CHAPTER 135

The Response of Gravel Beaches in the Presence of Control Structures

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Abstract

Much of the research on beach control structure concentrates on sand beaches. Relatively little work has been done on gravel beaches though they are important features along temperate and sub-arctic coastlines, particularly in the UK. Until recently attempts to manage these beaches and to control erosion have relied on traditional approaches such as timber groynes, designed by past experience and engineering judgement. To improve this situation HR Wallingford have undertaken a programme of flume and wave basin physical model studies followed by the development of numerical models and design guidelines.

The most recent phase of this programme has been an investigation of beach response to detached breakwaters. This work was undertaken at a scale of 1:50 in a mobile bed, random wave basin. The breakwaters varied in length, crest elevation and distance offshore. The sea conditions varied in wave period and water level. Measurements were taken of wave heights around the structure, beach planshape development and longshore transport rates. For each breakwater configuration and sea condition an efficiency value has been derived which relates the longshore transport rate in the presence of the structure to the potential open beach rate. These efficiency values can be used to tune breakwater dimensions to the actual long term drift rates of a specific beach, thereby providing a stable beach.

This paper briefly reviews some of the previously published work, then discusses the major findings of the recent model tests and suggests methods for applying the results. The results are, at present, only applicable to a limited range of sea and beach conditions, but it is anticipated that the approach will be developed for general application. The findings are also related to a recently completed detached breakwaters and beach recharge scheme on the UK south coast.

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Introduction

Detached breakwaters, unlike groynes, have not been used for beach control in the UK until quite recently. Existing design guidelines are based on experience in other areas of the world, with the most recent publication being the CERC Technical Report "Engineering Design Guidance for Detached Breakwater as Shoreline Stabilization Structures ". The majority of research and experience available relates to sand beaches in microtidal environments (range <2m) with shore normal wave attack, and are therefore not directly applicable to many UK situations. In addition, much of the research concentrates on determining the shape and extent of the beach development in the lee of the structures; this approach only considers the symptoms and not the causes, as it does not directly consider the sediment transport regime at the design site.

A number of morphodynamic models are available which go some way towards predicting sand beach response. Unfortunately these models are not applicable to the shingle beaches found along much of the UK coastline. To improve this situation HR Wallingford undertook an ongoing research programme, utilizing physical models, with the aim of developing design tools and guidelines.

Powell (1990) investigated the response of shingle beaches to random waves in a flume at a scale of 1:17. The physical model results were used to develop a parametric model which predicts beach response to wave and water level conditions. This work was extended to a large scale (1:17) wave basin study in which the hydrographic conditions included oblique long crested random waves (Coates and Lowe, 1993). Modifications to the original parametric model were derived which accounted for beach profile response under oblique waves. In addition, the cross-shore distribution of longshore transport along an open shingle beach was derived; this distribution was used by Brampton and Goldberg (1991) in conjunction with Powell's parametric model to improve an earlier one-line beach plan shape model (Brampton and Motyka, 1985).

Brampton and Goldberg's model was able to predict beach plan shape development in the presence of groynes, but the authors recognised that the model over-simplified a number of aspects relating to beach response. This response was investigated by further physical modelling at a scale of 1:17 and, more recently, in a different facility at a scale of 1:50 (Coates and Lowe, 1994; Coates, 1994a; Coates, 1994b). Some of the results of the larger scale model have been incorporated into a new mathematical model (Huang, 1993) which includes an improved method of predicting profile development within a groyne field as well as transport over and around groynes. The smaller scale study investigated beach response within a multiple groyne system and it is this work that is described in this paper. The approach adopted was to investigate sediment transport, rather than morphological development, in the lee of single breakwater, followed by a preliminary investigation of beach response to pairs of breakwaters.

Model studies

The test programme was conducted at HR Wallingford in a 23m by 24m wave basin with a maximum working depth of 0.4m. The model was designed at an undistorted scale of 1:50, according to Froude's relationships. The basin was equipped with:

- a 15m long, mobile wave paddle capable of generating waves at up to 45° relative to the beach;
- nine wave monitoring probes;
- an oblique angle camera;
- a manual sediment recirculation system.

The model layout is illustrated in Figure 1. It was designed to simulate a typical UK sand lower/shingle upper beach. The lower portion of the beach was constructed from a rigid moulding at a slope of 1:50. The upper portion comprised a mobile bed formed of crushed and graded anthracite coal formed at an initial slope of 1:7½ with a crest above the limit of wave action.

The mobile bed was designed to be similar to those used in previous model studies within the programme and to simulate typical UK beaches. The scaling relationships used in selecting the model beach material are based on a well established method which attempts to satisfy three criteria: the permeability, which governs slope, the relative magnitude of onshore or offshore movement, which determines whether erosion or accretion will occur, the threshold velocity of particle motion.

