Joint Defra/EA Flood and Coastal Erosion Risk Management R&D Programme

Understanding barrier beaches

R&D Technical Report FD1924/TR

Environment Agency

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Understanding Barrier Beaches

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Statement of use
This report scopes the state-of-the-art understanding of barrier beaches. It sets the foundations for improving the science and application of management tools, and for developing sound management guidance. Such guidance will help flood and coastal erosion risk management policy makers and practitioners make better decisions on barrier beach management practice. It should be noted that it does not constitute official government policy or guidance but provides information for understanding the geomorphological, physical and morphodynamic processes influencing barrier beach evolution and management. It will be of particular relevance for coastal managers and researchers engaged in enhancing understanding and management of these dynamic features.

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Executive summary

Barrier beaches around the UK are important, not only in terms of defences against flooding but also in their own right as important coastal geomorphological features. A lack of detailed understanding of how these beaches evolve and of models to predict their performance as flood defences, together with constraints on acceptable methods of intervention, make the successful and cost-efficient management of barrier beaches a challenging task for coastal managers. The need for better management of such beaches as both flood defences and natural heritage areas will inevitably increase in the face of rising sea-levels.

The essential feature of a "barrier beach" is that it has a distinct crest separating the seaward beach face and a well-developed back-slope. In many cases, such beaches have (or once had) an area of water on their landward side, whether an estuary, lagoon or brackish-water. Beaches with these characteristics may be further sub-divided into "barrier islands", "spits" and "barrier beaches", depending on whether they have none, one, or both of their ends attached to a land mass. Distinctions such as this, however, can become blurred although this may not be particularly important from the viewpoint of managing such features, as opposed to the geomorphological approach to describing and classifying them.

Barrier beaches often form a natural flood defence to low-lying land behind them. However, man's chosen land-use, e.g. for residential and business properties, can often mean that the natural standard of defence afforded by the barrier is inadequate. Barrier beaches are often overtopped by large waves, they leak, can roll-back landward, and ultimately may breach. All of these events can give rise to unacceptably high flood-risks, and are likely to become more frequent as sea levels rise further. These flood-risks can justify intervention to improve the standard of protection that barrier beaches provide, but the natural heritage interests of such barrier beaches can constrain what type of intervention is acceptable.

At present there is scant guidance available which enables a balance between intervention and natural heritage interests to be achieved, and coastal managers are sometimes left to struggle through on a trial and error basis when seeking solutions.

This study has provided justification and scope for further research into the performance of barrier beaches as flood defences. The Performance-based Asset Management Systems (PAMS) research programme, funded by Defra and the Environment Agency, has furthered performance-based flood-risk assessment through, for example, the concepts of fragility, resilience and deterioration. PAMS is specifically designed for the identification and prioritising of works needed to manage existing flood defences. Part of the reason for the lack of management guidelines for barrier beaches is our relatively poor understanding of the processes driving their short-term morphology and long-term evolution. The recommendations for further research, and the further research itself, will be embedded in the Environment Agency's Sustainable

Asset Management theme. As such, the proposed research, whilst still firmly and unavoidably centred upon improved process understanding, is placed into the context of performance-based flood-defence management. Ultimately, the research is expected to provide the framework for the evolution of a “Best Practice” guide.

The main outputs of this study include:

- A review of scientific literature and of existing predictive process methods for overtopping, through-flow and morphological change of barrier beaches has been carried out.

- A review of barrier beach management methods has been carried out in consultation with individuals responsible for management of the beaches. This review includes discussion on monitoring and appropriate analysis. Case histories detailing state-of-the-art at 12 selected sites have been presented.

- A catalogue providing site-specific information on barrier beaches around England and Wales, including an on-line GIS database that provides information relating to these beaches. Details such as description, location, dimensions, current management practice, photographic record and links to further information, are consequently held on one publicly available website www.barrierbeaches.org.uk. This web-site is also used to disseminate the findings of this scoping study.

- Recommendations for a future phased-research programme within the Environment Agency’s Sustainable Asset Management theme have been made. This programme is designed to enhance our knowledge and understanding together with our predictive capabilities. It will include monitoring their condition, performance and the flood-defence standard that they offer, as well as develop predictive tools. Ultimately a “Best-Practice” guidance for cost-effective and environmentally acceptable management of barrier beaches as natural flood defences will be produced.

Contents

Tables

Figures

Plates

1. Background and objectives of study

1.1 Statement of problem

Barrier beaches are wide spread around the coast of the UK. They are subject to rapid and large-scale changes in morphology, during episodic extreme storm events. Overtopping and overwashing may occur in such events, resulting in large scale flooding. Limited flood forecasting techniques are available to aid their management, yet these are often high-risk structures in context with flooding (Plates 1.1 and 1.2).

Plate 1.1 Salthouse flooding 1996 (Steve Harris)

Plate 1.2 Chesil beach overwashing at Chiswell 1979 (http://www.swgfl.org.uk/jurassic/chiswell.htm)

Barrier beaches have an important role as geomorphological features in their own right; they are often part of a coupled system, providing natural protection to extensive areas of wetlands. In some instances large scale rollback of the barrier can occur (Plate 1.3) with displacements of 80-100m occurring in a single storm event. This can cause instantaneous loss of large areas of environmentally important intertidal areas and may also have a dramatic impact on built infra-structure (Plate 1.4).

Plate 1.3 Hurst spit roll back 1989 (AP Bradbury)

Plate 1.4 Erosion of Slapton coast road - 1999 (Peter Wagstaff) http://www.saveslaptoncoastroad.co.uk

Occasionally, the formation of a permanent or semi permanent tidal breach may occur (Plate 1.5) when the barrier reaches a stage of breakdown; this may cause significant changes to the morphological and ecological evolution of the area to landwards.

Plate 1.5 Porlock tidal breach formation 1998 (Bray and Duane, 2001)

A wide variety of management techniques are used including recharge, recycling, beach scraping (Plate 1.6) and hard structures.

Plate 1.6 Beach scraping at Medmerry (Environment Agency)

1.2 Objectives of this research project

The objectives of this project are set out in contract CSA 6932 and are repeated below:

1. To undertake a scoping study to assemble and assess current understanding of barrier beach design.
2. To define the need for further research.

In particular, the research has specifically addressed the following:

a) threshold conditions for overwashing;
b) causes of breaching;
c) through-flow;
d) prediction of the profile development of the beach following the onset of overwashing;
e) processes that may result in the natural rebuilding of the crest;
f) quantification of overtopping rates;
g) quantifiable modification of the crest and back barrier by overtopping and overwashing; and
h) review field performance of a range of key sites.

In addition, the project was expected to:

a) gather information on and report current management methods;
b) estimate costs of current barrier beach management practice; and
c) understand and report constraints on management.

The work has also established a framework of studies to support better-informed and more effective management of barrier beaches.

Though not considered explicitly, barrier beach design is considered implicitly throughout the report, particularly through discussion of processes, geometry, modeling and management techniques.

During the course of this study, it was proven very difficult to estimate the costs of current barrier beach management practice. Coastal managers were generally unwilling, or perhaps unable, to contribute cost estimates to the scoping study. Because of this, that particular aim was not achieved.

It was recognised at an early stage in this scoping study that our general understanding of the performance and condition of sandy beaches (and barrier beaches) is far greater than our understanding of the performance and condition of mixed sediment and shingle beaches. Because of this, a strong emphasis was placed on the scoping of knowledge and application of techniques to mixed and shingle barrier beaches. Until such a balance of understanding is achieved, or close to being achieved, it is recommended that effort be concentrated in this vein.

1.3 Layout of report

This report consists of seven chapters.

1. Background and objectives of this study (this chapter)
2. Review of process understanding. This Chapter provides a thorough review of the current level of understanding of the processes associated with barrier beaches and spits. This is a detailed and technical chapter which also explains much of the terminology used with barriers. This Chapter highlights the processes which are reasonably well understood, as well as those which are not.
3. Methods of study. This Chapter presents a series of methods that are routinely applied to aid the study of barrier beaches, and highlights their relative infancy.
4. Application of state of the art. This Chapter presents a summary of how state of the art understanding is applied through modelling techniques. It highlights a low level of practicality.
5. Case histories. This Chapter provides review of a range of barrier sites around the coast. The case histories highlight some sites which have been well-studied, as well as some sites which have been less so.
6. Review of current management methods. This Chapter explores the management methods practiced on barrier beaches as summarized in the case histories, for example. It also provides a review of, and some guidance on, monitoring methods.
7. Conclusions and research needs.

2. Review of process understanding

This Chapter provides a thorough review of the current level of understanding of the processes associated with barrier beaches and spits. This is a detailed and technical Chapter which also explains much of the terminology used with regard barriers. Processes which are relatively well understood, as well as those which are not, are highlighted and described. The Chapter culminates in a brief summary which specifically addresses areas suggested for further research.

2.1 Barrier beaches – definitions

In order to discuss what does, or does not constitute a barrier beach, both for this specific study and as defined in other studies of such features around the world, it is useful to first introduce some explanation of the terms used by geomorphologists. By clarifying these specialist terms, and relating them to the ideas and terminology used in coastal engineering, it is hoped that the relevance of many previous reports and scientific papers dealing with such beaches can be made more relevant and useful in the context of their management by coastal engineers.

2.1.1 Barrier beaches and spits

The essential feature of a "barrier beach" as defined in this study is that it has a distinct crest separating the seaward beach face and a well-developed back-slope. In many cases, such beaches have (or once had) an area of water on their landward side, whether an estuary, lagoon or brackish-water. Beaches with these characteristics may be further sub-divided into "barrier islands", "spits" and "barrier beaches", depending on whether they have none, one or both of their ends attached to a land mass. Such distinctions, however, can become blurred, for example given a narrow beach that periodically breaches allowing water to pass into the area to landward. Further discussion of these distinctions is presented later in this section, although these may not be particularly important from the viewpoint of managing such features, as opposed to the geomorphological approach to describing and classifying them.

Figure 2.1 summarises some of the alternative plan-shapes of barrier beaches, spits, and barrier islands, as suggested by Cope (2004); it has been devised from a combination of various previous papers suggesting classifications (Carter and Orford, 1991; Forbes *et al.,* 1995; King, 1972; Swift, 1976; Zenkovitch, 1967) together with a literature review and field observations.

A barrier beach may be attached at both ends (see Group 1 of Figure 2.1) or remain offshore (see Group 3 of Figure 2.1), whilst a spit is attached to the coastline at its proximal end and detached at the distal end (see Group 2 of Figure 2.1). Cope (2004) notes that morphology alone cannot always be relied upon to indicate the origin of these forms. There are many instances where there is no clear distinction between barriers and spits as each may exhibit features common to the other and may alter rapidly from one to the other. This has resulted in conflicting and contradictory terminology (Price, 1951; Riddell *et al.,* 1998).

Because of this, it is important to establish whether or not a beach experiences a long-term net longshore drift of sediment, both from a geomorphological viewpoint and, more crucially, from a beach management perspective.

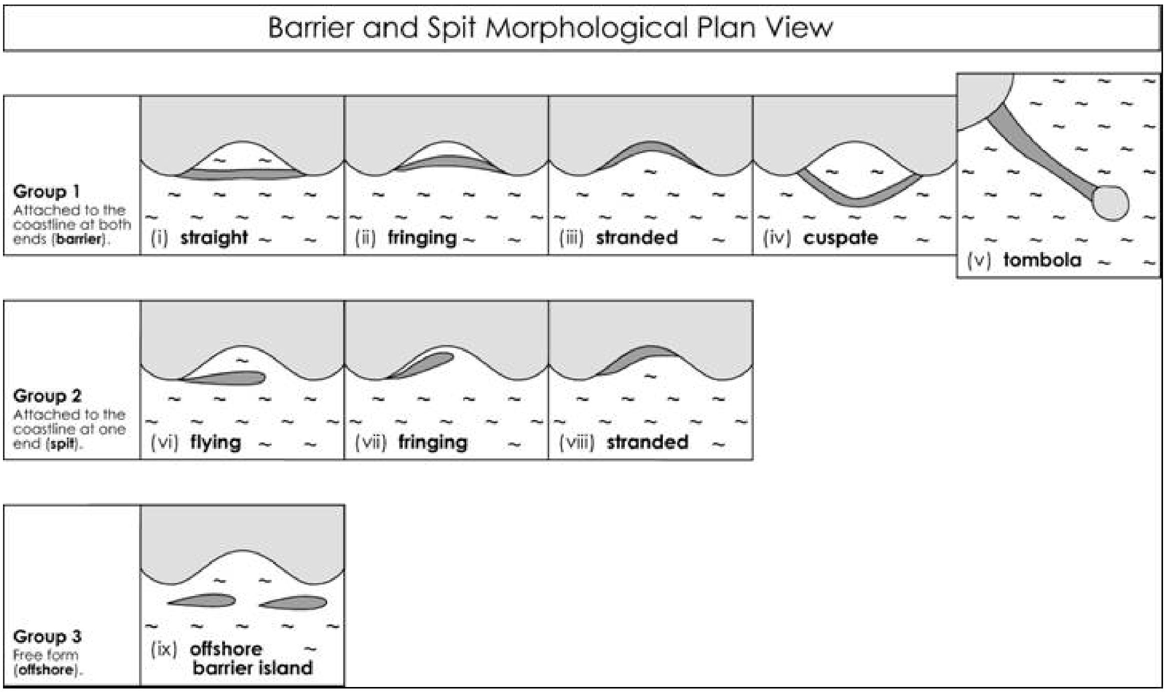


Figure 2.1 Plan shapes of barriers, spits and barrier islands (Cope, 2004)

2.1.2 "Drift aligned" or "Swash aligned"

The plan shape of any beach, and hence its orientation or alignment at any location, tends to vary with time, in response to changing wave conditions. In general, however, a beach will retain a fairly constant alignment in the long-term at any location unless the amount of sediment travelling along it is altered, either by human intervention or, more rarely, as a result of a change in the wave climate. This is true even if it gradually recedes landwards, or advances seawards.

Swash aligned

Where there is no possibility of any long-term addition from or loss of sediment to adjacent sections of coastline, for example in deeply indented pocket bay beaches, the beaches tend to adopt a plan-shape that ensures a zero net longshore sediment transport, at least when averaged over a long time period. Geomorphologists refer to such beaches as "swash aligned" and concentrate on the movements of sediment perpendicular to the beach contours, i.e. onshore-offshore transport as being the mechanism that brings about long-term changes in such beaches. Beaches that are strongly curved in plan shape are usually "swash aligned" although not all "swash aligned" beaches are curved.

In reality, beaches can have a zero net longshore drift even if their contours are not aligned with the crests of the breaking waves, for example in the lee of a breakwater or offshore island where there is a longshore gradient in wave heights. Even in this case, however, the term “swash aligned” tends to still be used, although there may be a substantial angle between the breaking wave crests and the beach contours.

It is worth making the point that in many situations, changing wave directions will produce a longshore drift along swash-aligned beaches, during the period taken for the plan-shape shape to adjust to the new circumstances. This behaviour is common on pocket beaches, for example in Cornwall, exposed to waves arriving from a wide range of directions. Because of this, “swash aligned” should only be interpreted in a long-term sense, i.e. over decades.

Drift aligned

In contrast to “swash alignment”, many beaches experience a persistent, i.e. long-term, net longshore drift of sediments and their plan shape, and hence alignment, is determined by and maintains the variations in that drift rate along the coastline. Such beaches are termed “drift aligned” by geomorphologists. To maintain their alignment, such beaches need a continuing supply of fresh sediment at their updrift ends, e.g. from an eroding cliff or (overseas) a major river.

The long-term evolution of such beaches will normally be dominated by changes in longshore drift rates, caused for example by variation in the sediment supply or in wave conditions. Onshore-offshore sediment transport processes still occur but are of secondary importance to the evolution of such beaches. If there is a continuing supply of fresh sediment to a beach, it will be more readily able to adjust to sea level rise or an increase in the heights of incident waves, e.g. by increasing its crest height, than will a beach with a fixed volume of sediment.

When considering spits, which in this study are considered as one member of the broad family of “barrier beaches”, the direction of sediment transport is usually, but not always, from the “proximal” end of the spit, i.e. where it attached to the main landmass to the “distal” end, i.e. the “free” end of the beach, which is often broader, and has a curvature inland.

2.1.3 Beach sediments

Barrier beaches may be formed of sand, gravel, pebbles and boulders or a mixture of these sediments. Most of these are typically derived as fragments of rock (at least in the UK) and are often referred to as “clasts” by geomorphologists. Thus a “coarse clastic” beach is used as a term denoting what coastal engineers would tend to call a beach of shingle, pebble or boulders. These types of beaches are normally found in areas of the world where, in previous Ice Ages, glaciers have produced and transported such sediments to the coastline. In parts of the world with no such glacial inheritance, barrier beaches still occur, but are of sand, i.e. “fine clastic” barriers. These may or may not have dunes on their crest.

2.2 Review of geomorphological classifications

The classification of spits and barriers has been addressed by many geomorphologists in the past. This section summarises a review by Cope (2004) of the approaches taken, and presents a table that suggests a characterisation of “barriers” and “spits” used in the remainder of this report.

Zenkovitch (1967) defined coastal depositional features on the basis of morphology and sediment sources, whilst King (1972) adopted a purely morphological approach to classification. King (*op. cit*) differentiated between spits and barriers, by defining the latter as being attached at both ends, whilst Zenkovitch (*op. cit*) categorised both into a single grouping. Swift (1976) and Carter and Orford (1991) have suggested similar definitions of beach-and-barrier coasts, by subdividing according to their plan shape: drift aligned and swash aligned coasts.

Barriers may be subdivided into swash- or drift-aligned structures (Orford and Carter, 1991, Orford *et al* 1995), depending upon their orientation relative to the incident waves. In the former paper, the authors suggest that drift-aligned coasts form when longshore processes dominate over cross-shore processes: swash-aligned beaches, in contrast, undergo little longshore development.

Barriers may take a number of forms, including recurved ridges which form adjacent to tidal inlets; alternatively, they may be attached to solid geology at either end, protecting brackish lagoons. Whitten (1972) defined a spit connecting two sides of a bay as a bar: such a feature is termed a barrier beach in this study. The definition of a spit adopted by Horn *et al* (1996) is “a detached beach that is tied to the coast at one end and free at the other, with a free end that often terminates in a hook”. Forbes *et al.,* (1995) classify barrier origin and evolution but only for barriers formed from a point source (spits).

As an example of the inconsistency that can arise, spits confining inlets, such as Pagham Harbour have been defined as bars by one author (Whitten, 1972) and as barriers by another (Bradbury, 1998). Hails (1982) uses the terms “barrier beaches, barrier spits, or barrier islands,” according to their particular morphological features, which is an effective distinction, but is based on fine-clastic depositional forms.

In response to this often confusing and contradictory terminology and classification, Carter *et al.,* (1987) call for a more analytical approach to barrier development, “to reinforce the many existing descriptions of barriers around the world”. A more recent classification (Cope, 2004) adopts a morphological approach but is based on plan-view, swash/drift alignment and origin**.** It has been developed from a combination of previous classifications outlined above. This classification provides a temporal and spatial context for the barrier beaches and spits under direct study and ensures consistency when referring to different morphological types. A summary of the main morphodynamic differences between natural barriers and spits is provided (Table 2.1). The aim is to provide a consistent differentiation of forms for future studies and to avoid past contradictions. It should be noted however, that the distinctions are idealised, whereas in reality transitional or intermediate forms can sometimes occur, e.g. during processes of barrier breaching or inlet sealing (Cope, 2004).

