

CHAPTER 82

Overtopping of sea walls under random waves

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Abstract

Over the last fifteen years a long running research programme has been undertaken at HR Wallingford to investigate the overtopping discharge performance of a wide range of sea walls. The research, which is funded by the Ministry of Agriculture, Fisheries and Food, is principally aimed at deriving methods to enable design engineers to determine the overtopping performance of a particular sea wall cross-section under a range of wave and water level conditions. The studies have generally used random wave physical model tests in order to collect data which can then be employed to derive empirical equations that describe the level of overtopping discharge.

1 Introduction

This paper describes recent research work at HR Wallingford, based on the results of physical model tests, aimed at quantifying the overtopping performance of recurved and vertical sea walls. The work is a continuation of a large research programme which has resulted in the derivation of empirical methods for assessing overtopping discharges on embankment sea walls.

2 Summary of previous work

Considerable stretches of the United Kingdom (UK) coastline are protected by a simple earth embankment, consisting of a sloping seaward face, a horizontal crest just a few metres wide and possibly a rear slope. These embankments are

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particularly frequent in rural areas, where the seaward face is often protected either by grass or pitched stone. In the late 1970's the then Hydraulics Research Station carried out an extensive research programme to determine the overtopping discharge behaviour of embankment type sea walls, culminating in the production of design guidelines (Owen, 1984).

The design method established for embankment sea walls is based on a dimensionless discharge parameter, Q_* , and a dimensionless freeboard, R_* . These two parameters are defined below:-

$$Q_* = Q / (T_m g H_s) \quad (1)$$

$$R_* = R_c / (T_m \sqrt{g H_s}) \quad (2)$$

where Q is the mean discharge overtopping the crest of the sea wall,
 T_m is the mean wave period,
 H_s is the significant wave height,
 g is acceleration due to gravity
and R_c is the sea wall freeboard (the height of the sea wall crest above still water level).

The dimensionless parameters are connected by the following exponential equation:-

$$Q_* = A \exp (-BR_*/r) \quad (3)$$

where r is a roughness coefficient
and A and B are empirically derived coefficients dependent upon the structure slope.

Typical values of these empirical coefficients vary from $A=0.00794$ and $B=20.12$ for a 1:1 slope to $A=0.025$ and $B=65.2$ for a 1:5 slope. Recommended values of the roughness coefficient vary from $r = 1.0$ for smooth impermeable slopes, $r = 0.85-0.9$ for turf, $r = 0.8$ for one layer of stone rubble on an impermeable base and $r = 0.5-0.6$ for two or more layers of rubble.

Further work on bermed sea walls (Owen, 1984) showed that equations (1) - (3) could also be applied to these type of structures by modifying the empirical coefficients A and B . This work illustrated that, in general, the most effective berm for reducing overtopping is located at or close to still water level.

Wave basin tests using long crested waves (Owen, 1984) indicated that overtopping can increase for angles of approach up to 30° off normal with the

worst overtopping occurring at about 15° off normal. Under short crested seas (CIRIA, 1991) the overtopping discharge remains roughly constant for wave directions between 0° and 30° off normal before tailing off at larger angles.

Allsop and Bradbury (1988) completed model tests in which measurements were made of the overtopping discharge for vertically faced crown walls mounted on top of rock revetments or breakwaters. A change to the relationship given by Owen (1984) was suggested with the introduction of a new dimensionless freeboard parameter, F_* , defined as follows:-

$$F_* = R_c^2 / (H_s^2 g T_m) \quad (4)$$

The equation connecting the dimensionless discharge and freeboard was also modified:-

$$Q_* = A F_*^B \quad (5)$$

where A and B are coefficients dependent upon the geometry of the structure cross-section.

3 Recurved Walls

In many urban areas the traditional embankment type sea wall frequently incorporates a wave return wall at its crest. This wall can be located either at the top of the seaward slope, or else it can be sited a few metres back allowing the crest berm to be used as a promenade. A series of physical model tests were subsequently undertaken to measure the overtopping discharges of a range of recurved wave return walls for different sea wall slopes, water levels and wave conditions (Owen and Steele, 1991).

