Effectiveness of recurved wave return walls

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Contract

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Summary

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Model studies have been carried out at a scale of 1:15 to measure the overtopping discharges for recurved wave return walls located on top of smooth, plain sloping seawalls. The measured discharges were compared with the expected values if the wave return walls had been absent, to derive a discharge factor representing the effectiveness of the return wall. These expected discharges were estimated from dimensionless expressions derived from many tests reported elsewhere.

The model tests were for a fixed recurve profile, and for seawall slopes of 1:2 and 1:4. A range of return wall heights, seawall elevations, return wall positions, and wave conditions was examined. Based on analysis of the results a design method has been proposed to enable the overtopping discharge for wave return walls to be estimated.

This study forms part of a continuing programme of research into the behaviour of seawalls, being carried out at HR Wallingford with support from the Ministry of Agriculture, Fisheries and Food under Commission 14A, Marine Flood Protection, Sea Defence Structures.

For further information about this study please contact the authors or Dr D M Herbert in the Coastal Group of HR Wallingford.

Notation

A, B	Empirical coefficients defining the discharge curve for a given Seawall profile
A_f	Adjustment factor
C_w	Width of crest berm (m)
D_f	Discharge reduction factor - Q^*_w/Q^*_b
g	Gravitational acceleration
H_s	Significant waveheight
QBAR	Mean overtopping discharge l/s/m or $m^3/s/m$
Q*	Dimensionless discharge QBAR/(Tg H_s)
QSOBAR	Standard deviation of discharge
Q^*_b	Dimensionless discharge at the base of the recurve
Q^*_w	Dimensionless discharge over the return wall
R_c	Freeboard at the top of the seaward slope
R_{cw}	Freeboard at the top of the wave return wall
R^*_c	Dimensionless slope freeboard - $R_c/(T_m (g H_s)^{½})$
R^*_w	Dimensionless wall freeboard - $R_{cw}/(T_m (g H_s)^{½})$
S	Sea steepness. In deep water $S = 2\pi H_s/g T_m^2$
SWL	Still water level
T_m	Mean wave period
W_h	Height of wave return wall from base to top
W*	Dimensionless wall height W_h/R_c
X*	Adjusted dimensionless freeboard R^*_c X A_f

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1 Introduction

By far the most common type of seawall in the UK, in terms of the length of coastline protected, is the 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 particularly frequent in rural areas, where the seaward face is often protected either by grass or pitched stone. In urban areas however the seawall 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.

In the late 1970's the then Hydraulics Research Station carried out an extensive research programme to determine the overtopping discharges for embankment type seawalls, culminating in the production of design guidelines and software for the prediction of overtopping (References 1, 2, 3 and 4). However virtually no information has been available to quantify the effectiveness of wave return walls in reducing overtopping discharge. As part of Hydraulics Research's continued interest in the design of seawalls, model tests have now been carried out to measure the overtopping discharges of a range of recurved wave return walls, for different seawall slopes, water levels, and wave conditions. This report describes the tests carried out (Section 2), the measurements made (Section 3), the analysis methods employed (Section 4), and the results obtained (Section 5). Finally the results are used to derive a method for estimating the effectiveness of recurved wave return walls during the design of seawalls (Section 6). Section 7 summarises the main conclusions of the study, and makes recommendations for the design of seawalls incorporating wave return walls.

2 Test variables

For the most part, the test conditions used in this present study have been based on those used for the earlier studies on embankment type seawalls (Reference 1), and on a parallel research programme to measure wave run-up and overtopping on seawalls with rough and/or porous seaward faces. This has enabled direct comparison of the results obtained with wave return walls with those obtained separately for flat-crested seawalls.

2.1 Return wall profile

Wave return walls with a very wide range of profiles have been constructed at different locations around the UK coastline. For this study, only the basic profile originally suggested by Berkley-Thorne and Roberts (Reference 5) and cited by Owen (Reference 3) has been used. This basic form is shown in Figure 1 which also includes some typical dimensions. The major feature of this profile is the very shallow angle (above the horizontal) at which the returning wave exits from the top of the recurved wall. This means that the returning wave is much less susceptible to being carried over the seawall by strong onshore winds, in contrast with a near vertical wave return wall. During this study, return wall heights have been used which represent the most likely range in practice.

2.2 Return wall position

At some locations, the wave return wall is positioned directly at the top of the seaward slope of the seawall, with the foot of the recurve joined tangentially to the slope. However in many coastal resorts the return wall is a few metres back from the top of the seaward slope: in this situation the crest berm is used as a promenade during calm weather. For this study the distance between the top of the seaward slope and the foot of the return wall was set at either 0, 4 or 8 metres. Figure 2 shows the general configuration for the model tests.

2.3 Seaward slope

The seaward slope of the main seawall which forms the base for the return wall was set at either 1:2 or 1:4. During the earlier tests on embankment type seawalls slopes of 1:1, 1:2 and 1:4 were tested. However the overtopping discharges for 1:1 seawalls were found to be very similar to those for 1:2 slopes, and the 1:1 slope was therefore omitted from these tests.

2.4 Berm geometry

Throughout these tests the seaward face of the seawall was a plain slope without any berm between the toe and the crest.

2.5 Crest elevation

For each of the seawall slopes tested, crest elevations of 0.5, 1.0 and 1.5 metres above still water level (SWL) were tested. In the model this was in fact accomplished by changing the water level by the required amount.

2.6 Wave conditions

For each combination of seawall and wave return wall, up to 5 wave heights were tested, giving significant heights at the structures of 1.25, 1.75, 2.25, 2.5 and 2.75m respectively. For all wave conditions a constant sea steepness of 0.045 was used (based on the mean deepwater wave length $gT_m^2/2\pi$).

3 *Test measurements*

3.1 Model description

The model tests were carried out at a scale of 1:15 in a random wave flume measuring 50m long with a nominal working depth of 0.61 metres. The overall width of the flume is 1.22m, which was divided into two channels. The working channel was 0.75m wide, and contained the seawall structure to be tested. The working channel was separated from the second channel by a perforated wall, with the porosity increasing towards the wave generator. By this means wave reflections from the seawall during the tests were dissipated in the second channel before reaching the wave generator. All the tests were carried out under deepwater wave conditions, with a horizontal bed extending from the wave generation section to the model seawall (see Appendix B).

The seawall and the wave return wall were both constructed mainly in wood with their painted surfaces giving a smooth finish

3.2 Wave measurements

Random waves were generated by a wedge-type wave paddle driven by a double-acting hydraulic ram, and controlled by micro-computer. Using software developed at HR, this system is capable of producing random waves with any desired energy spectrum and for a wide range of sequence lengths, but with repeatable sequences to allow the performance of different structures to be compared under identical wave conditions without statistical uncertainty. For this study the JONSWAP form of the wave energy spectrum was used for all tests, and a very long sequence length was employed (typically 3000 waves). The wave conditions during the tests were measured by twin wire wave probes located in the second channel of the flume, well away from the wave generator and a few metres off a shallow sloping shingle beach. At this location the measured waves were free of any reflection effects. The wave probes were connected to a micro-computer, and the signals were processed to give significant wave height and mean zero crossing wave period. During the initial calibration of the model the signals were also processed to give the wave energy spectrum for comparison with the required JONSWAP spectrum.

3.3 Overtopping measurements

For each test condition, five overtopping measurements were taken to enable the mean and the standard deviation of discharge to be calculated. Each measurement consisted of collecting all the water which overtopped the seawall during a period of 100 waves (defined as 100 times the nominal mean wave period). The resulting depth of water in the collecting tanks was measured, and using previously derived calibration data the total volume of water was calculated. Further details of these overtopping measurements are given in Appendix A.

4 Method of analysis

4.1 Dimensionless freeboard

The freeboard of a seawall is the difference between the crest elevation and the still water line. For seawalls with a return wall some confusion can arise over the definition of the "crest", especially if the return wall is set back some distance from the top of the seaward slope of the seawall. In this study two definitions were used for freeboard

- R_c, the freeboard at the top of the seaward slope,
- R_{cw}, the freeboard at the top of the wave return wall.

From all the previous research at HR on a wide range of seawalls it has been found useful to express the freeboard in dimensionless terms, defined as

$$R_{*c} = R_c/T_m\,(g\,H_s)^{1/2}$$

and $R_{*w} = R_{cw}/T_m\,(g\,H_s)^{1/2}$

where T_m and H_s are the measured mean zero crossing wave period and the measured significant wave height respectively. The physical significance of this dimensionless grouping can perhaps be appreciated better by noting that in deep water an identical definition is

$$R_{*c} = (R_c/H_s) \times (2\pi/S)^{1/2}$$

where S is the sea steepness.

4.2 Dimensionless discharge

From the test measurements, each overtopping discharge was calculated by dividing the volume of water collected by the actual duration of the measurement (nominally 100 x T_m). Each measured value therefore represented the average over 100 waves. Further to this, each measurement was taken 5 times : from these measurements the mean overtopping discharge, QBAR, and the standard deviation, QSDBAR, were calculated, both expressed in terms of cubic metres per second per metre length of seawall (prototype units).

In similar fashion to the freeboard, a dimensionless discharge can be defined as

$$Q_{*w} = QBAR / T_m \, g \, H_s$$

where Q_{*w} is the dimensionless discharge overtopping the wave return wall. All measured overtopping discharge results were converted to dimensionless values using this definition.

4.3 Dimensionless wall height

During the course of the analysis of the results it became clear that one factor governing the effectiveness of the wave return wall was the height of the wall relative to its position above the still water line. Accordingly the dimensionless height of the return wall was defined as

$$W_* = W_h/R_c$$

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 was at an identical elevation to the base of the return wall) and the still water line.

4.4 Return wall effectiveness

There are many possible ways of defining the effectiveness of wave return walls. Two options would be

- 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. This was the definition used by Allsop and Bradbury (Reference 6).

- the ratio of the measured overtopping discharge to the discharge which would have occurred if the return wall had been absent. In most cases this is equivalent to the ratio of the discharge which overtops the return wall to the discharge which arrives at its base.

For the present study this second definition has been used, since it seems a more direct indicator of the performance of the return wall, and also hopefully it should be much less dependent on the geometry of the seawall on which it is based.

4.5 Base overtopping discharge

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 level and seawall geometry. Measurements of these discharges were not made specifically for this study, but extensive measurements under similar conditions had been made during the earlier research programme (Reference 1), and these measurements had also been repeated and extended during a research programme running in parallel with this study, to determine overtopping discharges for seawalls with rough and/or porous seaward slopes. From all these measurements it had been found that for a given seawall geometry the overtopping discharge could be predicted from the expression

$$Q_* = A \exp(-B R_*)$$

where A and B are dimensionless coefficients whose values depend on the seawall geometry. For plain sloping seawalls with 1:2 and 1:4 gradients, as used in this study, the coefficients have the following values

Slope	1:2	1:4
A	9.39×10^{-3}	1.16×10^{-2}
B	21.6	41.0

These values are slightly different from those published in Reference 1, having been revised to include the results from all the latest tests.

