SEA LEVEL SET-UP BEHIND DETACHED BREAKWATERS

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ABSTRACT

This paper reports some of the results of a theoretical and experimental study of submerged, detached, segmented, trapezoidal rubble mound breakwaters. It focuses on the phenomenon of sea level set-up which occurs at these structures. A theoretical analysis is presented which gives an explanation for the occurrence of set-up and the experimental results so far confirm the proposed relationship, which is given by Equation 4. Various other checks are presented in the paper, such as a demonstration that the set-up can be eliminated by pumping out from behind the breakwater and the measurement of a net drift velocity offshore in the core of the breakwater.

INTRODUCTION

Any detached breakwater where the waves are able to pass over or through the structure will experience a rise in the mean sea level on the shoreward side. This "set-up" can have a significant effect on the performance of the structure, but it has been very little researched and is still not well understood. It is created because the wave inflow or overflow is greater than the backwash. The means of restoring equilibrium to the situation is either for the sea level to rise behind the breakwater in order to create more backflow or for a transverse current behind the breakwater to be created, if this is possible.

The results of our research into this set-up have shown, so far, that the magnitude of the set up is a maximum when the freeboard, R_c is zero i.e. when the breakwater's crest is at mean sea level. However we have not yet tested any impermeable breakwaters and these will exhibit slightly different behaviour displacing the maximum set-up position.

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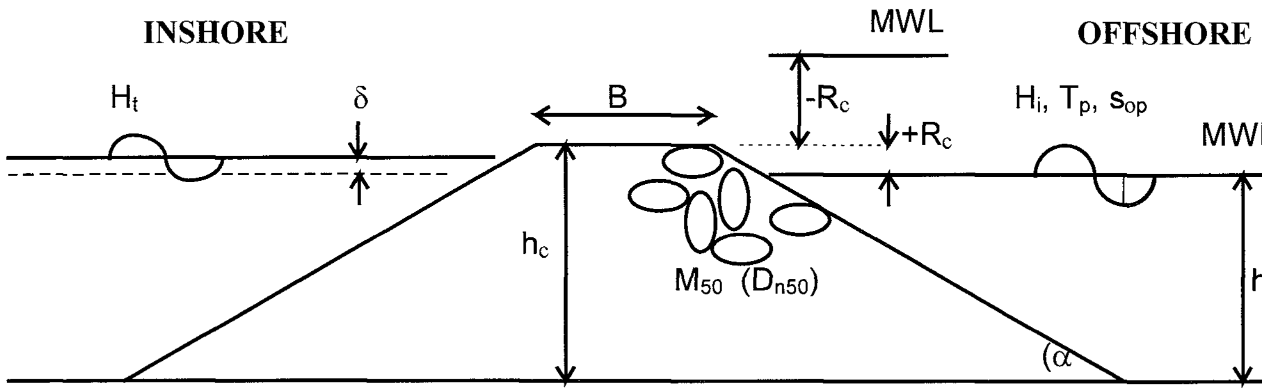
Detached breakwaters are now widely accepted as one of the three main methods of protecting eroding coastlines; the other methods being groynes and shoreline armouring. Detached breakwaters have greatly increased in popularity in recent years and many papers on their design have been reported at recent coastal engineering conferences. Few if any of them have discussed the phenomenon of (breakwater created) set-up. Yet we believe that an understanding of this set-up and the other effects it produces is essential to the correct design of detached breakwater schemes. Furthermore, as designs of detached rubble mound breakwaters have evolved, designers have tried to use lower crested breakwaters. This is because since the cost of the breakwater system is proportional to the volume of rock per km. of coastline it probably follows that the cheapest rubble mound scheme will minimise the crest height and width and maximise the side slopes. Indeed the first author recommended nearly 12 years ago (Loveless 1986) "From the evidence available it can be argued that a crest level over one metre below the MHWS level would be sufficient for many cases of coast and beach protection.".

Unfortunately, the recent experience of researchers (Murphy 1996) (Van der Biezen 1998) and practising engineers (Browder et. al 1997) when investigating or designing low crested or submerged detached breakwaters, has been that they produce large longshore currents which can result in more beach erosion instead of less. Some of these researchers are now advising against the use of detached breakwaters altogether. It is our view however that these longshore currents are mainly due to the phenomenon of set-up. Once all the mechanisms governing the creation of set-up have been identified it will be possible to produce designs which minimise it and so eliminate its adverse consequences on beach erosion and minimise overtopping of the final sea defences.