Table 2.1 Diagnostic morphodynamic features of barriers and spits (Cope, 2004)

Diagnostic feature	Beach type/ form	
	Barrier	**Spit**
Morphology	Linear accumulation of sediment with distinct crest and backslope, which fronts lowland, lagoon or bay. Attached at both ends to the coastline or remains offshore (see barrier forms - Figure 2.1).	Linear accumulation of sediment with distinct crest and backslope, which fronts lowland, lagoon or semi-enclosed bay. Attached to the coastline at proximal end and detached at distal end.
Sediment origin	Offshore/nearshore.	Mainland (point) source delivered by littoral drift.
Current dominant sediment supply	Onshore/offshore.	Mainland (point) source delivered by littoral drift.
Alignment	Swash-aligned	Drift-aligned.
Dominant sediment sorting	Cross-shore.	Alongshore.
Behaviour	Landward migration, overtopping, crest cut-back, over-washing and breaching.	Longshore migration (elongation), over-washing and breaching. Landward migration involving rotation into inlet.
Morphologic features attributed to behaviour	Steep crest due to dominant swash-aligned wave attack that results in overtopping. Flat crest where over-washing has taken place. Inlet formation following breaching.	Seaward and longshore progradation, where sediment supply is abundant or rate of sea level rise has declined. Curvature at distal end. Relatively low crest due to dominant oblique wave attack resulting in wash-over fans and flats and breaching
Geomorphological setting	Low lying coastal plain or bay.	Associated with tidal inlets and estuaries.

Numerous coarse-clastic depositional forms occur along the U.K coastline; these have been classified according to diagnostic features and processes. Table 2.2 provides a summary of publications which have made these classifications.

Table 2.2 Diagnostic features and processes operating on paraglacially derived coarse-clastic barriers (Cope, 2004)

Diagnostic features and processes	**Author**
Sedimentary organisation and tendency towards drift-aligned gravel barriers	Carter and Orford, 1991.
Breaching and sealing of stream outlets through mixed sand and gravel barriers	Carter *et al.,* 1984.
Responses to sea level and sediment supply changes	Carter *et al.,* 1987; Orford *et al.,* 1995*b*.
Gravel barriers, headlands and lagoons in an evolutionary context	Carter *et al.,* 1987.
Long-term morphodynamic evolution	Orford *et al.,* 1991*a*.
Self-organisation and instability	Forbes *et al.,* 1991.
Longshore spacing of overwash throats	Carter, 1984.
Lagoon closure and subsequent overwash	Orford *et al.,* 1988.
Gravel barrier retreat	Orford *et al.,* 1993
Modelling of barrier morpho- and hydro-dynamic interactions with respect to overtopping, overwashing, crest cut-back and breaching processes	Bradbury and Powell, 1992; Bradbury, 1998 and 2000.

2.3 Barrier distribution around the globe

This section summarises a review by Cope (2004) on the distribution and difference between coarse- and fine-clastic barrier-beaches and spits. Coarse-clastic barriers are distributed widely on a global basis, but are especially common in formerly glaciated areas (Forbes and Taylor, 1987; Forbes *et al*, 1993, 1995). Previous research has tended to concentrate on breaching of fine-clastic barrier beaches (Tanner, 1990; Forbes *et al.,* 1991), most of which are located in the low to mid latitudes, rather than their coarse-clastic counterparts (Shulmeister and Kirk, 1993; Forbes *et al*., 1995*a*; Leont'Yev and Nikorov, 1965; Carter *et al.,* 1987; Orford *et al.,* 1991*a*) of mid to high latitudes. Some studies of the evolution, breakdown and breach development of sandy barrier beaches on the eastern Atlantic shores of the U.S.A and other parts of the world is listed in Table 2.3. An understanding of the operative geomorphological processes regulating fine-clastic barrier beaches on the U.S Atlantic and Gulf coasts is useful as comparisons can be made with coarse-clastic barriers, thereby providing a wider context (Table 2.4).

The world-wide occurrence and morphology of individual barrier beaches and spits is very much influenced by continental topography (Inman and Nordstrom, 1971), sediment characteristics, tectonic setting, tidal range, sea level rise and wind/wave circulation patterns (King, 1972). In terms of tectonic and topographic locations, barriers are more prominent along passive, continental margins, characterised by an abundant sediment supply derived from inland sources such as rivers or glaciers. Additionally, a gentle offshore seabed gradient favours landward sediment progradation given a moderately (2-5mma^{-1}) rising sea level (King, 1972).

Table 2.3 Some global studies of barrier breaching (Cope, 2004)

Location	Author
Gulf of Mexico	Penland and Suter (1984).
Eastern Atlantic shores of the U.S.A	Leatherman et al., (1980); Morgan and Stone (1985); Pierce (1970); Suter et al., (1982).
Sebastian Bay, Tierra del Fuego, Argentina	Dujalesky et al., (1991).
Ebro Delta, Spain	Sanchez-Arcilla and Jiminez (1994).
Western Bight of Benin, West Africa	Anthony and Blivi (1999).

Table 2.4 Comparison between fine- and coarse-clastic barriers (Cope, 2004)

	Fine-clastic barriers	Coarse-clastic barriers
Sediment type	Quartz sand and carbonate.	Predominantly shingle and gravel but also sand.
Origin	Predominantly river sediment, also offshore sources such as shell banks and coral reefs.	Predominantly paraglacial sediment from offshore sources, integrated into the barrier system during the Holocene sea level transgression.
Sediment input rates (contemporary)	High	Low
Latitude	Low-mid latitude.	Mid-high latitude.
Beach planform and profile	Often drift-aligned. Ponce de Leon Inlet to St Lucie Inlet, Florida is up to 213 km in length and 4.8 km wide, characterised by a wide, dissipative profile.	Often swash-aligned. 4 - 24 km in length and 0.9 – 2.5 km wide, with a narrow, reflective profile
Profile response to increased wave energy	Relocation of sediment from berm, seaward to form offshore bar/bars and trough/troughs.	Relocation of sediment from beach face to crest or back-barrier area.
Tidal range	Micro-meso	Meso-macro
Wave conditions	Swell	Storm
Percolation rates	Low	High
Landward migration rate	Low	High
Elongation downdrift	High	Low
Inlet development	High, where tide dominated	Low, apart from where sediment supply is short.

Short (1999) notes that passive coasts in the low-mid latitudes are characterised by long, fine to medium quartz sand barrier beaches and spits, where not interrupted by hills and embayments. The barriers are extensive as sediment yields are high, coming from a number of the world's largest rivers, often accompanied by an input of carbonate material from coral reefs.

As can be identified from Table 2.4, barrier beaches and spits in Florida can be up to 213km in length. Both depositional forms are extremely rare on macro-tidal coasts (>4m) (Pethick, 1984; Reinson, 1992), therefore the micro-meso tidal range (King, 1972; Hails, 1982) and swell wave climates of the low-mid latitudes are most conducive to barrier formation.

The U.K coast is characterised by coarse-clastic barrier beaches consisting of paraglacially derived sediment, which comprises units of sand, gravel or cobble, inherited from previous glacial periods, which includes fluvio-glacial, marine glacial and periglacial conditions (Carter, 1982; Orford and Carter, 1984). Paraglacial barrier beaches are most commonly found on mid to high latitude coasts (Forbes *et al.,* 1991; Forbes *et al.,* 1995*a*; Orford and Carter, 1995) within the limits of the last major ice advances (Wisconsinan, Devensian) to the present day ice caps (Carter *et al*., 1987).

The supply of fine-grained suspended sediment decreases on mid-high latitude coasts due to there being fewer major rivers (Short, 1999) and lower rates of weathering. Therefore, barriers in the mid-high latitudes are usually coarser grained, smaller, shorter and steeper, than those of the low-mid latitudes and are often attached to the land, rather than remaining offshore. In the mid-high latitudes, sediment availability rather than sea level fluctuation (isostatic and eustatic) is predominant in influencing barrier evolution, whilst in the low-mid latitudes; tidal range and wave climate are dominant in influencing barrier evolution due to the abundance of sediment supply.

2.4 Characteristics of barriers

2.4.1 Barrier beach origin and sediment supply

This section summarises a review by Cope (2004) on worldwide barrier and spit origin. The majority of literature has focused on the origin of sandy, offshore barrier islands (Tanner, 1990), particularly on the Gulf of Mexico and eastern U.S.A coastline (Table 2.3). Several hypotheses and theories have been developed over the past 100 years, with a number of contradictory modes of origin being promoted for the same feature. With regards to development of U.K coarse-clastic barrier beaches and spits, sediment supply originates from;

Terrestrial sources, which include relatively small sediment input from rivers and glaciers and an episodic input (where sediment volume positively correlates with rate of relative sea level rise (Jennings and Orford, 1999) eroded from cliffs and shore platforms (Orford *et al.,* 1991*a*)). However, this sediment source cannot enter the system where hard defences protect areas of potential yield. Where a barrier migrates inland, the terrestrial store which outcrops the front of the beach may be available for re-working (Orford *et al.,* 1991*a*).

Seaward sources*,* which may have been significant throughout the Holocene, but are no longer considered to make a major contribution to the shoreline as stores are now largely depleted (Jennings and Orford, 1999).

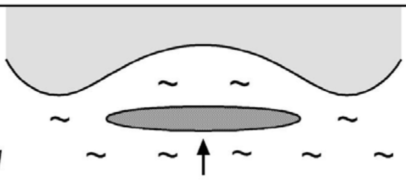
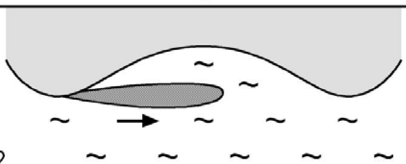
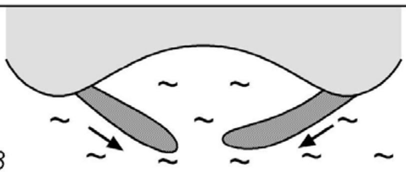
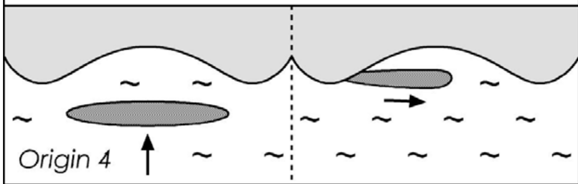
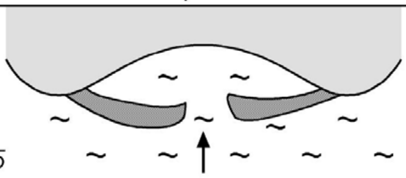
There are depositional forms that have had a composite mode of origin and continue to be fed by terrestrial and marine sediment sources, thereby experiencing cross-shore and longshore sediment sorting. Orford *et al.,* (1988) note that most south-east Irish barriers that migrated onshore, lost contact with their original shelf and coastal sediment sources in the mid-Holocene as the rate of local sea level rise stabilised. Even though sea level rise continued but at a much slower rate, the barriers became stranded against the present coastline and local cliff erosion became the dominant sediment source, thereby changing the beach morphodynamic regime.

Other examples where barriers have become stranded against the contemporary coastal slope and lost contact with the original offshore sediment source are presented in Table 2.5.

Table 2.5 Barrier beaches that have largely lost contact with their original offshore sediment source (Cope, 2004)

Barrier	Author
Porlock barrier, Somerset	Jennings and Orford (1999).
Loe Bar, Cornwall	Bird (2000); Hardy (1964 a).
Start Bay barriers, south Devon	Hails (1975); Morey (1983).
Chesil beach, Dorset	Carr (1978); Carr and Blackley (1973, 1972).
Hurst Spit	Nicholls (1985); Nicholls and Webber (1987); Bradbury (1998).
Medmerry barrier	Wallace (1996).

Figure 2.2 has been devised from existing literature on fine- and coarse-clastic barrier origin and field studies, in order to clarify the different modes of barrier and spit origin.

Origin	Explanation
 Origin 1	**Barrier may have migrated onshore with sea level rise to attach itself at both ends or one end to the contemporary coastline, or remain offshore.** Bird (2000) and Hardy (1964) - Loe Bar, south Cornwall, Carr (1978) and Carr and Blackley (1972, 1974) - Chesil beach, Healy (1995) - Marazion barrier, Mounts Bay, south Devon, Hails (1975), Morey (1983) - Start Bay barriers, south Devon, Leont'Yev and Nikorov (1965) - Black Sea, Soviet Union, Mercer (1966) - Slapton Sands, south Devon.
 Origin 2	**Spit may have formed from a longshore sediment source (river input, delta, drumlin or headland).**
 Origin 3	**Barrier may have formed from opposing spits (river input, delta, drumlin or headland) trending in opposite directions.** Carter *et al.* (1987)
 Origin 4	**Barrier may have a more composite or multiple stage evolution *(origin 1 and 2)*. The barrier may have migrated onshore and attached itself to the coastline at one end. Due to an influx of longshore sediment supply it may become characteristic of a barrier spit.** Orford *et al.* (1988) - Ireland.
 Origin 5	**Double or single, flying spits may have been formed by breaching of an attached barrier form. Additionally, an offshore barrier island may form from double breaching of an attached barrier form.**

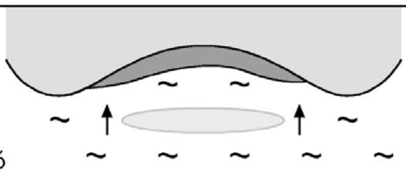
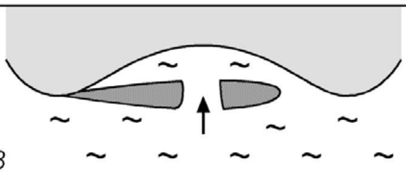
Origin	Explanation
 Origin 6	**Offshore barrier island may have formed by progradation of an offshore bar or shoal on a rising, stable or falling sea level.** De Beaumont (1845) - Northern Europe, Cooke (1968) - North Carolina, Georgia and Texas, Fisher (1968) - Middle Atlantic, Fisher (1973) - North Carolina, Johnson (1919) - Atlantic, Jones (1977) - Massachusetts, Leontyev and Nikiforov (1965) - Black Sea and Soviet Union, Merill (1890), Otvos (1970) and Tanner (1990) - Gulf of Mexico and Shephard (1960) - Gulf Coast.
 Origin 7	**Offshore barrier island may have formed by drowning of dune complexes on a rising sea level.** McGee (1890) - Michigan, Hoyt (1967) - Georgia, Texas and The Netherlands.
 Origin 8	**Offshore barrier island may have been formed by breaching of an elongating spit.** Evans (1942) - Michigan, Fisher (1968) - Atlantic and Texas barriers, Gilbert (1890) - Utah and Lake Michigan Lakes, Hoyt and Henry (1967) - Sapelo Island, Georgia and barriers in Holland, Steers (1946) - East Anglia, England (shingle barriers) and Weide (1968) - Gulf of Mexico barrier development.

Figure 2.2 Sedimentary origins of barriers and spits (Cope, 2004)

2.4.2 Stratigraphic structure

The geological record presents a confused interpretation of barrier and spit preservation (Bradbury, 1998), primarily because of the difficulty in differentiating between the various types of beach on the basis of the analysis of the internal sedimentary structure (Nielson *et al*, 1988).

Spits and barriers can both be characterised by large-scale landward-dipping sedimentary structures, which form as a result of progradation. Structures demonstrating a coarsening upwards sequence, with washover deposits at the crest, could be argued to fit into the category of beach, barrier or spit.

The most obvious characteristic of these structures is their plan-shape, which can rarely be seen within a geological exposure. Kraft *et al* (1979) presented sedimentary models for coastal environments which differentiate “ocean-estuarine systems” from “estuarine barrier washover systems”, using type combinations of sedimentary sequence. Similarly, geophysical methods have been used to examine spit migration (Siringan and Anderson, 1993; Smith and Jol 1992 and Jol *et al,* 1994).

Description of the washover processes has been examined on the basis of vibracore studies (Hequette *et al*, 1995). Back barrier stratification has presented evidence for overwashing through landward-dipping wedge-shaped, intertidal sub-beach structures, with upward coarsening sediments at Carnsore, (Ireland) (Orford and Carter, 1982, 1984): here, washover processes are suggested as the main migratory process.

Hurst Spit is similar in broad terms, but there are some significant differences. Such differences include the shingle/sand ratio, and the mean shingle fraction of the beach size distribution which are both higher at Hurst Spit (Nicholls, 1985), than at Carnsore. Similar circumstantial evidence of overwashing is provided by Carr and Blackley (1973) at Chesil Beach, and by Leatherman and Williams (1977 and 1983) in sand barriers, on the basis of the description of the barrier stratification.

2.4.3 Morphology of seaward face

The profile and form of a shingle beach can be considered as part of a general morphodynamic model. It has been suggested that two main types of beach profile exist: a step or swell profile formed by waves of low steepness and associated with beach accretion; and, bar or storm profiles formed by waves of high steepness and associated with beach erosion. These two forms of profile (Figure 2.3) have been identified largely through regular wave model testing and have focused upon the shape of sand beach profiles (Powell, 1990).

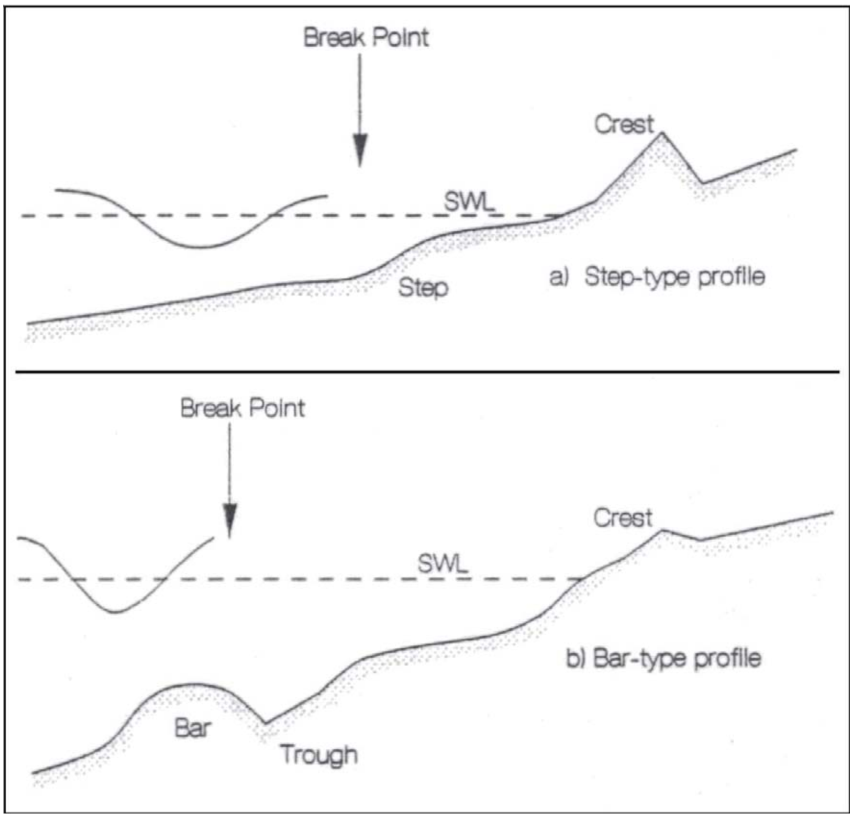


Figure 2.3 Idealised beach profiles (Powell, 1990)

The beach profile, to seawards of the barrier crest, can be defined in terms of its principle geometric components (Powell, 1990) using numerical descriptors for key points on the profile, linked by defined curves. A datum position is defined at the intersection of the seaward beach slope with SWL; this varies, with time, as the profile develops or the water level changes. The remaining features are described by co-ordinates relative to this datum (Figure 2.4). Crest position (p_c) and crest height (h_c), formed by run-up, are usually clearly-defined descriptors. The transition step position lies directly beneath the breaking wave location; it is a clearly developed feature, although it is less well-defined than the beach crest. This particular location is the most mobile and variable point on the profile.