For each test in the present study, the overtopping discharge to be expected without the wave return wall was calculated from the above expression, with the appropriate values of the coefficients, and using the measured significant wave heights and mean wave period. The measured discharge overtopping the wave return wall, expressed in dimensionless terms as Q_{*w} could then be compared with this dimensionless base discharge Q_{*b} to derive the discharge reduction factor

$$D_f = Q_{*w}/Q_{*b}$$

5 Test results

5.1 Data presentation

In choosing the method of presenting the data, consideration was given to the way in which a designer could use the information to calculate the discharge overtopping a wave return wall. Figure 3 shows the form of presentation which was finally selected, in this case for a seawall with a 1:2 seaward slope, and with the wave return wall placed directly at the top of the slope (ie $C_w = 0$). In this graph the abscissa is the dimensionless crest berm freeboard R_{*c}, as defined in Section 4.1, 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_* (Section 4.3), which can be determined from the actual wall height and the actual freeboard to the top of the seaward slope of the seawall. Knowing the values of dimensionless freeboard and dimensionless wall height, the discharge factor can therefore be read from the graph. The overtopping discharge at the base of the return wall can be calculated from the freeboard and the wave conditions (Section 4.5), and the discharge overtopping the wave return wall is then obtained simply by multiplying the discharge factor.

5.2 Effects of crest elevation and wall height

Figure 3 shows that the discharge factor increases as the dimensionless crest elevation decreases: in other words the wave return is more effective when there is less water arriving at its base. When very large quantities of water arrive at the return wall it becomes "drowned", and has very little effect on the overtopping discharge. Figure 3 also demonstrates the very strong effect of the return wall height on the discharge factor, which is to be expected. At first it was expected that the best way of non-dimensionalising the wall height would be by division by the wave height. However this did not produce any consistent pattern in the results. Dividing by the crest freeboard is in fact displaying the message that return walls which are low in relation to the quantity of water which reaches them are less effective at reducing the overtopping discharge.

5.3 Effect of seawall slope

Figure 3 is for a 1:2 seawall slope, with the return wall at the top of the slope. Figures 4 and 5 show the results plotted in the same form for return walls placed 4 and 8m respectively from the top of slope. Figures 6-8 show the results for a 1:4 seawall slope for the three different crest widths tested. On each graph the lines joining results for constant dimensionless wall height have been fitted using the method of least squares.

Comparison of Figures 3 and 6 shows the effect of seawall slope. For the same dimensionless freeboard and dimensionless return wall height a return wall based on top of a 1:4 seawall appears to be more effective than one on a 1:2 seawall. For example, taking a dimensionless crest elevation of 0.04 and a dimensionless wall height of 1.0, the discharge factor for a return wall on a 1:2 slope is about 0.1: on a 1:4 slope it is about 0.025. Similar reductions in discharge factors also occur for the other crest widths. This increased effectiveness is explained in part by the fact that for a given dimensionless crest height the overtopping discharge for a 1:4 slope is less than for a 1:2 slope, and as mentioned previously a wave return wall is more effective at low discharges. This suggests that replotting Figures 3 and 6 using Q_* as the abscissa instead of R_* might collapse the 1:2 and 1:4 data onto the same line. However replotting in this fashion, while it brought the data closer together, still indicated that wave return walls are more effective when based on top of a 1:4 seawall.

5.4 Effect of crest width

Comparison of Figures 3, 4 and 5 shows the effect of moving the wave return wall back from the top of the seaward slope for a 1:2 seawall, and Figures 6, 7 and 8 show the same for a 1:4 seawall. For a 1:2 slope there is a noticeable improvement (reduction) in the discharge factor when the wave return wall is retarded by 4m, with very little further reduction at 8m. For the same dimensionless crest elevation and dimensionless wall height as exampled previously, the discharge factors are about 0.07 for both crest berm widths, compared with about 0.1 for a return wall directly at the top of the seaward slope.

For a 1:4 slope, there is much less consistency in the effects of the crest widths on the discharge factor. For low values of dimensionless wall height (W_h/R_c = 0.3 and 0.5) a return wall placed 4m back from the top of the seaward slope is more effective, but for larger values of dimensionless wall height there appears to be a slight worsening of the discharge factor. When the crest width is increased to 8m, the discharge factor improves (reduces) significantly for the lower dimensionless wall heights, and also improves noticeably for larger wall heights. This compares with the 1:2 slope tests where there was very little difference between the 4m and 8m wide crest results.

6 Design method

Figures 3 to 8 can be used directly in the design of a wave return wall, knowing the dimensions of the return wall and the wave conditions, and provided that the crest width and the seawall 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. This section of the report attempts to address these questions.

6.1 Design graph

Examination of Figures 3 to 8 showed that for a given dimensionless return wall height the slopes of the lines were almost constant irrespective of the crest width or the seawall slope. Given the scatter of the results, and the fact that fewer than the ideal number of tests were carried out, it was decided to investigate whether a standard slope could be fitted to all lines having the same dimensionless wall height. Clearly the intercepts of the lines on the axes would be different according to the crest width and seawall slope.

In concept, the method of determining the slopes and the displacements of the lines was as follows. Firstly, the results obtained for a 1:2 seaward slope were taken as the baseline conditions. The equivalent graph for the 1:4 slope was then overlaid onto the standard graph: by displacing the overlay to the right, both groups of data tended to form single groups of data for each dimensionless wall height. The method then is to move the 1:4 data by an amount x along the R_{*c} axis, and calculate the best fit line to the combined 1:2 and 1:4 data using the method of least squares. This was then repeated for displacements x +/- Δx until the highest overall coefficient of correlation was found for the data groups. This overall correlation was taken as the average of all the coefficients of correlation of all the data groups for different dimensionless wall heights. The linear displacement necessary to achieve this best fit was noted: because the x-axis is logarithmic, this displacement can be expressed as a factor to be applied to the dimensionless crest elevation to derive the adjusted dimensionless freeboard X_*,

where $X_* = R_{*c} \times A_f$

and A_f is the adjustment factor

The pairs of Figures 3 and 6, 4 and 7, 5 and 8 were each treated in this way, to give adjustment factors to combine the 1:4 slope results with the 1:2 results for crest widths of 0, 4 and 8m. A similar process was then used to combine all the 8m crest width results with the 4m results. Finally all the 4m and 8m results were adjusted to the 0m crest width results. Here however it was found that there were significant differences in the adjustment factors necessary to obtain high correlations for all the different dimensionless wall heights. Two different adjustment factors were therefore adopted dependent upon the value of the dimensionless wall height.

Using these methods, a single graph was produced, and this is shown in Figure 9. For each value of dimensionless wall height there are now approximately 20 data points, through which a straight line (using logarithmic scales) has been fitted by the method of least squares. For most wall heights there seems to be a good fit to the data, although some wall heights suggest a slight curvature, with the discharge factor reducing more rapidly for higher values of adjusted freeboard. Figure 9 should be used in conjunction with Table 1, which gives the values of adjustment factor to be used for any particular combination of seawall slope and crest width.

6.2 Example problem

Given	Seawall slope 1:4 Crest elevation 5.0m OD Crest width 8.0m Return wall height 0.8m
Find	Overtopping discharge when:- Still water level 4.2m OD Significant wave height 1.2m Mean wave period 3.64s
Solution	Dimensionless crest elevation $R_{*c} = R_c/T_m\ (gH_s)^{½} = 0.8/3.64\ (9.81 \times 1.2)^{½} = 0.064$ Dimensionless base discharge $Q_{*b} = A \exp(-B\ R_{*c})$

From Section 4.5, the values of A and B for a simple seawall with a 1:4 slope are given as 1.16×10^{-2} and 41.0 respectively.

$\therefore Q_{*b} = (1.16 \times 10^{-2}) \exp(-41.0 \times 0.064) = 8.41 \times 10^{-4}$

Hence the dimensional overtopping discharge at the base of the return wall is

$Q_b = Q_{*b}\ T_m\ g\ H_s = 8.41 \times 10^{-4} \times 3.64 \times 9.81 \times 1.2 = 0.036 m^3/s/m.run$

This is a rather high overtopping discharge (36 litres/s/m) which would probably not be tolerated if pedestrians regularly walked behind the seawall, hence the need for a wave return wall. For the return wall, the dimensionless wall height is

$W_* = W_h/R_c = 0.8/0.8 = 1.0$

With an 8m wide crest, and a 1:4 seawall slope, Table 1 shows a crest elevation adjustment factor of 1.33, for dimensionless wall heights ≥ ⅔.

Therefore the adjusted dimensionless freeboard is

$X_* = R_{*c} \times A_f = 0.064 \times 1.33 = 0.085$

From Figure 9, the discharge factor D_f for $X_* = 0.085$, $W_* = 1.0$ is read off as 3.7×10^{-3}. The actual overtopping discharge over the wave return wall is therefore

$$\begin{aligned} QBAR &= D_f \times Q_b = 3.7 \times 10^{-3} \times 0.036 \\ &= 1.33 \times 10^{-4}\ m^3/s/m.run \end{aligned}$$

which is very large reduction.

Because the location of the return wall and the slope of the seawall are standard values which were actually tested, the discharge factor could in this case have been read directly from Figure 8, using the un-adjusted dimensionless freeboard $R_{*c} = 0.064$. The slightly different value obtained ($D_f = 3.3 \times 10^{-3}$) arises from the different number of data points used in the linear regression.

For the example given, the dimensionless wall height corresponds exactly with a tested condition: some interpolation between lines will usually be necessary. In many cases it may also be necessary to extrapolate the lines to higher values of X_*: this should be done with extreme caution, although it is likely that the resulting estimate of overtopping discharge will be too high, ie conservative. It was not possible to extend the range of results in the model study because the overtopping discharges became too low to measure accurately. If accurate estimates of overtopping discharge are required for this situation, then consideration should be given to carrying out model tests for the specific seawall design, with special measures to record the very low discharges (eg collecting the overtopping water for a period of 1000 instead of 100 waves).

6.3 Application to other seawalls

The model tests described here were carried out only for simple seawalls with smooth seaward slopes of 1:2 and 1:4. Testing of additional seawalls was not possible within the research budget. Some interpolation/extrapolation will therefore be necessary for the application of the results to other seawalls. For simple seawalls with seaward slopes between 1:1 and about 1:2½, the overtopping discharges at the base of the wave return wall will be very similar for the range of dimensionless crest elevations used in the tests, and therefore it seems reasonable to use the same discharge factors as for the 1:2 slope. For slopes between about 1:2½ and 1:4 the overtopping discharge decreases almost linearly, and linear interpolation between the discharge factors for 1:2½ (taken equal to 1:2) and for 1:4 would therefore be appropriate.

None of the tests in this study involved seawalls with a berm located partway between the toe and the crest of the seaward slope. Therefore there must be some uncertainty about the way in which these results could be used for that situation. The most logical way would be to convert the bermed slope into an equivalent plain slope (which will always be flatter), which for the same wave conditions, water level and crest level would give the same overtopping discharge. The discharge factors appropriate to this equivalent plain slope would then be used for designing the wave return wall. Often however the equivalent plain slope will turn out to be flatter than 1:4, the most shallow slope which has been used in these tests.

For seawalls which are rough but impervious, the most logical method to proceed would again be to convert the rough slope into an equivalent plain smooth slope, giving the same overtopping discharge, and using the appropriate discharge factor.

From the above discussion, it will be seen that there are likely to be occasions when the only accurate method of determining the overtopping discharge for a bermed or a rough seawall will be to commission specific model studies.

6.4 Rock revetments

All the tests in this present study were for plain smooth and impervious slopes. However tests had been carried out by Allsop and Bradbury in an earlier study (Reference 6) in which measurements had been made of the overtopping discharges for crown walls mounted on top of rock revetments or breakwaters. In all cases the tests were carried out only for a seaward slope of 1:2, and most of the crown walls had a vertical faces on their seaward side. A few tests had a recurved face, albeit of different profile to the present study, illustrated in Figure 10. No tests were carried out with the crown walls completely removed.

