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A LABORATORY STUDY OF OVERTOPPING AND BREACHING OF SHINGLE BARRIER BEACHES

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Abstract

Physical model tests were undertaken at HR Wallingford to address gaps in the knowledge of the failure process of shingle barrier beaches. During these tests, numerous factors were considered such as sediment characteristics, the type of waves (storm or swell), and beach crest geometry (height, width, back slope).

INTRODUCTION

Shingle and mixed sand / shingle beaches are widespread in many parts of the UK and Europe. These beaches are highly efficient and practical forms of coastal protection with high ecological, amenity and aesthetic value. However a shingle beach in common with any other type of beach, can suffer erosion and subsequent landward retreat of the shoreline. Consequently over a period of time a beach which was originally of satisfactory dimensions may be reduced to such an extent that it no longer constitutes an acceptable 'line of defence'. Anticipating this state is clearly important if shingle beaches are to be managed effectively, and landward structures are not to be damaged by flooding.

The classic dynamic equilibrium shingle beach profile has been described using the parametric model of Powell (1990). In theory, a dynamic equilibrium profile should develop for any given combination of wave conditions assuming that there is sufficient time and sediment available for the profile to form. However this limitation means that the model is not valid for the prediction of overwashing and breaching of shingle barrier beaches, though it has been used to provide a first estimate of profile performance in these circumstances (Buijs et al., 2005).

An alternative empirical framework, based on extensive fieldwork and physical model data was developed to predict the threshold for breaching of shingle barrier beaches by Bradbury (2000). The field and model data used to develop the model related only to the shingle barrier at Hurst Spit. Bradbury et al (2005) found that model did not work so well when applied to other sites and concluded that use of the model outside the valid predictive range would result in the under prediction of overwashing. Further data was therefore required to test and extend the range of validity of the Bradbury model.

PHYSICAL MODEL

Physical model tests were performed in one of the wave basins at HR Wallingford at a scale of 1:15 to study the overwashing and breaching of shingle barrier beaches. The physical model consisted of 4 separate bays each 2m wide and 15m long, with the shingle beach represented by crushed coal according to the scaling adopted by Powell (1990). Bay 1 consisted of a lower sand layer and an upper coal layer with a prototype grain diameter of 16 mm. The sand layer was used to simulate the effect of a relatively impermeable core on the threshold for breaching. Bay 2 contained sediment of the same size of as bay 1 so a direct comparison between a beach with and without an impermeable core could be made. Bay 3 & 4 much contained coarser sediment with a d50 of 42 mm and 53mm respectively. This allowed the effects of beach permeability on the threshold for failure of barrier beach to be observed.

One of the other main objectives of the study was to investigate the effect of the barrier width on the threshold for breaching. To do this three different crest widths were investigated (5m, 10m, & 15m prototype). Two different wave steepness were used (S=0.06, 0.01) to study the different effects of storm and swell waves. The geometry of the barrier also has a significant effect on the threshold for breaching and as a result two extra tests were made. The first was a barrier beach fronting an elevated hinterland and the second was a barrier with the same volume as a previous test but with an elevated free-board. Table 1 gives details for each of the test conditions.

The initial profile of each shingle beach was a slope of 1:7. Irregular waves with a significant wave height of 2m (prototype) were run for 1000 waves to generate an equilibrium profile. The barrier width was defined as the distance between the crest of the initial equilibrium profile and the back face of the barrier. The rear face of the back barrier for the majority of the tests was cut back steeply at a slope of approximately 1:2 to the floor of the basin. An example of an initial profile and set-up is shown in Figure 1. Once the initial profile had been generated the bathymetry was recorded using the laser scanner. The wave height was then increased incrementally by 0.25m for bursts of 1000 waves until the barrier failed. After each burst of a 1000 waves the new position of the crest was recorded. Once the barriers had failed in all four of the bays the basin was drained and the bathymetry was again recorded using a laser scanner.

Trimble GS200 3D laser scanning system was used to measure the bathymetry of each bay before and after each test to an accuracy of ± 1mm. Each scan measured the (x,y,z) locations of the bathymetry on a 10mm by 10 mm spacing (model scale). An example of one of the outputs from the laser scanner can be found in Figure 2 with the wall separating the two bed sections clearly visible. A wave probe array was placed offshore to measure the incident wave conditions.