This paper reports on a 2-D flume study of detached trapezoidal rubble mound breakwaters which was carried out to investigate the nature of the set-up. In most detached breakwater schemes the potentially very large levels of set-up do not in fact occur because they are relieved by 3-D current circulations. However, in this study, a way of transferring the data obtained from the 2-D flume research was devised so that the advantages of control and increased scale offered by the 2-D flume research could be retained. The new technique enabled the applicability of these 2-D results to be extended very simply so that they could be used to predict the performance of the kind of 3-D installations common in the field. The method used was simply to install a pump behind the breakwater in the flume so that a relationship between the pumping rate and the reduction of set-up could be obtained. The paper also reports briefly on some 3-D experiments in the UK's National Coastal Research Facility at HR Wallingford.

A detailed report of the original research is obtainable from the Department of Civil Engineering at the University of Bristol (Debski & Loveless 1997) and a previous paper (Loveless & Debski 1997) presents the findings with respect to wave transmission which were found to be generally as predicted by earlier researchers.



Variable	*Definition*	*Units*	*Variable*	*Definition*	*Units*
B	Breakwater crest width	m	H_i	Incident wave height (H, H_s, H_{rms})	m
R_c	Breakwater freeboard, - when $R_c<0$,	m	H_t	Transmitted wave height (H, H_s, H_{rms})	m
	referred to as the submergence of the breakwater		H_r	Reflected wave height (H, H_s, H_{rms})	m
h_c	Breakwater crest height	m	H_s	Significant wave height	m
h	Water depth at offshore toe	m	H_{rms}	Root-mean-square wave height	m
α	Offshore slope angle	degrees	K_t	Transmission coefficient, $K_t=H_t/H_i$	-
M_{50}	50% value of rock mass distribution curve	kg	K_r	Reflection coefficient, $K_r=H_r/H_i$	-
ξ	Surf similarity number, $\xi=\tan\alpha/\sqrt{(2\pi H_i/gT^2)}$	-	T	Wave period (regular monochromatic waves)	s
δ	Inshore water level set-up	m	T_p	Peak wave energy period (wave spectra)	s
L	Local incident wavelength	m	s_{op}	Deep water wave steepness, $s_{op}=2\pi H_i/gT_p^2$	-

Figure 1 : Definition sketch and major variables for breakwater cross-section and analysis

PROBLEM DEFINITION

The problem studied in the research was the performance and design of submerged, detached, (trapezoidal) rubble mound breakwaters. A definition sketch, which also shows the major variables involved in the problem, is shown in Figure 1. The freeboard, R_c, which is a key variable in the problem, is negative when the crest is submerged.

The main purpose of a detached breakwater is to reduce the wave energy between it and the shoreline. The incident waves transfer quantities of water inshore over and through the breakwater. Since the waves inshore are smaller and the return flow resistance is generally greater, the return velocities over and through the breakwater are less than the inshore velocities and this results in a net transfer of water to the inshore zone at the breakwater. However, the shoreline behind the breakwater is usually relatively impermeable so there can be no net flux of water inshore. The excess flow must therefore be driven back offshore either through the breakwater (if it is permeable) or over the breakwater (if it is submerged) or around the ends of the breakwater (if it has gaps) or by a combination of these three flow paths. All three routes offer resistance to a flow of water and require a head difference to drive the flow. Therefore a rise in water level will occur inshore to maintain a zero net flux of water transfer in the near shore zone.

It can be shown that the volume of flow backwards and forwards in half wave period of a progressive wave is HL/πT. At a breakwater (or other bar like structure) there is greater resistance to the return flow than the overflow. Therefore there is a net inflow which must be balanced (in 2-D) by a set-up of the water surface behind the breakwater (or inshore of the bar).

If the resistance is dominantly turbulent (which it is for both bars and breakwaters) then the set-up, δ should be proportional to u_o^2, where u_o is the mean offshore set-up driven velocity. If h is the water depth then it follows that

$$\delta \quad \alpha \quad \left[\frac{H_i L}{hT}\right]^2 \qquad \text{Eq. 1}$$

and this was found to be true in the experimental results.

Because the resistance to flow is dramatically reduced when the breakwater is submerged it is also likely that the relative submergence R_c/h_c will be an important variable and since the resistance to flow within the breakwater is a function of D_{n50} the stone size (or permeability of the breakwater) will also affect the set-up.

If the set-up height is non-dimensionalised using the crest width, B we have a hydraulic gradient δ / B which is related to u_o by means of the Forcheimer equation.

Hence,

$$\frac{\delta}{B}=f\left[\left(\frac{H_i L}{hT}\right)^2,\frac{R_c}{h_c},D_{n50}\right] \qquad \text{Eq. 2}$$

or alternatively

$$\frac{\delta}{B}=f\left[\frac{(H_i L/hT)^2}{8gD_{n50}},\frac{R_c}{h_c}\right] \qquad \text{Eq. 3}$$

So far we have found that for our admittedly limited range of tests that

$$\frac{\delta}{B}=\frac{(H_i L/hT)^2}{8gD_{n50}}\cdot e^{-20(R_C/h_C)} \qquad \text{Eq. 4}$$

best describes the results.

