MINISTRY OF AGRICULTURE, FISHERIES AND FOOD

CSG 15

Research and Development

Final Project Report

(Not to be used for LINK projects)

Two hard copies of this form should be returned to:
Research Policy and International Division, Final Reports Unit
MAFF, Area 6/01
1A Page Street, London SW1P 4PQ
An electronic version should be e-mailed to c.csgfinrep@csg.maff.gsi.gov.uk

Project title	Design and Performance of Submerged Offshore Breakwaters
MAFF project code	**FD0706**
Contractor organisation and location	University of Bristol Dept of Civil Enginering,Queens's Building Bristol BS8 1TR
Total MAFF project costs	£ 100,983
Project start date	01/01/98
Project end date	01/03/00

Executive summary (maximum 2 sides A4)

SUMMARY

This summary describes a two year flume and wave basin study of detached rubble mound breakwaters carried out between January 1998 and January 2000. The study was carried out at Bristol and HRWallingford and met all its objectives while also providing extra results not included in the original proposal.

In the flume studies ten 2D model cross-sections of submerged rubble mound breakwaters were tested by the authors during 1998 and 1999, thus extending the data set presented in Loveless and Debski [1997]. The influence of rock size, core impermeability, crest width, wave climate and water level on the magnitude of inshore set-up was revealed. Pumping tests were performed to reduce set-up and to calculate the discharge imbalance across the breakwater, which generates the set-up when waves attack it. The tests have shown that set-up is significantly increased when the core of the breakwater is impermeable suggesting that designs should be made fairly permeable if possible .

The wave basin of the UK Coastal Research Facility (CRF) at HR Wallingford was used to perform a series of 3D fixed bed model studies of the Elmer breakwater system in February 1998. The crests of the model breakwaters were lowered by 2.0m compared to those built at Elmer so that they were submerged at water levels equivalent to those generated during high water spring tides. This was done to investigate the effect of reducing crest level on the hydrodynamic and morphodynamic behaviour of the Elmer breakwater system. Results were gathered from the 3D fixed bed studies to demonstrate the influence of wave climate and water level on the inshore hydrodynamic conditions. The results showed that the very large set-up (observable behind a constrained 2-D breakwater configuration when the water level is near to the crest)disappears, but that it is converted into large currents in the lee of the breakwaters.

A series of 3D mobile bed model studies of the submerged Elmer breakwater system were also performed in the CRF in October 1998. The influence of wave climate and water level on sediment transport in the inshore region behind submerged breakwaters was demonstrated. Comparisons were made between the morphological features developed within the model submerged breakwater/beach

system and those within the model and prototype Elmer breakwater/beach systems. A novel prediction method was developed to calculate the magnitude of the currents developed by integrating the discharge imbalance at the breakwaters as determined from the 2-D tests(see section 6.9). The method was shown to gives accurate results when water levels are close to crest levels for the Elmer model tests. The main conclusion of the mobile bed studies was that the design with the substantially lowered crest would still have provided a satisfactory design, without excessive shoreline retreat for the beach at Elmer West Sussex.

The authors present a preliminary design protocol that they have developed, as a first step towards facilitating the assessment and design of submerged breakwaters by UK coastal engineers. The authors suggest that the adoption of this design protocol may improve the chances of harnessing the most beneficial features of submerged breakwaters. Much further development is required however before submerged breakwater systems can be built in which the designers can have full confidence in the certainty of their performance over the long term.

Some of the results of this research have been presented in a paper at the ICCE 1997 conference in Copenhagen, Loveless et al [1997], at the MAFF 1999 conference in Keele, Loveless and MacLeod [1999], and at the Coastal Sediments 1999 conference in Long Island, USA, Loveless and MacLeod [1999]. The authors are composing a journal paper summarising all the results from this research.

The intellectual property in this report would not be easily commercially exploitable.

