CHAPTER 206

THE SHORT TERM PROFILE RESPONSE OF SHINGLE SPITS TO STORM WAVE ACTION

Bradbury A P [1] and Powell K A [2]

Abstract

The profile performance and the conditions giving rise to overtopping and overwashing of shingle barrier beaches have been examined in an extensive programme of field work and physical model studies. The hydraulic performance of Hurst Spit, on the south coast of the UK, has been examined in detail, and the studies have been used to design a management programme for this barrier beach. This has included the design of a beach replenishment scheme and quantification of overtopping thresholds and alarm conditions, for a range of beach geometries. An extensive series of 3-dimensional physical model tests are discussed and a dimensionless parametric framework is suggested, which provides a method of estimating the conditions that will result in crest accumulation, breaching and crest lowering of shingle barrier beaches, for a wide range of beach geometries.

Introduction

Shingle spits often provide the only natural defence from wave attack to the areas of low lying land in their lee. They are hydraulically efficient structures, maximising wave energy dissipation through the high permeability of the shingle. Barrier beaches are particularly important at sites where they protect low lying land, often at the mouths of large tidal inlets. Important sites around the UK include Hurst Spit, Chesil Beach and Orfordness. These beaches are typically less than 100 metres wide above mean water level and may have a crest width of less than 5 metres.

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[1] Coast Protection Group, New Forest District Council, Lymington, S041 9ZG, UK
[2] Coastal Group, HR, Wallingford, Oxon, OX10 8BA, UK

The natural beach crest is generally formed at a level which is just below the maximum level of wave run-up. This is controlled by a combination of wave conditions, and water levels, and the shingle size and grading. The crest of a shingle barrier beach may be exceeded by wave run-up, resulting in overtopping under extreme conditions. The beach crest is often able to dissipate energy as waves pass over the crest, restricting both water and shingle movement to the seaward side of the beach crest. Under these circumstances any shingle that is pushed or carried to the crest by the wave run-up is deposited at the crest, thus causing the crest level to increase. The high permeability of the beach usually ensures that most wave energy is dissipated before it reaches the lee face of the beach. When the combined effects of freeboard, crest width and wave conditions reach a critical condition however, often during periods of extreme water levels resulting from tidal surges, the beaches become susceptible to overtopping which cannot be confined within the crest. This may result in crest rollback, crest lowering, fan formation and subsequent sudden, and often spectacular failure of the shingle bank. Whilst the leeward position of a shingle beach with a seawall or cliffs on its landward side is fixed, a barrier beach with no retaining wall on its landward side is free to move landwards or "rollback" when waves reach the leeward crest of the beach.

Previous field studies of shingle barrier beach development have discussed various stages of crest evolution (Nicholls, 1985) resulting in:

a) crest level raising by overtopping;
b) crest rollback:
c) crest level lowering by overwashing;
d) breaching;

Field observations of the performance of barrier beaches have been discussed in some detail by several authors and the sedimentology of the crest features of sand barrier beaches has been studied extensively (Leatherman and Williams, 1977). Whilst the field studies have provided a valuable description of the development of these beaches, they have not quantified the complex interaction of wave and water level conditions, sediment size and the geometry of the beach, which result in the formation of these features. There is a particular dearth of information with respect to shingle barrier beaches, which typically have a D_{50} grain size in excess of about 5mm.

Recent examples of severe storm activity on Hurst Spit, UK, have demonstrated that catastrophic failure of shingle barrier beaches may occur, resulting in extensive flooding and damage to low lying land in the lee of the beach. The problems of diminishing shingle supply to Hurst Spit present a significant problem to the management of the beach and remedial action was urgently required to maintain the integrity of the beach. This problem presented the opportunity to examine at full scale a barrier beach which was becoming subject to overtopping under storms of increasing frequency.