Van Hijum and Pilarczyk (1982) and Powell (1986) conducted physical model studies, designed to develop profile descriptors for shingle beaches. A similar approach to the profile description was adopted in both sets of investigations. Profiles were schematised as two hyperbolic curves: one from the beach crest to the step; the other from the step to the lower profile limit. In a similar manner to van der Meer (1988), Powell (*op. cit.*) has described the beach profile by three hyperbolic curves: from beach crest to the static water level shoreline; static water level to the top edge of the step; and the top edge of the step to the lower limit of profile deformation. Figure 2.4 illustrates the schematisation of the beach profile and defines the co-ordinates descriptors for the three curves. The resulting schematisation is characterised by the following parameters relative to the still water and shoreline axes:

p_r - the position of the maximum run-up (-ve);

	h_c	- the elevation of the beach crest	(+ve);
	p_c	- the position of the beach crest	(-ve);
	h_t	- the position of the beach step	(+ve);
	p_t	- the elevation of the beach step	(-ve);
	h_b	- the elevation of the wave base	(-ve);
and	p_b	- the position of the wave base	(+ve)

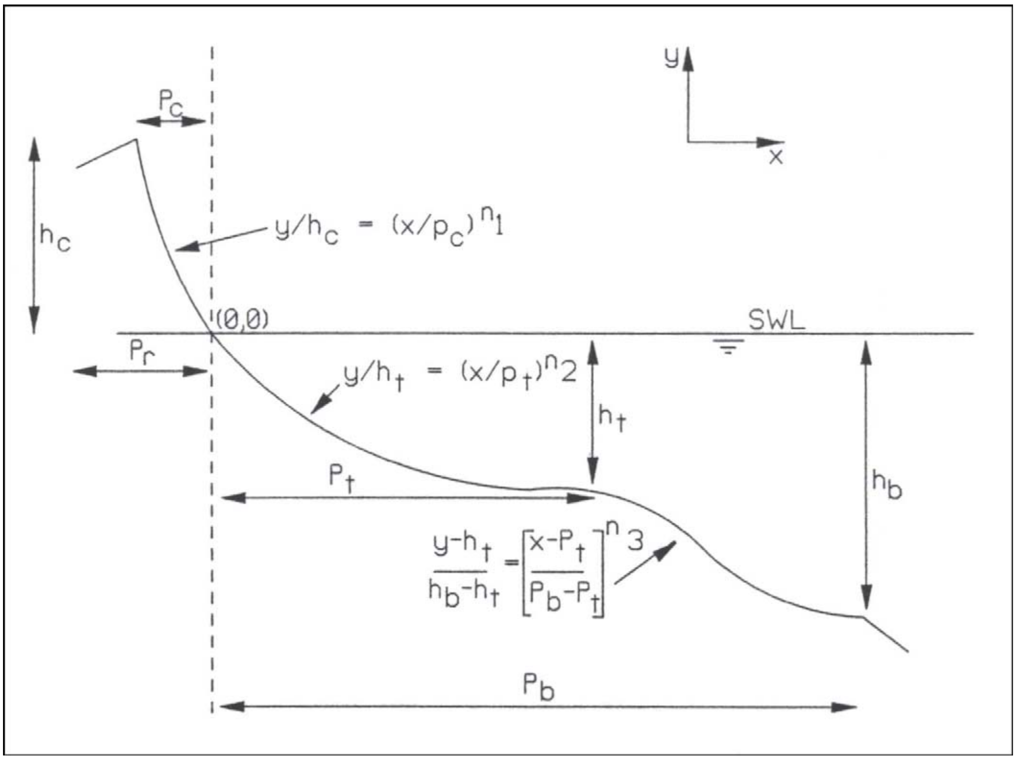


Figure 2.4 Schematised beach profile (Powell, 1990)

The schematisation shown in Figure 2.4 provides an appropriate description for both fringing beaches and also barrier beaches. The descriptors do not, however, identify the location of several key locations over the crest and the back of a barrier beach.

2.4.4 Morphology of crest and back-barrier

The barrier profile is characterised by additional features to those identified by Powell (1990). Common features to all the profiles, which can be defined in relation to the same zero as suggested by Powell (1990), are presented in Figure 2.5 in a modified schematic barrier profile (Bradbury, 1998).

The maximum crest level of the barrier is a key feature of the profile: likewise, a series of ephemeral crest ridges co-exist commonly, on shingle barrier beach profiles. The highest and most landward crest is that resulting from the most severe combination of wave and water level conditions. The lowest crest is that which has occurred as the result of the most recent wave activity. The maximum barrier level lies often above the clearly-defined maximum wave run-up crest; this may be due to any combination of a range of circumstances as outlined below

(a) Undermining of the beach crest may result from short period but high amplitude waves.
(b) The crest ridge may be degraded by aeolian processes such as wind and rain.
(c) Human activity may result in the degradation of the beach crest.
(d) Beach recharge may raise the barrier crest, artificially, above that normally formed by hydrodynamic processes: the beach profile may not show a clearly- defined wave run-up crest, until the beach has become well sorted and a new dynamic equilibrium profile has formed under storm action.
(e) Erosion of the beach crest may result in: the formation of washover deposits on the crest; deposits leeward of the crest; and erosion of the crest due to hydrodynamic processes, such as overtopping and overwashing.
(f) A relict degraded beach crest may result during periods of falling relative sea level.

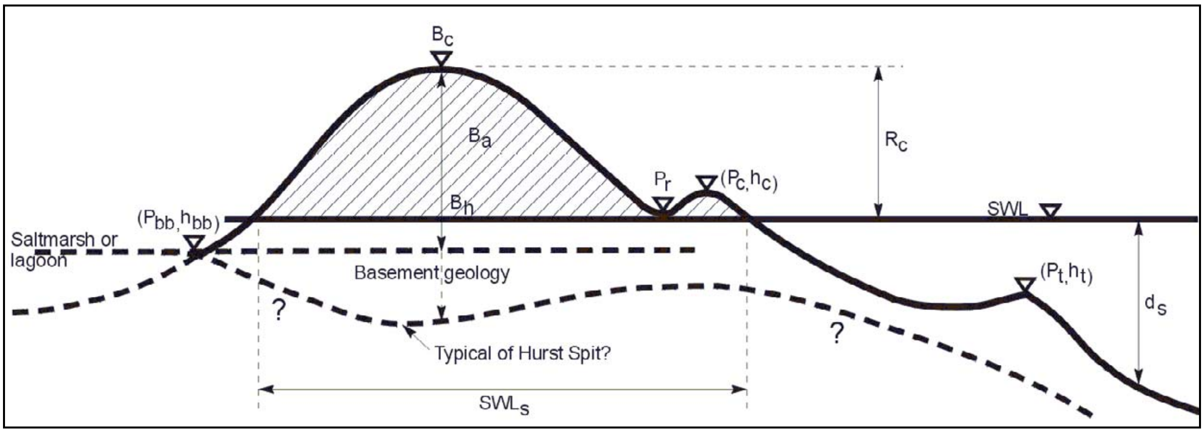


Figure 2.5 Schematic barrier beach profile, showing levels and definitions (Bradbury, 1998)

The barrier-crest, freeboard and its position can be defined by simple profile descriptors, co-ordinated relative to the still water zero datum (p_{bc}, h_{bc}). Although the highest point on the beach crest can be defined simply, it is not always representative of the crest geometry. The lee crest (p_{lc}, h_{lc}) can be defined as the point of maximum wave run-up, to landwards of the barrier crest; it is often marked by a ‘strand line’ on the beach, which is ephemeral and often difficult to identify, and is used within the definitions of barrier crest elevation.

The crest width is a useful variable to consider in analyses, and is widely used in field assessments by engineering managers of beach sites, although it is not easily defined. Qualitative analysis of natural profiles has indicated that the profile is associated generally with a single turning point over a parabolic crest section. The single turning point at the crest has no width; the only exception to this is likely to be the presence of a flat crest berm formed artificially by beach recharge. This situation is not appropriate for the general definition of the barrier crest, but may be relevant when assessing the design of beach recharge.

The landward limit of the back-barrier (p_{bb},h_{bb}) is used often to identify the rate of barrier progradation (Orford *et al*, 1991a). Evolution and location of the back-barrier toe is influenced by the basement topography, as well as the hydrodynamic variables.

Qualitative analysis of model tests of barrier beaches has indicated that the beach span at SWL and the surface emergent cross-sectional area (CSA) are geometric variables which can influence overtopping or overwashing. The SWL span (SWL_s) is defined as the barrier width at the zero datum level; similarly the CSA of the surface emergent profile can be calculated by integrating the area above this limit.

2.4.5 Geometry of barrier beaches

The geometric variability of the structure shape and size has a significant effect on barrier response, in addition to the hydrodynamic conditions. A summary of geometric variables considered by various investigations is presented in Table 2.6. It should be noted that data shown does not consistently relate barrier geometry to clearly defined tidal elevations. Examples of the relative barrier geometry for some of the case history study sites (Chapter 5) are shown in Figure 2.6.

Table 2.6 Typical coarse clastic barrier geometry (Bradbury, 1998)

Location	Width (m)	Height (m)	Investigator
Story Head (Nova Scotia, Canada)	40-60	4	Orford et al (1991)
Olympic National park (Washington, USA)	20-30	5-6	McKay & Terich (1992)
Louisburg (W.Ireland)	80	4-5	Carter & Orford (1993)
New Harbour (Nova Scotia)	30-40	4	Carter & Orford (1993)
Ballantree (Ireland)	80	10-12	Carter & Orford (1993)
Porlock (Devon)	40-60	10	Carter & Orford (1993)
Carrs Pond, (Nova Scotia)	40-50	4	Carter & Orford (1993)
Hurst Spit (Hampshire)	30-50	4-5	Nicholls (1985)

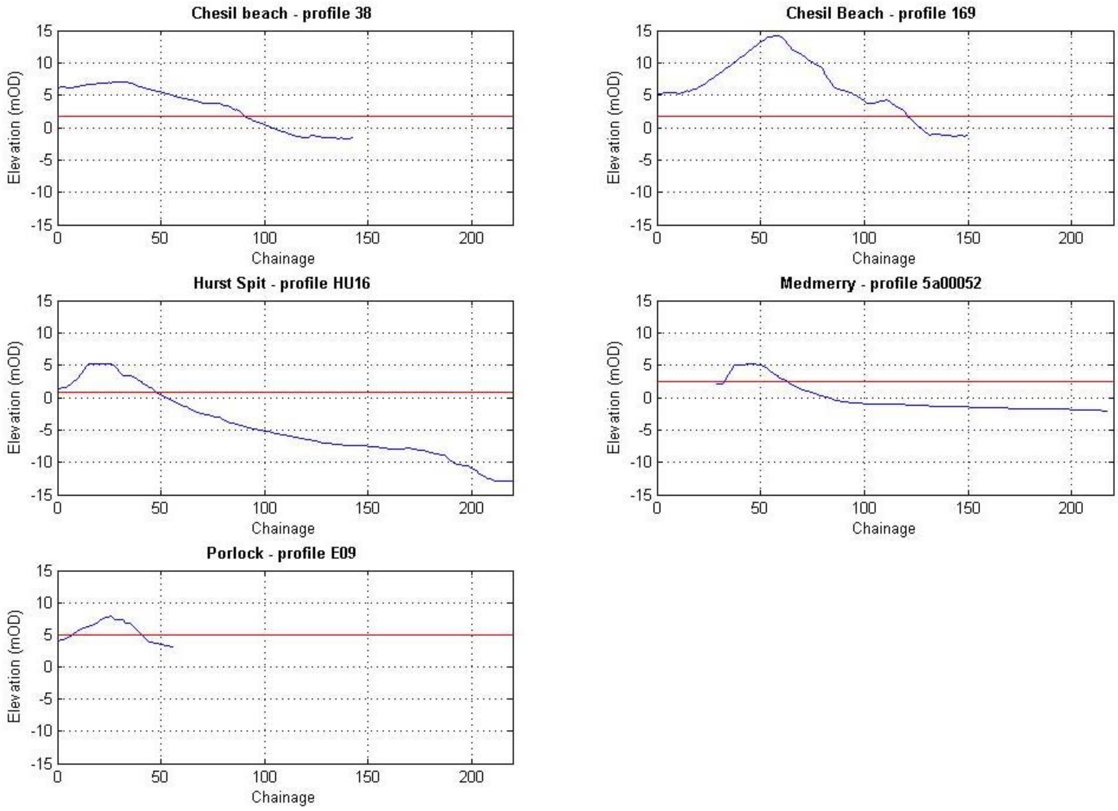


Figure 2.6 Examples of the relative barrier geometry for various case history (Chapter 5) study sites

2.5 Barrier beach profile features

2.5.1 Mode of development

Barrier islands are often associated with increases in sea level following periods of glaciation (so-called marine transgressions). Under these dissimilar conditions (rising sea level, for example) waves may transport sediment over the shore-platform in an attempt to create the equilibrium profile. Barrier islands are formed when the slope of the shore-platform is flatter than the gradient corresponding to the equilibrium profile.

Waves propagating over a relatively steep shore-platform are able to transport sediments all the way to the shoreline. Over a shallow platform, however, they may lose their energy before reaching the shoreline itself. As a result, deposition may take place some distance offshore, thereby creating a barrier.

The above are essentially cross-shore processes. In addition to these, material brought by longshore transport processes may also contribute to the sediment budget. Thus barrier islands are often linked and part of spit systems. Should the distal (down-drift) end of the linked island or spit system reach land, effectively enclosing an area of low-lying land, then a barrier beach is produced.

While the development of a barrier beach is linked to sea level rise it is usually short-term events that bring about change. During exceptionally severe wave/tidal conditions a barrier island may be breached or over-washed. This results in beach material being transferred from the seaward to the landward side, producing a net landward displacement of the feature (without necessarily losing volume).

The fate of barrier islands is often also intimately linked to the tidal inlets, which separate them. The tidal currents through these inlets are often sufficiently strong to transport material either landwards or seawards, allowing ebb and flood deltas to be formed. These deltas usually rely on the barrier islands for a supply of material, which is provided by the littoral drift along the seaward face of the island.

It is thus evident that barrier beaches are active systems, which cannot be fixed in place without affecting their natural “regime”. When attempts are made to manage a barrier beach, so that it cannot evolve naturally, then severe erosion problems may result (Mangor, 2001).

Morphodynamic changes in barrier beach profiles are as a result of wave and tidal conditions and the associated sediment transport. Understanding these changes is important for the quantify barrier condition, and assessing their performance as flood defences. Such assessments enable barrier design and consequent improvements to the standards of defence afforded by barriers to be made. The following sub-sections describe influences on barrier morphology.

2.5.2 Initial beach slope

The dependence of the profile development on the initial slope profile, prior to wave action, has been discussed widely. The consensus of most investigations on both sand and gravel beaches suggests that initial slopes within the range 1:5 to 1:10 had no effect on the development of the beach profile.

2.5.3 Berm formation

Berm formation is the most frequently-occurring process, which reshapes the supra-tidal section of the beach; it occurs close to the limit of wave run-up, when the swash fails to reach the crest. Beach deposits arising from this process are ephemeral and berms may only exist for a single tidal cycle, depending upon the prevailing wave conditions (Nicholls, 1985).

Berm deposits are virtually always composed of the coarser fraction of the beach; these may be preserved occasionally, if the beach progrades. Evolution of swash ramps, associated with run-up that cannot reach the barrier crest has been examined by Orford and Carter (1984); it is suggested that their formation results from the development of supra-tidal terraces by spilling wave conditions during storm surges. Hypothetical wave conditions have been examined to hindcast ramp-forming conditions based upon empirical relationships between run-up and wave height and the basic analysis of local breaking wave conditions (Carter and Orford, 1981).

Shingle beaches are often reflective, with steep upper faces, dominated by plunging wave conditions over a narrow surf zone (Carter, 1988). A series of stages of beach form, ranging from ‘totally reflective’ to ‘dissipative’ conditions, have been recognised (Short, 1979). Reflective shingle beaches show a characteristic highly reflective form under storm wave conditions (Kirk, 1980; Carter and Orford, 1984): these are characterised by a shallow gradient offshore profile, a steep linear beach face and a high crest berm. Bars may develop during storm conditions (Kemp, 1963), particularly when the lower foreshore comprises a finer sand fraction (Short, 1979). Shingle beach profiles are often described as stepped, due to the distinctive inflexions on the profile. Formation of a step-berm at the breaker point on the cross-shore profile is reported to induce premature wave breaking. This results in partially reformed spilling breakers running high up the beach to form ramp and overtop deposits (Orford and Carter, 1985).

2.5.4 Sediment characteristics

The importance of sediment characteristics such as grain size and grading has been examined (Powell, (1990); van der Meer, 1988). Sediment grain size appears to have more effect than grading. It is suggested, however, that there is a strong correlation between the characteristic wave steepness and the mean grain size, when analysing the profile response. Both of the above studies have indicated that there is little variation in the beach profile response, due to sediment grading, although this does not accord well with field observations. Subsequent research (Powell, 1993) has examined the effects of a wider range of sediment sizes and gradings. Regrettably, the results cannot be related back to the original profile prediction methods, as a much simpler form of analysis, (based upon the mean beach slope) was adopted.

Shingle beaches are characterised by both sediment size and hydrodynamic response characteristics. Their morphological development is controlled primarily by wave action: it has been suggested, for example, that the beach responds critically to the proportion of the wave energy dissipated (Wright and Short, 1984). Movement of shingle is less influenced by tidal currents, than sand, as shingle moves primarily in bed load as opposed to suspension (Velegrakis, 1994).

McLean and Kirk (1969) suggest a linear relationship between grain size sorting and foreshore slope. Kirk, (1980) extended this work to identify the zonation of barrier profiles. Distinctive cross-shore shape sorting is sometimes apparent (Bluck, 1967; Orford, 1975) with spherical material concentrated over the lower segment of the profile. Large clasts can be stranded preferentially at the berm crest (Carr, 1969).

Working and sorting of sediments is possible only within a confined zone on a simple beach backed by cliffs; energy absorption is confined to reflection and dissipation within the voids. In contrast, flow can pass over the crest of a barrier beach, causing modification of the crest and the lee slope profiles of the beach, creating a new depositional regime (Carter, 1988). A cross-shore correlation between sediment size and cross-shore elevation has been proposed for gravel barriers in Washington State, (U.S.A) (McKay and Terich, 1992).

The nature of overwash deposits results often in the deposition of a mixture of the finer and coarser sediments; consequently the permeability of the beach is reduced due to this layering effect. Similarly, beach recharge operations may influence the sorting of the beach, often resulting in much reduced permeability (McFarland *et al*, 1996). Hydraulic or mechanical processes used for placement of beach recharge materials results in artificial mixing of beach material; semi-cohesive recharge deposits may “cliff”, forming nearly vertical supra-tidal slopes under these circumstances.

2.5.5 Through-flow

Seepage is also noted as a significant process. This may result in the formation of wash out canns or channels as discussed for sites at Slapton Ley (Devon) (van Vlymen, 1979), Chesil Beach Arkell, (1955); Carr and Blackley, (1974) at Dungeness (Eddison, 1983) and at Hurst Spit (Dobbie and Partners, 1984; Nicholls, 1985). Carter *et al* (1984) quantified stream seepage through coarse clastic barriers in SE Ireland, identifying a relationship between maximum potential head, discharge and barrier geometry. Barrier seepage through-flow is suggested to occur in the range 0.25-1.8 x 10^{-3} m^3s^{-1}, assuming seepage velocities calculated by reference to Darcy’s formula:

$v = k\Delta h / \Delta\ell$, where

K is the permeability coefficient and $\Delta h / \Delta\ell$ is the hydraulic energy loss per unit length. Differential water levels often occur on either side of the barrier, resulting in varied hydraulic gradients. Cross-barrier differential water levels of 1.5m were observed at Padre Island, (U.S.A) during hurricane Allen (Suter *et al* 1982); these may have encouraged breaching to occur.