The model tests were carried out in a wave flume at a nominal geometric scale of 1:15. Smooth impermeable sea wall slopes of 1:2 and 1:4 were tested under a range of wave and water level conditions but always with a constant sea steepness (based on the mean deep water wave length) of $s=0.045$. Although wave return walls with a very wide range of profiles have been constructed at different locations around the UK coastline, only the basic profile originally suggested by Berkeley - Thorne and Roberts (1981) was used in this study. However, the distance between the top of the seaward slope and the foot of the wave recurve was varied throughout testing. Figure 1 illustrates the general configuration of the model tests.

Two options were investigated as a means of defining the effectiveness of wave return walls. The two alternative definitions were:

- the ratio of the measured overtopping discharge to the discharge which

would have occurred if the return wall had been removed, and the seaward slope had been extended up to the same elevation as the top of the return wall.

- the ratio of the measured overtopping discharge to the discharge which would have occurred if the return wall had been absent.

The second definition was the most appropriate as it appeared to be a much more direct indicator of the performance of the return wall. During the course of the analysis it became clear that one factor governing the effectiveness of the return wall was the height of the wall relative to its position above the still water line. Accordingly the dimensionless height of the wave return wall was defined as:-

$$W_* = W_h/R_c \quad (6)$$

where W_h is the height of the wave return wall from its base to its top and R_c is the freeboard between the top of the seaward slope (which is at an identical elevation to the base of the return wall) and the still water line.

Using the above definition of the effectiveness of the wave return wall, it is necessary to know the overtopping discharge which would have resulted during the tests if the wave return wall had been absent, for identical wave conditions, water levels and sea wall geometry. Measurements of these discharges were not made specifically for this study but used the results of the earlier research programme (Owen, 1984).