To make use of the results obtained in that earlier study, Allsop and Bradbury's experimental equipment was reinstated for this study, and a series of measurements made for a plain rock slope only, without any crown wall present. The results obtained are plotted in dimensionless form in Figure 11. Unlike a smooth impermeable seawall it is impossible to measure overtopping discharge directly at the top of a rough porous slope. For the stability of the slope a crest width of a least two rock diameters has to be allowed, in this case equivalent to a crest width of 2.2 metres, and the overtopping discharges were therefore measured at this distance back from the top of the slope. Even so the significant scatter in the results shown in Figure 11 indicates the difficulty of measuring overtopping discharges for rock slopes, and also indicates the variability in overtopping due to the different degrees of energy absorption on the slope and of drainage into the crest for different wave conditions and water levels. The results are plotted in Figures 12 and 13, where they are compared with a 1:2 plain smooth slope for crest widths of 0 and 4 metres respectively.

Figures 12 and 13 show that the discharge factors for a return wall mounted on top of a rock slope are very much better (lower) than for a smooth slope. The recurved profile shown in Figure 10 would be expected to be less effective than that given in Figure 1, and the reduction in discharge factor must therefore be due to the effects of the rock slope. The probable explanation is as follows. As the wave runs up the slope and onto 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 the increased depth of water on the crest causes a greater head difference from the crest to the bottom of the slope, increasing the reverse drainage down through the armour layer and, to a lesser extent, the underlayer. The remainder of the wave run-up therefore finds it more difficult to overtop the wave return wall.

7 Discussion

7.1 Crest raising versus return walls

The results of the tests have shown that recurved wave return walls can have a very dramatic effect on the overtopping discharges of seawalls. For some test conditions the discharge was reduced by almost three orders of magnitude compared to the expected situation without the return wall. Of course some reduction would have been obtained simply by raising the basic seawall by the same amount as the height of the return wall. However calculations showed that, for the same tests conditions, the reduction achievable by this method is only about one order of magnitude. This point is well illustrated by the two examples shown in Figure 14 (Reference 7). For either a 1:2 or a 1:4 seawall, the figure shows a plot of the overtopping discharge against the total height of the seawall, for a 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 dotted line shows the reduction obtained by raising the crest height, without any wave return wall. For a given total height of seawall, the incorporation of a wave return wall greatly reduces the overtopping discharge compared to simply raising the crest.

7.2 Dimensionless overtopping expressions

To calculate the effectiveness of the wave return walls, the measured overtopping discharges have been compared with the expected discharges if the return wall had been absent. The estimation of these expected discharges was based on the use of the dimensionless overtopping expressions.

$$Q_* = A \exp(-BR_*)$$

where the coefficient A and B depend on the seawall geometry. In this study the values of A and B used for the smooth 1:2 and 1:4 plain slopes differ slightly from those quoted in earlier reports and software (eg References 1 and 4). This is because a large number of extra tests have been performed and these results have been combined with the earlier ones to produce revised estimates of the coefficients. Extra tests have also been carried out for many of the bermed seawalls, and all these revised values will be published in a separate report, and will no doubt be incorporated into the next version of the software when it is produced.

7.3 Recurved wall profile

Almost all the tests in this study were carried out for a fixed type of recurved wave return wall. Under conditions when the return wall is almost drowned out (ie when it is least effective) the exact shape of the recurve probably makes very little difference. Under those conditions Figure 9 could therefore probably be used whatever the design profile. However for high wave return walls on top of a seawall with large freeboard the recurved profile is very important, since it defines the trajectory of the returned water jet. The profile shown in Figure 1 is probably one of the most effective, since the water is returned seaward at a very shallow angle above the horizontal. Vertical wave return walls are probably very much less effective

Compare with Vertical wall results?

8 Conclusions

1 A series of model tests has been carried out at a scale of 1:15 to measure overtopping discharges for a standard design of recurved wave return wall, mounted on top of a plain sloping seawall. The tests covered two seawall slopes (1:2 and 1:4), and a range of seawall heights, return wall heights and positions, and wave conditions.

2 The measured overtopping discharges were compared with the expected discharges if the wave return wall had been absent, to derive a discharge factor which expressed the effectiveness of the return wall in reducing overtopping. The expected discharges were calculated using dimensionless expressions derived from many tests performed earlier.

3 The results showed that the effectiveness of a wave return wall depended strongly on its dimensionless height, and also on the dimensionless freeboard of the seawall itself. As would be expected, the lowest discharge factor was obtained when a high return wall was mounted on top of a seawall with large freeboard and flatter slope. For some of the tests the overtopping discharge was reduced by almost three orders of magnitude by the presence of the return wall (discharge factor about 1.4×10^{-3}).

4 Purely in terms of reducing the overtopping discharges, it is much more effective to add a wave return wall of given height on top of an existing embankment-type seawall than to raise the crest of the seawall by an equal amount.

5 Previously reported tests to measure overtopping discharges for crown walls on top of rock revetments or breakwaters were extended during this study to measure overtopping without any crown wall. Comparisons were made of the results for the recurved crown wall, with the results of the present study. Although the recurves had a different profile, the comparison showed that wave return walls on top of rough, porous rock revetments are even more effective at reducing overtopping discharge.

6 Based on analysis of all the results obtained for the smooth seawall tests a design method has been produced to enable engineers to estimate the overtopping discharges for any wave return wall within the range of variables tested. Some guidance has also been given on suitable methods of interpolation/extrapolation to obtain approximate overtopping discharges for other types of seawalls. However there will continue to be many seawall designs for which accurate predictions of overtopping discharge can only be obtained by specially commissioned model studies.

9 References

1. Design of seawalls allowing for wave overtopping, Hydraulics Research Station. EX 924, June 1980.

2. Overtopping of sea defences. Proc. of Conf. on Hydraulic Modelling of Civil Engineering Structures, BHRA; Coventry, 1982.

3. Hydraulic design of seawall profiles. Proc. of Conf. on Shoreline Protection. ICE, Southampton, 1982.

4. SWALLOW - a seawall design package for micro computers. Distributed by HR Wallingford.

5. Sea defence and coast protection works. R Berkley-Thorn and A C Roberts. Published by Thomas Telford Ltd, 1981.

6. Hydraulic performance of breakwater crown walls. Hydraulics Research Limited. Report No SR 146, March 1988.

7. Research on beaches and coastal structures. Proc. of Conf. in Coastal Management. ICE, Bournemouth, 1989.

Tables

Table 1 Adjustment factors

$W_h / R_c \geq 2/3$ $> 1/2$

1:2 slope ,	0m	crest	1.00
1:2 slope ,	4m	crest	1.07
1:2 slope ,	8m	crest	1.10
~~1:4 slope ,~~	~~0m~~	~~crest~~	~~1.27~~
1:4 slope ,	4m	crest	1.22
1:4 slope ,	8m	crest	1.33

$W_h / R_c \leq 1/2$ $< 2/3\,?$

1:2 slope ,	0m	crest	1.00
1:2 slope ,	4m	crest	1.34
1:2 slope ,	8m	crest	1.38
1:4 slope ,	0m	crest	1.27
1:4 slope ,	4m	crest	1.53
1:4 slope ,	8m	crest	1.67

Appendix A

Overtopping measurement

Appendix A Overtopping measurement

The method of measuring overtopping discharge was consistent for all the tests and was based on a standard procedure employed at HR. A set of five overtopping intervals are recorded where water discharging over the seawall is collected in a calibrated tank. Here a magnetostrictive float monitors the difference in water level from which the volume can be calculated. The overtopping sequences are separated by periods during which water overtopping the wall is returned to the flume. This diversion of water is controlled by a hinged, manually operated flap gate. During these intervals the water collected in the tank is allowed to settle before the higher level is recorded. If necessary the water is then pumped out and returned to the main flume. Three calibrated tanks are available for these measurements in the flume used for this study. This allows a range of volumes from 64-556 litres (model) to be recorded in any one interval. The resolution is 0.04 litres (model).

The length of each overtopping and recording interval was based on the nominal mean wave period (T_m) of the wave spectrum being used. Each overtopping interval was $100T_m$ seconds with each recording interval being $200T_m$ seconds. Each test was preceded by a $300T_m$ 'run-in' time. Five overtopping intervals were recorded allowing a mean and standard deviation of volume (and hence discharge) to be calculated. The full test sequence was thus:

$300T_m$	:	Run-in	
$100T_m$	:	overtopping	1
$200T_m$	:	recording	1
$100T_m$	:	overtopping	2
$200T_m$	:	recording	2
⋮			
$100T_m$	:	overtopping 5	
Stop			

giving $1600T_m$ seconds for the whole test. With the range of input conditions used, these tests lasted between 29 and 41 minutes.

During each overtopping interval the number of 'overtopping events' (waves) discharging over the seawall was recorded. An overtopping event was considered to be any wave which caused a sheet of water at least 75% of the width of the wall to be discharged. The number of overtopping events was later expressed as a percentage of the number incident on the structure during the overtopping interval. The number of incident waves was defined as $100T_m$ using the measured value of wave period.

The sequence length set on the wave generator (see Appendix B) for each test was considerably longer than the total recording time and throughout the test wave data was recorded using a wave counting technique. This is described in Appendix C. This data gave values of significant wave height and mean wave period which were used in the subsequent analysis of the data.

Appendix B

Physical model test facility

Appendix B Physical model test facility

All the model tests for this study were carried out in a wave flume or channel measuring 50m long by 1.22m wide by 1.1m deep and having a nominal working depth of 0.61m. The wave generator is a wedge type random wave paddle powered by a double acting electro-hydraulic ram controlled by a BBC micro-computer. This system was developed at HR from an older "hard wired" wave spectrum synthesizer. This combination of synthesizer and wave generator is capable of producing any required deep water ocean wave spectrum that can be described by 16 spectral ordinates. The BBC micro computer wave spectrum synthesizer produces a random wave spectrum by digitally filtering a white noise signal via a shift register. Varying lengths of wave sequence can be produced on this shift register which is used in conjunction with a clock pulse generator (Reference 1). This allows a repeatable pseudo-random sequence of outputs to be generated creating sequences of waves with repeat times varying from a few minutes to several tens of years depending on the test requirements.

This wave flume is divided along its length into two channels by a vertical splitter wall which increases in porosity as it approaches the generator end of the flume. This porous divide wall helps prevent the generation of cross waves as well as dissipating any energy reflected back from the structure being tested. The smaller of these two channels (0.47m wide) is of constant depth and ends in a shingle spending beach of 1:5 gradient. This channel is used to measure the "deep water" wave conditions produced by the generator. The wider channel (0.75m wide) contained the model and sea bed profile under test.

References

1 Wave spectrum synthesizers. E&ME Tech Memo 1/1972, Hydraulics Research Station, June 1972.

Appendix C

Spectral analysis and wave counting programs

Appendix C Spectral analysis and wave counting programs

The BBC micro computer wave spectrum synthesizer produces a random wave spectrum by digitally filtering a white noise signal via a shift register. Varying lengths of wave sequence can be produced on this shift register which is used in conjunction with a clock pulse generator (Reference 1). This allows a repeatable pseudo-random sequence of outputs to be generated creating sequences of waves with repeat times varying from a few minutes to several tens of years depending on the scaling parameters.

For this study two types of wave sequence were used, ie using either a short repeating sequence of about 10 mins duration or one of between 4 and 5 hours. The former was used for the wave flume calibration checks whilst the long sequences were used for wave overtopping tests.