DESCRIPTION OF THE 2-D MODELS

The experiments were performed in a large random wave flume, 15m long, 1.5m wide and 1.1m high with a 5m long, 0.5m wide test section. A total of six different model breakwater cross-sections were tested utilising three different sizes of carefully graded rock. A schematic layout of the flume showing the position of the breakwater models is shown in Figure 2 and a picture showing the three rock sizes used is presented in Figure 3. All the breakwaters were 500 mm high, but the crest widths varied from 200 to 600 mm. A range of wave and water level conditions were tested with incident waves ranging from 50 to 200 mm height and water depths being varied between 400 and 650 mm giving both submerged and emergent conditions. Both random and regular waves were tested.

As can be seen in Figure 2 a submersible pump was located behind the wave absorber at the end of the flume so that the set-up could be artificially lowered in order to simulate 3-D conditions and investigate the nature of set-up.

RESULTS FOR SET-UP IN 2-D MODELS

An example of the results obtained is given in Figure 4 where the set-up for one of the breakwater models is plotted against the incident wave height for various water levels. The set-up was found to be a maximum when the water level is just below the crest of the breakwater.

As can be seen from Figure 4 when $R_c = 0$, the set up is greatest. For a regular wave of height 3.0 m the maximum (unrelieved) set-up was found to be 1.0 m. The results for random waves were found to given equivalent amounts of set up provided that the mean wave height rather than the significant wave height was used. Figure 5 shows

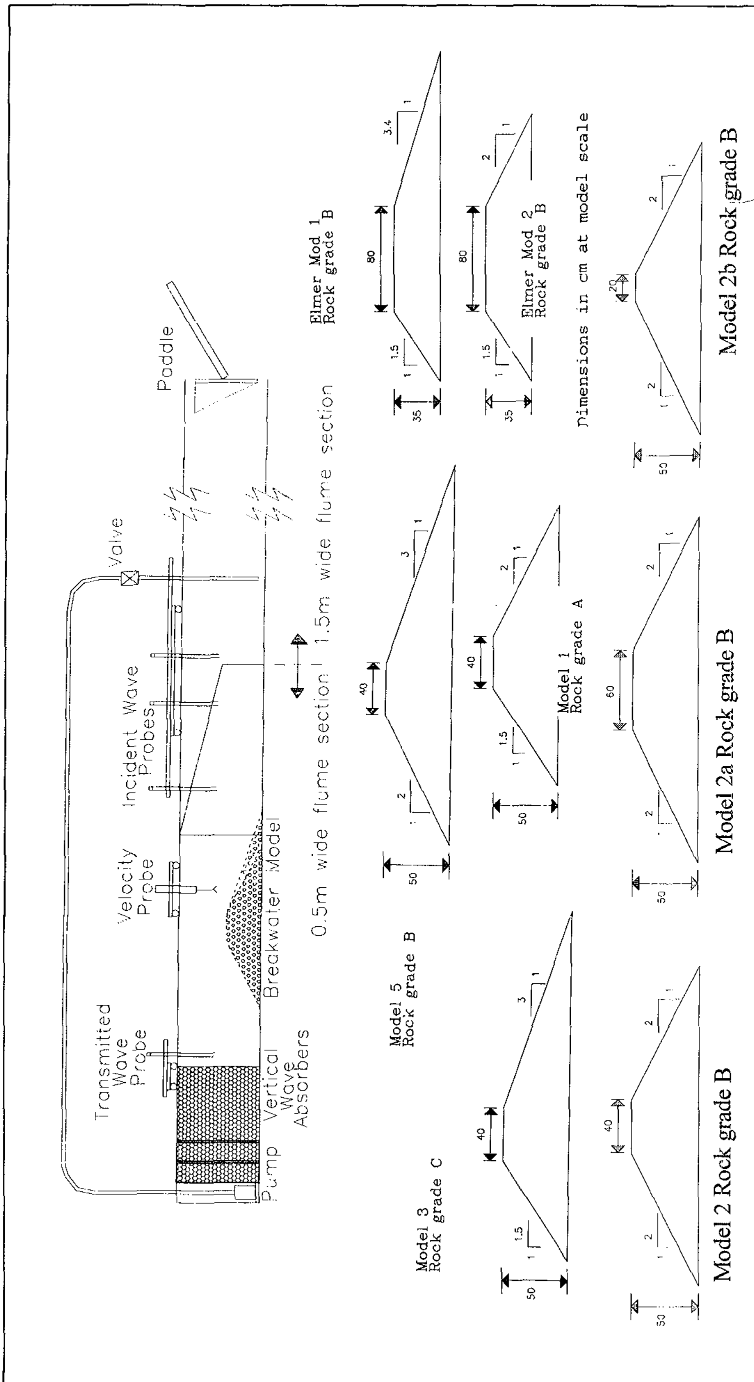


Figure 2 : Schematics of the experimental flume and some breakwater cross-sections