Limited investigations have been undertaken on this process, despite its significance in both evolution and flood defence terms. This probably reflects the difficulty of obtaining field measurements and the possibility of reproducing the beach permeability correctly within physical models. Studies of relevance to this process, but with the focus on internal flow within the beach include investigations at Slapton (Austin and Masselink, 2005) and the Grosse-Wellen-Kanal (Lopez de San Román-Blanco *et al*., 2006).

Site specific investigations have been conducted by HR Wallingford (1984) to examine the internal flow within Chesil Beach, with a view to design of a beach drainage structure. Observations made of instrumented boreholes are recorded by the Environment Agency at Chesil Beach (Riches, 2005) but these investigations produce inconclusive results, suggesting that whilst the flow data is of some considerable management value, there is a requirement to modify the measurement programme further to include forcing variables such as wave conditions and also to added further measurement control. Such an opportunity is likely to arise with the anticipated introduction of a waverider buoy off Chiswell in autumn 2006, as part of the southwest regional monitoring programme. There is some merit in pursuing the monitoring of this system in a robust scientific manner, as there is a significant risk of flooding arising from percolation at Chiswell (see Chesil Beach case study in Chapter 3).

2.5.6 Crest level

The crest level of a shingle barrier beach is one of the most critical parameters in defining its stability (Nicholls, 1985) and is dependent upon wave run-up and sediment availability. Landwards recession occurs when wave conditions exceed the unconfined crest: this can occur on barriers that lay many metres above mean sea level. For example, the maximum crest height of Chesil Beach was 14.7mOD (Carr, 1969); this was reduced to 13.7mOD due to overwashing (Carr, 1982).

Orford (1977) examined hypotheses proposed by Palmer (1834) and Lewis, (1931), which suggested that shingle beach crests were deposited by plunging breakers, but concluded that spilling breakers in combination with a storm surge were a more likely mechanism. Orford and Carter (1984) suggest that edge waves may form a significant role in the crestal and overwash processes on drift-aligned barriers.

2.5.7 Crest elevation reduced by foreshore widening (cut back)

Bradbury (1998, 2000) notes that crest elevation can be reduced through cut back of the profile and foreshore widening between the wave breaking point and run-up crest, to form a more dissipative foreshore. This process is more commonly occurring on beaches that have been managed, and where the crest elevation is artificially high relative to the usual environmental conditions.

Where swash does not exceed the barrier crest height during a storm, undermining or cut-back of the beach crest may occur, thereby initiating collapse and crest height reduction. This process is exacerbated by inefficient percolation rates, promoting stronger backwash. An example of crest-cut back is shown in Plate 2.1 at Porlock barrier, Somerset. This process is also clearly evident on the managed beaches at Cley, Medmerry and Hurst.

Plate 2.1 Porlock barrier, Somerset. A vulnerable section that experienced crest cut-back, 3rd September 2001 (Cope, 2004)

The width of the barrier is also reduced (Plate 2.1) and break-through breaching may be initiated as waves push through the barrier (Bradbury *et al*., 2005) as opposed to traditional breaching arising from overtopping and overwashing (Bradbury and Powell, 1992).

Bradbury (1998, 2000) notes that crest cut back has not been recognised previously in the literature (although undercutting is mentioned in Orford *et al*., 1991*a*) but considers it to be one of the most important processes leading to overwashing and breaching, especially on managed barriers. This process is particularly notable at sites where management activities, such as re-profiling of the beach with mechanical plant, have altered the beach sorting, or where the crest elevation is artificially high.

2.5.8 Crest reformation

Bradbury and Powell (1992) and Bradbury (1998) note that overwashing can also result in the post storm crest elevation being raised, relative to the pre-storm profile. The process and response is controlled essentially by the surface emergent cross section of material available, sediment supply, back barrier geometry and the shape of the pre-storm beach profile (Bradbury, 1998, 2000).

The initial response is usually a reduction in crest elevation arising from the initial overwashing waves, but the crest may rebuild further to landwards of its original position if there is sufficient material within the cross section to allow a dynamic equilibrium profile to occur with reformation of the post-storm crest, at a higher level than the pre-storm crest elevation (Figure 2.14c). This process occurs where there is sufficient sediment supply, marginal pre-storm conditions compared with the overwashing threshold, if the beach rolls back onto rising land, or where a barrier outflanks a topographic low (Bradbury, 2000). This form of overwashing is characteristic of low, wide barriers.

2.5.9 Foreshore level

The influence of the foreshore level, relative to the toe of the beach, is significant in influencing the development and change in beach profiles - particularly below static water level. The water depth at the toe of most beaches is shallow, in contrast to the deep water at the toe of the beaches modelled by Powell (1990). There are a few exceptions, notably Chesil Beach, which has relatively deep water at the toe of the beach, under certain tidal conditions.

There is considerable scope for an improvement of the profile prediction methods for shallow water conditions; these are much more typical of beaches around the UK. Wave breaking in shallow water complicates the evolutionary processes, but little work has been carried out into such conditions, particularly under random waves. The location of wave breaking clearly has a significant impact on the distribution of energy dissipation and the consequent profile response. As the water becomes shallower, the waves break farther offshore; consequently, they have a smaller impact on the upper beach profile. Assuming that the waves offshore follow a Rayleigh distribution, then depth-limiting is assumed to occur when $H_s/D_w > 0.55$. Van der Meer (1988) suggests that the effect of a reduction in the foreshore depth resulted in a shortening of the beach profile below still water level, over the range $0.56 < H_s / D_w < 0.74$.

2.5.10 Overstepping

Barrier overstepping is the condition by which a barrier remnant is left on the shoreface, whilst the upper part of the sediment body moves rapidly onshore, often with short-term rapidly rising sea level (Forbes *et al.,* 1991), in the form of surge-generated overwashing (Orford and Carter, 1995). The condition is a component of progressive barrier landward migration, which may be irregular or episodic (Forbes *et al.,* 1991; Bray, 1997). Orford and Carter (1995) investigated inter-annual, sub-decadal and decadal (mesoscale) processes affecting barrier overstepping at Story Head, Nova Scotia. There is a strong dependence upon the nature of the foreshore solid geology. Muddy estuarine deposits capping sands or clays frequently occur in such environments.

Oversteepening of the barrier profile due to overtopping allows waves to break closer to the shore and can be attributed to meso-scale events. Therefore, the unstable steep barrier needs only a low magnitude storm event to completely plane off the crest leaving behind a relict barrier, particularly when backed by compacted sediment. The remaining overstepped barrier would have sustained rapid onshore migration due to a reduction in cross-sectional area (Forbes *et al*., 1991).

Orford and Carter (1995) suggest that the role of micro-scale storm events may be conditioned by the preceding meso-scale pattern of storm events, which condition the barrier morphology. They believe that barriers which build a high but thin crest in response to an extended period of low magnitude storms may sow the seeds of their own destruction. Alternatively, periods of high magnitude events produce a more dissipative profile through wide beach faces and low crest elevations following overwashing, offering a more resistant profile to barrier retreat.

2.5.11 Overtopping

Overtopping occurs in response to appropriate combinations of wave and water level conditions, and beach geometry (Figure 2.7). Orford and Carter (1982) and Orford *et al*. (1991a), suggest that where the volume of unconstrained run-up is small, sediment deposition tends to be confined to thin veneer overtop deposits; this results in vertical crestal accretion, when wave energy is inadequate to pass over the crest. Such deposits occur as virtually horizontal open-work shingle.

Swash returns to seawards by percolation through the permeable shingle. Nicholls (1985) has identified maximum accretion of the beach crest of 0.45m due to this process; this is primarily during storm surges, suggesting that this process occurs more frequently than overwashing. Shingle overtopping, without overwashing, has also been recorded at Chesil Beach (Dorset) and Llanrhystyd (Wales) (Orford, 1979). Landward thinning deposits predominate at Carnsore, demonstrating a preference for barrier crest build-up under these conditions (Orford and Carter, 1982).

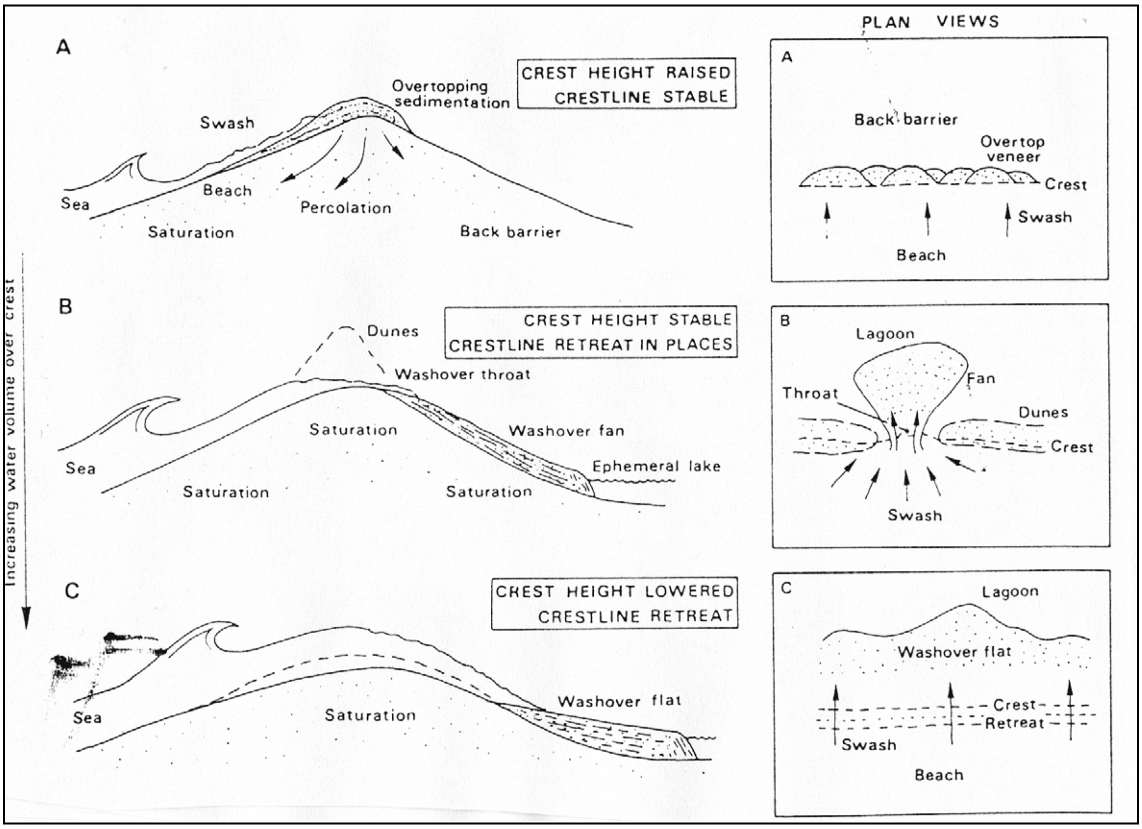


Figure 2.7 Contemporary barrier beach processes - the continuum of overtop-washover sedimentation as a function of the increasing volume of water passing over a barrier crest during a severe storm (Orford and Carter, 1982)

Crest level raising by overtopping results where wave swash is sufficient to reach the crest but is insufficient to wash over the crest (Bradbury and Powell, 1992; Bradbury, 2000). The swash carries pebbles up to the crest where, due to efficient percolation, they are deposited causing the barrier to accrete vertically at the swash limit. The process deposits thin layers of shingle which form a run up berm (Bradbury, 2000) thereby reducing further water and sediment movement to the back-barrier side (Bradbury and Powell, 1992). Plate 2.2 provides an illustration of the overtopping process in operation. Crest height may increase but the width often decreases under overtopping conditions, thereby making the system more susceptible to breakthrough breaching during the next major storm event.

Overtopping is characteristic of coarse-clastic, as opposed to fine-clastic barriers, due to larger clast size promoting higher permeability and thus creating a steep reflective seaward profile. Little research has been carried out on overtopping and overwashing events for coarse grained barriers (Bradbury and Powell, 1992; Orford and Carter, 1982). However, Bradbury's (1998) conclusions from studying the profile response of Hurst Spit, Hampshire, to extreme hydrodynamic forcing conditions provide new insights into these processes.

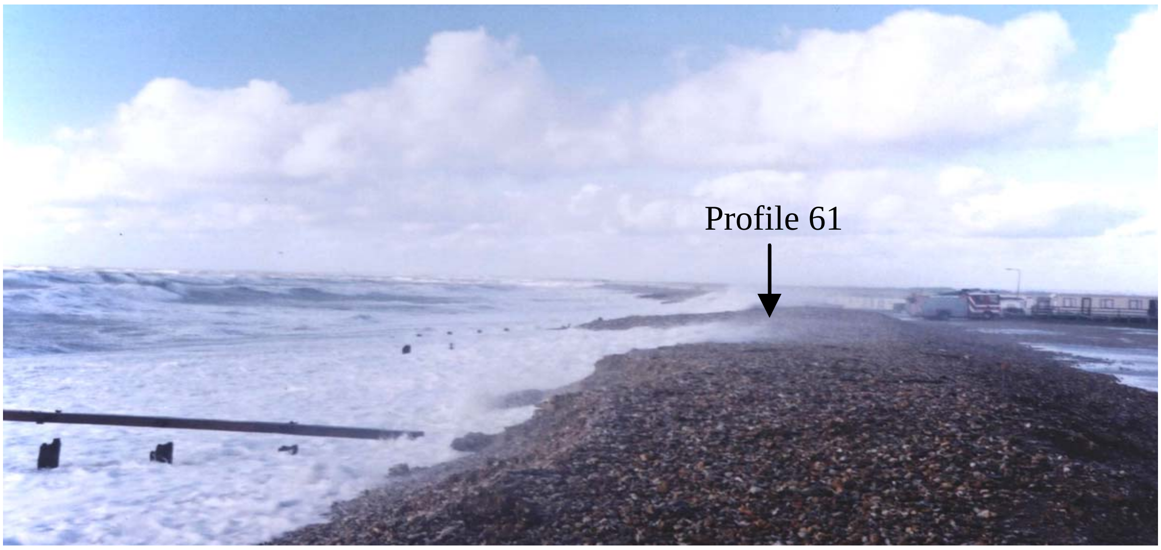


Plate 2.2 Overtopping of the Medmerry barrier, West Sussex, looking NW. 1st February 2002 (Cope, 2004)

2.5.12 Overwashing

Overtopping and overwashing are the processes that drive landward rollover for swash-aligned barrier beaches and spits. Orford *et al.,* (1991*b*) note that through time and with sea level rise, the crest gradually builds to the height of extreme swash run-up. The rate of rollover is dependent on the rate of sea level rise, degree of storminess in relation to the basement condition, nature of the material and the geometric and volumetric properties of the barrier (Bradbury and Powell, 1992; Bradbury, 1998; Carter *et al.,* 1987; Orford *et al*., 1991*a*), with sediment availability dominating over sea level rise. As a consequence, rollover rates vary. Sediment sorting is a good indication of rollover, with poorly sorted barriers experiencing fast rollover rates and well sorted barriers experiencing slower rollover (Orford *et al*., 1991a).

Overwashing takes place when swash continues over the unconfined crest, onto the back crest of the beach. Differentiation between the processes on sand and shingle barriers relates to the higher permeability of shingle beaches (Nicholls, 1985). Whilst coarse clastic barriers exhibit high permeability, the nature of overwash often results in the mixing of coarse- and fine-grained materials in the washover deposits (Carter and Orford, 1993). Hayes and Kana (1976) suggest that overwashing is associated only with topographic lows, in sand barriers. Carter and Orford, (1981) and Orford and Carter, (1982, 1984) suggest a similar control on the coarse clastic barriers of SE Ireland. Leatherman *et al*. (1977) suggests a barrier elevation overwashing threshold of 2.5m above sea level on a sand barrier system at Assateague Bay (U.S.A), but does not reference this to wave conditions. Overtopping gives way to discrete overwash and the formation of throat-confined washovers, often at topographic lows, when run-up exceeds crest height; this can develop further, as wave intensity increases, leading to sluicing overwash (Orford *et al*., 1991a). The entire barrier crest may be displaced in surge-like swash flow, under such circumstances (Figure 2.8).

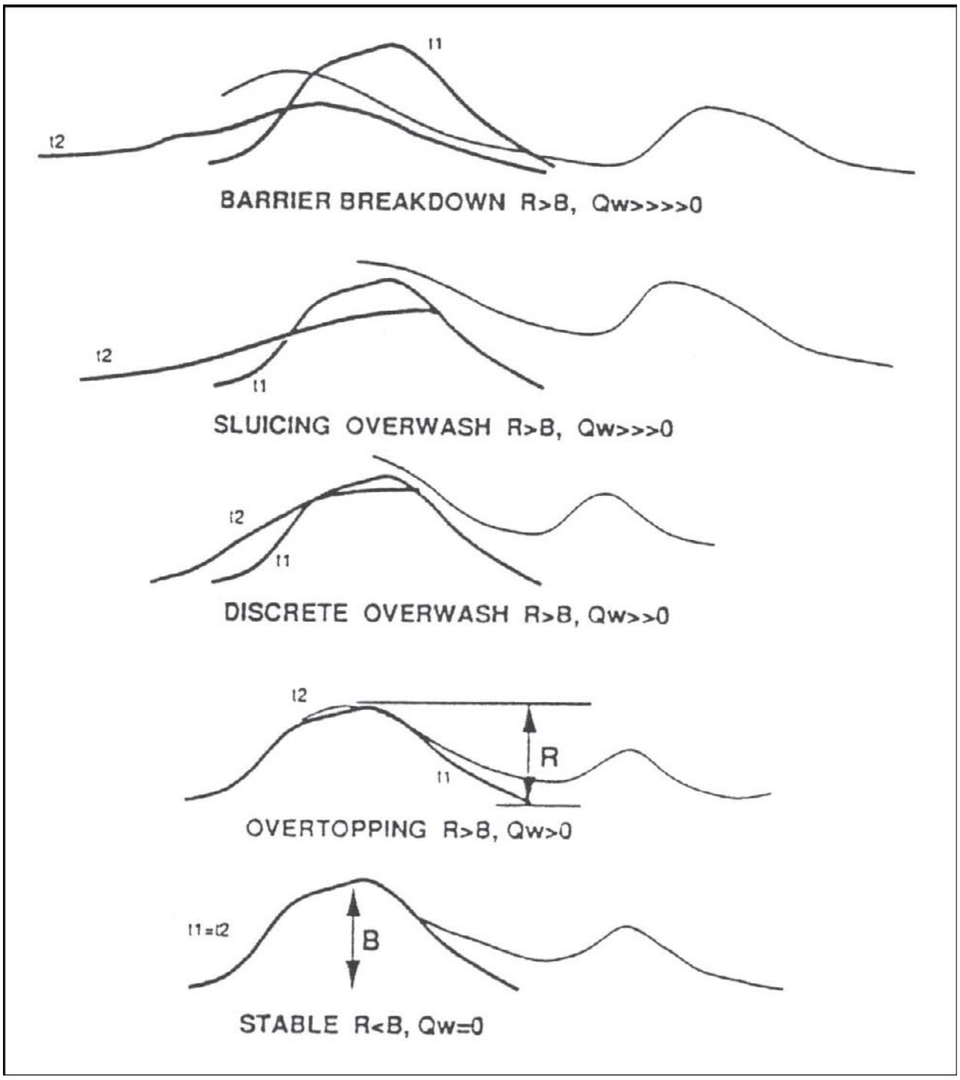


Figure 2.8 The continuum of overtopping and overwashing modes by which gravel barrier crest migration may occur: crestal profiles before and after storm generated overtopping and/ or overwashing run-up (Orford *et al*., 1991)

Orford and Carter (1982) have discussed the possibility of simultaneous overtopping and overwashing at various locations, under the same hydrodynamic conditions; it is suggested that throat-confined overwash fan formation may result. A periodicity of spacing of throats has been observed by Carter *et al* (1990) at a spacing of 15-25m on the drift-aligned barrier at Story Head, (Canada).