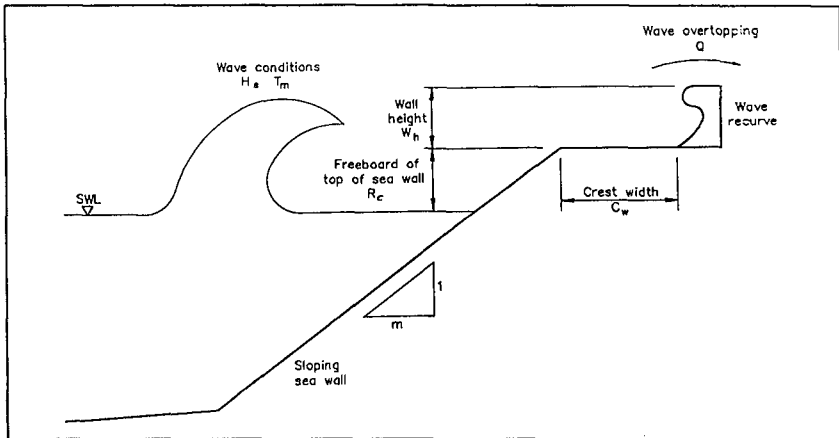


Figure 1 Definition of parameters, recurve wall

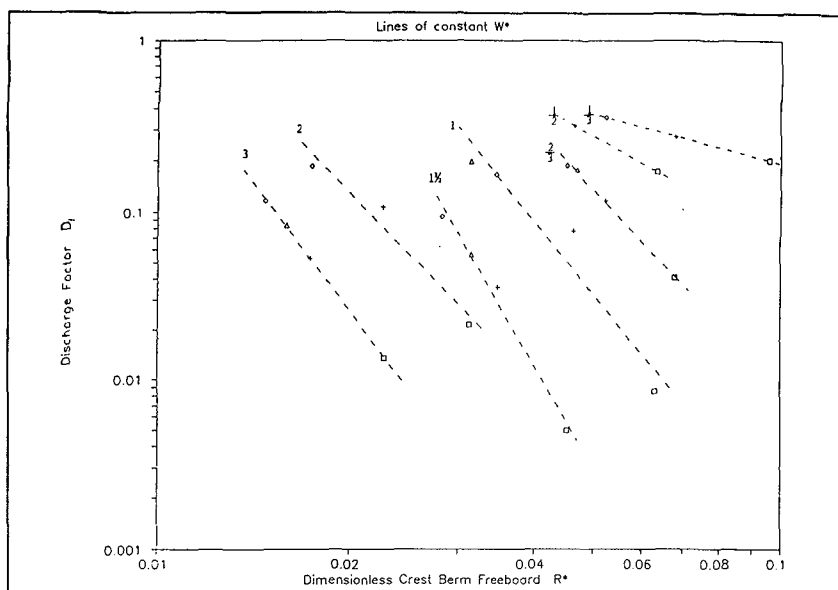


Figure 2 Recurve wall results, 1:2 slope, 0m crest width

For each test in the present study, the overtopping discharge to be expected without the wave return wall was calculated using equations (1) - (3). The measured discharge overtopping the wave return wall, expressed in dimensionless terms as Q_{*w} , could then be compared with the dimensionless discharge at the crest of the sea wall, Q_* , (ie the recurve has been removed) to give the discharge factor, D_f . Thus:-

$$Q_{*w} = Q_w / (T_m g H_s) \quad (7)$$

$$D_f \approx Q_{*w} / Q_* \quad (8)$$

where Q_w is the mean discharge overtopping the wave recurve.

In selecting a method of presenting the data, consideration was given to the way in which a designer could use the information. Figure 2 shows the form of presentation which was finally selected, in this case for a sea wall with a 1:2 slope and with the wave return wall placed directly at the top of the slope (ie the crest width, $C_w = 0$). In this graph the abscissa is the dimensionless crest berm freeboard, R_* , as defined in equation (2), which can be calculated from the actual freeboard and the wave height and period. Each line on the graph represents a constant value of the dimensionless wave return wall height, W_* , which can be

determined from the wall height and the actual freeboard. Knowing the values of R_* and W_* allows a discharge factor D_f to be established from Figure 2. Use of equation (3) to calculate the dimensionless discharge at the crest of the sea wall, Q_* , then enables Q_{*w} to be determined from equation (8). The mean discharge overtopping the wave recurve, Q_w , may be determined from equation (7).

The method outlined above allows the overtopping discharge of a sea wall with a recurve wall to be estimated provided that the crest width and sea wall slope are equal to one of those combinations tested. However, a single design graph would be preferable, together with some means of estimating the overtopping discharge for conditions not specifically tested. Given the scatter of results in Figure 2, and the fact that fewer than the ideal number of tests were completed for each structure cross-section, it was decided to investigate whether a standard slope could be fitted to all lines having the same dimensionless wall height.

All of the individual graphs were overlain and, using the 1:2 slope with a zero crest width as the baseline, the data sets displaced in the horizontal direction. With the appropriate displacements the individual W_* data sets tended to collapse on to a straight line. An iterative procedure was used to find the displacements which, using the method of least squares to find the line of best fit, gave the highest overall coefficient of correlation for all the data sets. This overall coefficient of correlation was taken as the average of all the coefficients of correlation of all the data sets for different dimensionless wall heights.

The result of the analysis described above was a single design graph which is illustrated in Figure 3. In this figure the abscissa is the dimensionless adjusted crest berm freeboard, X_* , which is defined as follows:-

$$X_* = R_* A_f \quad (9)$$

where A_f is an adjustment factor dependent on the structure cross-section.

Hence R_* may be calculated from equation (2), whilst typical adjustment factors are given in Table 1. Thus a discharge factor can be obtained from Figure 3 and the overtopping discharge calculated as before (see equations (1), (7), and (8)).