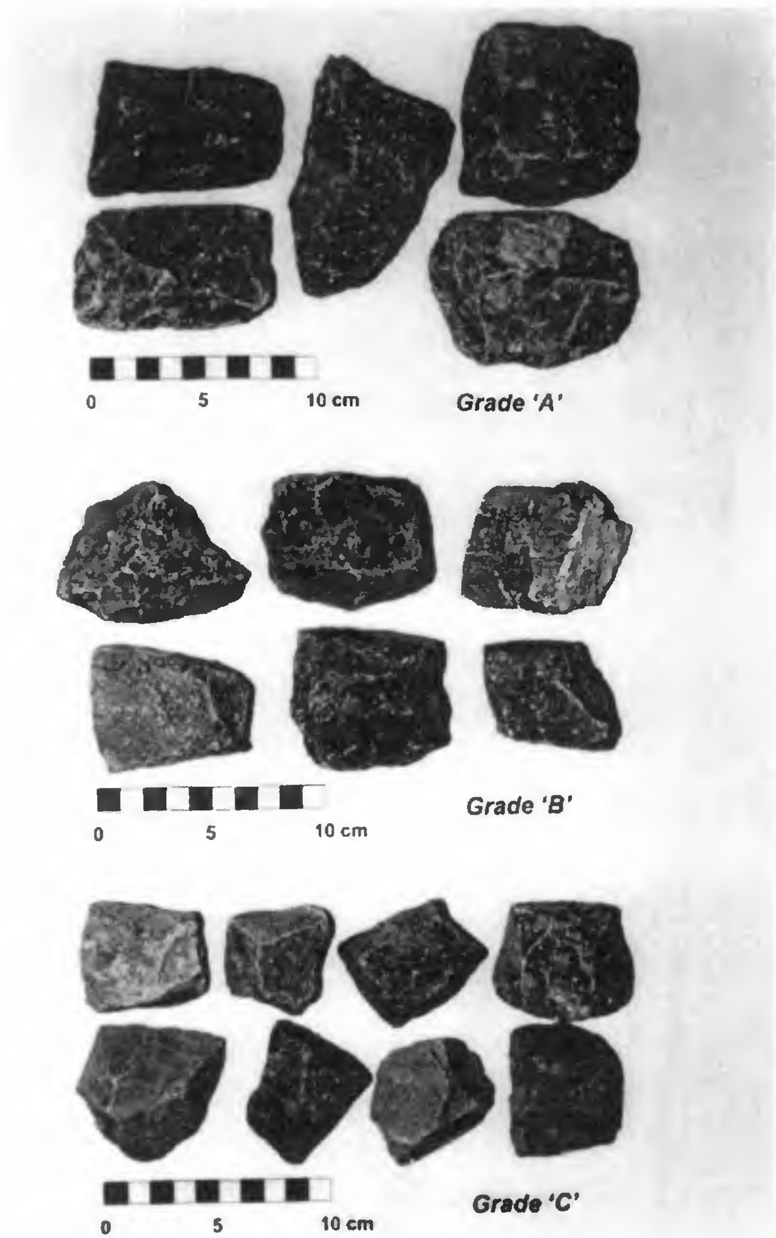


Figure 3 : The three grades of rocks used in the breakwater models

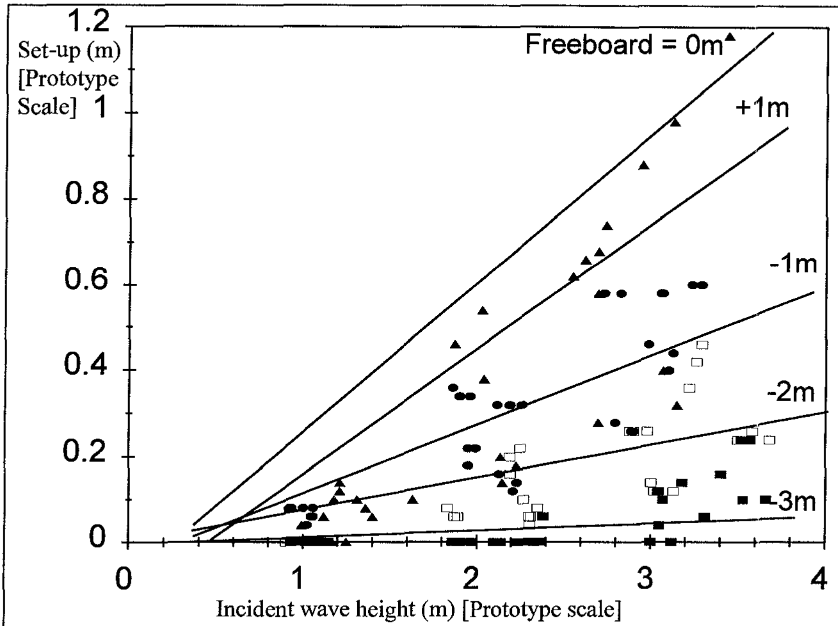


Figure 4 : Set-up inshore of breakwater at prototype scale (averaged for three different models: 2; 2a; 2b for T = 11.2s)

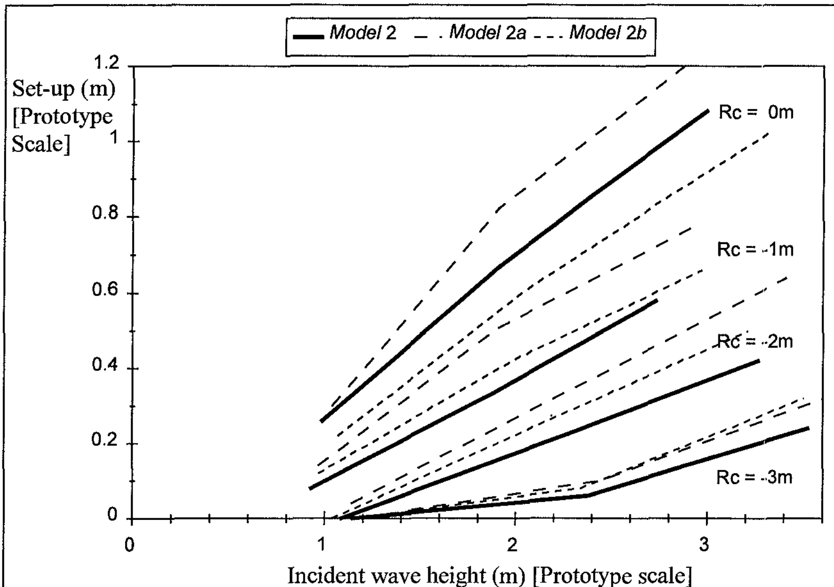


Figure 5 : Set-up inshore of breakwater at prototype scale for T = 11.2s (Models 2, 2a, 2b shown separately)

how set-up is affected by the crest width. Model 2b has the narrowest crest and 2a has the widest. It can be seen that set-up generally increases with crest width.

The only previous study of set-up at a trapezoidal rubble mound breakwater was that of (Diskin 1970). However, the results of our experiments, which are shown against Diskin's curve on Figure 6 gave much lower results. This was attributed to the fact that we had used a larger stone size in our tests. From all the results we have obtained so far the best fit curve is that given by Equation 4 which, as explained earlier, also has a firm theoretical basis.

PUMPING TESTS AND VELOCITIES

Besides the measurement of set-up the research in the 2-D flume provided accurate data on the quantity of flow necessary to suppress it. Thus, by reference to Figure 7, it may be seen that, with model 5, a set-up of 0.5 m could be reduced to zero with a rate of pumping of 1.8 m^3/s/m width of breakwater.

