid; and for what types of structures the model can be used.

One-dimensional (1D) shallow water equations were originally developed for near horizontal, free-surface channel flows. The equations describe water depth and velocity in time and space, but assume a small bed slope, typically incorporate a simplified bed shear stress term, and ignore wind shear *etc*. Implicitly these equations are also only applicable when the main flow occurs in one direction and the structure can be assumed to be alongshore homogeneous. Vertical velocity, as a variable, is neglected, being assumed small in comparison to horizontal velocity. The pressure distribution in the vertical is assumed to be hydrostatic. Non-linear shallow water (**NLSW**) equations are deduced from the Navier-Stokes equations by averaging over depth and applying the aforementioned hydrostatic assumption. A broken wave is represented as a bore (i.e. a near-vertical face). At the crest of a sea defence structure, these models must be able to continue computing as the flows either side of the crest separate, overtop or return.

In two plan dimensions (2D), NLSW models can simulate overtopping of more complex (non-homogenous) structures and complex (non-normal) sea states, although they are still limited by the shallow water assumptions.

Physical model tests suggest that a sea state represented by 1000 random waves will give reasonably consistent results, but that shorter tests may show significant variations in extreme statistics. Any numerical model should therefore be capable of running similar numbers of waves, a requirement that is relatively easy to satisfy by models based on NLSW equation (e.g. ANEMONE, AMAZON, and ODIFLOCS) , but probably not yet possible for models based on more complete representations (e.g. Volume of Fluid, VOF, or Smoothed Particle Hydrodynamics, SPH).

Setting up the models

As waves propagate towards the shoreline, their wave heights and lengths transform due to physical processes. The relative depth ratio *h/L < 0.05* (where h is the water depth and L the inshore wavelength) defines the range where NLSW equations are valid, see Figure 1. This limit places severe restrictions on the use of models based on these equations, and researchers have recently begun to investigate to what extent the limit (*h/L < 0.05*) might be increased. One key consequence of the condition is that the seaward boundary (sometimes termed the inflow boundary) of the numerical model must be relatively close to the structure, sometimes even further inshore than the structure toe, particularly for seawalls of shallow slopes or compound / bermed form.

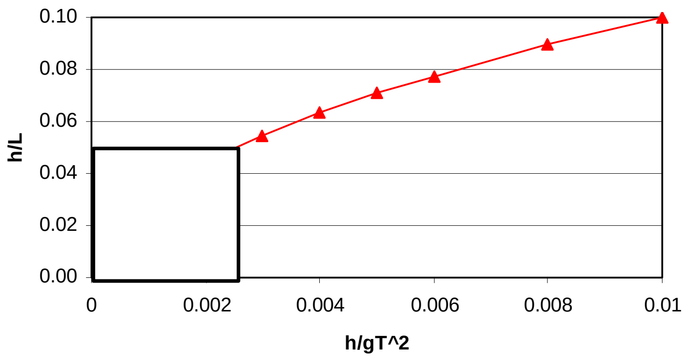

Figure 1 Theoretical limits for NLSW equation models are shown inside the box, h/L < 0.05

Physical model tests at Edinburgh University were modelled by Richardson et al (2001, 2002) using the AMAZON-CC NLSW models at Manchester Metropolitan University under the VOWS project (see: www.VOWS.ac.uk). In common with all NLSW models, these waves were not dispersive. Propagation of waves that were generated in relative water depths of h/gT^2 = 0.03 to 0.08 (equivalent to *0.17 < h/L < 0.28*) was not simulated accurately using the NLSW wave models. When the comparison was made with waves measured and

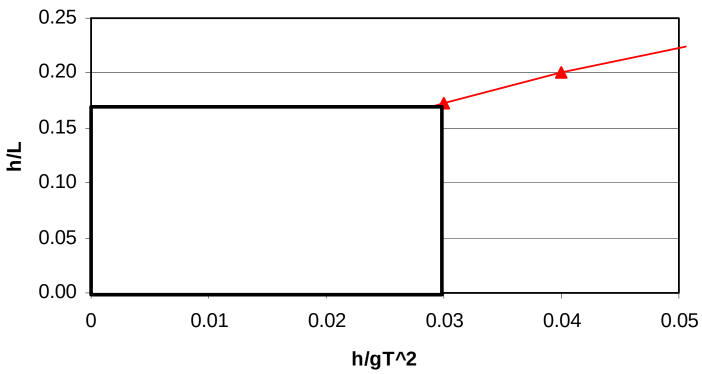

Figure 2 Practical limits for NLSW equation models suggested by the VOWS comparisons are given by the box, note change of scale.