Scientific report (maximum 20 sides A4)

Introduction

The layout of a system of detached segmented breakwaters may be described in terms of the segment length, L_s, the gap length, L_g, and the distance offshore from the original shoreline, X. Generally the segments are placed parallel to the original shoreline since placing them at an angle will increase the effective gap length or require the segments to be longer to achieve the same level of protection. After some time the shoreline will reach an equilibrium profile as shown in Figure 1

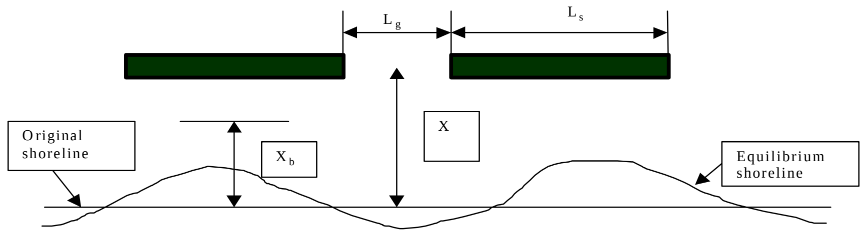

Figure 1 : Definition Sketch of the Layout of Detached Segmented Breakwaters

The cross-section of a detached segmented breakwater (DSB) may be defined as shown below in Figure 2. The relevant variables are the depth of the water at the toe, h, the height of the breakwater, h_c, the freeboard, R_c, which is positive for an emergent structure and negative when the structure is submerged, the core height, C, and the size, D_{n50} of the stone used to construct the breakwater. Inshore of the structure the mean sea level is set-up by the action of the waves on the structure and this set-up is termed,δ.

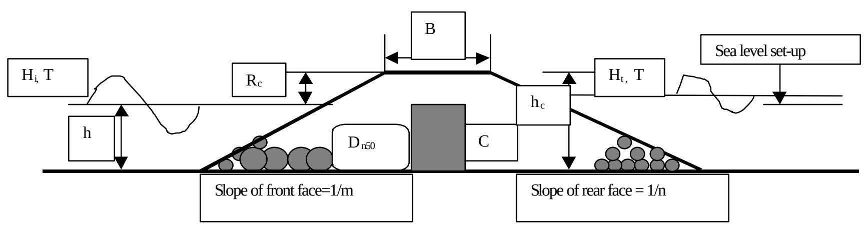

Figure 2 : Definition Sketch of the Section of a Detached Breakwater

DSB’s began to be built in large numbers around the world from around 1965 and by 1990 many thousands had been built protecting over 1000km of coastline. The first DSB’s to be built in the UK were constructed on the Wirral in 1981. After 1990 designers began to experiment with lowering crest levels to save money, but at the same time began to observe certain design problems. The recent research at Bristol has identified the source of some of these problems as the set-up effect and has explored this phenomenon extensively.

In order for a DSB scheme to be implemented it must first be shown that it is the most cost effective solution to the coastal problem. Then the functional design assessment can proceed in the three principal areas of performance:

(a) The response of the beach and shoreline to the DSB system;
(b) The wave and water levels inshore at the final defence;
(c) The structural stability and configuration of the breakwaters themselves.

Beach and Shoreline Response

When the first DSB’s were constructed it tended to be the policy of the designers to minimise wave transmission into the inshore region behind the breakwater line. This, it was later discovered, had three potentially negative effects. Firstly, it dramatically reduced the rate of littoral drift, secondly by creating a system of small coves it risked creating zones of poor water quality near the shoreline and thirdly it blocked the open view of the sea from the beach because the breakwaters were high and the gaps were small.

An obvious question was therefore: To what extent can the crest of a detached breakwater be lowered while still maintaining the shoreline at a satisfactory equilibrium position? The object of this research was to show whether cost savings could be made to conventional surface piercing detached breakwater schemes by reducing their crest levels.

Flume Studies

The first phase of tests reported by Loveless and Debski [1997] investigated the hydraulic performance of a number of differently configured permeable 2D rubble mound breakwaters under a variety of regular and random incident waves and at a number of still water depths. Hydraulic performance was assessed in terms of the magnitude of the set-up generated, the magnitude and direction of the local velocities and the relationship between the magnitude of the incident, reflected and transmitted waves. Loveless and Debski [1997] demonstrated a link between the performance of the breakwaters under regular and random waves thus facilitating the use of only regular waves in subsequent tests.