Recent research on restrained shingle beaches by Powell (1990) and van der Meer (1988) has provided parametric empirical frameworks to predict the profile performance of restrained shingle beaches, but these studies have not examined the evolution of barrier beaches. The processes which result in the dramatic evolution of these beaches are difficult to quantify and as a result coastal managers have little guidance on their management. If natural or renourished shingle barrier beaches are to be used as effective coast protection structures, it is essential that the hydraulic performance of the crest evolution is clearly understood.

Objectives

The studies were commenced with the intention of providing an understanding of the hydraulic performance of the Hurst Spit shingle bank, and to provide a management strategy to ensure that the spit would continue to provide protection to the land in its lee. This required identification of the conditions that cause crest lowering of the beach, the design of a major beach replenishment scheme and the identification of appropriate alarm conditions for future maintenance of the beach.

In the course of the studies the objectives were extended to investigate the hydraulic performance of shingle barrier beaches of a wide range of geometries, subject to a wide range of wave and water level conditions, thus providing an empirical method to allow the crest processes of these beaches to be quantified.

The study has been divided into three main phases of experimental work.

a) field studies;

b) wave climate studies;

c) 3-dimensional physical model studies.

Field work

The main focus of the studies has been on the hydraulic performance of Hurst Spit, UK. The spit which is approximately 2.5 kilometres long, protects an extensive area of low lying land, and has a long history of rapid evolution (Nicholls,1989). Reduced shingle supply to the spit by longshore transport has resulted in a declining beach volume. Consequently, the frequency of overtopping events has increased. The features which result from wave run-up and the overtopping processes discussed in this paper have all been observed, and detailed measurements of beach profile response have been recorded on a regular basis. The field work and wave climate studies were carried out at Hurst Spit, UK, over a five year period between 1987 and 1992.

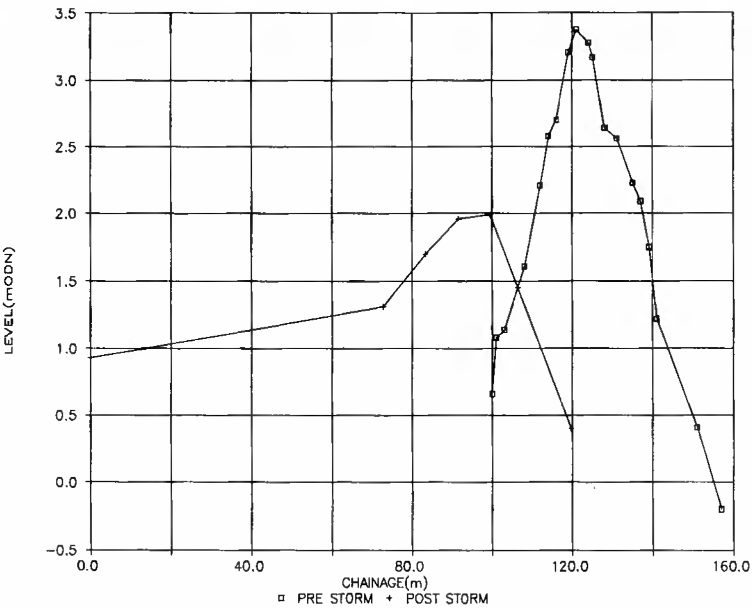
The beach is surveyed quarterly and also following severe storms along 20 profile lines, at approximately 100m intervals. Wave conditions and tidal conditions have also been monitored at this site, thus allowing the beach response to be quantified with respect to the hydraulic parameters.

The beach is characterised by a coarse grain size D_{50} of about 16 mm. The beach crest is generally between 2 - 4 metres above mean high water, with a crest width of 3 - 10 metres. Wave rider records indicate a nearshore wave climate with storms regularly occurring with significant wave heights in excess of 2.5 metres. The complex offshore bathymetry results in different incident wave conditions along the length of the spit. The beach profile response is therefore different at each profile position. The significant wave height of the 1:100 year return period storm varies from 1.8m to 3.5m (in 7 metres of water), from east to west.