Two types of wave analysis program were also used, one for each type of wave condition. For the short tests a spectral analysis was used where data recording takes place over one complete wave generation sequence thus eliminating any statistical uncertainty in the results. The water level at the twin wire wave probe (Reference 2) is recorded by the mini-computer at every clock pulse of the synthesizer, typically every 0.1-0.2 seconds. A maximum of 4096 data points can be collected from up to 16 probes at one time using this program. The analogue output of the wave probe, representing a displacement relative to a static water level, is first converted to a digital form by an A-D convertor and then to an elevation in prototype metres via the model scale. Hence at the end of sampling a series of water level elevations are known for every clock pulse, ie up to 4096 points. This program then uses a Fast Fourier Transform technique (Reference 3) to convert the time base data into the frequency domain and then splits the data into individual sine waves to extract the energy content of each frequency component. From this data the energy/frequency spectrum can be set up from which values of significant wave height, H_{mo}, and average wave period, T_{ij}, can be estimated using various moments of the spectrum.

The second type of analysis is a wave counting program and is used during the long sequence tests where, as on this study, recording may last up to 50 mins. Here an initial 'mean value of water level' is calculated by sampling the water surface elevation several hundred times during the first few waves of the test, eg a value of 1000 points is not unusual. This value of mean water level is regularly updated throughout the run which then continues with the sampling of waves for analysis where measurements of surface elevation are made relative to the mean value of the water level. Although the sampling rate of the wave probe is about the same as that of the spectral analysis program, only about 5 points are used to define each 'wave', where a wave is defined as lasting between two successive down crossings of the mean water level. All these values of elevation are squared and summed throughout the recording of each 'batch' or group of waves, the number of which is specified by the user. Typically 5 or 6 batches each of 200 or 300 waves would be used. At the end of each batch 'wave heights', calculated from the sum of the maximum departure above and below the mean in each wave, are sorted into descending order from which statistical values of $H_{1/3}$, HMAX, H10 etc can be found. The total number of points recorded in each batch is divided by the number of waves and combined with the sampling rate to give the mean wave period TBAR. Results for this batch are then output and the data discarded before moving onto the next batch.

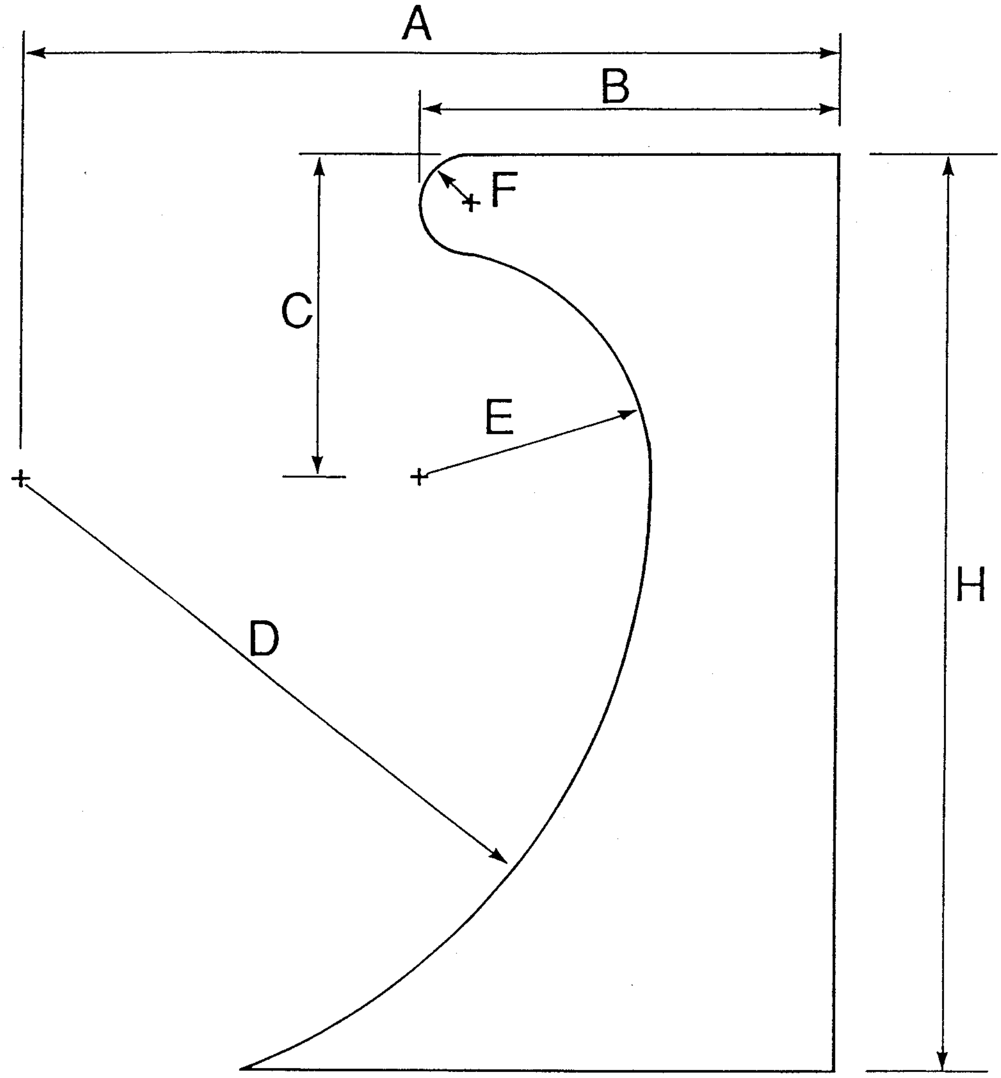
At the end of sampling the arithmetic mean and standard deviation of $H_{1/3}$ and TBAR over all the batches is calculated and printed out. It is these two values which are then used to represent the wave conditions of the test. The value of $H_{1/3}$ is defined as being the average height of the one third highest waves and is generally quite close to the spectral analysis equivalent of H_{mo}. Similarly TBAR and T_{ij} are also considered to be comparable. This data is followed by two histograms based on wave height giving wave height class and the number of waves in that class. One histogram is accumulated throughout the batch and its totals are added into the other histogram which is accumulated over all batches. Since they are too lengthy to be output during sampling only the histogram for the final batch is output at the end. All the above information applies to each of the wave probes being monitored during the test.

References

1 Wave spectrum synthesizers. E&ME Tech Memo 1/1972, Hydraulics Research Station, June 1972.

2 Twin wire wave probe modules. Tech Memo 3/1974, Hydraulics Research Station, October 1974.

3 The fast Fourier transform with applications to spectral and cross spectral analysis. Hydraulics Research Station, Int Report 100, December 1972.



H	0.61	0.91	1.22	1.52	1.83
A	0.53	0.79	1.05	1.33	1.63
B	0.38	0.50	0.61	0.72	0.84
C	0.33	0.42	0.52	0.60	0.67
D	0.31	0.52	0.75	0.99	1.24
E	0.15	0.23	0.30	0.38	0.46
F	0.10	0.10	0.11	0.11	0.11

(metres)

H	33	66.7	100
A	30.5	56.7	87.3
B	22.7	35.3	47.7
C	19	30.7	40
D	16.3	38.7	64.7
E	8	16.3	25
F	6.7	7	7.3

After Berkley Thorn & Roberts.
Model (mm)

Figure 1 Basic form of recurved wall profile.

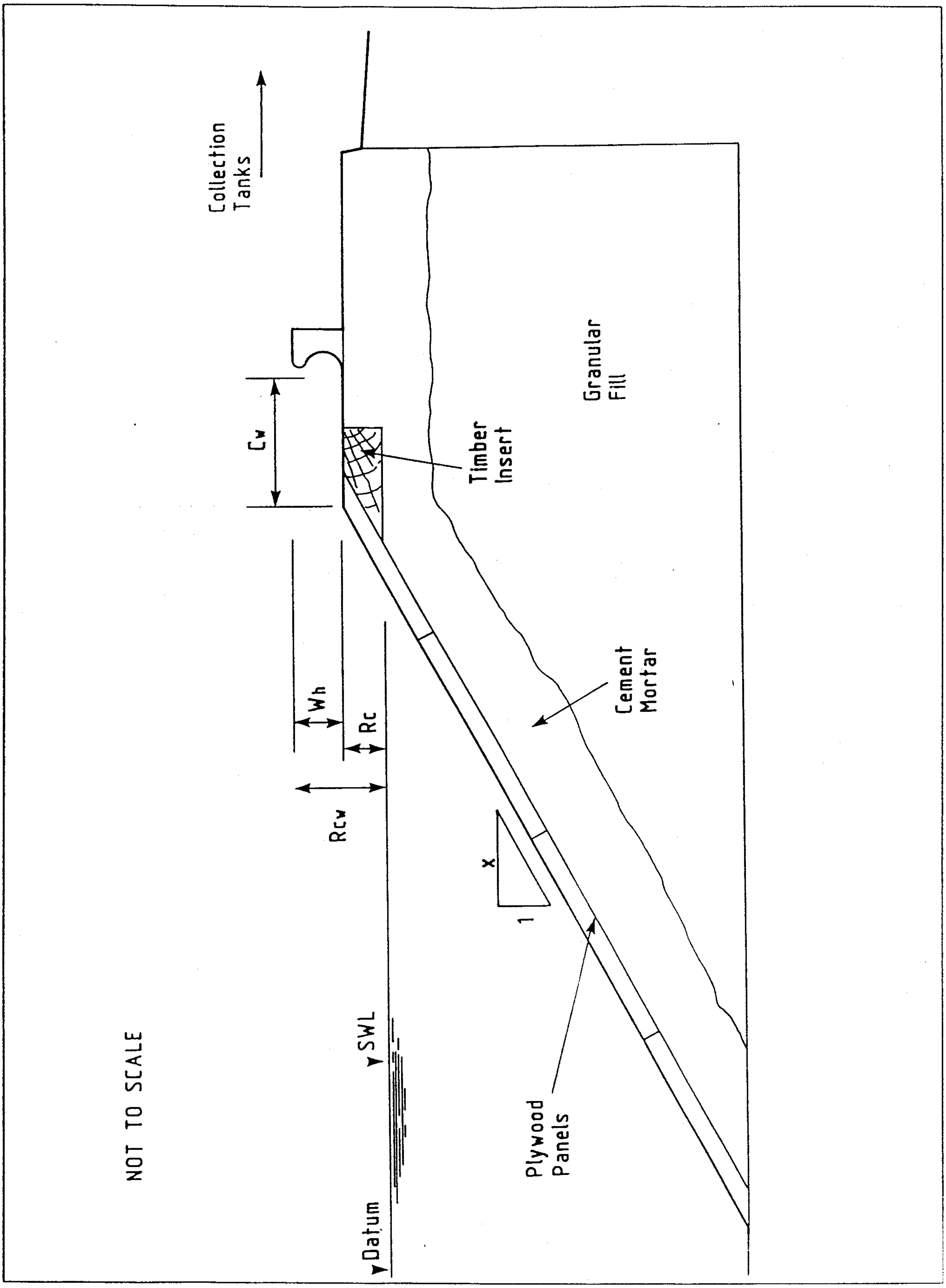


Figure 2 Configuration for model tests.