The methods published by Yalin (1963) which relate slope to a non-linear function of the voids Reynolds Number are used to satisfy the permeability requirement. The fall velocity parameter H_b/wT proposed by Dean (1973) is used to satisfy the onshore/offshore criteria while the relationship of Komar and Miller is used for the threshold of motion. At the model scale of 1:50, an assumed prototype D_{50} of 15mm and a density of 2.65T/m^3 then the required model sediment should have a D_{50} of 2.5mm and a density of 1.41T/m^3. Anthracite coal has a density of 1.39T/m^3 and can be obtained in a range of grades. It is therefore an appropriate material for use as the mobile bed.

The sea condition variables were restricted to wave steepness (S) and water level (SWL). All tests were run with a 2m H_s wave height and a 30° offshore wave direction. Wave steepness values were either 0.02 (swell) or 0.06 (storm) and water levels varied form 3.0m to 4.5m relative to the initial beach toe (each test was run at a fixed SWL). Wave induced currents were investigated in a single test using S = 0.02 and a range of water levels from 1m to 4m. Tidal currents were not investigated.

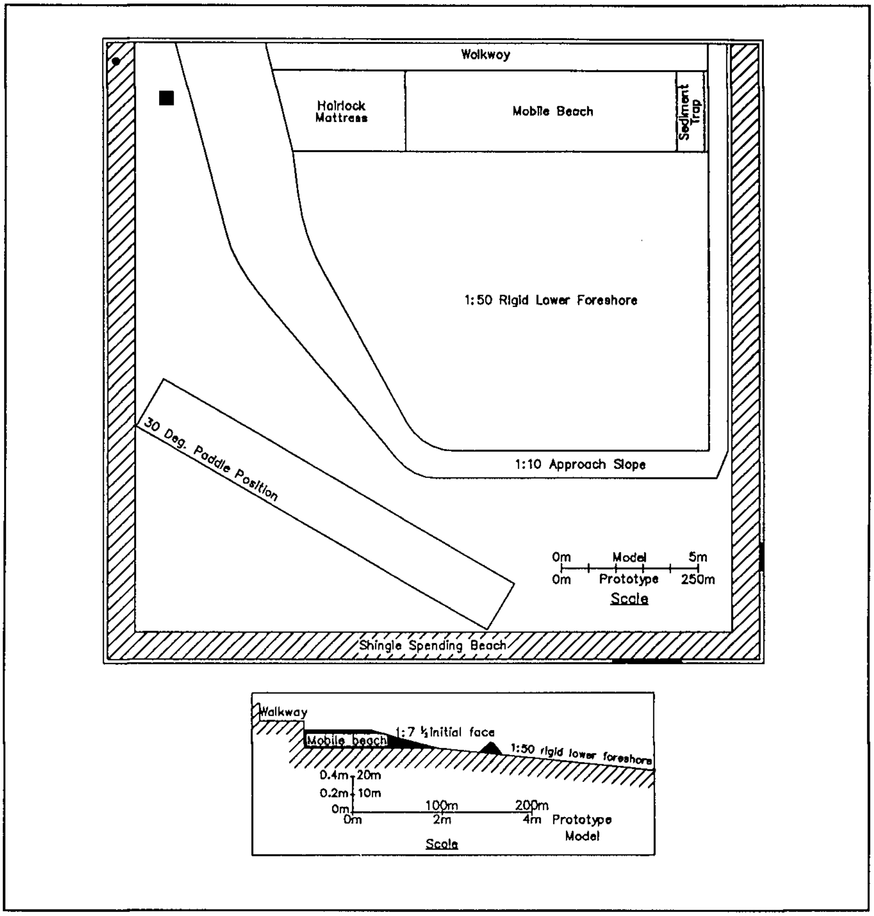

Figure 1 The wave basin facility

The test programme concentrated on single breakwaters. Structural variables included crest length (L_s), freeboard (R_c), offshore distance (X_s). A short series of tests was run at the end of the programme to investigate pairs of structures. These tests investigated the influence of gap length (G_s). A total of 34 tests were run, with an average duration of 2-3 days.

The basic structural design comprised a crest width of 4m, side slopes of 1:2 along the body of the structure and slopes of 1:3 at the ends. The rock grading was selected for minimal damage. The initial median rock weight was 6.55 tonne, with a linear grading from W_0 = 4.0 tonne to W_{100} = 9.1 tonne. During construction larger rocks were preferentially placed in the areas of potential damage, based on the findings of earlier research (Jones & Allsop, 1993). Figure 2 indicates the breakwater layout and the variables. Table 1 summarizes the structural variables.