The authors conceived and performed the Phase 2 tests (1998-1999) to extend the data set illustrating the performance of rubble mound breakwaters. A range of configurations of both permeable and impermeable 2D rubble mound breakwaters were tested to determine their response to impact by waves with similar characteristics to those used in Phase 1. The main parameters studied in Phase 2 were core permeability, crest width and stone size. A complete listing of all the results from Phase 2 cannot be shown here but it is presented in the main report of this research, MacLeod & Loveless(1999). A selection of results are presented and discussed below in order to illustrate the influence of the main structural and wave parameters on the performance of rubble breakwaters under wave impact.

The influence of wave period

Figure 3 shows the variation of set-up and h/hc for increasing wave periods and incident wave heights equal to 1m and 3m for Model 5P2. For large wave heights, Hi=3m, the wave period has a significant influence on the magnitude of set-up. Set-up for the lowest period waves, 4.5secs is less than half that of the highest period waves, 11.2secs, with waves having intermediate periods generating increasing set-up as period increases. For low wave heights, Hi=1m, the same trend in relative set-up magnitude is maintained for all levels of submergence but the magnitude of the set-up difference is much reduced compared with Hi=3m. Again the phenomenon of set-up reduction to zero is observed for the majority of incident waves when h/hc = 0.8. In one exceptional case however, when Hi equal 3.0 and the period is equal to 8.9secs, a significant volume of water is clearly passed over the breakwater. This performance results in the set-up building up until, having overtopped the impermeable core, an equal volume of water per unit time is discharged back by weiring into the offshore region.

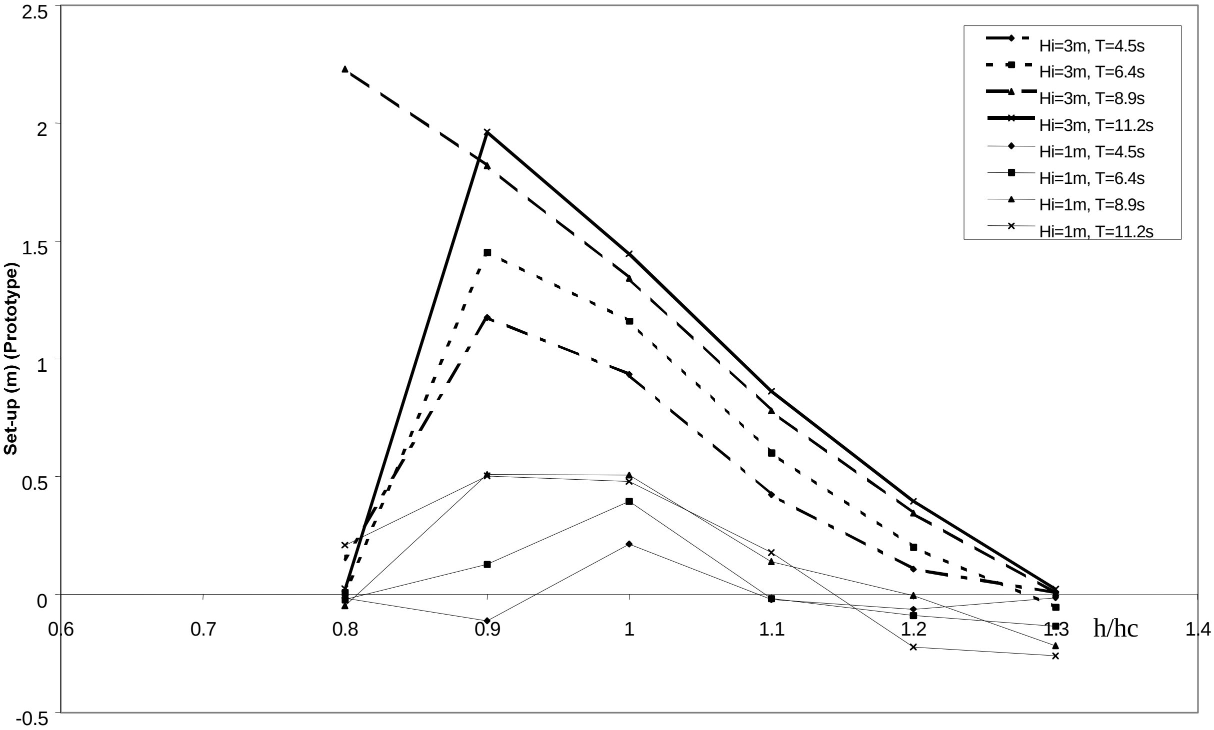

Figure 3 Set-up versus h/hc for impermeable breakwaters (Variable T)