Nicholls (1985) identified two types of overwashing:

(i) Type 1 Overwashing - without a reduction in crestal height;
(ii) Type 2 Overwashing - with a reduction in crestal height.

Type 1 Overwashing is characterised by the deposition of open-work shingle, on the lee crest of the barrier; it conforms to other authors definitions of overtopping. Such deposits consist of thin layers, of approximately 0.1m with a dip similar to the leeward face of the beach; this is typically at a slope of 5-13°, but also reaching a steepness of 19° in places on Hurst Spit (Nicholls, 1985). The deposits are characterised also by steep fronts, probably resulting from sudden cessation of the flow due to percolation.

Type 2 Overwashing occurs less frequently; it results when the combination of wave and water level conditions are severe, relative to the beach geometry. A range of features may result from this process; these include throat-confined overwash fans, or more wide-spread sluicing overwash. Examples of both of these processes (Plate 2.3) were observed and documented by Bradbury (1998).

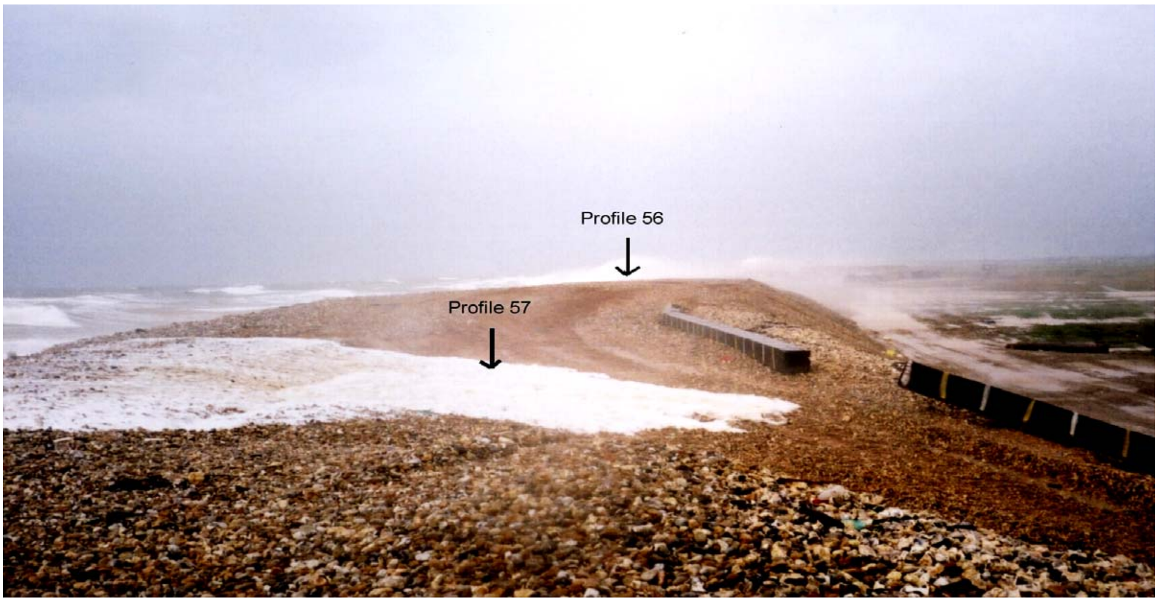


Plate 2.3 Examples of Type 2 (Nicholls, 1985) overwash (Cope, 2004)

Development of such features can occur within a few minutes. Once the crest level has been reduced the wave energy required to overtop the crest is reduced; hence, the frequency of overtopping events increases. This process is the most significant in terms of the volumetric movement of material onto the back crest of the barrier.

Nicholls (*op. cit*) observed Type 2 Overwashing on at least 13 occasions, between 1980-1982, at Hurst Spit; it should be noted that at this stage Hurst Spit was in a serious state of decline. Washover fans and flats generated during these events were up to 1.5m thick; one extended up to 100m to landwards of the crest. Fans are characterised generally by a steep face at their landward end, where the fan intersects the lagoon. The composition of sediments within the fans can be highly variable, and they may comprise of a mixture of open-work shingle, sandy open-work and sandy-shingle: landward dips vary at between 3 – 6° at Hurst Spit (Nicholls, *op. cit*). The maximum reduction recorded by Nicholls (*op. cit*) was 2.5m. A 30m wide throat formed on 10/4/83; this widened to 100m on 2/9/83. Beach face erosion accompanied overwashing, resulting in crest elevation reduction of 0.5-1m over a 100m length of the beach. The landward dip of the washover deposits were typically at a slope angle of 6-10^{O}.

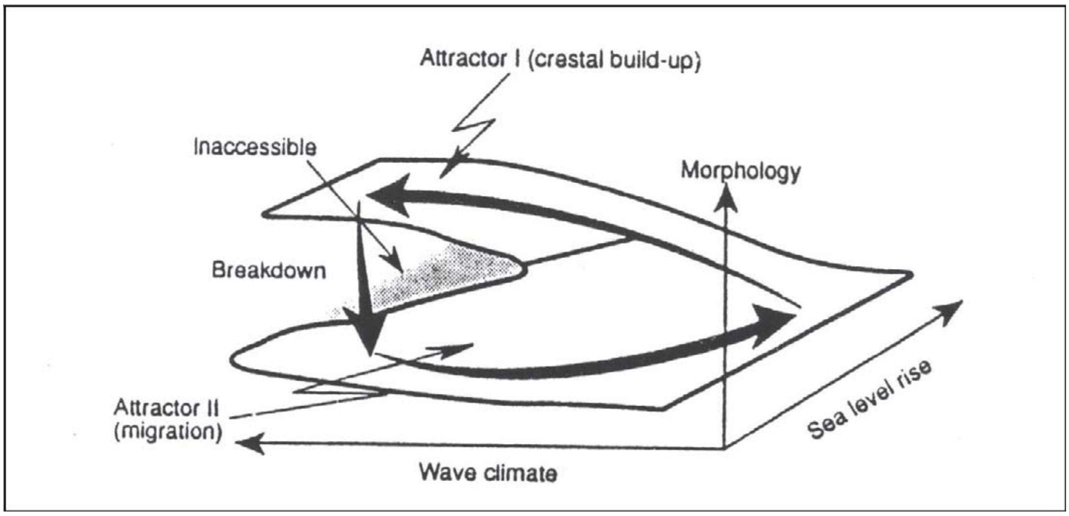


Figure 2.9 The two attractors for gravel barriers under the influence of rising sea level (Carter and Orford, 1993)

The jump from crestal build-up to barrier breakdown may involve little additional forcing (Carter and Orford, 1993) (Figure 2.9). Transitional zones may occur between these triggers, when a certain mode of development persists; the sorting and stability of the structure improves during these periods (Figure 2.10) (Carter *et al*, 1993). Despite numerous field-based studies which have examined cross-shore response processes, none have quantified crest evolution with respect to hydrodynamic forces.

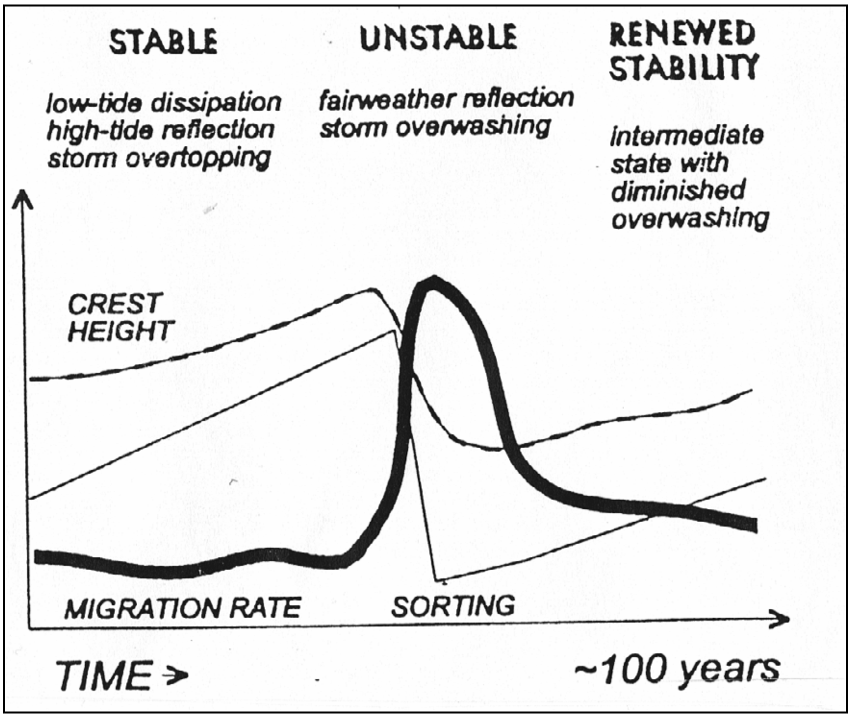


Figure 2.10 Schematic representation of catastrophic transition from stable to unstable hydrodynamic states (attractors I and II of Carter *et al*., 1993) for swash-aligned single-ridge gravel structures

Crest lowering by overwashing (Figure 2.7b) can occur at lower crestal elevations under similar hydrodynamic conditions that give rise to overtopping and rollback resulting in raised crest elevation. Rollback occurs as the crest elevation is reduced by waves that exceed the crest limit (Bradbury, 2000). Rather than reforming a post-storm crest that is higher than the pre-storm crest elevation, crest lowering by overwashing results in deposition in the backbarrier area in the form of washover fans (Figure 2.7b), or on a more intense scale, washover flats (Figure 2.7c). Where a barrier crest is wide the speed of formation and size of washover fans will be slower and smaller respectively. With major storm surges the barrier can be almost completely submerged and relocated landward by sluicing overwash (Figure 2.7c) from which adjacent washover fans merge to form a washover flat. In the event that the barrier is characterised by a relatively small cross section or if the beach rolls back into a channel the likely response is of a crest reduction.

2.5.13 Breaching

Cope (2004) notes that an exposed coastal breach can be defined as an entrance through a barrier or spit protecting low-lying land, bay, lagoon or estuary which is characterised by tidal flow. Such a process is infrequently occurring. Recent examples of sustained breach formation within the past 50 years are evidenced at Porlock (Plate 2.4), which has remained as an open breach since 1996 and at Sowley in the western Solent, which has remained open for about 50 years. The plan shape characteristics of the two sites are very similar, with the channel becoming flanked by two spits running inland. Breaching may be singular or multiple (Mehta, 1996), which will affect whether the breach will be ephemeral or permanent (Figure 2.11).

Plate 2.4 Example of Porlock barrier, Somerset that breached 26th October 1996 (Environment Agency, 1999)

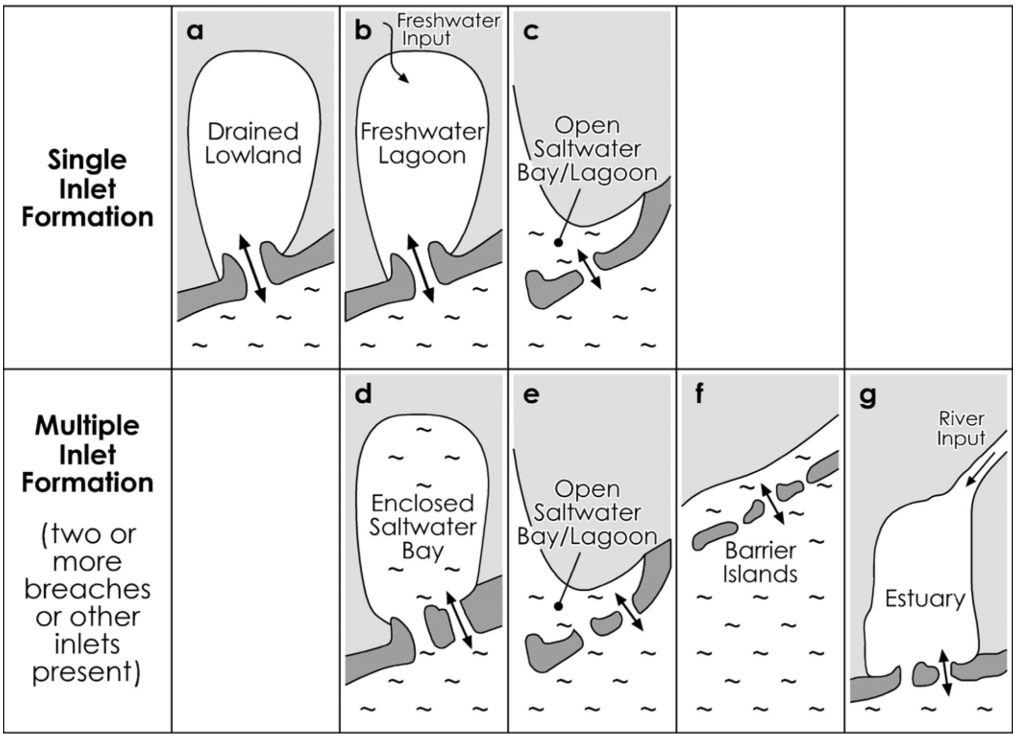


Figure 2.11 Singular or multiple breaching (Cope, 2004)

The majority of barrier beaches and spits along the south coast of England are in the ultimate phase of barrier breakdown (Carter and Orford, 1984). This is due to an imbalance between their *forcing* hydrodynamic and *resisting* morphodynamic factors (Cope, 2004).

Breaching can best be understood in an evolutionary context by studying Figure 2.12 (Orford *et al.,* 1991*a*), which integrates conditions of equilibrium and critical thresholds (Greenwood and Keay, 1979; Jennings and Orford, 1999). For swash-aligned barriers, backed by lowland, wetland or lagoon, stability is achieved by maintaining an adequate sediment volume in relation to the rate of sea level rise. This needs to be sufficient to sustain a steadily increasing crest elevation and maintain sediment sorting. If these conditions are satisfied, migration rates should remain slow (Forbes *et al*., 1995*a*) (Figure 2.12).

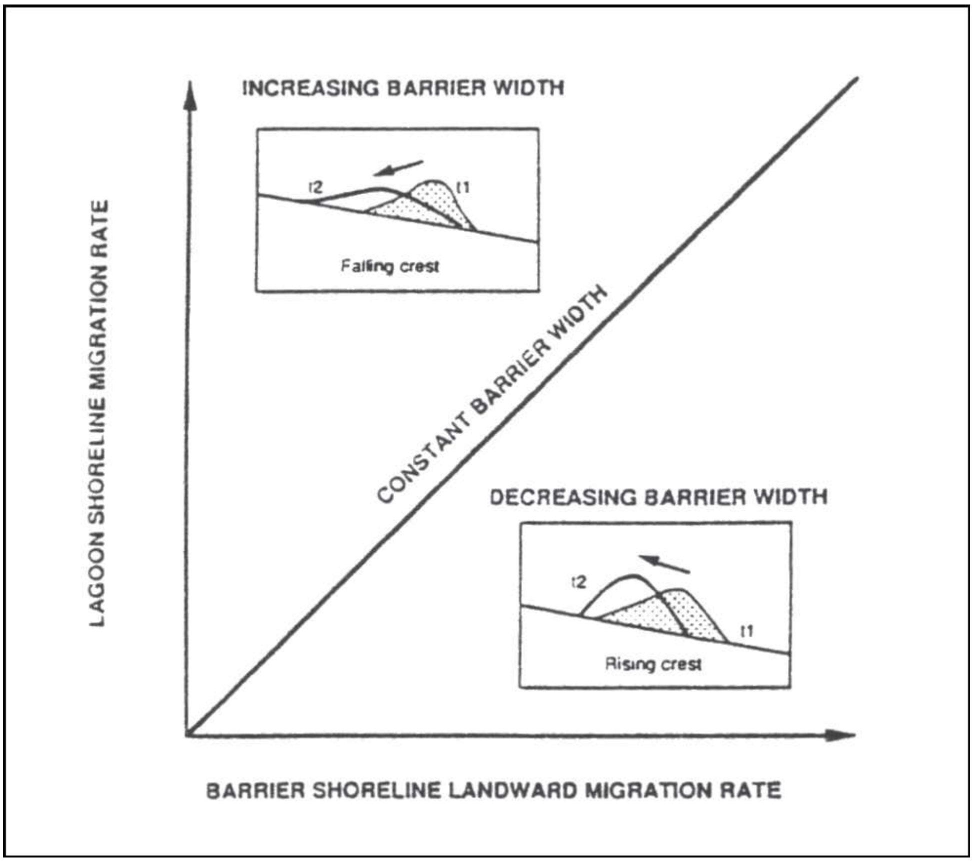


Figure 2.12 Schematic view of barrier crestal stability domains as a function of seasonal and back-barrier shoreline migration (Orford *et al*., 1991a)

This would have been the situation throughout the early Holocene, due to an abundance of paraglacially derived sediment. However, the latter part of the Holocene transgression is marked by a depletion of sediment due to reworking of the finite source within coastal systems (Jennings and Orford, 1999). These sediment shortages cause landward migration and possibly breakdown, where sea level continues to rise and the barrier morphology attempts to adjust (Jennings and Orford, 1999). As sediment supply declines and/or rate of sea level rise increases, barrier crest height cannot be maintained with respect to tidal levels and becomes vulnerable to processes such as overwashing that further lowers the crest.

Reworking and overwashing of the crest reduces the sediment sorting of the barrier and its permeability to swash, thereby promoting enhanced overwashing and crest cut-back (Bradbury, 1998). This results in rapid landward migration rates and where a critical stability threshold is surpassed, breaching may be initiated. The first recorded tidal breach of Hurst Spit occurred during December 1989 but was rapidly filled using mechanical plant.

On a micro-scale, ranging from one day to a year (Orford and Carter, 1995), a landward-directed breach may be initiated directly by a storm surge, whilst a seaward-directed breach (Penland and Suter, 1984) may be triggered by a rapid influx of freshwater into the backbarrier area (Cope, 2004). However, it is macro-scale factors, operating over a 100-1,000 year timescale, such as a decline in Holocene sediment supply, accompanied by increasing sea level rise, which promotes barrier breakdown (Orford and Carter, 1995; Orford and Jennings, 1998; Jennings and Orford, 1999), paving the way for breach events. The same principle applies to spits, which are even more dependent on sediment inputs.

Once a barrier crest has been lowered, through overwashing or crest cut back, the probability of waves reaching the crest increases along the overwashed section thereby promoting further crest reduction (Bradbury, 1998). Bradbury (1998) notes that development of a washover fan or breach is affected by the cross-sectional geometry of the beach, which in turn is influenced by the backbarrier topography as it migrates onshore. Where sediment is displaced into a low lying backbarrier area the overall barrier cross-sectional area will be reduced and is therefore prone to further crest reduction and breaching. The most important factor governing a breach location is antecedent barrier height as the water from seaward and/or landward will take the path of least resistance. Therefore, breaching and subsequent inlet formation may result from localised breaching or barrier inundation (Basco and Shin, 1999), in a landward or seaward direction, or a combination of the two (Figure 2.13).