The storms of the winter of 1989 have provided data of some considerable interest. Large scale overwashing of the spit occurred in December 1989, resulting in crest lowering along an 800 metre length of the beach. The seaward toe of the beach was rolled back by as much as 60 metres whilst the leeward toe was moved by as much as 80 metres in a single storm event. Crest lowering in the same event resulted in reduction of much of the beach below the level of the peak water level of the storm, a reduction in crest level by as much as 3 metres. Whilst offshore wave conditions were only moderate during this storm, an extreme tidal level resulting from a storm surge at the peak of the storm resulted in a very low freeboard and increased wave conditions at the toe of the beach. This dramatic event provided valuable data about the response of Hurst Spit under extreme conditions and provided excellent data for calibration of the physical model. More severe wave conditions monitored on other occasions at lower water levels also resulted in overtopping and crest rollback, thus providing calibration data for events closer to the threshold of crest roll back.

Further field work is ongoing to provide additional validation of the empirical methods developed in the laboratory and to assess performance of the proposed beach renourishment, which has been designed in conjunction with these studies. The conditions that occurred during the field work programme provided the rare opportunity to carry out concurrent measurements of waves, tides and beach profiles, for a series of severe storms. Whilst the field studies provided a considerable amount of quantitative information about the processes, the restricted number and random nature of the natural storm events provided insufficient data alone to generate a statistically valid method for the prediction of the beach crest evolution. It was therefore necessary to examine the beach response in a more controlled environment to enable threshold conditions to be defined.

Overwashing of Hurst Spit in December 1989



Profile response of Hurst Spit to storm of December 1989

Physical Model Tests

Recent developments in physical modelling techniques for shingle beaches have provided an appropriate method of study. The use of distorted sediment scaling to produce model beaches with appropriate permeability and sediment movement characteristics to reproduce the correct profile response of the beach has been discussed by Powell (1989). Since detailed physical model studies had not however previously been used to examine the profile response of overtopping shingle barrier beaches, it was necessary to carry out a series of calibration tests to prove the validity of the modelling techniques for this type of beach.

The model tests were conducted in a 3-dimensional wave basin under random wave conditions. The model was constructed at a scale of 1:40, with an active beach frontage of 1000 metres. Model variables included a range of significant wave heights from 1 - 4.5 metres, wave periods of between 6 - 12 seconds and four static water levels. The four water levels provided a variable freeboard between the beach crest and the static water level and also provided the opportunity to examine the effects of varied breaking wave conditions at the toe of the structure. Many of the tests were carried out with normally incident waves, although a large number of tests were also carried out with the beach subject to oblique wave attack, within the range 5-15°. The material used to represent the model beaches was a graded anthracite, scaled to reproduce both the correct beach permeability, threshold and direction of sediment motion and thereby the correct profile response of the beach.

A number of hydraulic variables were kept constant throughout the test programme. These included sediment size and grading, spectral shape (JONSWAP), and storm duration.

The test programme was divided into three phases

a) validation of the test methodology

b) evaluation of the hydraulic performance of Hurst Spit

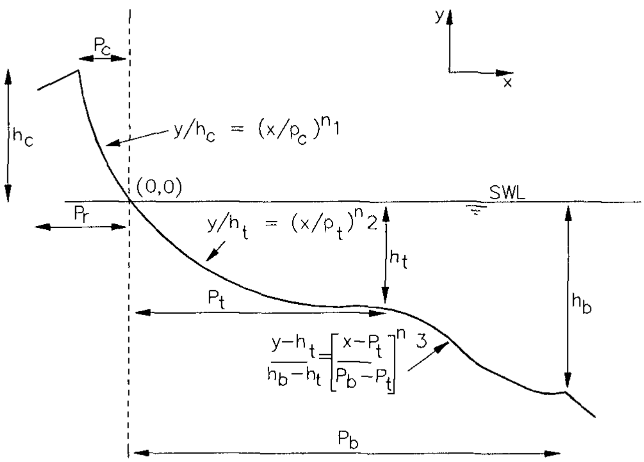
c) evaluation of the hydraulic performance of an extensive range of barrier beach geometries.