simulated at or close to the structure toe in relative depths of $h/gT^2 = 0.01$ to 0.03 (equivalent to $0.1 < h/L < 0.17$, see Figure 2), correlations of both wave processes and overtopping have been shown to be very good.

Other users have sometimes started the numerical model some 10's or 100's of metres seaward of this limit. Early tests with one of these models in the study of Coastal Defence Vulnerability 2075 by Sutherland & Wolff (2001, 2002) showed variations in waves and overtopping as the seaward point of the overtopping model was moved inshore from approximately 100-200m seaward of the structure (equivalent to $0.09 < h/L < 0.11$). Overtopping waves and discharges were generally highest when the numerical overtopping model was started near the structure toe, reducing if the overtopping model was started farther out, even when all other input parameters remain fixed.

It is clear therefore that overtopping rates vary with the position of the seaward boundary, and that the position of the seaward boundary of the numerical model should be as close to the structure toe as possible. That does however have the disadvantage that the wave conditions must also be correctly specified at that position, see discussion on wave conditions below.

Structure configurations

Theoretically, NLSW models can only be used for shallow slopes where the vertical component of the wave flow is relatively small. In practice, such models have been run for very steep slopes including walls battered at 5:1 or 10:1, well beyond those for which the underlying equations should be valid. Hu *et al.* (2000) and Richardson *et al.* (2001) discuss theoretical limitations in applying NLSW equation models to such structures and present overtopping predictions for steep walls with remarkably good agreement to physical model results for some conditions. It is not yet clear, however, how far these may safely be extrapolated, or how this expansion of their theoretical capabilities might interact with the positioning of the seaward boundary of the numerical model.

Hubbard & Dodd (2002) have examined overtopping at off-normal incidence using a 2D NLSW model, and find reasonably good agreement with wave basin experiments on 1:4 and 1:2 structures subject to incidence at a range of angles up to 30 degrees. These comparisons were run with the numerical model starting at the toe of the 1:2 or 1:4 slope.

At a scientific level, Alcrudo & Benkhaldoun (2001) examined the analytical solution of the NLSW equations in the presence of a vertical step in the bed and demonstrated that a solution could be found. Zhou et al (2001, 2002) also successfully extended their treatment of the bathymetry for use in the AMAZON-CC numerical scheme. Whilst these papers consider submerged vertical steps, they suggest that the numerical and theoretical treatments may be applied to steep faced sea walls.

Wave conditions

These numerical models are most useful if they can be run to simulate random waves (1000 x T_m is suggested). For practical design cases, an offshore wave condition is most easily available, either as simple values of H_s, T_m or T_p, and perhaps with a spectral shape. Such parameters can be used as input to the simulation of a sea state, but reproducing waves so far away from the overtopping structure would not conform with the advice discussed earlier on the wave generator position. At any “offshore” condition, often taken seaward of a -10m contour, waves will be dispersive and far from depth-averaged, and tests so far suggest may lead to under-estimates of the overtopping.

It has been noted previously that numerical models based on NLSW equations require shallow water wave conditions as input. Those wave conditions must therefore include effects of shoaling, refraction, depth-induced breaking and related non-linear effects. Various techniques and models are available to transform wave conditions inshore, often outputting simple wave parameters of H_s, and T_m or T_p, but sometimes outputting the full transformed wave spectrum. In certain cases these models have been coupled directly with NLSW models. There is however no way of transforming a shallow-water non-linear wave spectrum into a shallow-water non-linear wave time series, as the phase-coupling between components of the spectrum is insufficiently understood. Coupling of phase-averaged non-linear wave models with NLSW wave models is therefore normally performed by generating a linear time series from a spectral significant wave height and average (or peak) wave period. This process throws out most of the non-linear information gained by using a nonlinear phase-averaged wave model.