The results of the model tests showed that recurved wave return walls can have a very dramatic effect on the overtopping discharges of sea walls. For some test conditions the discharge was reduced by almost three orders of magnitude compared to the expected situation without the return wall. Although some reduction would be obtained by simply raising the basic sea wall by the same amount as the height of the return wall, calculations indicated that only one order of magnitude reduction in overtopping could be expected. This point is well illustrated in Figure 4. For either a 1:2 or a 1:4 sea wall, the figure shows a plot of the overtopping discharge against the total height of the sea wall, for a

Sea wall slope	Crest width, C_w	$W_r/R_c \geq 2/3$	$W_r/R_c \leq 1/2$
		Adjustment factor, A_f	Adjustment factor, A_f
1:2	0	1.00	1.00
1:2	4	1.07	1.34
1:2	8	1.10	1.38
1:4	0	1.27	1.27
1:4	4	1.22	1.53
1:4	8	1.33	1.67

Table 1 Adjustment factors

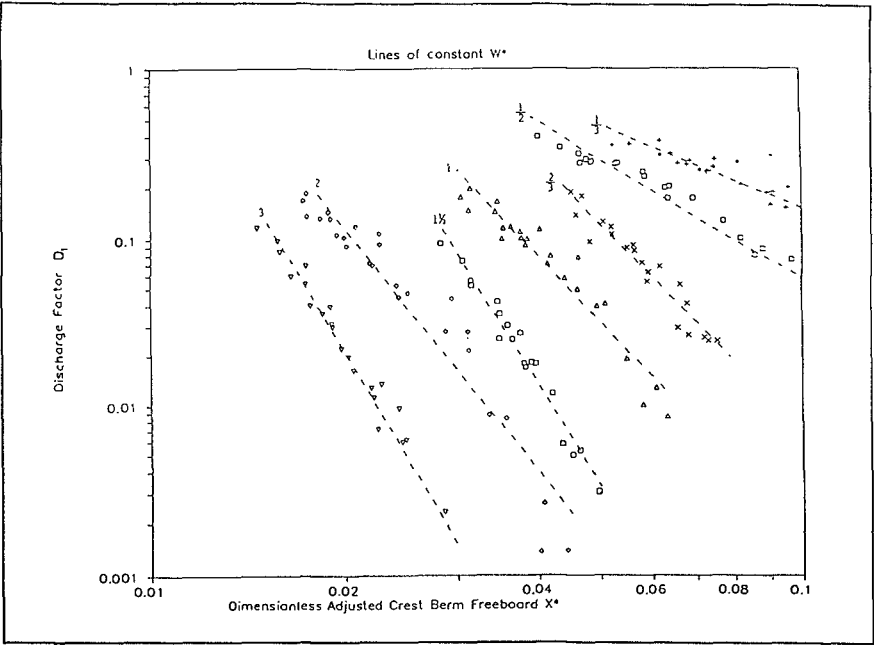


Figure 3 Design graph, recurve wall

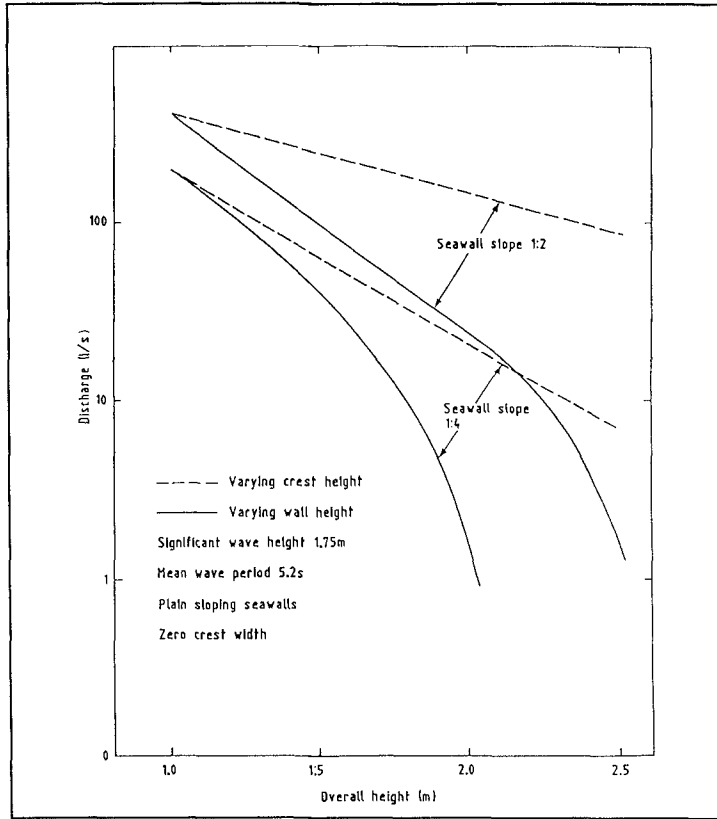


Figure 4 Effect of raising the crest

particular wave condition and water level. Starting from a crest elevation of 1.0m with no wave return wall, the solid lines show the reduction in discharge which is obtained by adding a return wall of gradually increasing height. The broken line shows the reduction obtained by raising the crest height, without any wave return. For a given total height of sea wall, the incorporation of a wave return wall greatly reduces the overtopping discharge compared to simply raising the crest.