Field data collected from Hurst Spit provided the basis for the design of the first two phases of laboratory tests, ensuring that model beaches, foreshore geometry, sediment size characteristics, wave and water level conditions were all modelled correctly. This provided a typical structure for validation of the test methodology and for examination of the processes identified by field observations.

The test conditions were extended to include a wider range of variables as the studies developed. The third phase of testing provided a means of examining a much wider range of conditions and beach geometries, which might be applicable to sites elsewhere. A wide range of wave conditions, water levels and shingle bank geometries have been examined in a systematic series of tests on the profile response of an extensive range of beach geometries. In excess of 3000 beach profiles were measured in more than 200 physical model tests. Concurrent measurements were also made of wave and water level conditions

The seaward slope geometry has been described by the three stage schematised shingle beach profile proposed by Powell (1990). Additional beach geometry characteristics appropriate to barrier beaches have been defined in terms of beach cross section area, crest level, crest width and beach span at each of a number of beach levels. The primary geometric variables which have been used in the analysis of the beach profile performance are the beach crest level and the crest width. A range of freeboards from 0.5 metres to 6 metres were examined, with beach crest widths ranging from 2 metres to 20 metres.

Pre and post storm profiles were recorded by surveying the beach before and after each test. Profiles were recorded using a computer driven incremental bed profiler, sampling levels at a constant survey interval of 2 metres along each profile line. A number of profiles were recorded along the beach frontage for each test. This provided the basis for the examination of the spatial variation of the beach performance. The tests were generally run for a prototype duration of 3 hours. Video recordings were made of each test from an aerial camera. These recordings were later used in the analysis of the evolution of the crest features.



Schematised shingle beach profile (after Powell, 1990)

Results

The physical model was validated by reproduction in the model of severe storms that occurred on Hurst Spit in 1989. Field measurements of wave and water level conditions during the storms, and beach profiles monitored before and after the storms have provided the basis for the comparison of the performance of the physical model with Hurst Spit. The results of the physical model provided a remarkably good reproduction of the profile response of Hurst Spit to these storms and demonstrated the validity of the modelling methods for this type of structure.

Observations made during the test programme provided an explanation for many of the features formed during the tests. A number of threshold conditions were identified, both visually and by parametric analysis of the beach profile response to various hydraulic conditions. The first threshold condition occurs when the combination of wave conditions, water level and beach geometry causes crest accumulation. This process is described well by earlier studies (Powell,1990) which have quantified wave run-up and berm formation. Crest accumulation occurs when waves reach the crest with sufficient energy to spill onto the crest, but with insufficient energy to wash the whole of the way over the crest. The hydraulic conditions giving rise to this process are confined to a very narrow band for any given beach crest width. The validity of the run-up formulae are further restricted by the initial beach crest width which may prevent the predicted beach profile from being reached, if the combination of crest position and crest height require a wider beach to achieve the dynamic equilibrium profile. The formulae do however provide an excellent method of defining the lower boundary conditions for thresholding of damage by overtopping, by reference to both the initial profile and the post storm predicted profile. An appropriate alarm condition for each profile geometry can therefore be derived by reference to the conditions causing crest accumulation. This method has been used to provide an empirical assessment of the alarm conditions for Hurst Spit, thus providing management guidance on the maintenance requirements following beach renourishment. Similarly the threshold geometry for given wave and water level conditions can be calculated to provide the required profile to withstand any event. Accumulations of material at the crest of up to 1.2 metres were measured during the model tests.