Storm versus swell conditions

The observed failure mechanisms were very different under storm and swell wave conditions. Under storm waves the crest elevation remained the same but the crest position continued to retreat back with increasing wave height. The retreat of the crest was generally caused by erosion from the front face which would cause a gradual steepening and then slope failure would occur. When the beach reached a minimum crest width flow, through the beach would cause rear slope failure and the beach would eventually breach. Under storm conditions the breach tended to occur over a small section of the beach which would heal itself if the subsequent waves were less energetic.

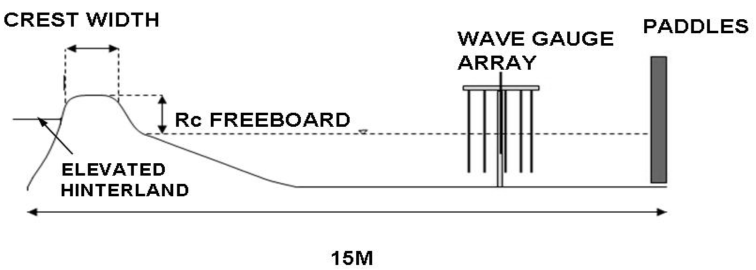

Figure 1 Wave basin set-up

Table 1: Test conditions

Test No.	Crest Width (m)	Wave Steepness	Type
1	5	0.06	Cut Back
2	10	0.06	Cut Back
3	15	0.06	Cut Back
4	15	0.01	Cut Back
5	10	0.01	Cut Back
6	10	0.06	Elevated Hinterland
7	5	0.06	Elevated Freeboard
8	5	0.01	Cut Back

Under swell conditions initially the crest position would build up as well as move back. Sediment would be drawn from offshore and deposited on the crest. However when the waves became sufficiently large overtopping would cause the rear slope to slump thus reducing the crest elevation. As the crest lowered the barrier would continue to be overtopped until it eventually failed. As one would expect there was significantly more flow through the beach than when compared to the storm conditions. It was also observed that under swell conditions the failure of the barrier tended to occur over the entire width of the beach and once the barrier failed generally it never recovered again. The finer material beach was found to be more vulnerable to the storm wave conditions. Conversely the coarser beach failed first under the swell conditions.

Barrier geometry

The effect of barrier geometry was investigated through two additional tests using the storm condition. The test which had a raised, sandy, hinterland was more vulnerable than a barrier of the same width but with the lower hinterland employed for the other tests. Flow through the beach resulted in the pooling of water behind the barrier. When the barrier was overtopped the shingle would be carried much further horizontally which meant that this sediment was no longer available to reinforce the back face of the beach. With the lower hinterland level sediment washed over the beach crest was deposited on the immediate rear slope and therefore remained part of the beach crest structure. Consequently the nett landward loss of sediment from the barrier beach system was much less for the reduced hinterland level.

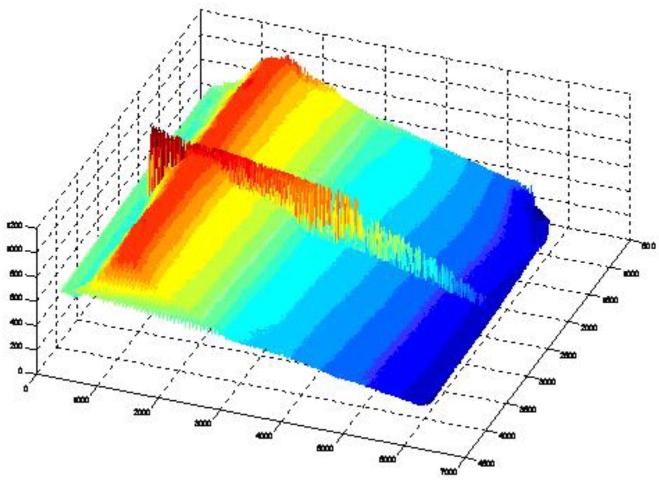

Figure 2 Shingle beach profile for test 8 for bays 1 & 2.