There are two key problems with these procedures. Firstly, the detailed specification of wave conditions in shallow water is neither well-developed nor well-validated. Such wave conditions are complicated by energy shifting away from the original spectral peak and related non-linear effects. Changes to wave height distributions are also very complex. Even the simple task of estimating the significant wave height in depth-limited conditions is still far from routine. Secondly, the task of simulating a time series of nearshore wave conditions is complicated, and usually ignores wave asymmetry and related effects, unless a phase-resolving non-linear wave model (such as a Boussinesq model) is used to transform the offshore wave conditions into shallow water.

Example models

At HR Wallingford, Advanced Nonlinear Engineering MOdels for the Nearshore Environment (ANEMONE) were initially developed by Dodd (1998) and have been applied by Richardson *et al.* (2002) and Clarke & Damgaard (2002). There are several models within the ANEMONE suite, of which ANEMONE OTT-1d and OTT-2d (Hubbard & Dodd, 2002) simulate wave run-up and overtopping on 1D and 2D smooth slopes respectively, and OTTP-1d (Clarke *et al*., 2003) simulates flows over and within porous slopes using an approach based on that of Van Gent (1994), but incorporating a more sophisticated and rigorous treatment of the bed shear stresses. At Manchester Metropolitan University, the Centre for Mathematical Modelling and Flow Analysis (CMMFA) has developed the AMAZON suite of one- and two-dimensional models, described by Mingham & Causon (1998) and Causon *et al.* (2000). At Delft Hydraulics, ODIFLOCS (One-DImensional FLOw on and in Coastal Structures, van Gent, 1994) models the action of irregular waves on and within coastal structures. ODIFLOCS consists of an external flow model and a porous flow model coupled to simulate flow between the two regimes.

Each of the models summarised above have been validated or calibrated to some degree (e.g. Niemeyer *et al.*, 2002), and attempts have been made to develop guidance on some of the issues raised here. Since the basic mathematical equations are essentially the same for each model, then any guidance for one will generally apply to the others, with clear advantages for researchers and model users.

Future use of these models

Coastal engineers often need to predict overtopping for situations where there are no calibrated empirical methods available. Alternative approaches are then to use physical or numerical model simulations. Physical model tests remain the most reliable and robust approach, and recent work by Pearson *et al.* (2002) has demonstrated that scale effects on mean overtopping discharges are probably minimal, even for impulsive overtopping.

In some circumstances, however, it will be more convenient to use a numerical model, and this is likely to become more frequent when clear guidance is available on how to use NLSW models to predict overtopping with confidence. Even so, use of such models, and interpretation of results will still require specialist experience.

One potential area for significant improvement might be to use those models to extend empirical methods in the Overtopping Manual (Besley, 1999). Extended coefficients could be derived for structure configurations outside of the range studied in physical model or field tests.

Summary of present guidance

Numerical models of wave overtopping based on NLSW equations should be operated with the seaward boundary as close to the seawall as possible, with input wave conditions adjusted for shallow water effects. Present research suggests that careful use of such a model, run with 1000 random waves, may give reasonable estimates of mean overtopping discharges (that is within half an order of magnitude), even when the structure itself is relatively steep.

Some research suggests that elements close to vertical (but not recurved) may be included without introducing major errors, although this does breach key underlying assumptions. The research so far indicates that these models might be used for such structures, but this may involve some uncertainties.

The operation of such numerical models with a seaward boundary starting well beyond the structure toe leads to significant errors in the waves at the toe of the structure in the numerical model, and therefore in the wave-by-wave overtopping. It is not clear to what extent those differences are then reflected in the calculations of mean overtopping discharge, although initial data suggest overtopping may be under-estimated unless the overtopping model is started very close to the structure. Further tests are required to clarify this effect.

There remain uncertainties as to the inherent variabilities of the overtopping process as simulated by these models, and to the difficulties in simulating random waves at the seaward boundary without detailed knowledge of shallow water effects on those waves.

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