Table 1 Structural variables

Length (m)	Distance offshore (m)	Crest elevation (m)	Gap length
60	120	4.5	N/A
60	90	3.5	60 & 90
60	90	4.5	N/A
90	150	4.5	N/A
90	120	3.5 & 4.5	N/A
90	90	3.5 & 4.5	N/A
90	60	4.5	N/A
120	120	3.5 & 4.5	N/A
120	90	4.5	N/A

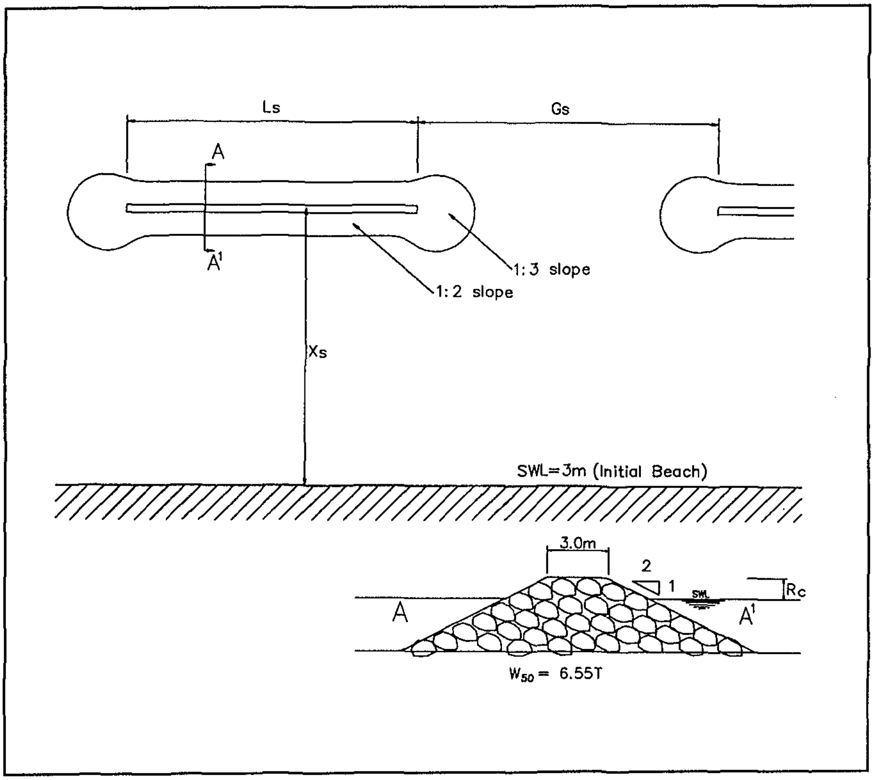

Figure 2 Basic breakwater layout

During calibration, the model was run with an open beach, fed by an unlimited supply of sediment. The transport rates and cross-shore distribution of transport were measured for each wave and water level combination. The transport rates were used to determine standard sediment input rates for all subsequent tests; these are referred to as the *calibrated* rates.

Test procedures included:

- continuous monitoring of downdrift sediment output;
- regular measurement of the cross-shore position of the crest, SWL and toe on 12 section lines to monitor planshape developments;
- regular overhead photographs of beach development; and
- measurement of wave heights seaward and landward of the breakwater.

The terms salient, tombolo, efficiency, potential drift rate and actual drift rate are used in the discussions. Salient refers to the seaward development of the beach in the lee of the breakwater; if this development reaches the breakwater at the water line then the salient is referred to as a tombolo. Efficiency, denoted as η, refers to the ratio of output drift relative to the potential or calibrated open beach drift for the model beach under a given sea state as expressed by the equation

$$\eta = \left(1 - \frac{Q_o}{Q_p}\right) \times 100$$

where Q_p is the potential drift rate for a beach under a given set of wave and water level conditions and Q_o is the output drift rate immediately downdrift of the structure. Potential and actual drift rates are important in relation to breakwater design. The potential drift rate is the volume of beach material that the incident wave condition could move along an open beach if the supply of material is unlimited - in the model this is the calibrated input rate. The actual drift rate on a given beach is often much less than the potential rate due to a lack of available material or the influence of artificial structures or channels; erosion of the foreshore often occurs as a result of the difference between these two rates.

Tests were run in two stages. During the initial stage the beach was supplied with drift material at the calibrated rate. This material was transported into the lee of the breakwater where a percentage was deposited to form a salient, while the remainder was transported to the downdrift sediment trap. When the downdrift transport rate stabilized after 12,000 to 16,000 T_m then the second stage commenced. The updrift sediment input position moved to a point immediately updrift of the breakwater, and the input rate was reduced to the stabilized output rate of stage 1.