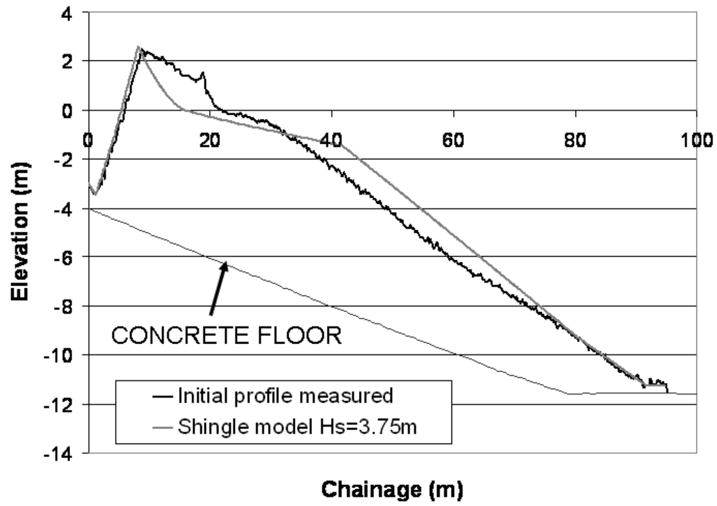

Figure 3 Shingle model prediction for test 2, bay 2 (D50=16mm).

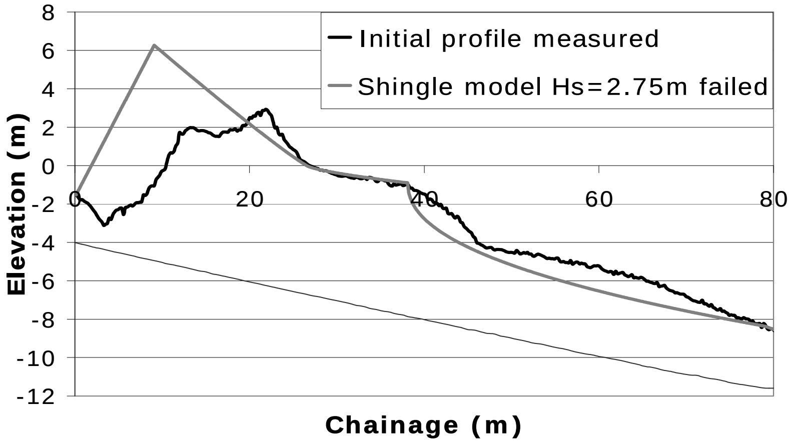

Figure 4 Shingle model prediction for test 5, bay 2 (D50=16mm).

A second test was performed with an elevated freeboard ie a narrower, higher barrier of the same cross sectional area. This was tested for storm conditions and was found to be more vulnerable than the lower but wider barrier. This was because the crest still retreated back by similar amounts which resulted in the barrier failing at an earlier stage relative to the wider barrier. This would suggest that as long as the crest of the barrier is above the maximum run-up position then having a wider barrier is more effective than a super elevated barrier. However it is important to note that this may not be the case under swell conditions.

The presence of a reduced permeability core

Another factor that has an important influence on the shingle volume in the profile is the presence of a less permeable compacted core within the shingle beach. Due to a compacted core less shingle volume is available to adjust to severe hydraulic loading conditions and the dissipation of wave energy due to flow through the beach is concentrated in the relatively more permeable surface layers. The net result being the increased mobility of the active shingle layer. To simulate this behaviour a layer of shingle with a d50=16mm (prototype) was placed over a sandy core. The sediment used was identical to that used in the adjacent bay so that a direct comparison could be made with a beach without a reduced permeability core. The depth of the shingle layer was 3m (prototype scale). As expected the beach with a low permeability core was much more vulnerable and under the majority of the tests and in some cases the sand layer became exposed at the toe of the profile.

Failure threshold of shingle barrier beaches

Varying definitions of breaching have been used in connection with shingle barrier beaches within the geomorphological and engineering communities. The definition commonly used within an engineering context, and within the current investigation, describes breaching as the short-term lowering of the barrier crest, resulting from wave induced overwashing (Bradbury, 2000). The classical dynamic equilibrium shingle beach profile (SHINGLE - Powell, 1990) develops whilst conditions are sufficiently benign that wave run-up cannot exceed the crest; this provides a distinct berm, breaker-step and toe at the seaward limit. The dynamic equilibrium profile can be predicted reliably for given combinations of wave, water level and sediment size under such conditions. Using the results from the hydraulic model tests we were able to assess the suitability of the method of Powell to predict the failure of shingle barrier beaches. By using the initial measured profile as input to the model the significant wave height was increased incrementally until the crest position moved beyond the rear face of the shingle barrier. This was deemed to be the point at which the barrier would fail, Figure 3 gives an example from test 2 which shows the initial measured profile and the predicted profile from the SHINGLE model at the point where the crest position is at the rear face of the barrier. In this case the SHINGLE model predicts the threshold for failure at a significant wave height of 3.75m.