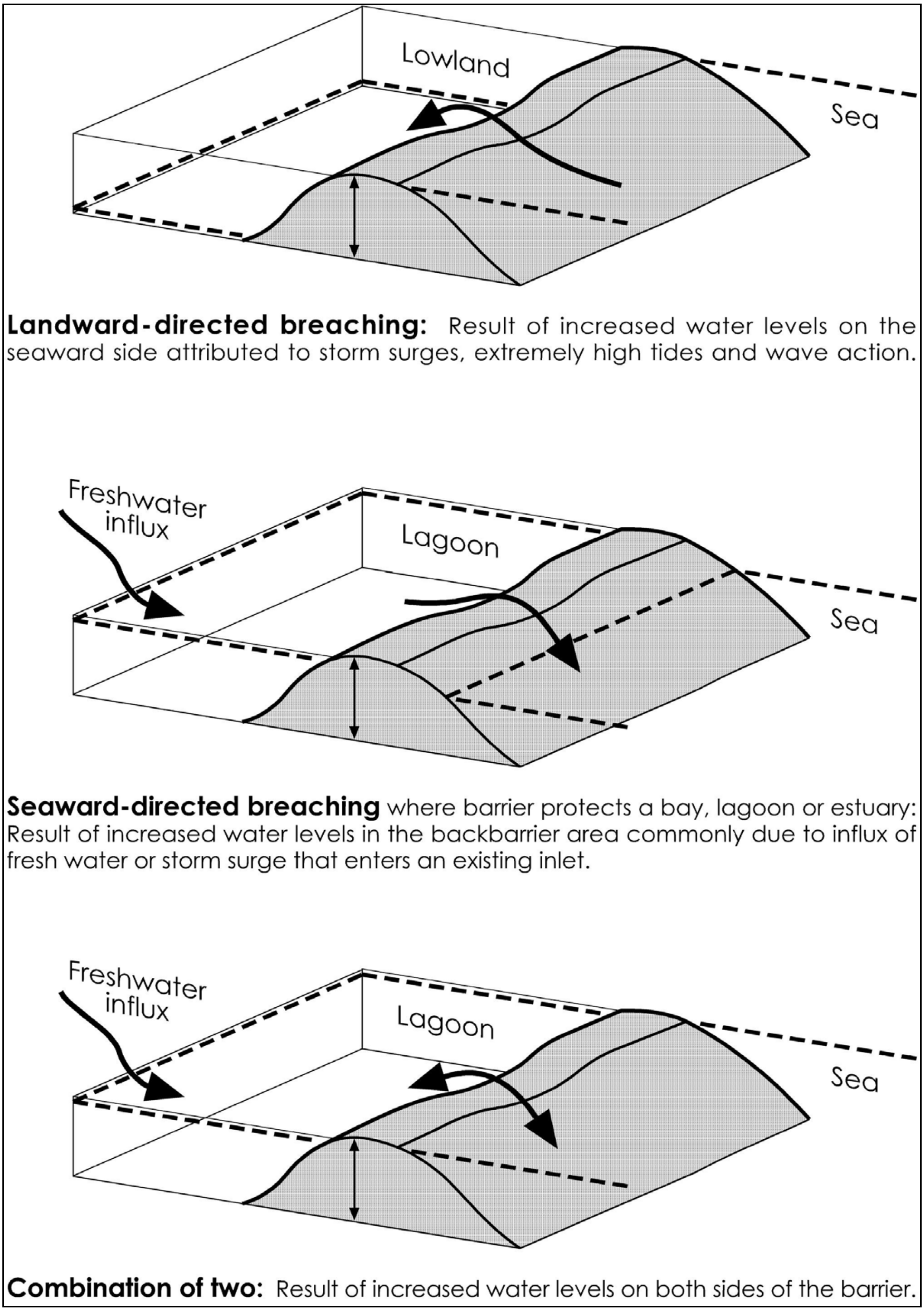


Figure 2.13 Landward and seaward-directed breaching (Cope, 2004)

Localised breaching may occur along areas of barrier systems below the maximum storm surge flood elevation or below the maximum freshwater discharge elevation. It is often a requirement of the evolutionary system to maintain cross-shore discharge routes associated with tides or stream flows (Carter *et al*., 1987). The location of seepage or surface channels (breaching) will depend on the discharge of the backbarrier lagoon, inflowing river/stream discharge regime, barrier size, sediment texture, permeability and wave field (Carter *et al*., 1984). Where barrier permeability is high relative to discharge, seepage through the barrier will dominate. However, where permeability of the barrier is poor or unable to sustain seepage then a surface channel may form. This is a process of seaward directed breaching but also applies for landward directed breaching. For both landward and seaward directed breaching, the surface channel will occur along the lowest sections of the beach crest (Carter *et al.,* 1987) such as wave focusing points, overwashing areas or the low sections of beach cusps. New inlets may form at the end of a storm event where ebb flow through localised breach points scours a channel (Basco and Shin, 1999).

Barrier inundation can also produce breaching and subsequent inlet formation where the barrier is completely submerged by the maximum storm surge flood elevation. The ebb flows, following one or more storms, will again take the path of least resistance through the lowest breaching locations, possibly forming new inlets (Basco and Shin, 1999). This latter scenario would apply more to sand barriers and spits (Taylor *et al.,* 1986), rather than shingle and gravel barriers, as they tend to lower their crest height (pre-storm profile) in response to increased storm surges (Basco and Shin, 1999).

2.5.14 Inlet closure

Cope (2004) notes that smaller inlets (<150m width) are more prone to closure from increased sediment supply due to the rapid reduction of inlet depth with width, which enhances susceptibility to changes in flow regime (Mehta, 1996). Large inlets are generally more stable as they are characterised by a large tidal prism and corresponding equilibrium cross-sectional area (where draining one basin tidal prism). They therefore require a large influx of sediment supply or reduced tidal prism (possibly through basin margin reclamation) to cause instability and closure. Consequently, small tidal inlets may be open (stable), rarely open or subject to periodic closure (Goodwin, 1996). Inlets will permanently close when a bar with an elevation above extreme tide levels cuts off water exchange (Goodwin, 1996).

Inlets subject to periodic closure will generally close on neap tides when the tidal prism and tidal flows are reduced. However, some small inlets are neither sealed nor open as during low tide and/or low wave energy conditions, tidal inundation may be prohibited by a low bar, whilst during high tide or high energy storm wave conditions, full tidal exchange occurs (Goodwin, 1996).

2.5.15 Categorisation of barrier crest evolution

The sequence of crest evolution scenarios is synthesized by Bradbury (2000). Observations have identified a series of alternative beach crest responses to hydrodynamic conditions, and are outlined below.

(a) No change occurs to the crest elevation and the profile is contained to seawards of the barrier crest. The beach responds in a similar manner to that described by the functional relationships observed in earlier studies (Powell, 1990).

(b) The crest elevation is raised in response to overtopping and limited overtopping of the barrier occurs. The waves modify the crest, by depositing thin layers of shingle and building a run-up berm in a similar manner to (a), above. Finally, the supra-tidal beach becomes higher and, usually, narrower.

(c) Roll-back occurs and crest elevation is reduced by overwashing. The waves exceed the crest-line, resulting in destructive modification of the profile. The crest is lowered and migrates landwards whilst deposition occurs on the back barrier and further to landwards.

(d) Roll-back occurs and the crest is raised by overwashing. The waves exceed the crest level, resulting in destructive modification of the crest profile. The crest is lowered initially, and then migrates landwards; it subsequently rebuilds in a new position, at a higher elevation than the pre-storm level.

(e) The crest elevation is reduced, due to beach widening and cut-back of the barrier crest, with no overtopping. Waves do not exceed the barrier crest, but the active profile widens (between the run-up crest and the breaking point). Undermining of the beach crest occurs and the beach crest level reduces; this dynamic profile response is similar to that observed in earlier investigations (Powell, 1990).

The evolutionary modes discussed above can be described in numerical terms using profile descriptors. The criteria used to define the thresholds, for each evolutionary stage is as outlined below.

(a) Crest elevation raised (build-up) by overtopping	$h_{bc(pre)} < h_{bc(post)}$, $p_{lc(pre)} <= p_{lc(post)}$, $h_{lc(pre)} <= h_{lc(post)}$
(b) Crest elevation reduced, by foreshore widening (cut back)	$p_{bc(pre)} > p_{bc(post)}$, $h_{bc(pre)} > h_{bc\ (post)}$ $p_{lc(pre)} = p_{lc(post)}$, $h_{lc(pre)} = h_{lc(post)}$
(c) Crest elevation raised and roll-back, by overwashing	$p_{bc(pre)} > p_{bc(post)}$, $h_{bc(pre)} < h_{bc\ (post)}$ $p_{lc(pre)} > p_{lc(post)}$
(d) Crest elevation lowered and roll-back, by overwashing	$p_{bc(pre)} > p_{bc(post)}$, $h_{bc(pre)} > h_{bc\ (post)}$
(e) No crest change	$p_{bc(pre)} = p_{bc(post)}$, $h_{bc(pre)} = h_{bc\ (post)}$

$$p_{lc(pre)}=p_{lc(post)},\ h_{lc(pre)}=h_{lc(post)}$$

These evolutionary categories are illustrated, with reference to test conditions, in Figure 2.14.

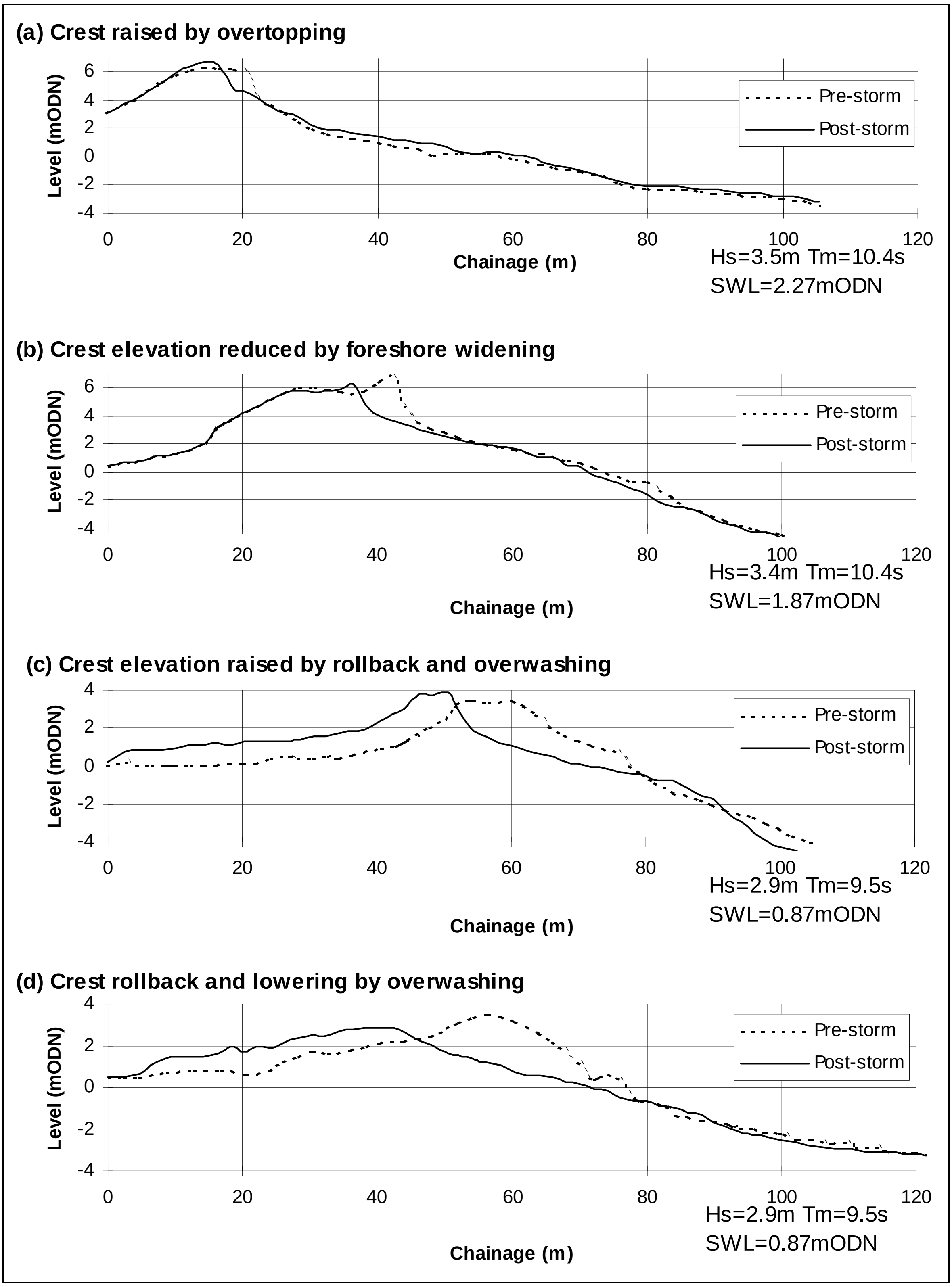


Figure 2.14 Barrier crest evolution categorisation (Bradbury,1998)

Category (b) conforms to the ‘Type 1 Overwashing’ mode (Nicholls, 1985), and the ‘overtopping’ category (Orford and Carter, 1982). Categories (c) and (d) provide subdivisions for ‘Type 2 Overwashing’ (Nicholls, *op. cit*) or ‘overwashing’ (Orford and Carter, 1982 *op. cit*).

2.6 Longshore processes specific to barriers

Drift-aligned barrier beaches and spits are inherently unstable as they are dependent on maintenance of longshore sediment input. Carter *et al.,* (1987) note that elongation of a drift-aligned barrier will continue for as long as sediment can be delivered to the distal end by the transporting wave gradient (Figure 2.15A). However, the probability of an individual clast reaching the distal end decreases as the length of the spit corridor increases (Carter and Orford, 1991). Sediment is then remobilized from the proximal end, which results in proximal thinning (Figure 2.15B). To combat the threat of possible breach formation, swash-alignment may occur by deposition against a confining structure or through sub-cell development (Figure 2.15B) (Orford *et al.,* 1991*a*).

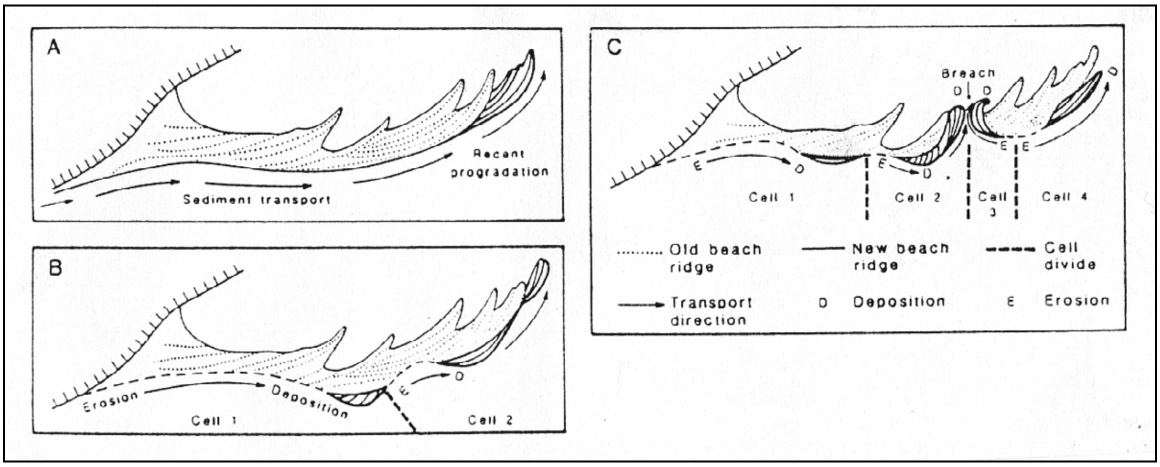


Figure 2.15 “Movement from a drift dominated barrier (A) to a system of barrier longshore reworking (B) and eventually barrier breaching (C) as cannibalisation leads to incident wave-defined sediment sub-cells and washover sites (after Orford *et al.,* 1991*a*).” (Orford and Jennings, 1998)

Where sub-cell development prevails, distinct erosion/accretion cells ranging from a few metres to kilometres in length may form along the barrier (Figure 2.15C) (Orford *et al.,* 1991*a*). This serves to halt transportation of sediment to the distal end (Carter *et al.,* 1987). Examples of this occurred on Flat Island, Newfoundland, where six sub-cells developed, and Fisherman’s beach, Nova Scotia, where three sub-cells developed at the proximal end (Orford *et al.,* 1991*a*). However, the updrift areas of the sub-cells will continue to be drift-aligned and due to their reduced cross-sectional area will be prone to overwashing and breaching (Carter *et al.,* 1987).

The same response condition applies to swash-aligned barrier beaches. Orford and Jennings, (1998) note that cannibalisation of the concave, attached, Porlock gravel barrier system, following depleted sediment input, produced sub-cell development. A concave attached barrier may be both swash and drift-aligned, with the centre being swash-aligned and the flanks being drift-aligned (Figure 2.16). The zone of change between swash and drift-alignment may be marked by a concave hinge, across which less sediment moves and landward migration increases. Theoretically, this starves the middle section of the barrier, so that in time the barrier may breach (Carter *et al.,* 1987). Once the breakdown process has been initiated, swash–aligned areas are prone to overtopping.

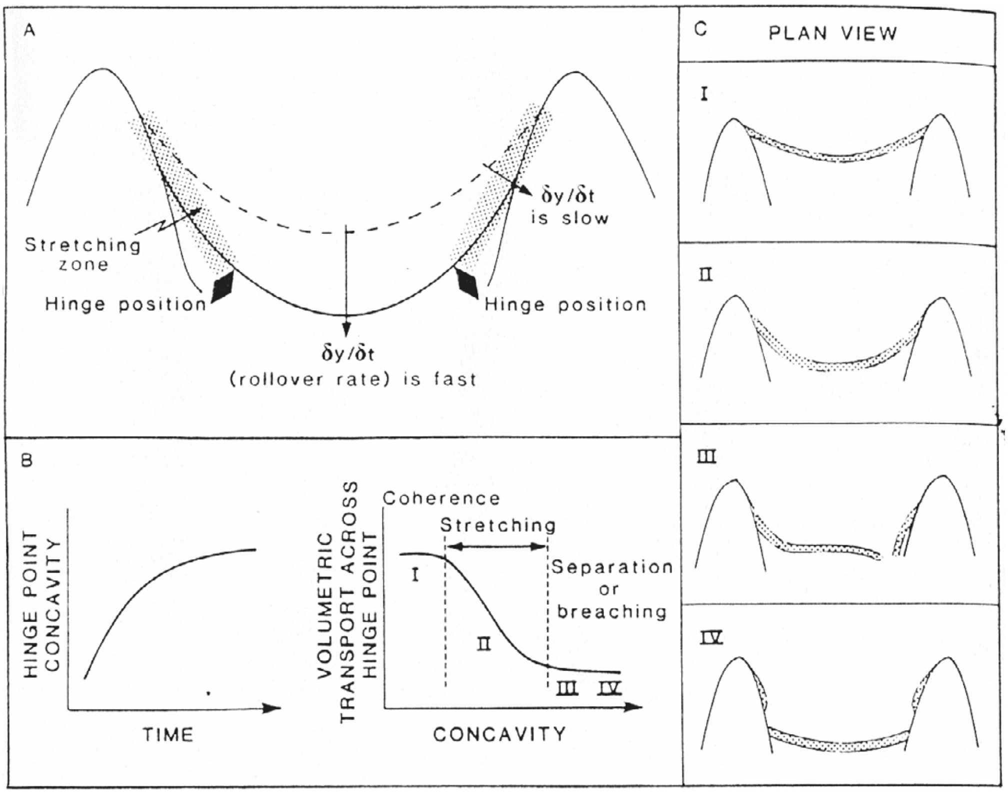


Figure 2.16 Barrier stretching between two headlands (Carter *et al.*, 1987)

2.7 Evolutionary timescales

Management of barrier beaches requires consideration of a range of evolutionary timescales (Figure 2.17). The focus is generally at the mesoscale and micro scale.