Assuming that this rate had been properly defined, then no further material accreted at the salient and the output equalled the new input. Stage 2 continued for about 12,000 T_m or until the input/output equilibrium was confirmed. The relationship between the stabilized output and the original calibrated input was then used to determine the efficiency of the particular structure.

Discussion of results

The test results are presented in a series of figures. Figures 3-8 illustrate the influence of various seastate or structural variables on breakwater efficiency. The lines join directly comparable data points to illustrate the strongest relationships. Where several similar lines are presented on the same plot, they indicate the influence of secondary variables. Figure 9 presents a summary plot combining all of the structure variables.

Effect of test duration on efficiency and beach development

The standard test procedure was followed during all tests except the first test, which was run with a constant input at the calibrated rate for over $100{,}000T_m$. The test was continued until a tombolo had built out to the breakwater and transport had resumed along the seaward face. Although this final result was not of use to the establishment of a η value, it did serve to illustrate the potential impact of a breakwater that is too effective relative to the actual beach drift rate. The structure efficiency reached a stable level of 86% after about 20,000 T_m, but as the beach continued to develop η increased to almost 100%. This increase in efficiency over time is an important consideration in design, as a structure that is too effective will block longshore transport, causing downdrift erosion until the beach has built out sufficiently to allow bypassing to occur.

Effect of wave steepness on efficiency

Tests were run with wave steepnesses of 0.02 and 0.06. Figure 3 illustrates that the structures were about 30% more efficient under shorter period waves.

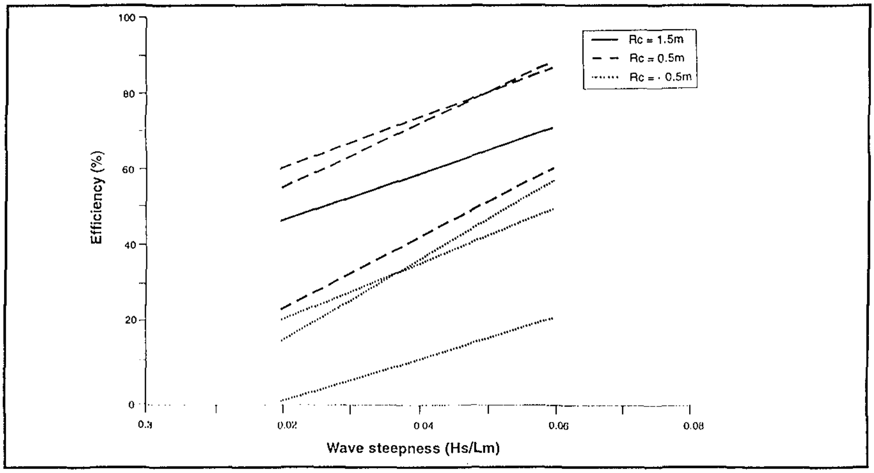

Figure 3 Effect of wave steepness on efficiency

Effect of offshore distance (X_s) on efficiency

X_s was measured from the structure centre line to the 3m contour line on the initial standard beach. Most of the tests were run with X_s values either 90m or 120m with only one test run with each of 60m and 150m.

Figure 4 illustrates that structures set at 60m, 90m and 120m had very similar impacts on efficiency. The only distance which showed a significant effect was 150m, which caused a substantial decrease in efficiency.

Insufficient data was collected for low and high values of X_s to establish any definite trends, however it is apparent that over the limited range of 90m and 120m X_s is not a dominant factor.