Velocities both within and around the model breakwaters were measured using an ADV velocity probe. Figure 8 shows an example of the net velocity vectors obtained. From this figure it is possible to identify clearly both the circulation cell just beyond the crest of the breakwater and the presence of a horizontal jet in front of the breakwater. Figure 9 shows the velocity vectors at the four key points of the wave cycle and clearly shows that the velocities within the core of the breakwater are subject to a net drift velocity offshore due to the set-up. In fact the magnitude of the drift velocity in the core of the various breakwaters was found to be as predicted by the Forcheimer equation and this result is shown in Figure 10.

SET-UP IN 3-D

All these flume measurements of set-up are those that would occur where it is not possible to relieve the very large rises in mean sea level by transverse currents or some other means. i.e. the pure 2-D situation. In 3-D the set-up is frequently dissipated by being converted into strong transverse currents so that, in many 3-D configurations, only a very small amount will occur. Nevertheless all the set-up drivers are still present and when converted into these strong currents may result in beach erosion and scour at the ends of the breakwaters. These unwanted results, as mentioned earlier, have been reported by a number of researchers and practitioners. We have also observed them in our own 3-D studies in the UK's National Coastal Research Facility. Figure 11 shows that the maximum set-up that we measured in one test was only 70mm (prototype scale) ,but the current measurements taken indicated, as expected, strong transverse flows. It is our view though that they can be eliminated once appropriate remedial action has been taken by the more appropriate design of submerged breakwaters.