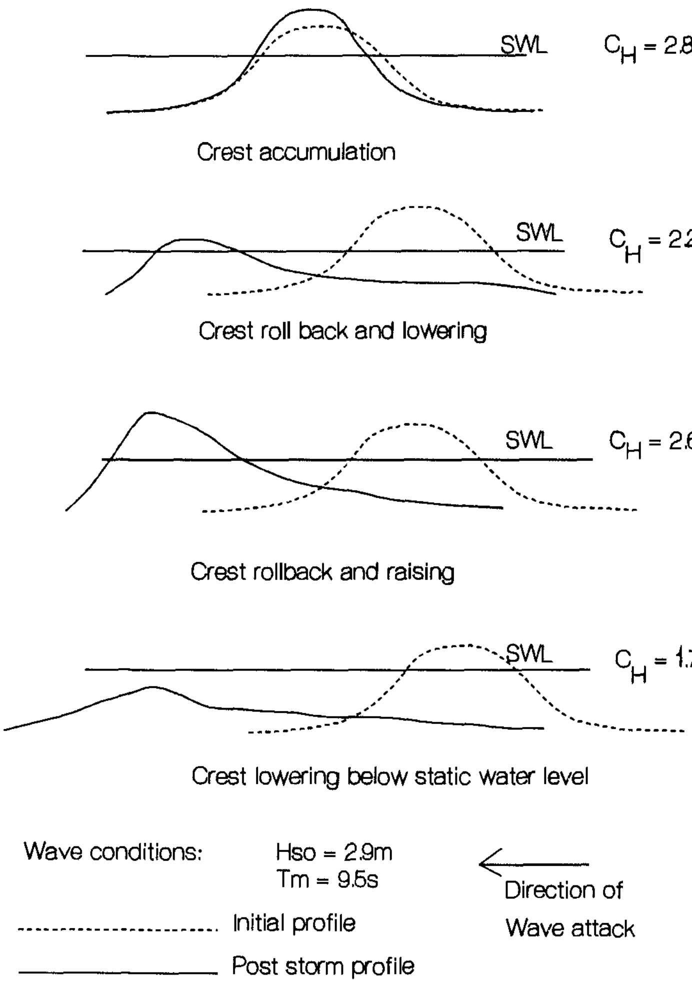
A further threshold, resulting in crest lowering, is reached if the combination of wave and water level conditions and beach geometry reach the lee face of the beach. The numeric definition of this threshold is however somewhat more complex, as there is a strong dependency on the crest width as well as the wave conditions. Tests indicated a wide spatial variation in the conditions that resulted in crest lowering and also some scatter in the results of profile response close to this threshold. The spatial variation may be a function of very small localised variations in the initial beach geometry. The beach crest is extremely sensitive to small changes in freeboard and crest width and caution should therefore be exercised when assessing the vulnerability of the beach to crest lowering.

Process development observed in the model helps to explain some of the scattered results measured close to the crest lowering threshold. Small washover fans normally form at first, unless the storm water level is very extreme. These are followed by the formation of more extensive breaches through the bank as the storm progresses, resulting in large scale crest lowering. The size and method of formation of a washover fan is dependent upon the crest width, crest level and wave and water level conditions. If the crest is very narrow a wide breach may form quite rapidly, as waves are able to overtop the bank and lower the crest quickly. If however the crest is rather wider, the rate of formation and size of the overwash fan will be significantly slower and smaller. Once the crest level has been lowered the rate of crest lowering increases rapidly, as the frequency of waves reaching the crest increases. The width of the initial overwash fan ranged in size from approximately 4 metres to 25 metres wide, depending on the shingle bank geometry immediately prior to overtopping. Similarly, the length of the initial fan could extend landwards from approximately 1 metre to 15 metres. The rate of landward progression of the overwash fan is also strongly dependent on the topography of the land in the lee of the beach. Following from the initial formation of an overwash fan the beach plan shape may evolve very rapidly. The fans are quickly widened by outflanking of the upstanding ridges formed on either side of the fan, by overtopping waves. The fan itself provides a preferential flow path for the overtopping waves and a gully forms through the centre of the fan. As the waves flow across the mound the sediment fan is liquified and flows with the wave to the lee side. The fan may then extend landwards very rapidly, moving as much as 10-15 metres in a single wave. As the fan reaches the base of the shingle bank it tends to spread and forms an enlarged head area which accumulates as more material is washed across the crest.