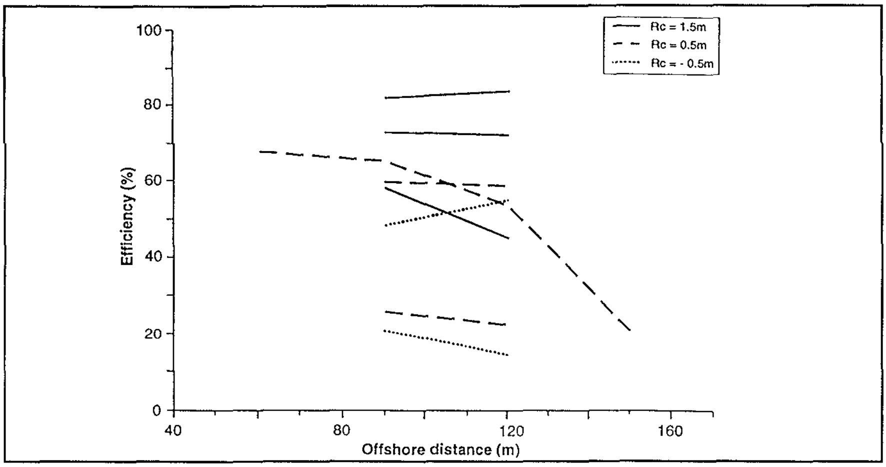

Figure 4 Effect of cross-shore location of breakwater on efficiency

Effect of freeboard (R_c) on efficiency

R_c is the difference in elevation from the structure crest to SWL. Changes in R_c influence the amount of wave energy that can pass over or through the breakwater. By varying both the structure crest elevation, from 3m to 4m, and the SWL from 3.0m to 4.5m, R_c values from -0.5m to 1.5m were tested. Figure 5 illustrates the effect of R_c on η. The results suggest that R_c is an important factor in beach response to breakwaters.

It is apparent from Figure 5 that R_c has a non-linear effect on efficiency. η increased by 30 - 40% as R_c increased from -0.5m to 0.5m. As R_c increased further to 1.5m then η only increased by 10 - 30% with the higher values being associated with the shorter structure lengths.

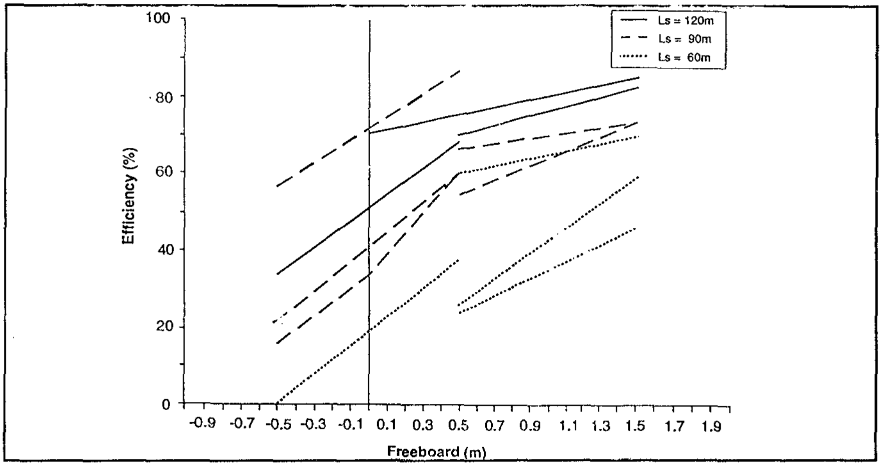

Figure 5 Effect of freeboard on breakwater efficiency

Effect of crest length (L_s) on efficiency

L_s values of 60m, 90m and 120m were tested and were found to be an important factor in determining beach response. Figure 6 illustrates the relationship between L_s and efficiency, with the lines joining data points of equal R_c.

Beach response to L_s was again non-linear. Changes in L_s from 60m to 90m resulted in efficiency increases of between 20 - 40% while changes from 90m to 120m resulted in increases of only 10 - 20%.

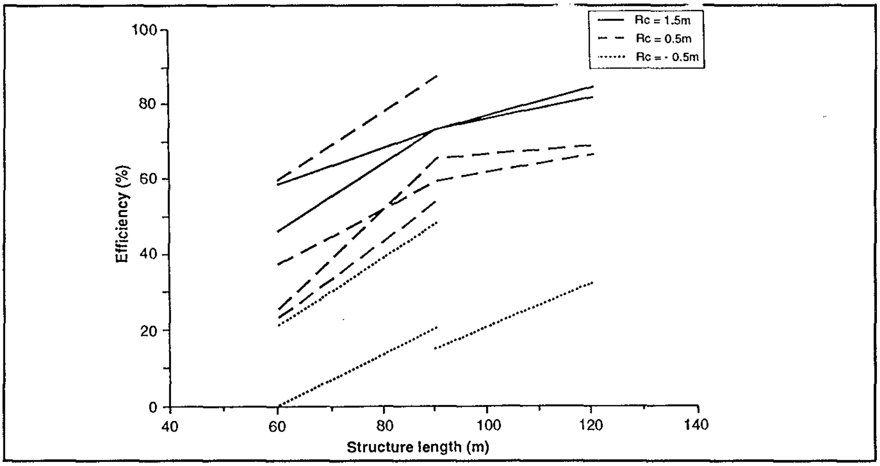

Figure 6 Effect of structure length on breakwater efficiency

Relationship between wave length and structural dimensions

The model data for wave steepness (or wave length), X_s and L_s were further investigated in combinations to determine any relationships. Figure 7 illustrates the influence of L_m/X_s on η. There are insufficient results to form any definite conclusions but the figure suggests that η may reach a minimum when $0.4 < L_m/X_s < 0.9$.