The analysis outlined above was also applied to the data obtained by Allsop and Bradbury (1988) who measured the discharges overtopping a rock armoured slope topped with a crown wall. In all cases these tests were carried out for a seaward slope of 1:2 and a crest width equivalent to two rock diameters. The crown wall had a significantly less efficient profile than that proposed by Berkeley-Thorn and Roberts (1981) as used in the other recurved wall tests.

The results, which are illustrated in Figure 5, showed considerably more scatter than for the smooth impermeable slopes. This was thought to be due to the different degrees of energy absorption on the slope and of drainage into the crest for different wave and water level conditions. Also shown in Figure 5 are the discharge factors for the equivalent smooth impermeable slope with a crest width of $C_w = 4.0$ metres. The discharge factors for a return wall mounted on top of a rock slope are very much better (lower) than for a smooth slope despite the less effective recurve. The reduction in discharge factor must therefore be due to the effects of the rock slope.

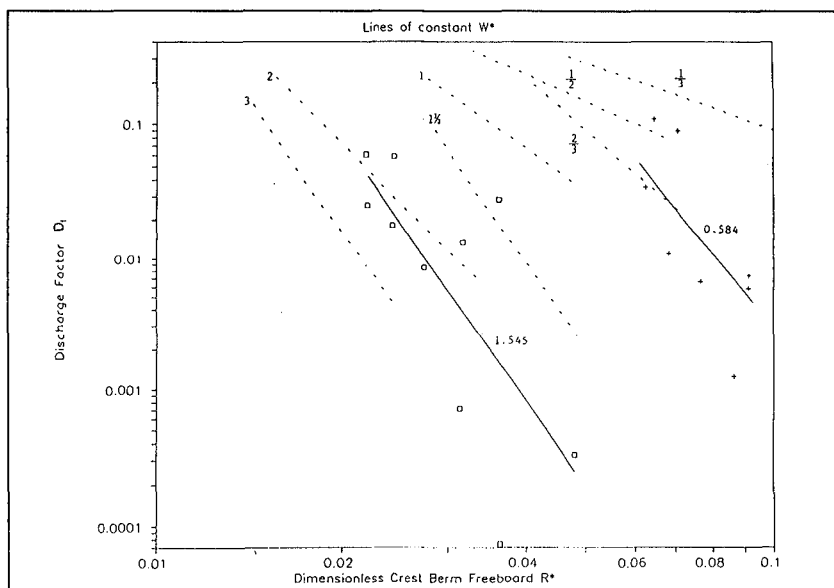


Figure 5 Rock slope compared with 1:2 slope with recurve, 4m crest width

The probable explanation for the lower discharge factors on the rock slope is as follows. As the wave runs up the slope and on to the crest, its forward progress is arrested by the return wall, increasing the depth of water on the crest. For an impermeable slope the remainder of the wave run-up to some extent rides over this cushion of water and a fraction overtops the wave return wall. On a permeable slope water reaching the return wall is able to drain away through the armour layer thereby limiting the depth of water at the crest. Hence wave run-up finds it more difficult to overtop a wave return wall on permeable than an impermeable slope.