The wave grouping following the initial fan formation is also extremely important. If the waves following the initial fan formation are small, a ridge may form seawards of the crest ridge. This in turn is pushed upwards towards the crest, and can prevent further overtopping. In many instances the wave grouping enables a fan to form, but permits the initial breach to heal. This demonstrates the need to study these processes with random as opposed to regular waves.

As the storm progresses, the width of the shingle bank is reduced, due to the beach attempting to reach a dynamic equilibrium profile for those conditions. Undermining of the beach crest occurs and the crest width is consequently thinned. This may occur even when waves do not reach the crest of the shingle bank, thus demonstrating the importance of both the span and the crest level of the shingle bank. In certain instances the bank can be cut back to such an extent that a breach may form as a result of waves breaking through a narrow ridge as opposed to the more normal failure mechanism occurring by overtopping.

In addition to overtopping, percolation of water occurred through the permeable model shingle bank, resulting in the formation of streams and washout fans on the lee side of the spit. This process has been noted at a number of locations on Hurst Spit. This condition occurs most commonly where the spit has been cut back by wave action to a small cross section profile, but where waves are too small to reach the crest of the beach. The fact that this process was observed in the model serves as an additional marker that indicates that the physical model is able to reproduce the permeability of the bank correctly.



Profile response of model barrier beaches

Parametric analysis

Many of the test conditions resulted in the formation of a run-up berm below the level of the beach crest. The measured post storm profiles for these conditions were initially compared with the functional relationships derived by Powell(1990) in earlier studies on restrained shingle beaches. This comparison confirmed the validity of the parametric framework for these conditions, with the results of the studies falling well within the error bands suggested for each of the functional relationships. The coarse grain size resulted in the formation of a steep upper beach slope below the crest berm. The parametric framework maintained validity even as the run-up level increased above the original beach crest, provided that the beach crest width was wide enough to allow deposition of beach material at the crest. The functional parameters to describe crest height (h_c) and crest position (p_c) given below however, became unstable as the wave and water level conditions became more severe, or as the crest width became narrower.

$$p_c D_{50} / H_s L_m = -0.23 \left(H_s T_m g^{1/2} / D_{50}^{3/2} \right) - 0.588$$

$$h_c / H_s = 2.86 - 62.69 \left(H_s / L_m \right) + 443.29 \left(H_s / L_m \right)^2$$

These equations are corrected for breaking wave conditions by use of the correction factors:

$$p_c \quad 3.03 \left(H_s / D_w \right) + 0.12$$

$$h_c \quad \left(H_s / D_w \right) + 0.41$$

The crest evolution changes from accumulation to crest lowering when the wave and water level conditions in combination with the beach geometry cannot reach a dynamic equilibrium profile within the existing beach cross section. There is therefore a complex relationship between the hydraulic parameters and the beach geometry which defines the threshold of overwashing resulting in crest lowering. This relationship is very sensitive to small changes in freeboard and crest width conditions.

The effects of each of the hydraulic variables are examined in turn. The main influence of the significant wave height is on the crest position (p_c). As H_s increases, the horizontal distance from the beach and the static water level intersection (0,0) is increased. If the dynamic equilibrium position for the wave conditions lies further landward than the width of the unconfined beach crest, the dynamic equilibrium crest position cannot be achieved and the beach crest will be breached, resulting in crest lowering. The situation is further complicated by the other hydraulic variables which in certain combinations may allow the beach crest to reform landwards of its original position.

The effect of increasing the wave period is to raise the crest level of the beach. If there is insufficient beach material available within the crest however, a raised crest will fail to form and overwashing will occur. The rate of overwashing increases dramatically with increasing period, due to the larger volume of water in longer period waves. Both the wave height and the period variables may reach a threshold condition whereby the beach is overtopped without the crest reforming above the peak run-up level. In this instance crest lowering may occur.