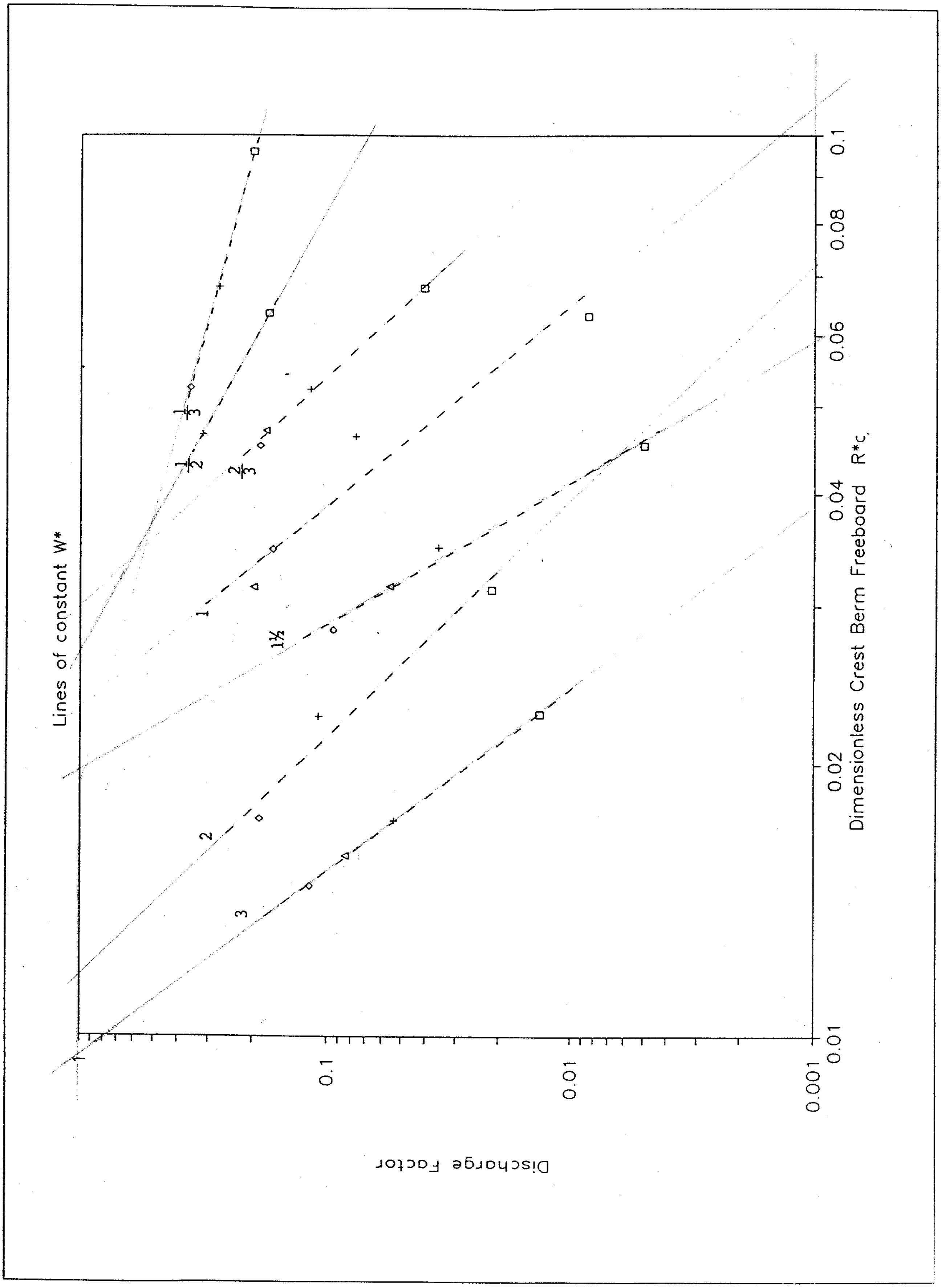


Figure 3 Results for 1:2 gradient. Crest width 0m.

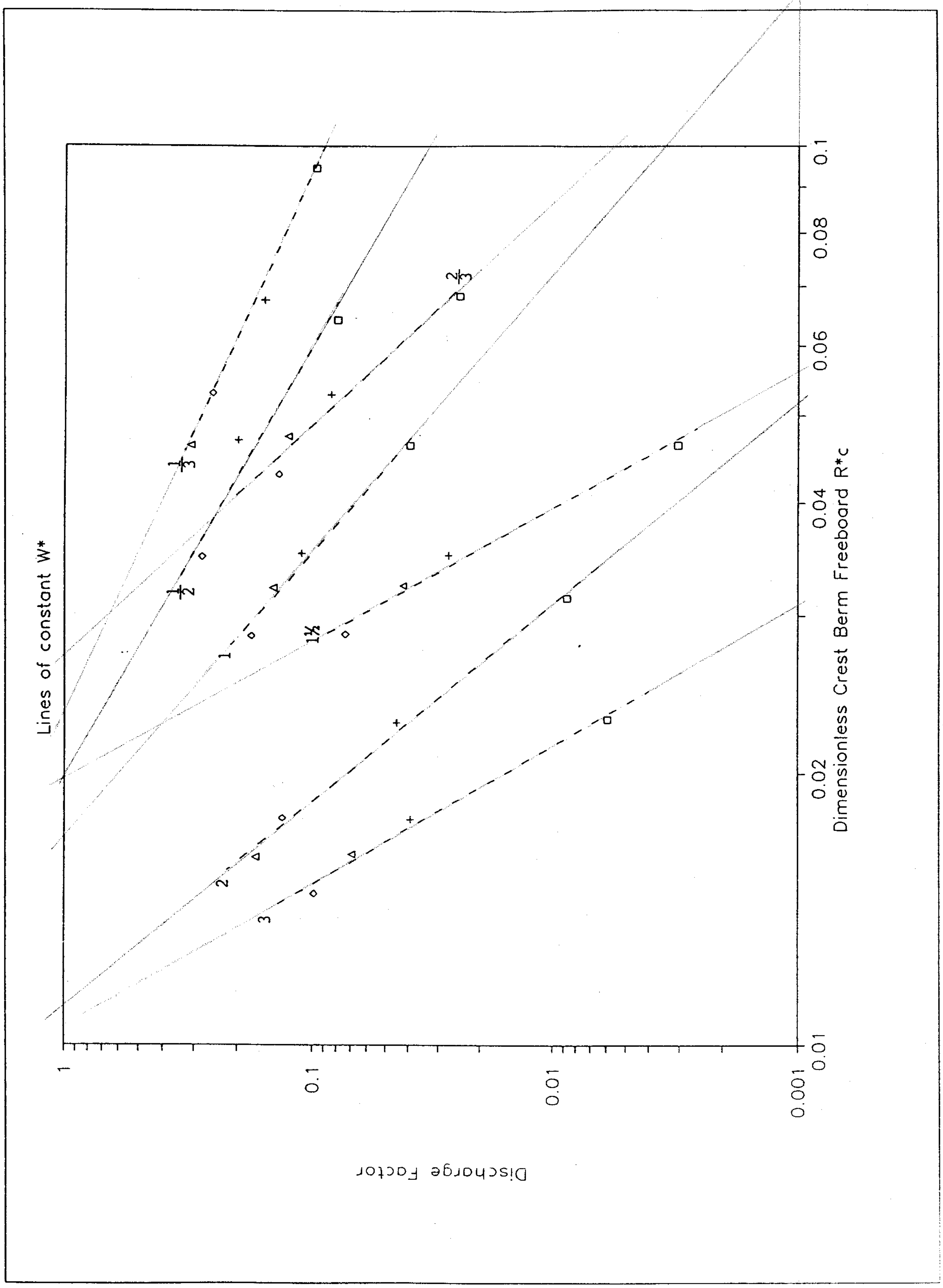


Figure 4 Results for 1:2 gradient.
Crest width 4m.

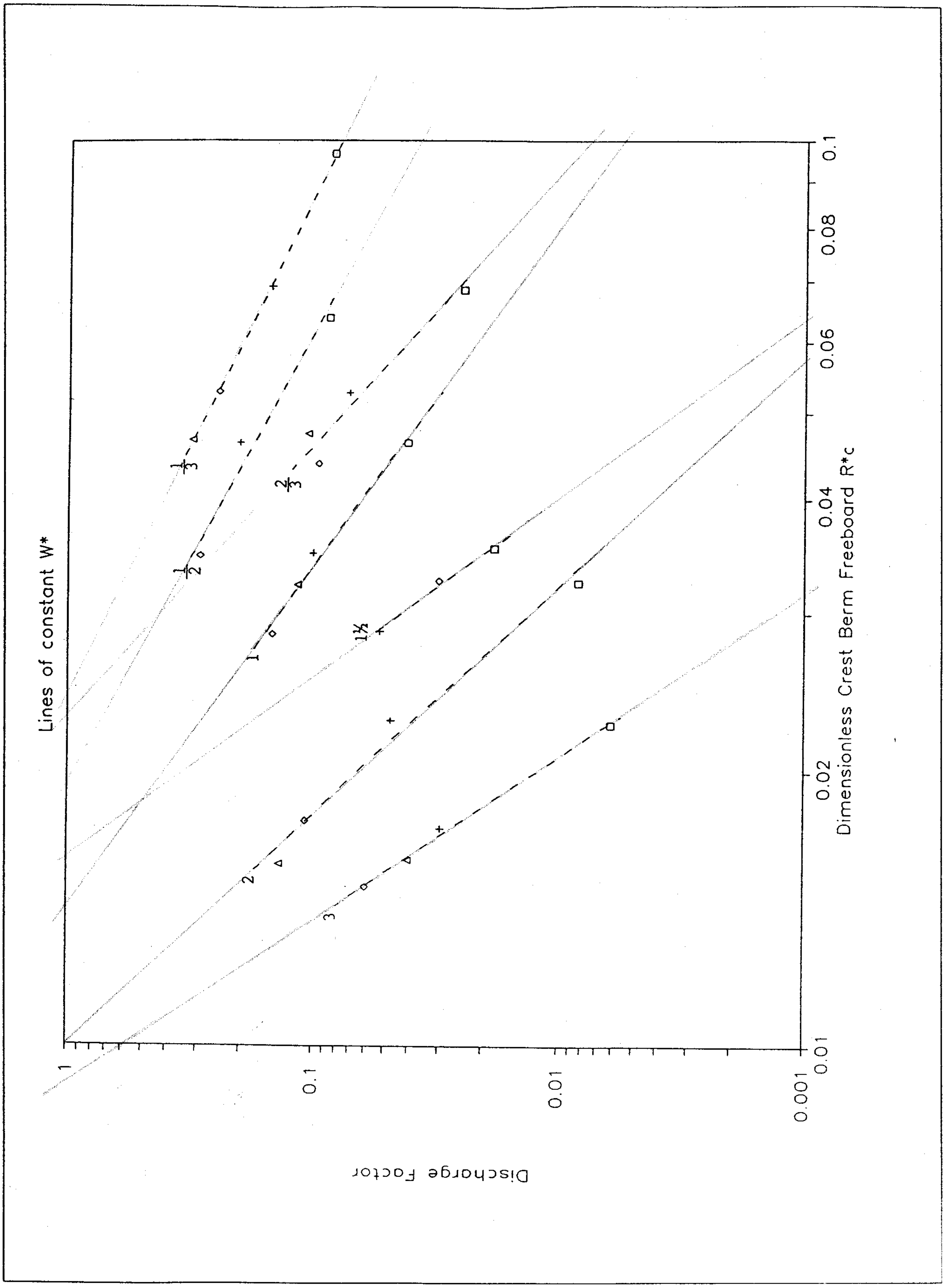


Figure 5 Results for 1:2 gradient. Crest width 8m.