The influence of packing and porosity

Figure 4 shows the variation of set-up with relative submergence, h/hc for 4 rock sizes (Models 1, 2 & 3 (Phase 1) and 10P2 (Phase 2)) and an incident wave height equal to 2m. The rock size for Models 3, 2, 1 and 10P2 in terms of D_{50} (m) were 0.035, 0.041, 0.050 & 0.067 respectively.

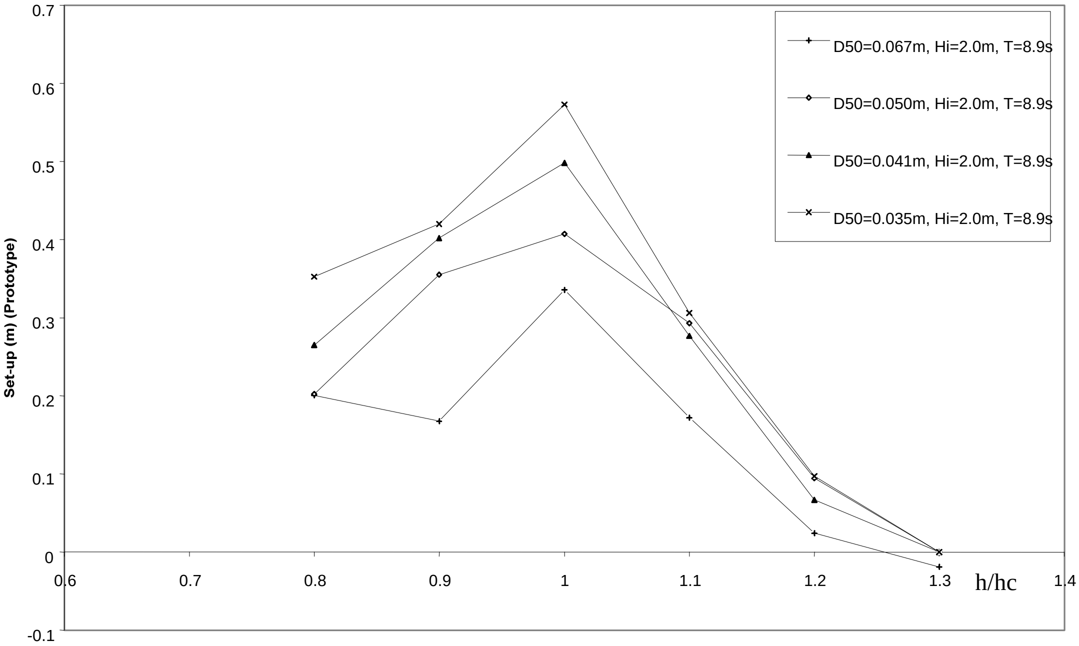

Figure 4 Set-up versus h/hc for breakwaters with different rock sizes (Variable Hi)

Figure 4 demonstrates that the magnitude of set-up decreases with increasing rock size. The authors attribute this behaviour to the porosity of the breakwater increasing proportionately with the average rock size. The interstitial velocity is approximately proportional to the cube of porosity so the back flow velocity that dissipates set-up reduces with decreasing porosity causing the set-up to be greater for breakwaters with smaller porosity. It should be noted however that porosity not only depends on rock size but also on its shape and type of packing.

Water Levels at the Final Sea Defence Line

A major contribution of this research to the understanding of the performance of detached segmented breakwater systems is the clarification of the nature of the set-up of the mean sea level behind the breakwater. Set-up is a very significant factor in the overall performance of detached breakwaters which has been studied very little and reported upon even less. Ignorance of its effects has led to at least one scheme being completely abandoned and other schemes have been designed without a proper understanding of its effects. However, this research has also shown that with a full understanding of its potentially damaging effects these can be entirely eliminated.