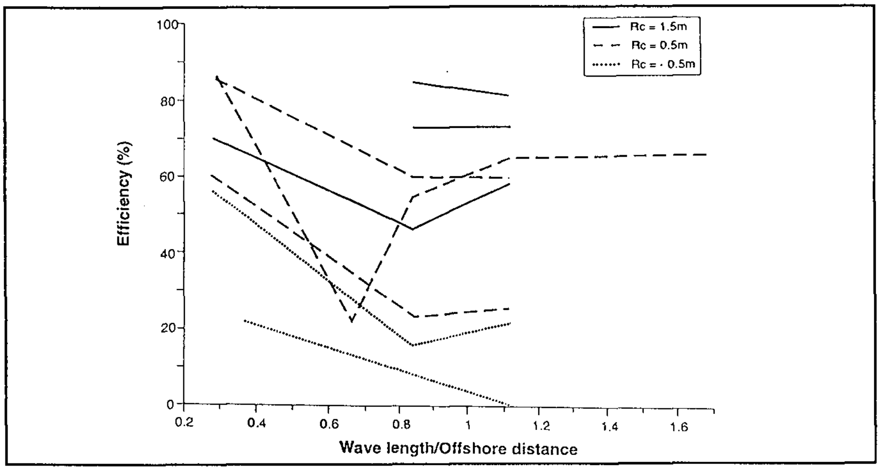

Figure 7 Effect of wave length relative to offshore distance of structure on breakwater efficiency

Figure 8 illustrates the influence of L_m/L_s on η. Though the results show some scatter, there is an obvious trend of decreasing η with increases in L_m/L_s.

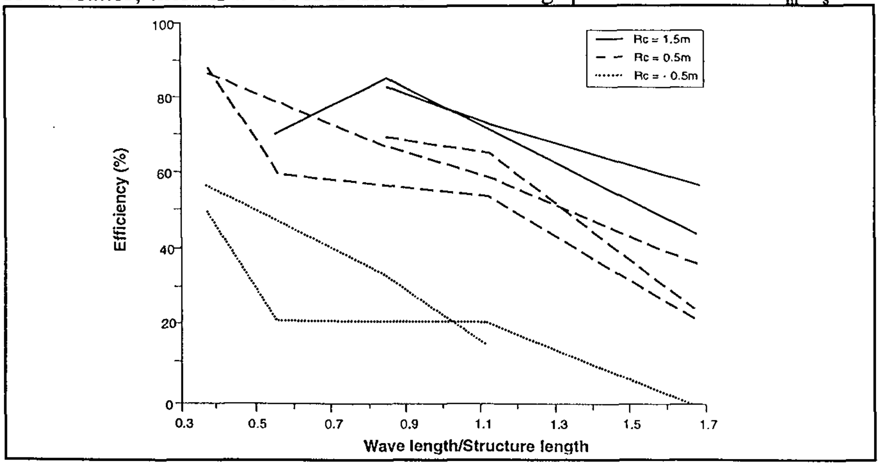

Figure 8 Effect of wave length relative to structure length on breakwater efficiency

Combined influence of structural variables

The individual effects of length, offshore distance and freeboard are combined into a single efficiency contour plot in Figure 9. The data set for the plot was extended by combining both the swell and storm values using the relationship η (swell) = η (storm) - 30; the plot is therefore set out for wave steepnesses of 0.02, and is only applicable to 2m H_s waves.