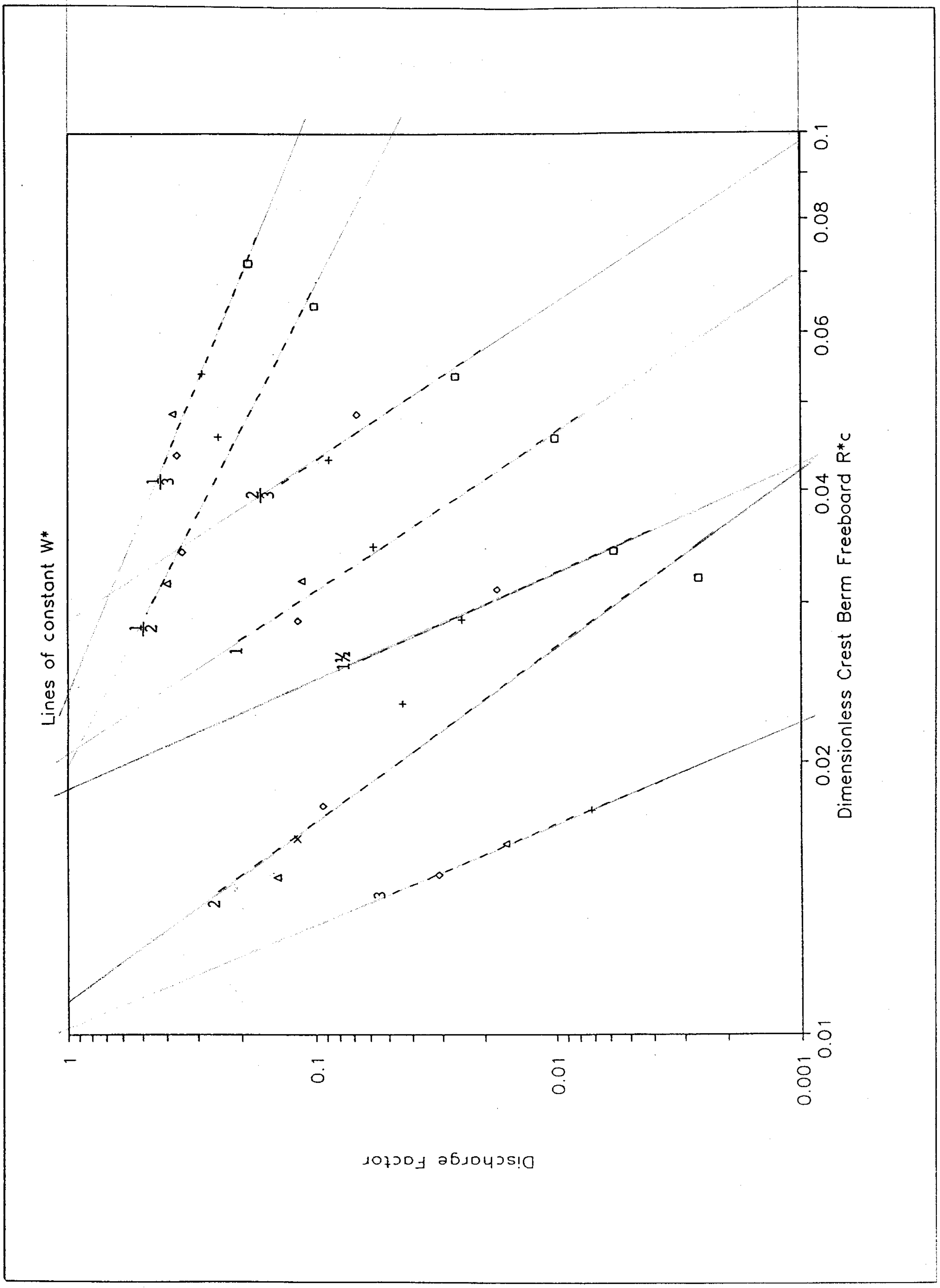


**Figure 6 Results for 1:4 gradient.
Crest width 0m.**

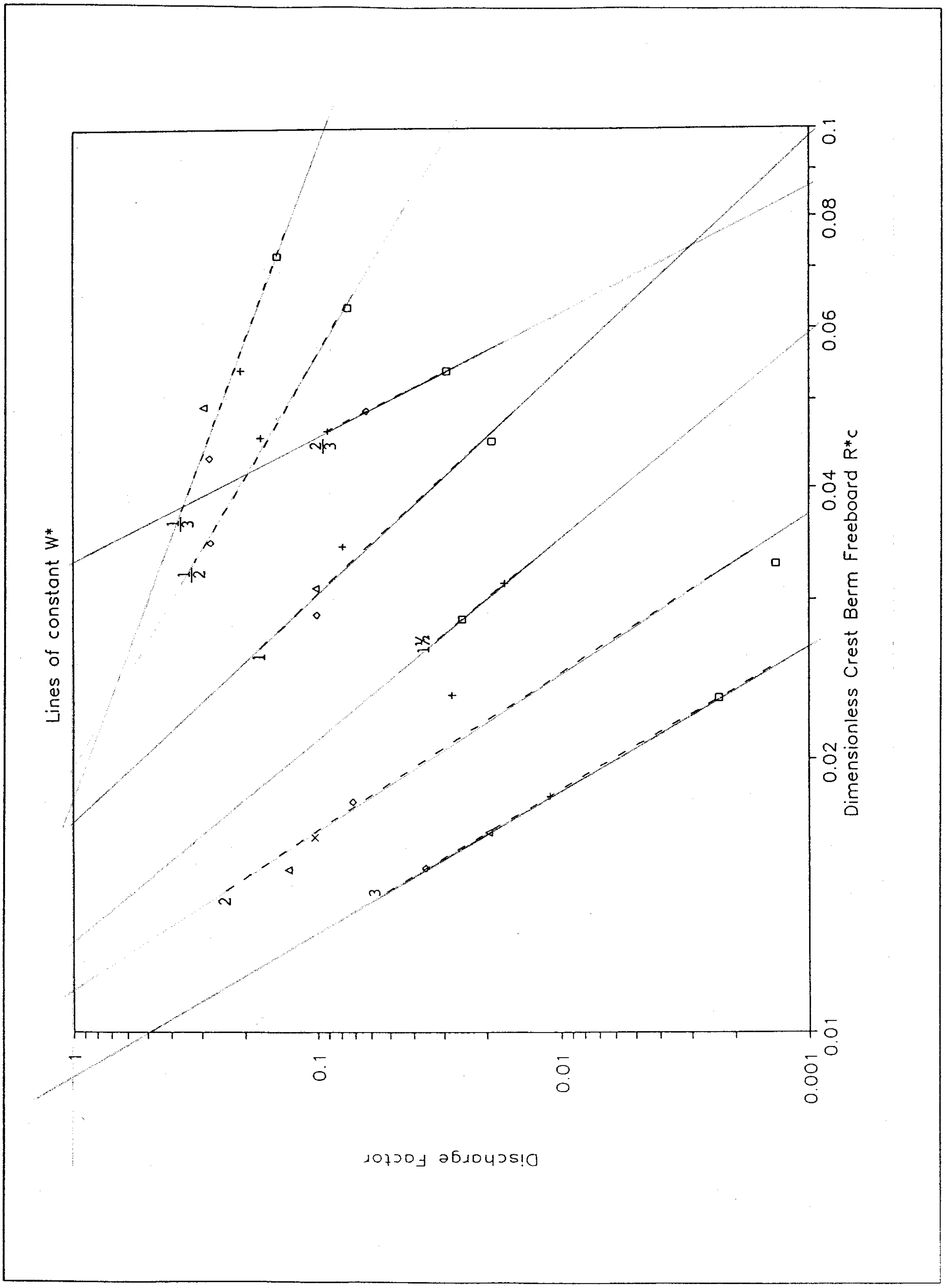


Figure 7 Results for 1:4 gradient. Crest width 4m.

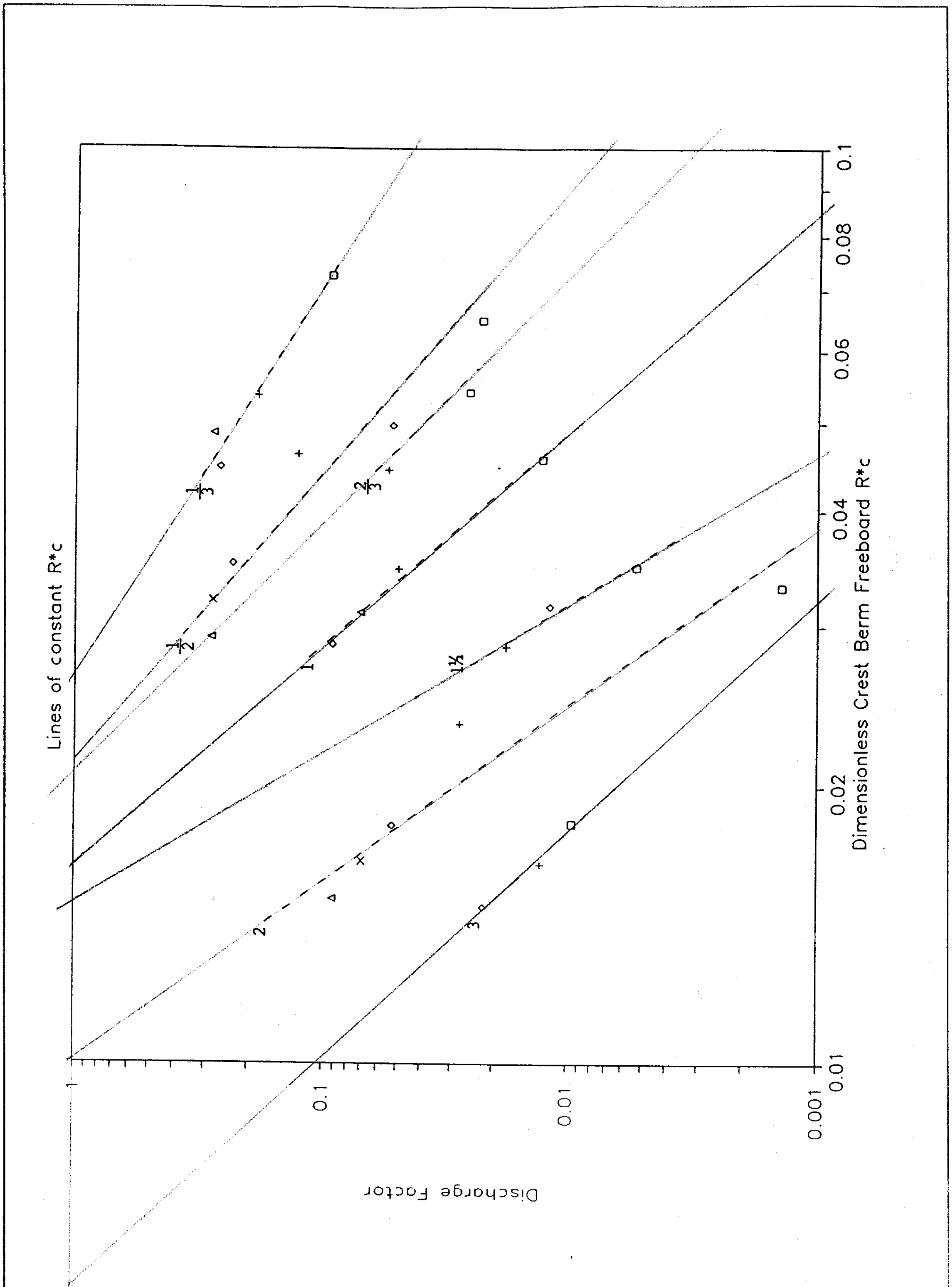


Figure 8 Results for 1:4 gradient.
Crest width 8m.