The phenomenon occurs because, when a wave passes over an obstruction such as a breakwater or a bar on a beach, there is more resistance to the backflow than to the forward flow. If the area behind the breakwater consists of a laterally constrained body of water the mean level will rise in this area until a back weir flow equals the excess overflow. In these laterally constrained cases the set-up can be enormous for example we have measured a 1.0m rise in mean sea level with a 3.0m wave.

The magnitude of the set-up (or the hydraulic gradient across the crest δ/B) is determined, in the main, by three factors:

1. The excess wave overflow rate relative to the water depth per unit width $=H_iL/h.T$, which is, in effect, a mean velocity across the full depth of water.
2. The relative submergence, R_c/h_c and
3. A backflow resistance factor given by the nominal stone size D_{n50} of the rubble mound.

It was found that from the full range of tests conducted with various rubble mound designs the results fitted the equation:

$$\frac{d}{B} = \frac{(Hi.L/h.T)^2}{8gD_{n50}} e^{-20(Rc/hc)^2}$$

from this equation it may be seen that set-up increases with the square of the incident wave height and that it is a maximum when the water level is at the crest of the breakwater (i.e. R_c=0). Also set-up is decreased the more permeable the breakwater is.
Generally, however, detached breakwaters are built with gaps between them and this allows the set-up to be almost entirely dissipated by currents which pass alongshore behind the breakwaters and out through the gaps between them. It is still nevertheless advisable to make the breakwaters themselves as permeable as possible so that the beach is not scoured by an excessive current. This is in fact what has happened in the past where many designs and models of detached breakwaters were built with impermeable cores. All the return flow required to dissipate the set-up was forced through the gaps giving rise to large rip currents and beach erosion. To test the significance of the core height on set-up a series of tests were carried out in a large 2D flume at Bristol. An example of the results obtained is given in Table 1 below.

Relative height of the core (C/h_c)	0	0.4	0.6	0.8
Measured set-up in (m)prototype	1.1	1.1	1.3	1.44

Table 1 : The Effect of Core Height on Maximum 2-D Set-up for a 4.1m ,8.9s wave when R_c=0 , B=8m and D_{n50}=1.0m

All this points to the need to maintain large gaps between the breakwaters, not to make the individual breakwater segments too long and for the breakwaters to be made as permeable as possible. In fact another test in the above series was carried out with a pipe installed in the breakwater and, for the same conditions as given in Table 1 above, the set-up was reduced to 0.77m at the prototype scale. When these conditions are met it is safe to reduce the crest level of the breakwaters. Significant cost savings are then possible and detached segmented breakwaters compare more favourably as a means of coastal defence and beach protection and enhancement than other methods. These recommended changes to design practice also go a long way to eliminating the various negative aspects of detached breakwaters that have been identified. First the magnitude of rip currents is reduced, second excessive beach erosion is eliminated, third with more open bays and lower crests the view of the horizon is less affected. Fourth the inclusion of pipes through the breakwaters near their centres will allow sand which accumulates in the tombolos or salients to be returned into the littoral zone offshore of the breakwaters so reducing the effect of the structures on the interception of littoral drift. Finally because the crests are lower waves overtop the structure during storms thus maintaining good water quality in the bays.

In order to explore the question of the effectiveness of submerged breakwaters in retaining and accreting beaches a 1:28 scale mobile bed model of the detached breakwaters at Elmer, West Sussex was constructed in the national Coastal Research Facility at HR Wallingford as shown in Figure 5. The shape of the breakwaters was identical to those used in the scheme except the level of the crest was lowered from 4.5mAOD to 2.5mAOD, just below the level of the Mean High Water Spring tides at 2.65mAOD at the site