CONCLUSIONS

We believe that coastal engineers in their search for cheaper forms of coastal defence will increasingly seek to deploy segmented detached trapezoidal rubble mound breakwaters with lowered crest elevations.

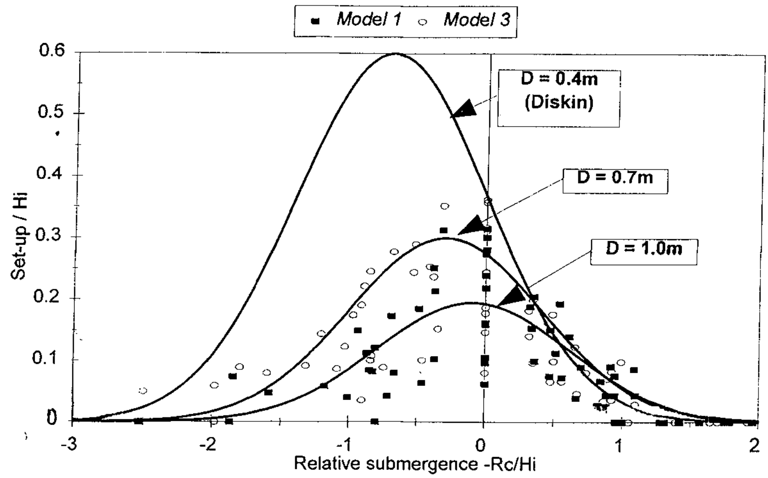


Figure 6 Diskin's curve for set-up compared to the new results

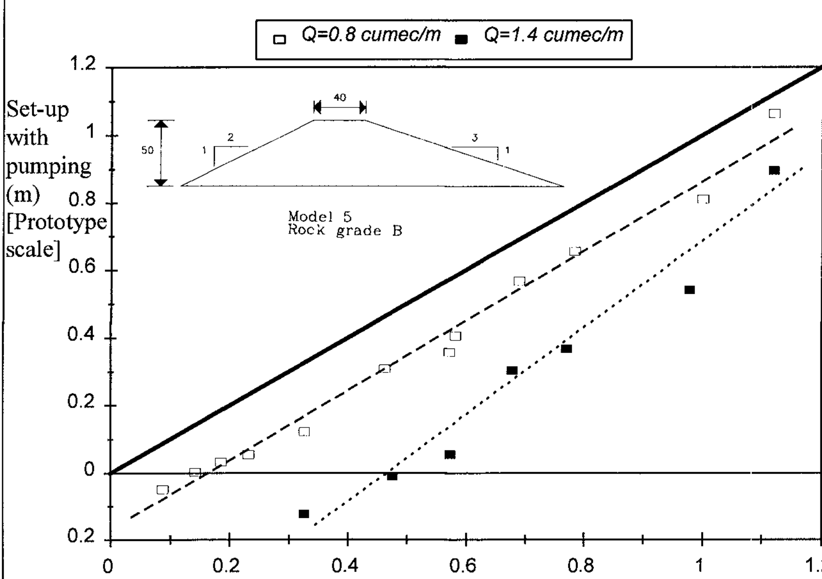


Figure 7 Reduction in set-up by pumping for Model 5 : $R_c = 0$m

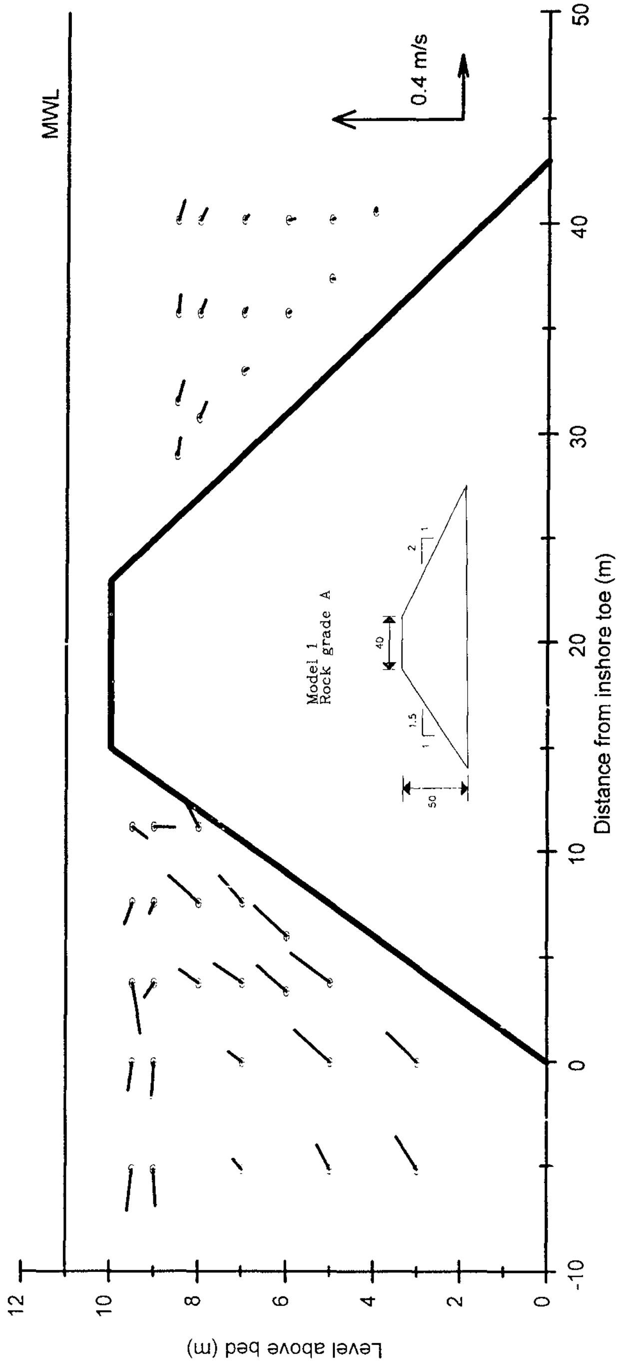


Figure 8 : Net velocity vectors for Model 1, T=6.4s, H_i=2.0m, R_c=-1m (K_t=0.55, δ=0.22m)