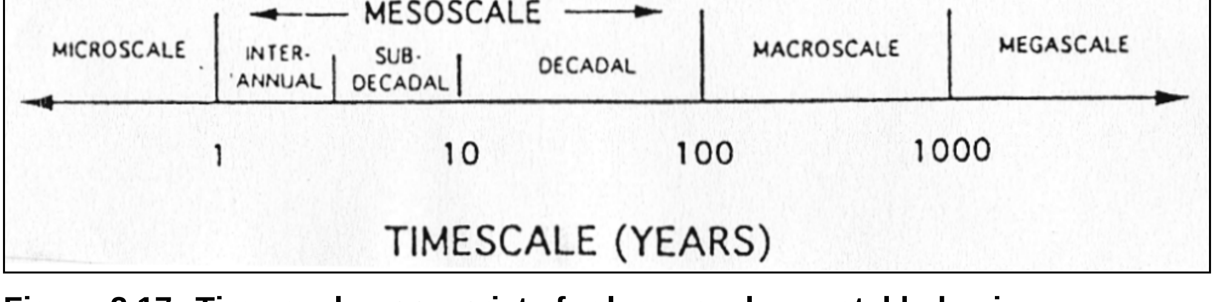


Figure 2.17 Time-scale appropriate for large scale coastal behaviour. Interest is centred on the mesoscale and its divisions (Orford and Carter, 1995)

2.7.1 Decadal scale evolution – links with sea level

Holocene developments within the North Sea have resulted in net sea level changes of about 20m over the past 8000 years (Shennan, 1987). This rapid change in sea level has enabled spits and barriers to evolve rapidly, through erosion of the shoreface and increasing washover (Hequette and Ruz, 1991; and Hequette *et al*, 1995). A number of features around the coast of the UK are attributed to formation at approximately 5000-6000BP. For example, Larcombe and Jago (1994) have suggested that the formation of the Mawddach estuary bar (Wales) was by sediment rolling landwards in response to sea level change: Nicholls (1985) suggests a similar evolution for Hurst Spit.

The effects of sediment supply on the evolution of Orford Ness (Suffolk), were examined by Carr (1970); a correlation with both sediment supply and changes in water level was demonstrated. Borrego *et al* (1993) examined evolution of the spit at the mouth of the Piedras River (Spain), on the basis of geophysical profiling. These investigators demonstrated the effects of episodic overwashing and changes to the sediment supply over the course of 4000 years. Boyd *et al* (1987) present a six stage evolutionary model for barriers; the first two of these phases relate to geological and oceanographical conditions for the initial formation; the latter stages conform broadly with evolutionary processes discussed by Orford and Carter (1982, 1991).

The balance between barrier crest build up due to overtopping, and crest breakdown by overwashing, dictates the rate of barrier migration (Orford *et al,* 1991). Differential response of the barrier crest and back barrier limit provides an indication of the evolutionary phase (Figure 2.12). If the seaward shoreline retreats faster than the back barrier, then the crest must be building. The opposite response suggests a falling crest elevation. These inferences suppose that net sediment transport is in balance and that the cross section of the barrier is maintained.

The balance between overtopping and overwashing is controlled by the frequency and magnitude of storms and storm surges, which are independent of water level. The theory that barrier migration is partially a function of sea-level rise is supported by Dillon (1970), who postulated that an increasing volume of material is required to maintain a stable barrier under sea level rise. Unless there is a longshore or offshore supply, this balance is unlikely to be maintained. It is suggested that barrier rollover will be spasmodic when sea-level is static: however, this theory does not allow for the effects of increased 'storminess'. Orford *et al* (1991) suggest that barrier migration must cease with time in a static sea level situation as the fetch limiting storm event approaches asymptotically: it is suggested that the large gravel barriers of SE Ireland fall into this category. This hypothesis could be applicable to fetch-limited situations, but does not allow for the influence of storm surges.

Carter and Orford (1993) suggest that the study of the short-term morphodynamics associated with coarse clastic barriers has been neglected; this reflects the difficulty of measurement within a high energy zone. As a result, information of a near instantaneous-scale dynamics of such barriers is rare. Horn *et al* (1996) confirms that detailed short-term process studies on spits are virtually non-existent.

2.7.2 Micro-scale hydrodynamic conditions

Whilst the influence of wave conditions is recognised by many authors: (Boyd *et al*, 1987; Forbes and Taylor, 1987; and Carter *et al* 1990); none provide local measurements of shallow water conditions. Orford *et al* (1991a), Forbes and Drapeau (1989), Orford and Carter (1982, 1984) and Fitzgerald *et al,* (1994); all provide information on the magnitude of offshore conditions but no details of nearshore storm-specific conditions. Carter *et al* (1990) have recognised the significance of these limitations. Carter and Orford (1993) emphasise the importance of wave conditions in the change of the morphodynamical status of the barrier, as the critical wave height to depth (H_s/d) and depth to wave length (d/L_o) change; this suggests that morphodynamic shifts occur at abrupt thresholds. Bradbury (1998) highlights further the significance of measurement of performance in episodic storm events and provides valuable data on several extreme events at Hurst Spit. These investigations provide a virtually unique record of beach response to defined storm events together with measured wave and tidal conditions (Bradbury and Powell, 1990, Bradbury, 2000).

A series of short term process investigations has been conducted on barriers at various locations e.g. Austin (2005). The episodic nature of the main evolutionary processes has often resulted in measurement of minor changes and has failed to capture any extreme events that have resulted in crest evolution. Since the key events arising in overtopping, overwashing and percolation processes are often associated with events of a return period of lower frequency than 1:5 years this is not surprising. The main exception to this programme is the work of Bradbury (1998) at Hurst Spit where several major storm events have been monitored. Field investigations at this site have enabled a range of events to be monitored, over a period of 15 years, with estimated return periods spanning a range of events up to the 1:50 year return period. This data is extremely valuable as it includes the response of both managed and recharged beaches to extreme conditions.

There are several instances where the response of major events has been captured, for instance at Slapton, Medmerry, Porlock and Cley (Chadwick, 2005; Cope, 2004; Bray and Duane, 2001; Bradbury and Orford 2006 (in prep)) but these have all been without the availability of wave conditions. The data sets at many sites are difficult to analyse since they fail to identify records associated with management activities (Box, 2005). The resultant analysis can be extremely misleading therefore. Data is available from many sites that fall within this category, particularly on the Anglian coastline, where long term records of beach profile response are available, but without detail of management activities and without usable hydrodynamic data. The response of overwashing events has been captured at Cley (Box 2005), but without supporting hydrodynamic data.

2.8 Recommendations for further work

Whilst it is clear from the content of this Chapter that there already exists a wealth of information relating to various processes, characteristics and features pertinent to barrier beaches, there is nevertheless scope for improving our understanding and study methods. This section highlights potential areas for future research into barrier beach processes which will ultimately contribute significantly to the assessment of flood defence condition and performance.

Most literature has focused on sandy, offshore barrier islands, and it is recommended that effort be concentrated on furthering the understanding of processes on mixed and coarse sediment barriers. Any recommendations for future work implicitly imply that sea level rise should be considered.

It is recommended that a national database of barrier beach field data be developed which enables, for example, the ability of a barrier to “self-heal” to be examined, or how through-flow might occur through barriers of different types. To supplement such a database, field monitoring programmes should be established/ modified to ensure generation of data useful for study. This might include defining a steer for the CCO monitoring programme. Few studies, for example, have been carried out which are able to link barrier crest evolution with hydrodynamic forcing conditions, or shallow water wave conditions and shingle beach morphology.

The database would ideally incorporate existing historical datasets held elsewhere in the UK in a piecemeal fashion. These datasets should include photographic records. The required database management could be achieved relatively easily through exploitation of GIS software.

It is unclear from the literature what effect the internal sedimentary structure of a barrier beach has on its morphodynamic behaviour, although a barrier is likely to behave differently to a fringing beach, and barriers of differing material composition are likely to exhibit differing response and resilience. Percolation, or perhaps the lack of percolation, may play an important role when considering the morphodynamic behaviour of a barrier beach, and it is recommended that this phenomenon is further investigated. In the first instance, a comparison of beach (both fringing and barrier) profiles along the same stretch of coast could be compared and any variations noted.

The role that sediment grading plays in the stability, or otherwise, of a barrier has yet to be studied in a robust and concerted manner. Furthering this understanding may enhance greatly the ability to successfully manage a barrier beach flood defence.

At this point, it should also be noted that, although this Chapter has sought to separate and define various processes, features, and characteristics of barrier beaches, there is yet to exist a single or modular predictive modelling tool which is capable of representing the inter-dependency of even a small proportion of them. This issue is explored later in Chapter 4, when application is discussed.

3. Methods of study

This Chapter outlines the various successful state-of-the-art approaches used when studying barrier beaches. Methods range from analysis of aerial imagery to detailed numerical modelling (mostly developed around the dynamics of sandy sediments). The importance of links between observations and models to achieve greater process understanding is highlighted; some methods requiring the exploitation of decades of measurements. This Chapter ably demonstrates the relative infancy of investigations into coarse-clastic barrier beaches such as those that predominate around the coasts of England and Wales.

3.1 Desk based investigations

Aerial photography provided the basis for investigations by Regnauld *et al*. (1993) at Sillon du Talbert (France); and Suter *et al*. (1982) and Penland and Suter (1984) on sand barrier islands of the Gulf Coast (U.S.A). Similar techniques have been used by Orford *et al* (1991a) to determine the rates of barrier migration at Story Head, (Nova Scotia), in parallel with long-term time-series of tidal records, (over periods of several decades) (Orford *et al* 1993). The relationship between back barrier and sea level margins has been used to infer phases of crest lowering and build up (Orford *et al*., 1991).

Digital orthophotos in combination with GIS have provided the basis for mapping of plan-shape changes over a range of epochs (Plate 3.1) on the south coast of England (Bradbury *et al*., 2005) and at Cley, Norfolk (Box, 2005). Such investigations have highlighted the rate of change of plan-shape evolution and have been particularly valuable for determination of decadal scale evolution. Since the photography available is often not taken at low water, imagery is often not suitable for derivation of detail on the lower beach. However, the back barrier location is a key feature which is generally visible. Evolution of plan shape features such as fans can often be identified. Imagery is widely available for many areas now, from a number of sources, for photography dating back to about 1940. It is typically possible to source appropriate scale data sets to provide coverage of many sites in England at a temporal interval of about ten years.

Photogrammetry has provided a useful method for determination of long term profile sets at a number of sites. Photogrammetric profile data is available for a number barrier sites within the EA southern region dating back to 1974. The data sets are of variable quality however and should be used with caution when assessing either changes in crest elevation or plan-shape. Site specific investigations of profile response have been determined from photogrammetry for several epochs in the mid 1990s for parts of Chesil Beach.

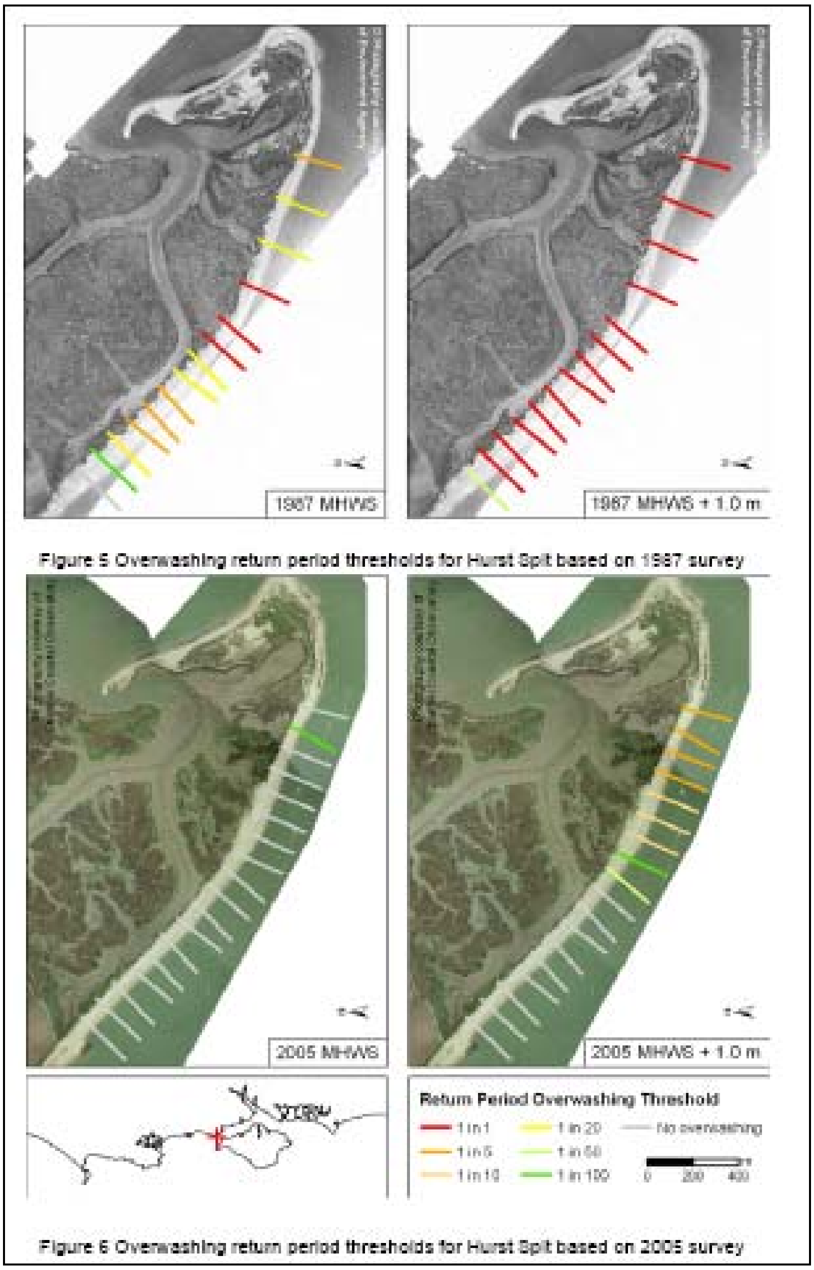


Plate 3.1 Predicted return period thresholds (Bradbury *et al*., 2005)

3.2 Field based investigations

The approach to field based activities can be broadly separated into three categories:

1. Profile and plan-shape response based upon topographic surveys, in combination with wave and tidal measurements
2. Internal and surface flow measurements, based upon instrumented boreholes or buried instrumentation
3. Ecological mapping of beach and back barrier environments

The Southeast regional coastal monitoring programme provides by far the most comprehensive data record of beach profile response covering all barrier beaches within the region between the Isle of Portland and the Isle of Grain on the North Kent coast and extends over the period 2002-2006. Longer term records are available for a number of sites including Reculver and Hurst Spit. The recently commenced (2006) southwest regional monitoring programme provides a similar approach but has not yet delivered any data. Both of these regional programmes have the advantage that hydrodynamic forcing data is also collected together with post storm profile information and also beach management data. An intensive monitoring programme is used in support of the EA PFI scheme at Pevensey; this includes measurement of beach response and utilises hydrodynamic data from the southeast regional coastal monitoring programme. The whole of the Anglian Region EA coastline is monitored with profiles spaced at approximately 1km intervals since 1991; this programme provides data for a number of beaches including Orfordness and Cley-Salthouse. Site specific investigations have taken place at Porlock (Bray and Duane, 2001) where profile data has been analysed since the formation of a tidal breach in 1996.

Data applications have generally focused upon determination of decadal scale evolution of the beach (e.g. Bradbury, 1998). A few investigations have attempted to use data to analyse the response of the beach to episodic storm events and to provide validation for empirical profile prediction models. Cope (2005) Bradbury *et al.* (2005), and Chadwick *et al*., (2005) have used limited field data to attempt validation of empirical predictive numerical methods.

More widely, beach profiling and sediment analysis has been used to investigate the profile response of gravel barriers in Olympic Park, Washington (McKay and Terich, 1992). Observations of the response of an artificial breach and natural closure of a shingle ridge to tidal currents and waves were made using tracers, topographical surveys and by reference to offshore waves by Walker *et al* (1991), at Batiquitos, San Diego.

The use of instrumentation is rare. Leatherman (1977) presents details of current measurements of overwash surge velocities, using electro-magnetic current meters and pressure transducers located in a throat on a sand barrier at Assateague Island (U.S.A). Velocities of 8ft/second were recorded. Similar overwash bore velocities were recorded by Holland *et al* (1991) on a sand barrier in Louisiana, using a video system and a series of capacitance wave staffs located in an overwash throat. Maximum storm event erosion depths have been analysed using plugs of dyed sand, by Fisher *et al* (1974); these investigations identified wave conditions resulting in profile and throat development on Assateague Island. Extensive throat-confined overwash-fans formed on the sand barrier: fans extending 100m, with a 30m wide head and 12m wide throat were recorded.

More sophisticated measurements of internal and surficial flow have been conducted with the aid of pressure transducers within the barrier beach at Slapton (Austin, 2005). This research has provided valuable insights into the flow characteristics within permeable gravel beaches in general, but does not specifically focus on barrier beach processes. Such an approach could be developed usefully to provide better information on flow characteristics on barrier beaches.

Internal flow has been examined rarely, but significant investigations have been made by
Nicholls (1985) at Hurst Spit, utilising stand pipes to monitor changes in head across the beach. A more sophisticated operational management system has been installed by the EA at Chiswell to monitor percolation flow through Chesil Beach (Riches, 2005). This approach is worthy of further investigation and refinement as a management technique.

The use of LIDAR has become more widespread during the past few years. LIDAR lends itself well to monitoring of barrier beaches, which are often characterised by rapidly varying topography. This technique has the advantage that rapidly varying spatial detail can be captured and features such as overwash fans and washout canns can be captured. The level of detail varies. Historically, plan-shape resolution of 2m was the norm, but more recently 1m resolution has become more usual. In the event that small scale features such as canns are monitored, a resolution of 0.5m is appropriate.

3.3 Modelling approaches

Four key approaches to modelling of barrier beaches have been identified and are further explored in this section. These approaches can be categorised thus:

Site specific physical models
Empirical models based on field observations
Empirical models based on physical modelling
Process models

3.3.1 Physical models

Site specific mobile bed physical models of barrier beaches are surprisingly rare. Investigations of Hurst Spit (Bradbury, 1992, 1998) have examined the response of existing and recharged beach management solutions subject to extreme conditions.

Although mobile bed physical models have been used previously to examine the response of restrained shingle beaches to storm waves, the published literature cites only one investigation in which the crest development of shingle barrier beaches has been examined (Bradbury, 1992, 1998, 2000). Earlier studies (Powell, 1990) have demonstrated that profile response of the shingle beach crest could be reproduced realistically on a restrained beach. More recently (during spring, 2007), a short series of experiments was carried out on a large-scale physical model of a barrier beach by HR Wallingford as part of the FLOODsite research programme (Obhrai and Powell, pers. comm.), although the findings have yet to be published.

The first recorded attempt to model a shingle barrier beach under random wave conditions with a mobile bed was undertaken in conjunction with the Hurst Spit beach management scheme (Bradbury and Powell, 1992, Bradbury and Kidd, 1998). Beach response calibration tests were based upon actual storm events and the pre-storm geometry of Hurst Spit (Bradbury and Powell, 1992).