The effects of static water level are twofold. Firstly there is a simple relationship which reduces the exposure of cross sectional area of the beach above static water level, as the water level rises. In this instance the dynamic equilibrium profile moves further landwards as the freeboard is reduced. There clearly comes a point where the freeboard becomes so small that virtually any wave condition may pass over the crest, resulting in rollback and crest lowering. The second relationship is more complex and relates to the breaking wave height at the toe of the beach. This clearly results in larger incident wave conditions at the shoreline as the water depth increases, for the same offshore conditions. Wave run-up is therefore increased. This is reflected by the adjustment factors for wave breaking given in Powell's model.

The test results were very sensitive to small changes in the model test variables and these have resulted in some scatter close to the threshold conditions. The effects of wave grouping appears to have a significant effect on the crest evolution and this appears to contribute to the lack of consistency of certain of the results whilst close to the threshold conditions. Whilst wave grouping was not quantified, it is certainly a feature which must be considered. Observations made during testing identified a number of instances when the beach would be overtopped, causing crest lowering in a period of severe wave activity. Later the same beach section would heal during the same test during periods of less intense wave activity, occasionally resulting in the formation of a higher run-up crest than the initial profile.

Further examination of the data resulted in the formation of a new parametric framework designed to examine the vulnerability of the beach to overtopping and rollback, and to define damage thresholds. Whilst Powell's model of profile response works well within a range of conditions, it becomes more unstable as the freeboard is reduced, the wave period is lengthened or the wave height is increased. The influence of the beach crest width is clearly very important but further work is required before the effects of this variable can be quantified with statistical validity.

The framework proposed for the analysis in this study considers the beach as a simple geometric structure subject to variables defined by freeboard, shallow water breaking wave height and shallow water wave length. A critical freeboard parameter C_f is proposed. The formula suggests limits for the safe use of Powell's earlier studies and suggests an inundation threshold value beyond which the level of the beach will be reduced below the static water level of the storm peak, for the range of conditions tested in these studies.

The threshold formula is given by:

$$C_f = \frac{C_H}{(H_{sb}^2 \; L_{ms})^{1/3}}$$

where C_f is the critical freeboard parameter

H_{sb} is the shallow water breaking wave height calculated from:

$$H_{sb} = 0.12 \; L_m \; [1.0 - \exp(-4.712 \; D_W \; (1.0 + 15m \; 1.33) \; / \; L_m)]$$

L_{ms} is the shallow water wave length given by

$$L_{ms} = T_m \; (g \; D_W)^{1/2}$$

C_H is the freeboard from static water level to the beach crest

When $C_f > 0.7$ no crest lowering will occur and Powell's parametric framework is valid for profile prediction

and where $C_f < 0.1$ and C_w (crest width) < 20m inundation of the beach will occur, lowering the beach crest below the storm peak static water level.

where $0.7 < C_f < 0.1$ the beach may respond by crest lowering or crest accretion, depending on the beach crest geometry.

The detailed beach crest evolution is clearly more complex than these simple equations can quantify at this stage . Further crest data analysis is required to refine the findings of these studies which have provided a first step towards quantification of the crest evolution of a barrier beach.

Conclusions

The processes resulting in crest development of shingle barrier beaches have been examined in field studies and a 3-dimensional physical model, and threshold conditions for each stage of development have been defined. The tests have indicated that the profile response of the crest of shingle banks is very sensitive to small changes in freeboard or wave conditions, when close to the critical condition which results in either raising or lowering of the crest level. A range of combinations of wave and freeboard conditions have been identified that result in either raising of the unconfined crest level by overwash deposition, or a reduction in the crest level due to wave overtopping. The complex form of the overwash features results in a wide spatial variation. Several types of feature including throat confined overwash fans have been examined in the model The crest width has been identified as an important parameter in determining the response of the crest geometry to wave action.

The physical model has been used as a design tool for a beach renourishment scheme at Hurst Spit and has provided alarm conditions and damage thresholds for the proposed beach renourishment.

The tests discussed in this study complement the results of an earlier study (Powell,1990) to predict the profile response of shingle beaches to wave attack. Data has been compared with the parametric empirical model of shingle beach profile response. A critical freeboard parameter has been defined together with suggested limits for the applicability of Powell's parametric model of beach profile response to shingle barrier beaches.

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