Recent work has concentrated on assessing the efficiency of recurve walls under

oblique wave attack. Physical model tests have been completed, at a scale of 1:25, using both short and long crested random waves with angles of wave attack ranging from 15° to 45° off normal. The preliminary analysis of the oblique data appears to indicate that discharge factors are larger, and hence the recurve is not as efficient, when compared to normal wave attack.

4 Vertical walls

Vertical or near vertical sea walls are common in urban areas and are often sited behind shingle or sandy beaches. Work detailing the overtopping performance of vertical walls had previously been completed by Goda (1975). Goda investigated approach slopes of 1:10 and 1:30 and offshore sea steepness of $s_{om} = 0.012, 0.017$ and 0.036. These conditions were considered to be unrepresentative of conditions around the UK coastline where the steepness of storm waves is greater and the bathymetry of approach is generally shallower.

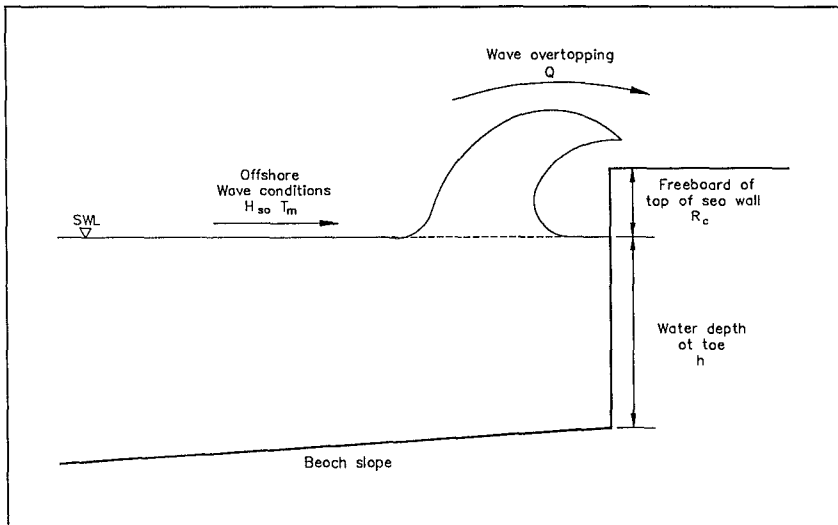


Figure 6 Definition of parameters, vertical wall

A series of physical model tests was therefore undertaken with the aim of confirming and extending the work of Goda so that it was more applicable to UK coasts (Herbert, 1993). Consequently three approach bathymetries of 1:10, 1:30 and 1:100 were used in the model with offshore sea steepness ranging from 0.017 - 0.060. Other parameters that were varied included the offshore wave height, the water depth at the toe of the sea wall and the freeboard of the wall. These parameters, which are illustrated in Figure 6, were varied to ensure that the model tests were completed in the zone of interest identified by Goda.