The SHINGLE model appeared to perform well under the storm wave conditions particularly for the finer sediment. Figure 3 gives an example where the SHINGLE model predicted the correct threshold for failure of the barrier beach. The original hydraulic tests used to derive the SHINGLE model were based on sediment of a similar size to the finer material that was used (D50=16mm). However SHINGLE was not calibrated to work with much coarser sediment similar to that used in Bay 4 (D50=57mm). It is therefore not surprising that it did not work so well for the coarser sediment used. Figure 4 shows the same test as the previous figure but shows the results for the coarser sediment. SHINGLE predicted the initial profile reasonably well but the position of the crest for the failed profile is beyond the back of the barrier. This implies that SHINGLE would have predicted failure too soon. Unfortunately SHINGLE did not perform well for the swell wave conditions but it is also important to note that the original tests from which the model was derived did not include many swell wave conditions. In general SHINGLE predicted a much higher crest elevation than was actually measured for the swell wave conditions.

Extensive 3-dimensional physical model investigations and limited fieldwork by Bradbury (1998, 2000) provided an empirical predictive framework and a preliminary estimate of the risk of breaching of shingle barriers of defined cross section. The conceptual approaches outlined by Bradbury (2005) have been developed to examine the short-term profile response, by reference to the wave climate, storm peak static water level datum, barrier freeboard Rc and the barrier cross-section-area above this datum. When combined, the two latter variables provide a barrier inertia grouping, which can be non-dimensionalised by wave height, to provide the dimensionless barrier inertia parameter (Bi), described by:

$$B_i = R_c B_a / H_s^3 \qquad (1)$$

Where R_c(m) is the barrier freeboard, B_a(m^2) is the cross-sectional area of the beach above still water level and Hs (m) is the significant wave height. The model is only valid in the range $0.015<H_S/L_M<0.032$. The predictive framework considers the morphodynamic response of shingle barrier beaches of varying geometry to a range of hydrodynamic variables and provides a preliminary estimate of the overwashing threshold under extreme conditions. The barrier inertia parameter is plotted against a dimensionless wave steepness parameter. The upper confidence limit for the barrier inertia parameter threshold is described by:

$$\frac{R_C B_A}{H_S^3} < 0.0006\left(\frac{H_S}{L_M}\right)^{-2.5375} \qquad (2)$$

Figure 4 shows a comparison between the threshold curve (2) and the field and model data used to derive the curve combined with new physical model data. Being below the curve implies that breaching will occur. It is clear from the new physical model data that extrapolation of the empirical model is not valid and that the predictive curve needs to be modified. This empirical model only includes the effects of wave steepness and barrier cross-sectional area. Results from the physical model tests indicate that the sediment size and the barrier geometry also have a significant effect on the threshold for failure.

The Bradbury model over predicts the threshold for breaching for swell waves and under predicts the threshold for steeper storm waves. By combining the previous field and model data with the new physical model tests a new more widely applicable model to estimate the threshold for the breaching of shingle barrier beaches can be developed. Figure 6 shows the combined field and laboratory data set with the new empirical curve to describe the upper limit for the threshold of failure of a shingle barrier beach. Three different types of regressions were investigated (Linear, Exponential and Logarithmic) and the results showed that all three were a significant improvement on the Bradbury threshold in view of the new data. However the simple linear fit provided the best description of the upper limit for the threshold for breaching and can be described as follows:

$$\frac{R_C B_A}{H_S^3} < -153.1\frac{H_S}{L_M} + 10.9 \qquad (3)$$

Valid for the range $0.01<H_S/L_M<0.06$

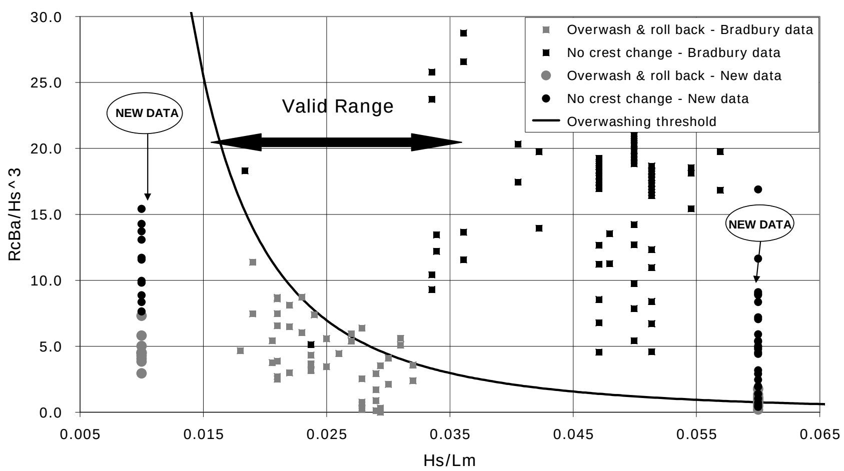

Figure 5: A comparison between the empirical approach of Bradbury (2000) and the combined field and model data.

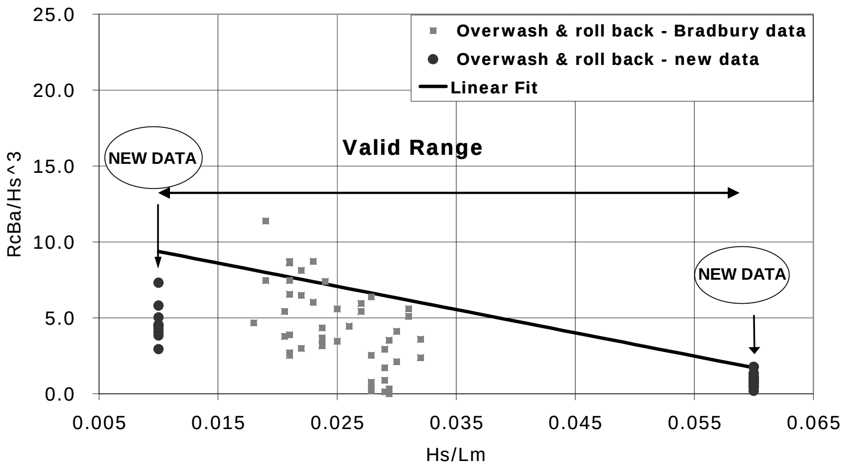

Figure 6: A comparison between the modified empirical curve and the combined field and model data.

Conclusions

This work has confirmed that there are several important factors which influence the failure of shingle barrier beaches. These include the wave steepness, the volume of sediment within the beach, the crest freeboard, barrier geometry and the permeability of the beach. The Bradbury approach has the advantage that it is a relatively simple method to apply to a limit state equation. However it does not take account of the effect of beach permeability on the failure process which was observed to be an important factor during these experiments. The new threshold curve does offer some improvement on the original Bradbury curve which is now valid over the range $0.01<H_S/L_M<0.06$. This should be viewed as an upper limit for the failure threshold as it does not include all of the processes involved. The SHINGLE approach does appear to work well under storm conditions particularly for the finer sediment. However it does not work well under the swell wave conditions or for the coarser sediment. This is not surprising as experiments used to design and validate the model did not include these types of conditions. It would be possible to extend and improve the validity of the model with further physical model tests but caution would need to be exercised in relying solely on physical model data as this can be subject to scale effects. It is not surprising that we obtain good agreement between these new experimental results and the SHINGLE model as they are both based on physical model tests using coal. It would therefore be desirable to further validate or calibrate the SHINGLE model using field data. Further work is also required to examine the influence of the hinterland (form and level) on the breaching process to ensure that any further developments are valid for the full range of conditions in the UK.

1) The approach original of Bradbury et. al. (2005) is not valid beyond the range $0.015<H_S/L_M<0.035$. It tends to over predict the threshold for breaching for swell waves and under predict the threshold for storm waves $H_S/L_M<0.035$.

2) The SHINGLE model was able to predict the threshold for breaching for the finer sediment case and the storm conditions but it was unable to predict the correct threshold for the swell wave conditions and the for the coarser sediment case.

3) A modified version of the Bradbury model was derived using the combined field and model data to provide an upper limit on the threshold for breaching that is valid over a wider range of wave conditions $0.01<H_S/L_M<0.06$.

4) The modified Bradbury model provides an upper limit for the threshold for breaching. However further work is required to include the effects of permeability, barrier and hinterland geometry on the threshold for breaching.

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