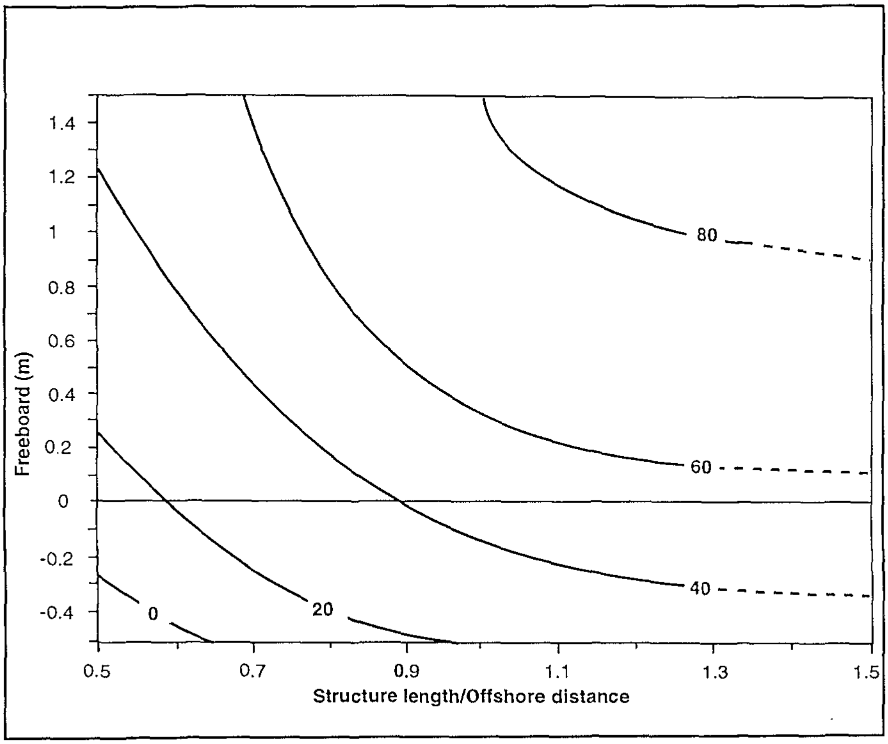

Figure 9 Contour plot of efficiency relative to structure variables for a single detached breakwater (Data points corrected for a single wave steepness of 0.02)

The contour plot can be used to determine the optimum combination of structural dimensions for a given situation. Although the model variables were limited, the method used to obtain the contour plot could be extended to form a full set of design plots for detached breakwaters. Alternatively, the information derived from the study could be used to calibrate existing numerical models which predict wave energy distribution in the lee of structures, although at present none of the available models are able to fully simulate wave/structure interactions.

Wave induced currents

Titanium dioxide was used as a neutrally buoyant tracer to monitor currents along the model beach and in the lee of the breakwaters. This work was subjective only.

Tracer was injected in the lee of a 90m breakwater during swell wave conditions with a range of water levels. Wave induced currents transported dye from the updrift beach and the immediate downdrift beach to the head of the beach salient. From this point the dye was transported intermittently seaward around the down drift end of the breakwater. The intermittent currents had a period of about 10 - 15 seconds (model), equivalent to 8 - 12 incident wave periods. The seaward current reached strong peak velocities, particularly at water levels of between 1m and 3m. Currents at higher water levels were less well defined.

These results qualitatively support available field observations. Further field work should be undertaken as areas of potential rip currents or deposition will be important to breakwater design.

Results of paired breakwater tests

Only three tests of breakwater pairs were carried out as a post-script to the single structure study. Variables included only gap width (G_s) and freeboard (R_c). The results are insufficient for rigorous analysis, but the following observations can be made.

As might be expected, a larger gap width resulted in a slightly lower efficiency, while a change of freeboard from +0.5 to -0.5 resulted in a substantial drop. Comparison against single structures indicated that two emergent 60m long breakwaters with a 60m gap have a η value approximately equal to a single 60m structure, indicating that the efficiency data collected for the single breakwaters may be applicable to multiple structures if appropriate gap widths can be determined. However, when submerged the pair of structures became significantly more effective than a single submerged structure suggesting that the relationships may not be straight forward.

Design applications

The results of the model study are not yet sufficient to form the basis of a design method for detached breakwaters. Only a limited number of conditions were tested, the programme concentrated on single structures and no field verification has been undertaken. However, a number of useful guidelines can be derived which will be of use to coastal engineers and managers.

Most shingle beaches in the UK are of a type known as shingle upper/sand lower, and are subjected to high wave energy within meso (range 2m - 4m) or macro (range $<$4m) tidal environments with the potential for substantial storm surges. It is these beaches which are considered here.

Detached breakwaters affect the beach by altering the inshore wave climate. Waves approaching the beach are either reflected from the structure, overtop the structure (usually breaking in the process) or diffract into the lee. The wave energy in the lee is less than on the open beach and therefore sediment which is transported along the open beach may be deposited. The percentage of deposition relative to the potential drift, for a sea condition close to those tested, can be derived from Figure 9. Efficiency at different tidal levels can be derived from the plot by varying R_c and X_s. If the actual open beach drift rate and the potential open beach drift rate can be derived from numerical modelling, desk studies and field measurements then the optimum layout can be derived.

The optimization may be based on factors apart from sediment transport efficiency. These might include cost of materials, cost of stabilizing the substrate, visual impact, navigational safety, desirability of providing safe mooring for small craft and possible use as an amenity platform for activities such as fishing. The design method assumes a strongly dominant drift direction. Substantial drift reversals driven by waves of lower energy than the design waves will cause material to be deposited in the lee of the breakwater from where it will not be eroded by subsequent conditions. This process will lead to tombolo formation and downdrift erosion. If this situation is likely then groynes may be more appropriate structures.