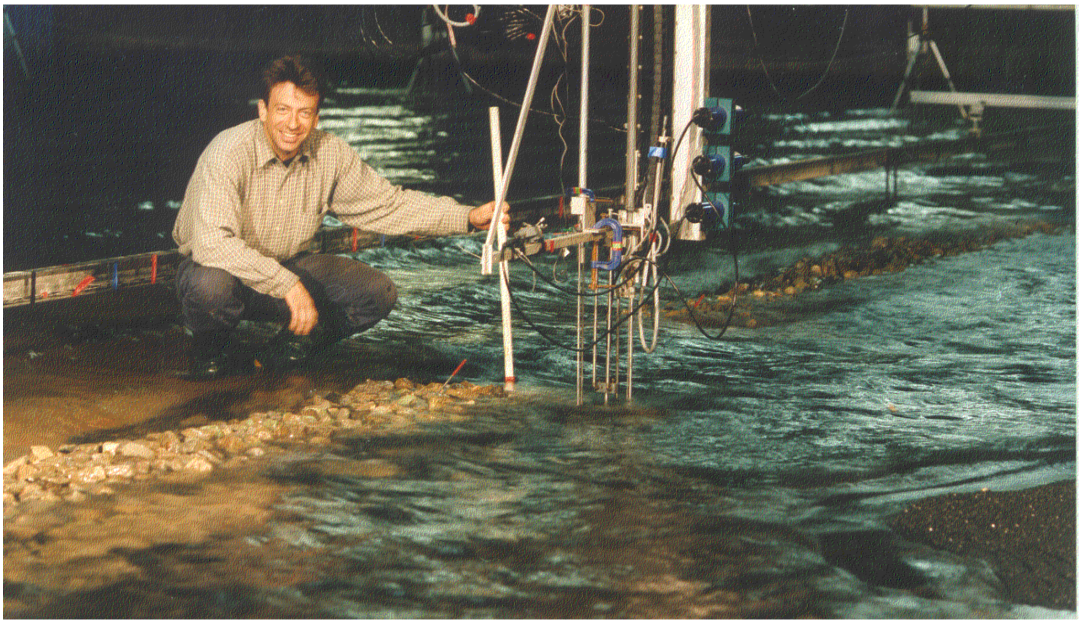

Figure 5 : Dr Breac MacLeod on the model of the Elmer Detached Breakwaters in the Coastal Research Facility

. The model was tested under a range of higher water levels combined with storm waves over a long duration to see if beach erosion or shoreline recession would become excessive. Figure 6 shows the results of a test in which the mean water level was set at 2.00mAOD (the level of the Mean High Water Neap tide) and the significant wave height was 1.76m. This wave height was close to the maximum wave height that could be obtained without substantial wave breaking offshore of the breakwaters. As can be seen from the change in the shoreline there was a tendency for accretion to take place in the lee of the breakwaters, but very little erosion was detected in the gaps.

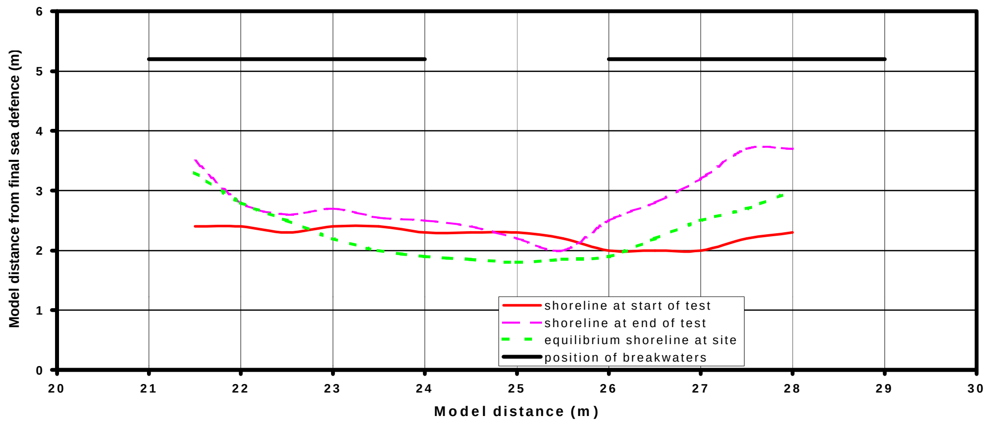

Figure 6 : Shoreline Change for the Lowered Crest Elmer Breakwater when subjected to 1.76m waves at a water level of +2.00mAOD

(Note: Only part of the breakwaters is shown in the figure.) This suggested that, for the majority of the time, the breakwaters with the lowered crest would produce much the same shoreline response as in the existing prototype.

The submerged breakwaters were then tested against a very severe storm and surge event. This had a mean water level of +4.0mAOD (equivalent to the MHWS level plus a surge of 1.35m) together with a significant wave height of 1.85m. In this test the shoreline in the lee of the breakwaters was seen to retreat by about 1.0m in the model ,equivalent to 28m in the prototype.