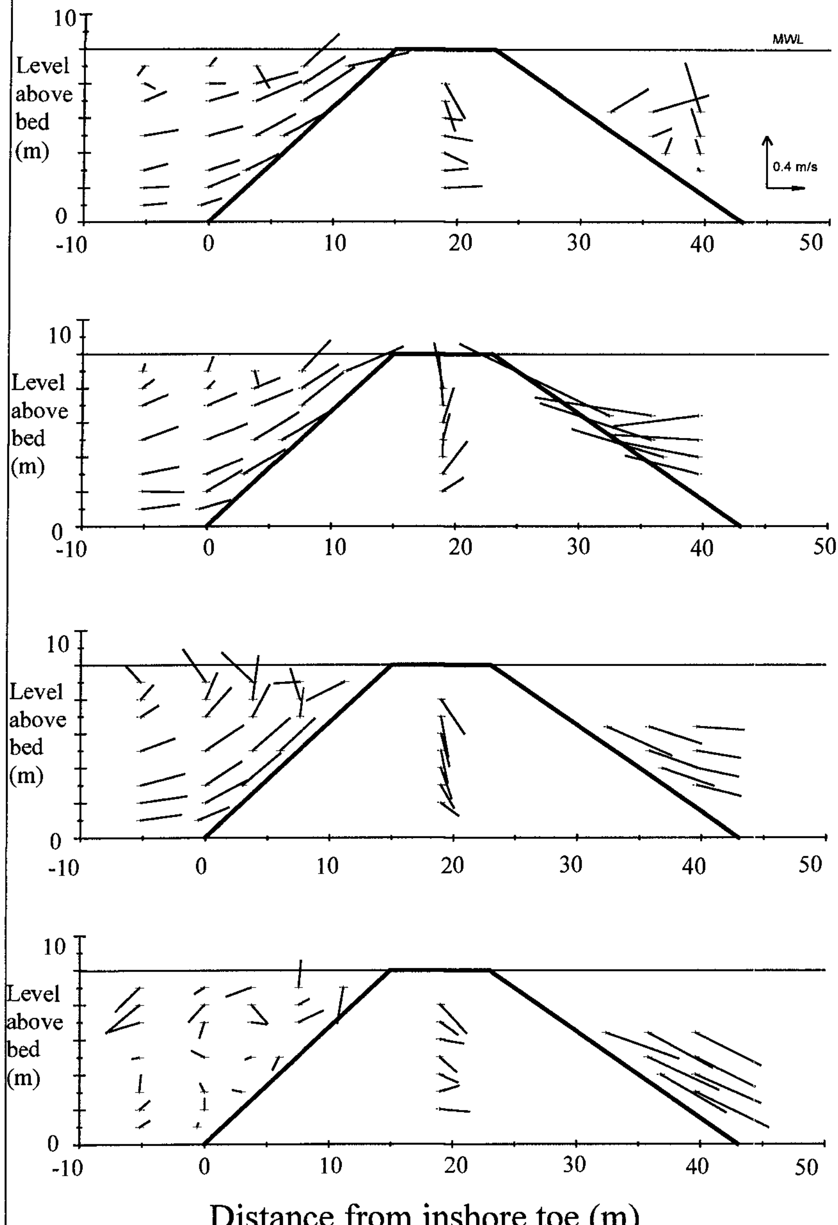


Figure 9 Velocity vectors at t=0, T/4, T/2, 3T/4 for Model 1, T=11.2s, Hi=3.2m, Rc=0m (Kt=0.45, δ=0.96m), Rock grade A

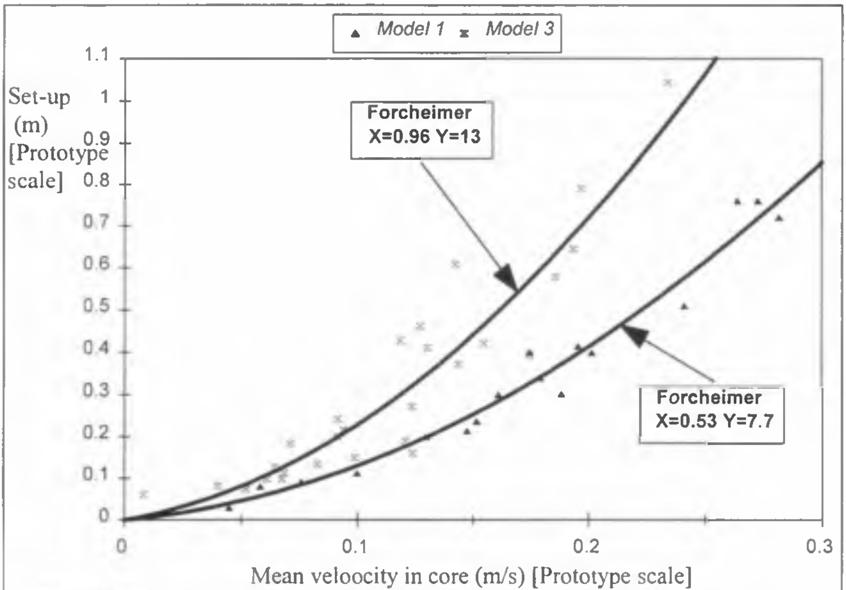


Figure 10 Forchheimer prediction curves and test data for Model 1 and Model 3

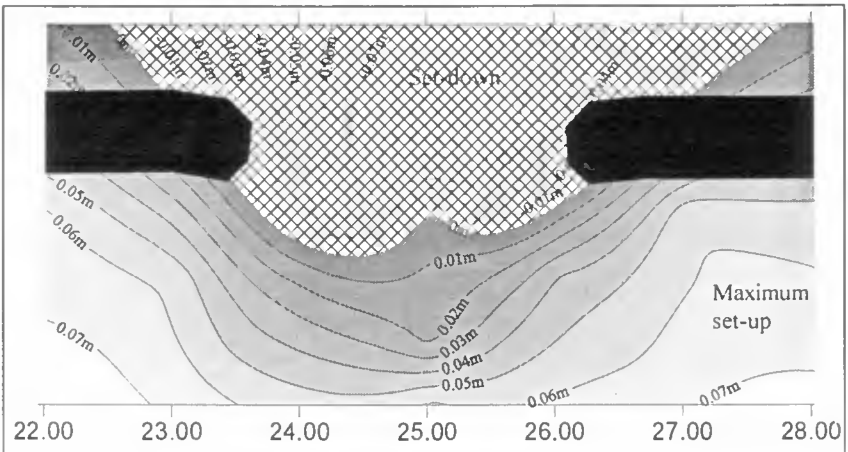


Figure 11 Set-up in the vicinty of Elmer breakwaters tested in CRF [Prototype scale]

In this paper we have shown that these designs are subject to the phenomenon of set-up of the sea level inshore and we have gone a long way towards defining exactly how this set-up is created.

In breakwater configurations where transverse currents are constricted or prevented the set-up can be very large. In most 3-D configurations however it is not readily observable, but the forcing elements are still present and they create instead strong currents which are capable of eroding the beach.

Further research, it is confidently expected, will lead to improved designs which will minimise these unwanted effects and make the use of detached breakwaters still more advantageous.

ACKNOWLEDGMENTS

This work was carried out under a Research and Development contract for the UK Ministry of Agriculture Fisheries and Food, Flood and Coastal Defence Division.

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