E A Report on
Overlapping uses
different curve than
Shown here for $W_*=2$

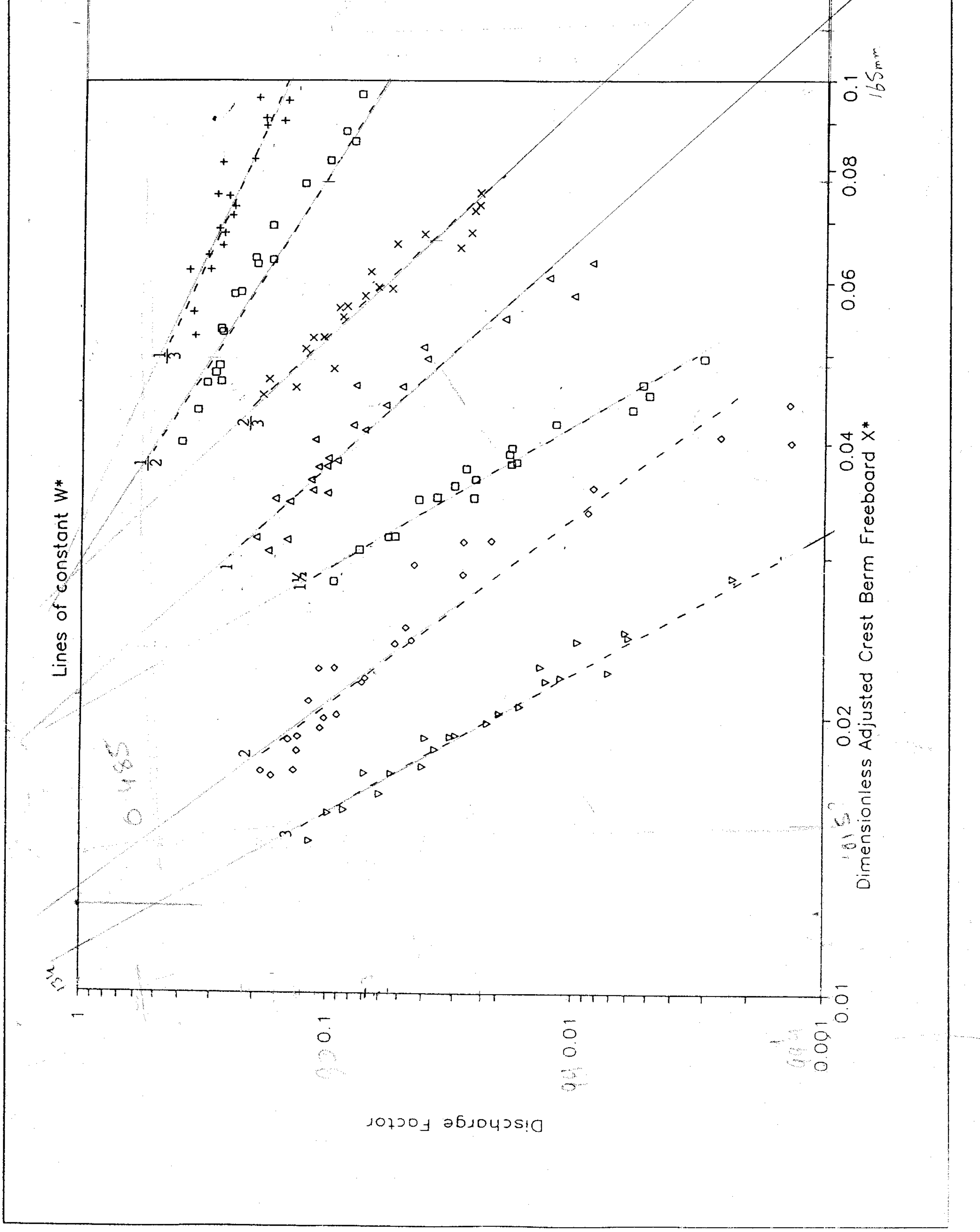


Figure 9 Final design graph.

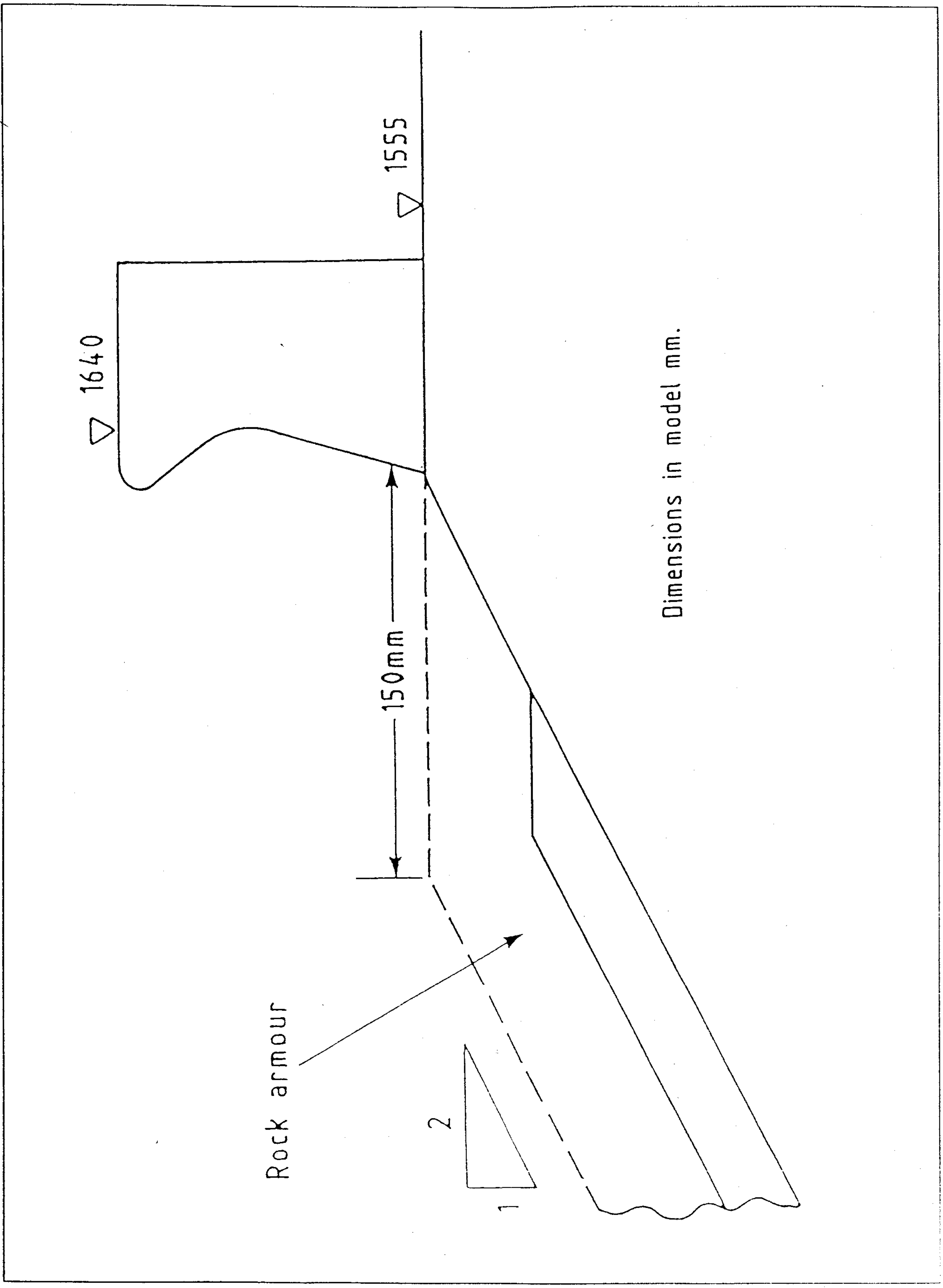


Figure 10 Rock revetment return wall.

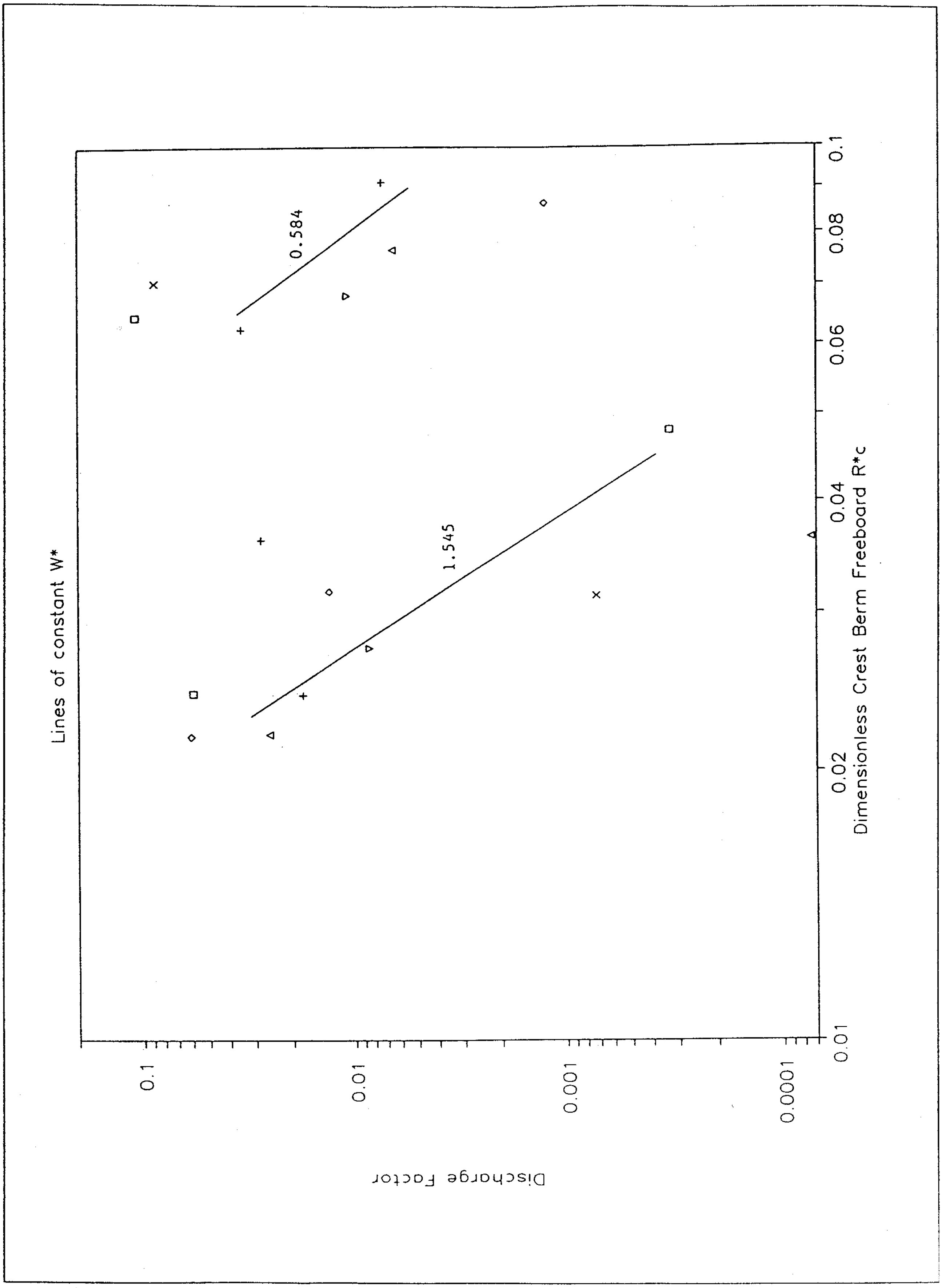


Figure 11 Rock slope overtopping curves.

EA Report has
a set of curves
for this type of
wall.