No single design will be satisfactory under the diversity of conditions found on most beaches, but satisfactory compromises should be possible.

Conclusions

Shingle beach response in the presence of detached breakwaters was investigated using a 1:50 mobile bed physical model as part of an ongoing coastal research programme at HR Wallingford. The study concentrated on single detached rubble mound structures, but concluded with a brief series of tests on pairs of structures Structural variables included length, crest elevation, distance offshore and gap width. The sea conditions only varied in wave steepness and water level.

The study conclusions are as follows.

1 The modelling work that has been completed provides a substantial data base for use in further physical modelling of single or multiple structures and for the development and calibration of numerical models based on wave transformation in the lee of breakwaters.

2 Breakwaters can be used to stabilize an existing or recharged beach where the natural drift has a strong dominant direction and both the potential and actual drift rates can be determined. Successful design depends on matching the breakwater geometry to the actual natural drift under the dominant wave and water level condition. Breakwaters on beaches with high gross transport, but

low nett transport are likely to cause unwanted areas of scour and accretion, as the structures can only be designed correctly for one drift direction.

3 An efficiency contour plot was derived from the study which relates the structure length/offshore distance ratio to freeboard. Application of this plot is limited as the range of sea conditions tested was restricted and no field verification data is available. However it provides a useful first step in developing design guide lines and can be used to optimize structural geometry with respect to construction costs and other design factors.

4 The relationships of efficiency with structure length (L_s) and freeboard (R_c) were clearly established. R_c is dominant when the structure crest is submerged due to the high level of wave energy transmission through and over the structure. L_s becomes the dominant factor as R_c increases above zero.

5 The influence of the offshore distance of the structures (X_s) was less conclusive. Most tests were run with an X_s of 90m or 120m. These distances were too similar to show any distinct trend. Comparison of efficiency with the ratio of wave length to offshore distance (L_m/X_s) suggests that there may be an efficiency minimum when the ratio is between 0.4 and 0.9. However the available data set is insufficient to confirm this possibility.

6 Tests on wave induced currents showed the potential for strong rip currents around the downdrift end of the breakwaters. The currents observed were intermittent at periods associated with 8 - 12 incident waves. They were also dependant on the depth of water in the lee of the structure, with peak currents being observed at depths of between 1m and 3m.

7 Breakwater efficiency also appears to be dependant on the size of the beach salient. As the salient extends out towards the structure the efficiency increases due to more of the wave energy being dissipated on the beach and less being available for transport. This applies particularly to structures with a large freeboard where the transmitted energy level is low. It is therefore important not to over design structures initially, but to allow for minor modifications after construction based on monitoring results.

8 The study did not investigate the potential for deposition of sand and fines in the lee of the structures, nor the influence of strong nearshore tidal currents. These aspects need further work.

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References

Brampton A H and Motyka J M (1985). Modelling the plan shape of shingle beaches. Lecture Notes on Coastal and Estuarine Studies, Vol 12, Springer Verlay, Berlin.

Brampton A H and Goldberg D G (1991). Mathematical model of groynes on shingle beaches. HR Wallingford Report SR 276.

Chadwick, A J, Fleming, C, Simm, J and Bullock, G N (1994). Performance evaluation of offshore breakwaters - A field and computational study. Coastal Dynamics '94 Conf, Barcelona, pp 950-961.

Coates T T (1994a). Phyiscal modelling of the response of shingle beaches in the presence of control structures. Coastal Dynamics '94, Barcelona, pp924 - 937.

Coates T T (1994b). The response of shingle beaches in the presence of control structures. HR Wallingford Report SR 387.

Coates T T and lowe J P (1993). Three-dimensional response of open and groyned shingle beaches. HR Wallingford Report SR 288.

Dean, R G (1973). Heuristic models of sand transport in the surf zone. Conference on Engineering Dynamics in the Coastal Zone.

Huang Z (1993). Mathematical model of shingle beaches with a groyne system. HR Wallingford Report IT 399.

Jones, R J and Allsop, N W H (1993), Stability of rock armoured beach control structures. HR Wallingford Report SR 289.

Komar, P D and Miller, M C (1973). The threshold of sediment movement under oscillatory water waves. J. Sediment Petrol., 43: 1101 - 10.

Powell K A (1990). Predicting short term profile response of shingle beaches. HR Wallingford Report SR 219.

Powell, K A (1992). Study of dissimilar sediment. MAFF Conf. of River and Coastal Eng., Loughborough, UK.

Yalin, S (1963). A model shingle beach with permeability and drag forces reproduced. Proc. 10th IAHR Congress 1963. London p.169.