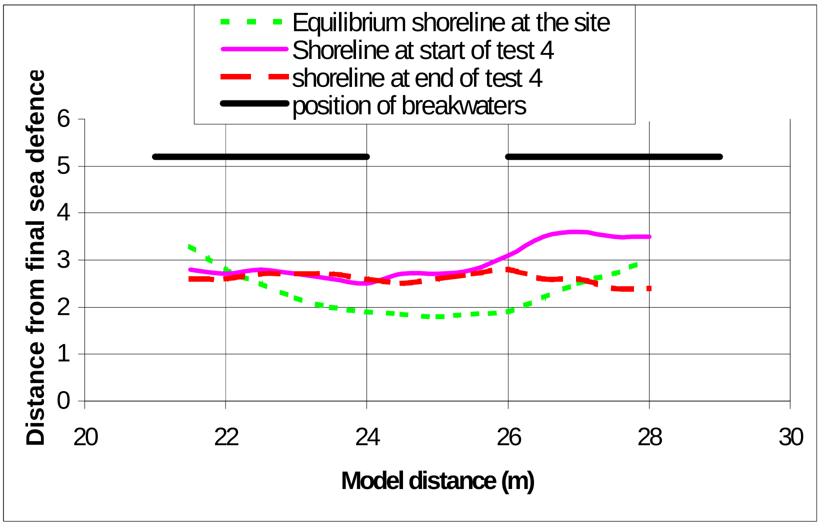

Figure 7 : Shoreline Change for the Lowered Crest Elmer Breakwaters when subjected to a wave of 1.86m and a water level of 4.0mAOD

However the material eroded was actually moved into the beach zone opposite the gap in the breakwaters thus providing more protection there. The resulting shoreline shape, as shown in Figure 7, was thus very favourable to the overall performance of the system in that it showed that the store of sediment which would build up behind the breakwaters in less severe events would be mobilised and become available for the protection of the shoreline opposite the gaps during the more extreme events. The general conclusion was therefore that it would be possible to lower the breakwater crests at Elmer to below the level of MHWS tides without losing the beach or suffering shoreline recession opposite the gaps.

Conclusions

This research on detached segmented breakwaters has shown that significant improvements to their design are still possible. Savings of millions of pounds in project costs are possible. These cost savings are made possible by reducing their crest level to at least 0.5m below the level of MHWS tides. Precautions however must be taken to avoid the set-up of the sea level behind the breakwaters. These include, ensuring that segment lengths L_s are slightly less than the distance offshore X,say, $L_s/X=0.9$; provision of generous gap lengths L_g and making the breakwaters as permeable as possible even to the extent of placing pipes through their centreline sections. The ideal crest width B appears to be around 8 –10m; any narrower and large waves are not destroyed any wider appears to be an unnecessary expense. Rock groynes and revetments put large rocks on the beach where they are most hazardous to users. DSB’s place them offshore out of harms way.

REFERENCES

1. Loveless J H, Debski D and MacLeod (1998) ‘Sea level set-up behind detached breakwaters’, ICCE 98, Copenhagen.
2. Loveless J H and MacLeod B (1999) ‘Influence of sea level Set-up driven currents on sediment movement behind detached breakwaters’, Coastal Sediments Conference, New York,
3. Loveless J H and MacLeod B (1999) 'The design and performance of detached breakwaters', MAFF Conference, Keele University
4. MacLeod B (1999) 'The influence of sea level Set-up driven currents on sediment movement behind detached breakwaters’, Internal Seminar, Bristol University.
5. Half-day symposium at the ICE London for the Maritime Board entitled "The UK Coastal Research Facility (CRF) 1994-2004" . Presented talk on 'The performance of submerged breakwaters in the Elmer model'
6. Loveless J H and MacLeod B (1999) 'The design and performance of submerged detached breakwaters', Journal of Water, Energy and Maritime, ICE, (In preparation).
7. MacLeod B &Loveless J H (1999) 'The design and performance of detached rubble mound breakwaters' Final Report MAFF Project CSA 4137 University of Bristol.

Please press enter