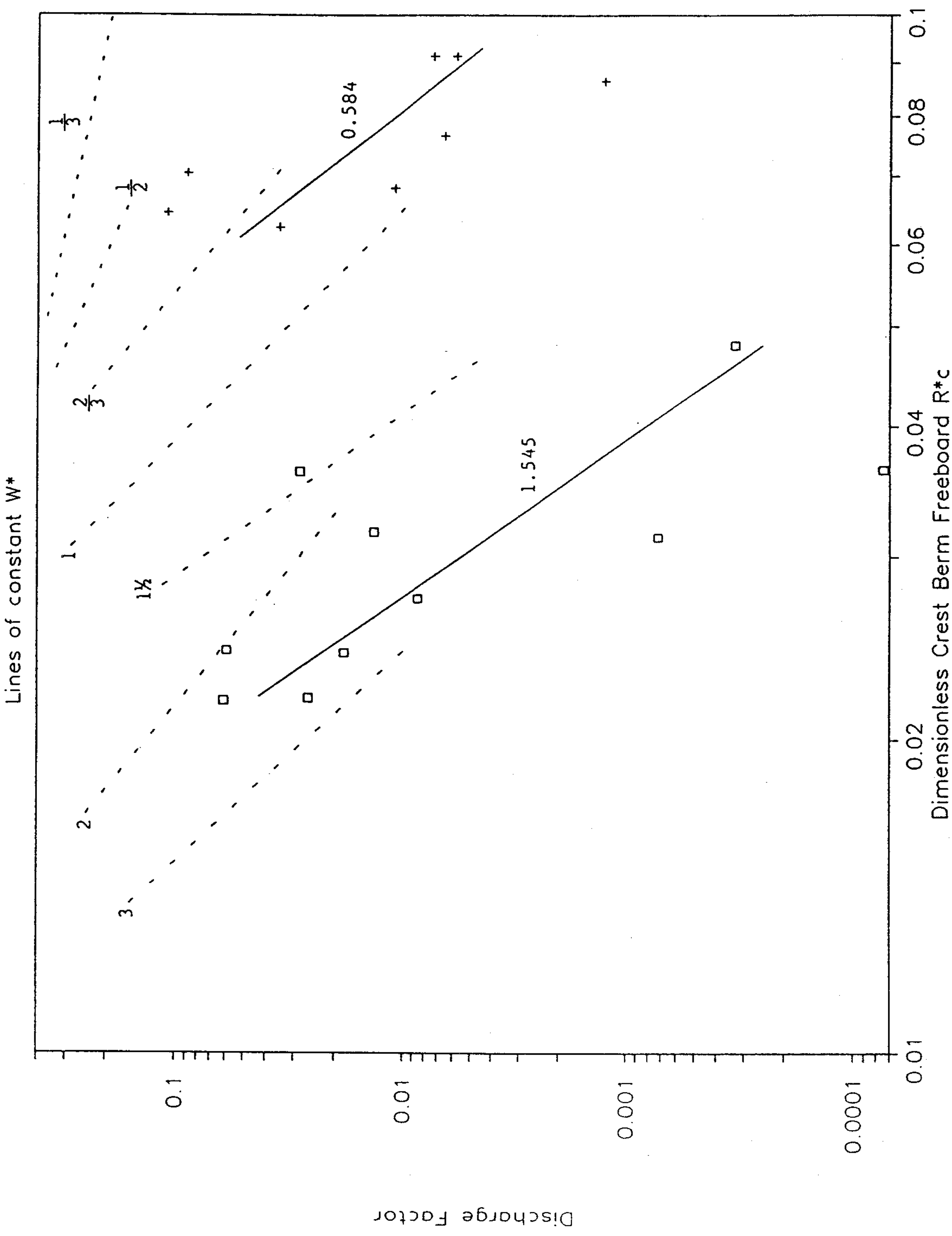


Figure 12 Rock slope compared with 1:2, 0m berm.

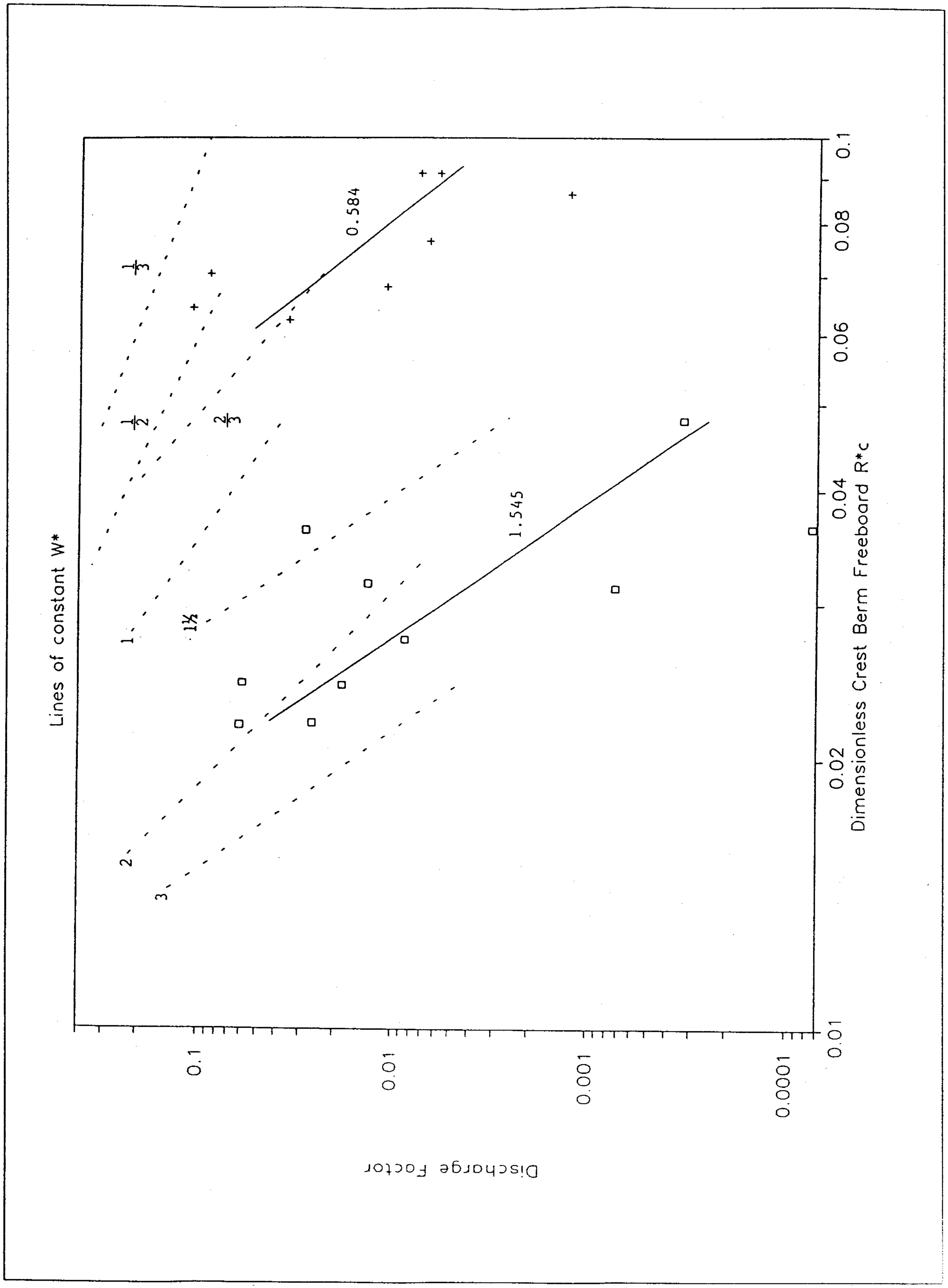


Figure 13 Rock slope compared with 1:2, 4m berm.

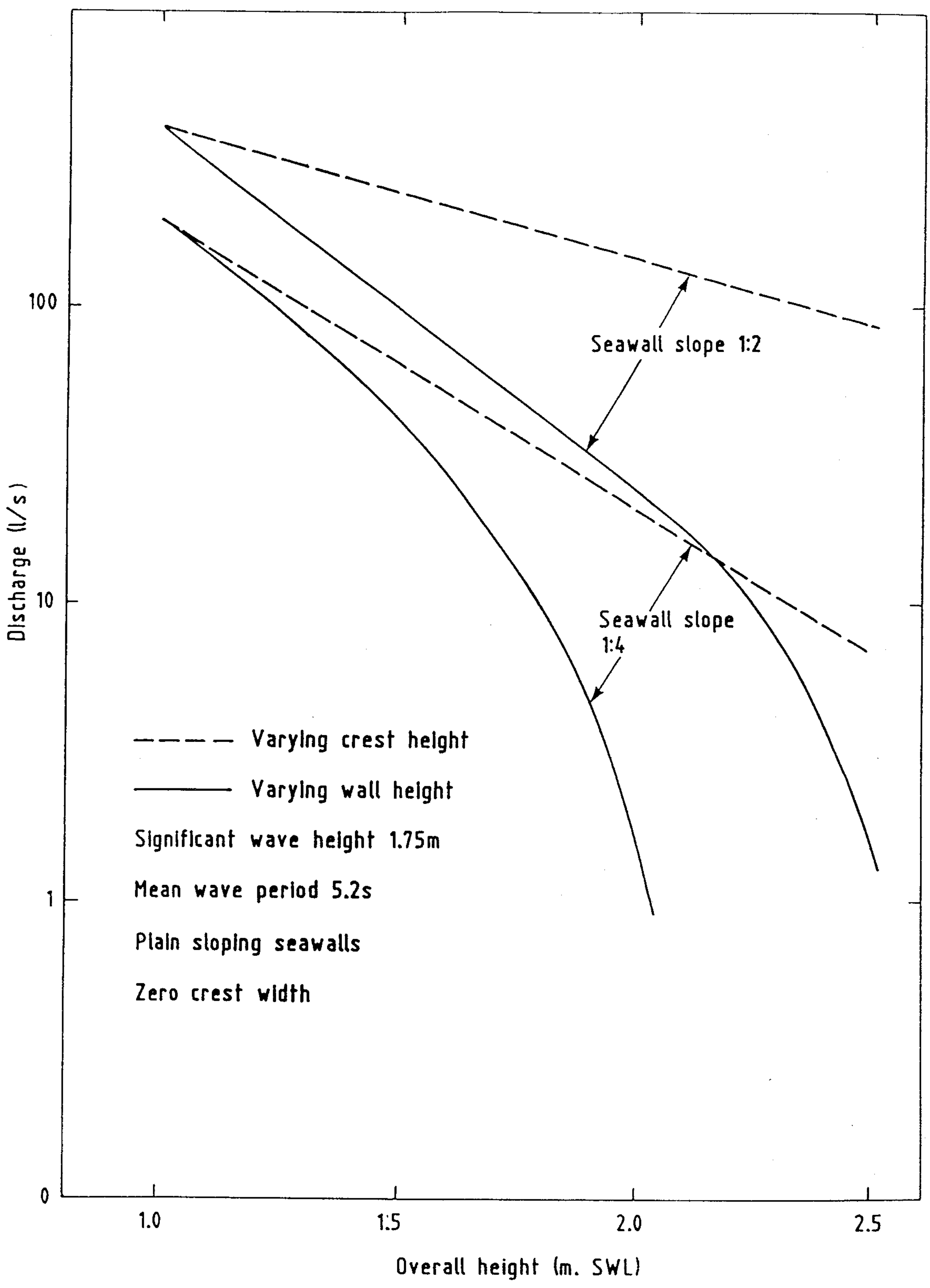


Figure 14 Effect of raising the crest.