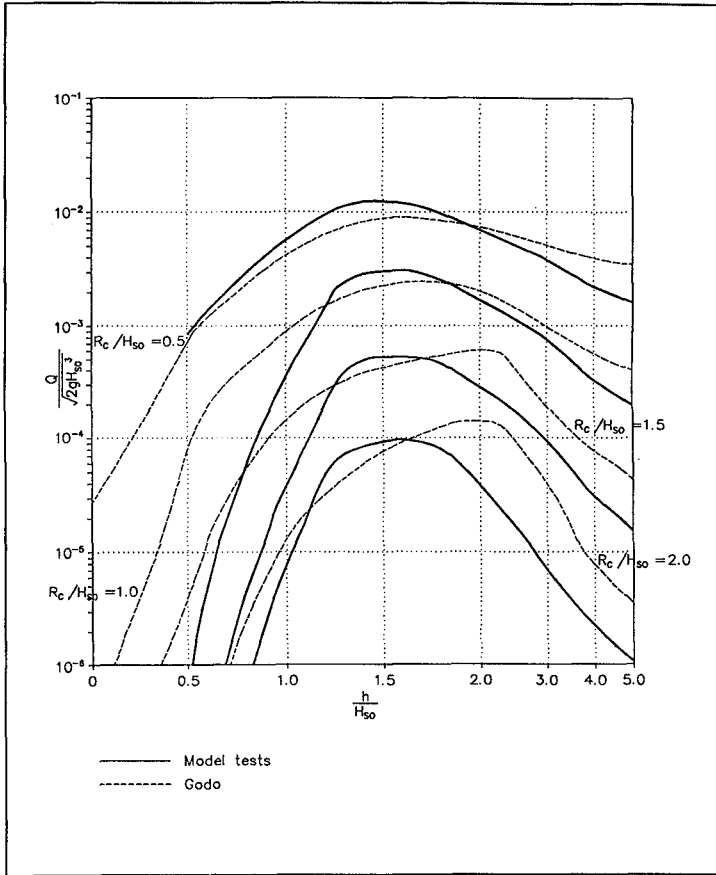


Figure 7 Vertical wall discharges, 1:30 slope, $s_{om} = 0.036$

The model data gave good agreement with the work of Goda. This is illustrated in Figure 7 where a dimensionless discharge, $Q^\#$, is plotted against a dimensionless water depth, h/H_{so} , where:-

$$Q^\# = Q / (2gH_{so}^3)^{1/2} \quad (10)$$

H_{so} is the offshore significant wave height
and h is the water depth at the toe of the structure.

Lines of constant values of the dimensionless freeboard, R_c/H_{so} , are illustrated on the graph where R_c is the height of the crest above still water level.

For a given dimensionless freeboard, maximum overtopping discharges occurred when $1.4 < h/H_{s0} < 2.0$. These conditions correspond to waves breaking immediately seaward of the structure toe. The breaking waves often pass directly over the crest of the seawall. For $h/H_{s0} < 1.4$ waves break before they reach the vertical wall. A considerable amount of energy is dissipated during breaking and hence overtopping is reduced. If $h/H_{s0} \ll 1.4$ the waves break farther offshore and overtopping is further reduced. Conversely, for $h/H_{s0} > 2.0$, waves are unbroken when they reach the structure and this also leads to a reduction in peak overtopping discharge. As waves travel into shallower water they steepen before breaking. When the water depth at the structure is large relative to H_{s0} little or no shoaling occurs and overtopping is commensurately lower. For unbroken waves, as the h/H_{s0} ratio increases the level of shoaling, and hence overtopping, is reduced. Eventually the effect of water depth at the structure and bed slope will become insignificant and overtopping will be a function of wave height and freeboard only. Therefore, for large values of h/H_{s0} , overtopping discharges will approach a constant value for a given dimensionless freeboard, R_c/H_{s0} .

5 Further work

A fieldwork deployment, designed to measure overtopping discharges at prototype sites, has been completed on the North Wales coast. This work is being analysed and compared with results from physical model tests. Furthermore measured discharges are being compared with present guidance on allowable overtopping with particular reference to vehicular and pedestrian safety.

Work is presently being undertaken to assess the performance of a wide range of sea wall cross-sections under oblique wave attack. The structure cross-sections, which are being physical model tested using both long and short crested waves, include simply sloping sea walls with and without recurved walls, bermed sea walls and vertical walls.

A design manual is being planned which will describe and detail the overtopping performance of sea walls and the standards to which they should be designed. It is anticipated that draft copies of this manual will be available in the latter part of 1995.

6 Acknowledgements

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