Initial investigations included reproduction of known overwashing events, which caused large scale morphological beach changes. This approach provided a quantitative means of comparison of the model and full-scale beach response, to similar prevailing hydrodynamic conditions. Measurements were made of the following profile variables, pre- and post-storm:

(i) crest level;
(ii) crest position;
(iii) lee slope angle;
(iv) incidence and extent of roll-back; and
(v) model construction accuracy (comparison of nominal to measured profiles)

Storms reproduced in the model were simulated for the full-scale equivalent of 3 hr duration, representing the peak of the storm. This approach ensured that the model represented profile development over the most intense period of storm activity, at water levels which permitted overwashing. Development of the profile was not reproduced following the storm peak, as it decayed and water levels fell. This decision impacts upon the development of the lower part of the beach profile, but should have only a limited effect on the crest evolution. Video recordings and observations of beach development in the model provided further qualitative details of the processes.

The model response appeared to provide a realistic reproduction of the beach development under defined storm events for a model at scale of 1:40 (Figure 3.1). Initial examination of the model profiles showed remarkable agreement with the full-scale measurements. Most of the key evolutionary features identified in geomorphological investigations were replicated during the model tests including the formation of washover fans, throat confined overwashing, ‘roll-back’ of the beach crest and sluicing overwash. The physical model investigations provide the opportunity to examine the development of barrier features on a wave-by-wave basis, using video analysis techniques. It is suggested that such physical modelling techniques provide an appropriate method of study for shingle barriers.

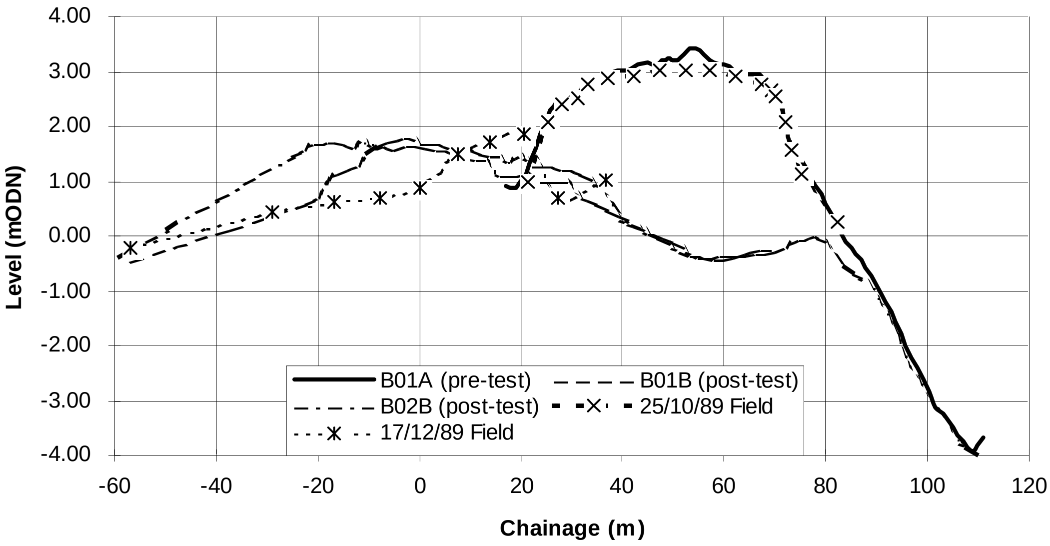


Figure 3.1 Comparative profile response of the model and field data, to the storm of December 17 1989, for profiles HU13. Test conditions: SWL=2.27mODN, Hs=2.5m, Tm=7.4s at an angle of 210° (Bradbury, 1998)

Analysis of post storm crest elevations from calibration tests suggests that the post storm crest level was reproduced in the model with a difference range of 0.2m-1.7m of those recorded in the field. Observations of crest elevations were consistently higher in the model. The model data suggests that the barrier crest will form approximately 0.1-0.3m below the storm peak water level, when sluicing overwash occurs. Field data show that the crest formed at a lower level than this, in some areas. Crest position reproduction was more variable: both the model and field data demonstrated crest roll-back. The extent of roll-back in the model and the field was variable, although the same general trends were observed; this may represent a limitation of modelling of the storm at a single water level. Some scatter in the data was expected, as overwashing waves can move the beach crest several metres within a single wave event. Lee slope angles were reasonably consistent, although the model slope tended to be steeper than at full-scale.

Further calibrations for a storm event much closer to the overwashing threshold were also undertaken for the storm of 25/10/89. Field data suggest that the beach response, measured following 25/10/89, is typical of events resulting in crest modification. Hence, in this particular respect, the storm provides ideal conditions for calibration of the model methodology. Roll-back levels measured were generally smaller in the model than in the field. Comparative results between the model and field data are generally more scattered, for this event. However, such events, which are close to the overwashing threshold, are likely to produce more scattered results.

Changes in crest elevation were reproduced reasonably well in the model, although the model tended to under-predict crest lowering. It may be suggested that this variance represents the fall in the tidal elevation, following the storm peak: sluicing overwash is likely to have continued for some period, resulting in further crest lowering. Most of the model crest elevation data lie within +/-0.5m of the field data levels. The extent of the crest roll-back (migration) follows similar trends, for both the model and the field data, although the data were scattered; this is not surprising, as individual waves can cause the crest to roll-back several metres. A maximum variance of 38m was measured, but most of the data varied by less than 15m. Field data controls are poor, relative to those within the physical model; consequently, the model and field data sets may be expected to differ.

Although it could be argued that the calibration phase of physical modelling of storm events lacked statistical rigor, in proving the methodology, the results obtained have provided compelling evidence that the beach response follows the same general pattern of change - in both model and at full-scale in the prototype. This conclusion is supported by: (a) observations made at full-scale during storms; (b) visual interpretation of profile data sets; and (c) the processes observed during the model tests. All of these considerations suggest that the overwashing processes are reproduced adequately. The scales of the washover features formed in the model were of similar dimensions and plan-shape to those observed at full-scale. Studies undertaken on restrained beaches have produced run-up crests with similar characteristics to the crest of shingle barriers (Powell, 1990). Although the calibration tests do not prove conclusively that overtopping and overwashing processes were replicated accurately by the model, the evidence obtained was such that this technique is appropriate.

Strategic research on shingle beaches (Powell, 1990) has highlighted a number of shortcomings in the published research undertaken to date. In particular, the study of the effects of oblique wave attack and the influence of longshore sediment transport on the profile response of a beach has been recommended. Subsequently, a limited experimental programme was carried out to examine the effects of oblique wave attack on profile response (Coates and Lowe, 1993); on the basis of this, modifications to the original predictive formulae were suggested. The influence of beach grading and a depth-limiting foreshore on profile modification has not been addressed fully; this is of some significance when analysing the performance of barrier beaches which are typically perched on a horizontal platform of marsh deposits. High quality field data is needed to validate the results of laboratory studies, in order that numerical models may be of direct use in beach design and management. Limited field investigations have been carried out to test the validity of the numerical methods (Coates and Bona, 1997), but further validation is still required. Likewise, the profile development and characteristics of an overtopping barrier beach, or spit, have been addressed in only one investigation (Bradbury, 1998).

Physical model tests (Obhrai and Powell, pers. comm.) were undertaken to address gaps in the knowledge of the failure process of shingle barrier beaches. Tests were performed in one of the wave basins at HR Wallingford at a scale of 1:15 (Figure 3.2) 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 an 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 d_{50} of 42 mm and 53mm respectively. This allowed the effects of beach permeability on the threshold for failure of barrier beach to be observed.

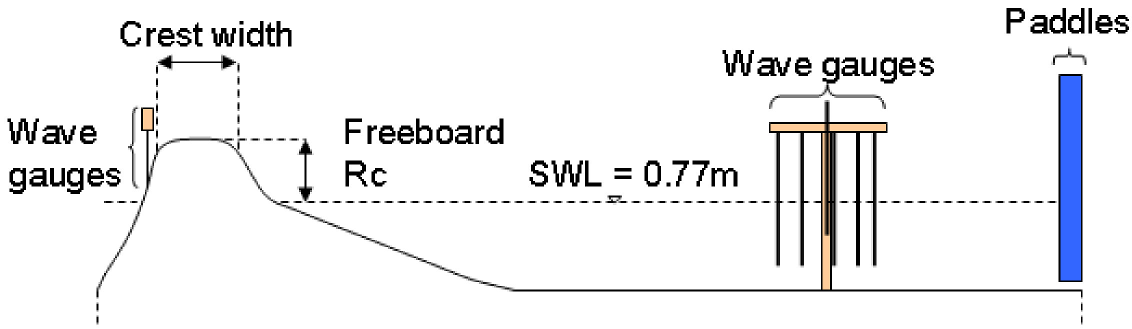


Figure 3.2 Experimental set-up – dimensions in model scale (Obhrai and Powell, pers. comm.)

One of the main objectives of the study was to investigate the effect of the barrier width on the threshold for breaching. To this end, three different crest widths were investigated (5m, 10m, & 15m prototype). Two different wave steepness were used (H/L=0.06 and 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. Details of the test conditions can be found in Table 3.1.

Table 3.1 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

The initial profile of each shingle beach was a slope of 1:7. Irregular waves with a significant wave height of 2m 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. An example of an initial profile is shown in Figure 3.3. A 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. Once the initial profile had been generated the wave height was increased incrementally by 0.25m for bursts of a 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 the laser scanner.

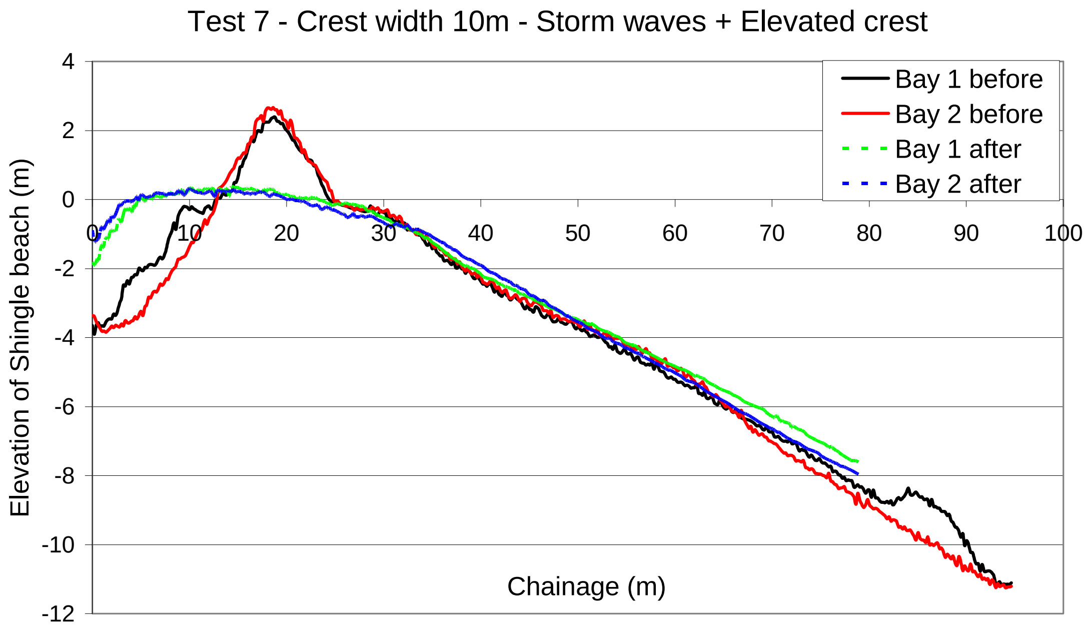


Figure 3.3 Example of an initial and failed shingle profiles from the physical model tests

3.3.2 Empirical models

A series of empirical frameworks have been established, that relate primarily to sandy barriers. The basic principles of these techniques may be worthy of further consideration in development of tools for managing shingle, or mixed-sediment, barriers with appropriate adjustments.

Morgan and Stone (1985) formulated a Storm Wave Susceptibility (SWS) Quotient for application to Florida's barrier islands following repeated comparisons of pre- and post-hurricane aerial photographs and beach profiles:

$$\text{SUSCEPTIBILITY} = \frac{E_I}{H2 + X_{BW} + \sqrt{X_{BF} + X_{EW}}}$$

Where;
H = primary dune height.
X_{BW} = beach width.
X_{SF} = shoreface width (extending to offshore longshore bar).
X_{EW} = effective island width.
The Energy Index (E_I) was assigned a value:
$1x10^4$ = high energy coasts.
$2x10^4$ = moderate energy coasts.
$3x10^4$ = low energy coasts.

The factors listed above are visually represented in Figure 3.4. The quotient was applied to barrier profiles (Figure 3.4) in order to approximate their susceptibility to wave power. A computer programme (STORMWAVE) was also written for SWS calculations at both mean and high tide.

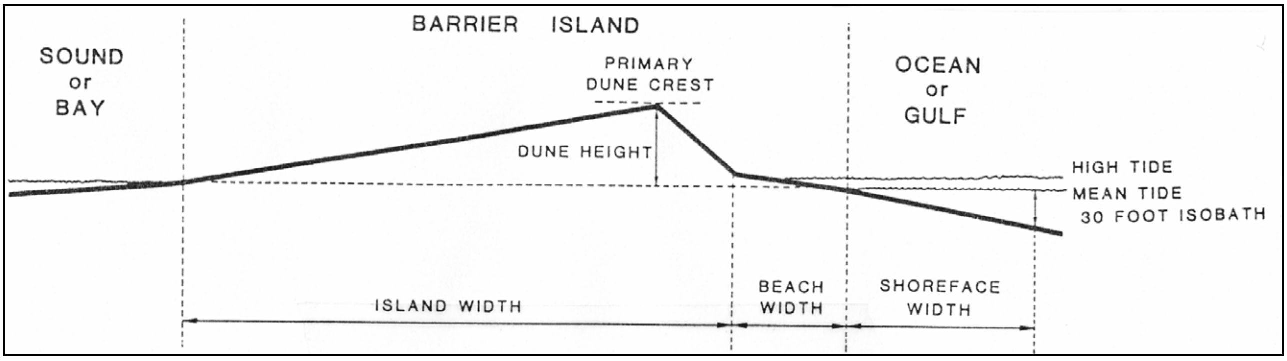


Figure 3.4 Profile of basic elements affecting Storm Wave Susceptibility (SWS) (Morgan and Stone, 1985)

Morgan and Stone (1985) found that dune height (H) was the most important factor with regard to vulnerability to wave power, whilst the shoreface width (X_{SF}) and the island width (X_{EW}) decreased in their relative importance. The criteria for the index were not precise, and therefore the number derived was only considered an approximation. Values are grouped:

70 = high storm wave susceptibility.
54-70 = moderate high storm wave susceptibility.
37-53 = moderate storm wave susceptibility.
20-36 = moderate/low storm wave susceptibility.
< 20 = low storm wave susceptibility.

As a development of the Storm Wave Susceptibility Quotient, Sanchez-Arcilla and Jimenez (1994) produced an Erosion Susceptibility Index (ES) for assessing the vulnerability of the Trabucador Bar, north-east Spain, to erosion (Figure 3.5):

$$\text{ES} = \frac{Z_t}{B_{2h} + \sqrt{(X_{bw} + X_b)}\,/100}$$

Where;
Z_t = Depth at top of longshore bar in front of barrier.
B_h = Barrier height above mean water level (MWL).
X_{bw} = Barrier width.
X_b = Distance between top of longshore bar and shoreline.

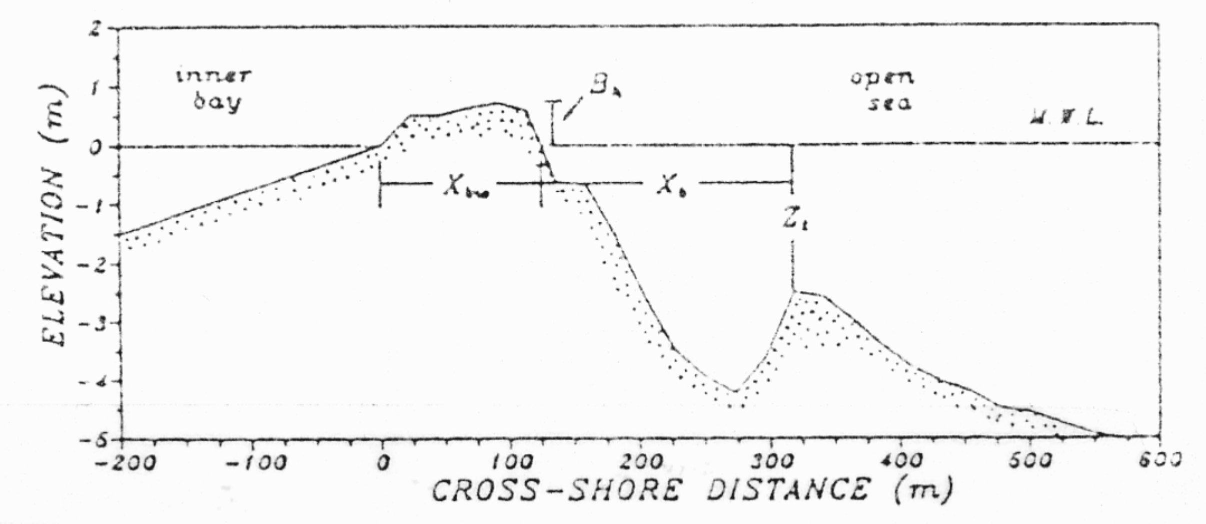


Figure 3.5 Typical cross section of the Trabucador Bar, North-east Spain. B_h, barrier height; X_{bw}, barrier width; Z_t, depth at top of longshore bar; X_b, distance between longshore bar and the shoreline. (Sanchez-Arcilla and Jimenez, 1994)

The higher the value produced from this index, the more vulnerable the barrier is to erosion. As with the SWS, the Erosion Susceptibility Index (ES) has a quadratic dependence on barrier height (B_h) (Sanchez-Arcilla and Jimenez, 1994) as it is considered to be the main parameter controlling over-washing. Barrier width (X_{bw}) and distance to the barrier (X_b) are also important but are less significant. Both of these formulae were designed for wave-dominated, sandy barrier islands, which have a totally different morphology and hydrologic environment compared with UK coarse-clastic examples.

Powell (1990) has provided the most recent and most relevant series of formulae to describe the shape of a shingle beach; these relate to an extensive series of random wave tests, undertaken in a 2-dimensional random wave flume. Powell (1990) describes the shingle beach profile by three hyperbolic curves: from beach crest to the static water level shoreline; static water level to the top edge of the step; and the top edge of the step to the lower limit of profile deformation.

The following variables were considered in development of the model

Significant wave height	Hs
Wave period	Tm
Grain size	D50
Wave length	Lm
Water level	SWL

Approach slope
Depth of sediment

A series of functional relationships were produced for the profile descriptors and, on the basis of dimensional analysis, three dimensionless parameter groupings were derived:

a) H_s / D_{50} , the ratio of wave height to sediment size

b) H_s / L_m, wave steepness, and;

c) $H_s T_m g^{1/2} / D_{50}^{3/2}$, the ratio of wave power to sediment size

A suite of empirical equations were derived, allowing wave run-up distribution, wave reflection coefficients and the beach profile response to be described. Development of a parametric profile model allows the quantification of shingle beach profile changes, due to onshore/offshore sediment transport. Profile equations for the upper segment of the profile are summarised in Table 3.2.

Table 3.2 Summary of functional relationships for use as beach profile descriptors (Powell, 1990)

Functional Relationship	Limit of Applicability
$p_r / H_s = 6.38 + 3.25 \ln(H_s / L_m)$	$0.01 < H_s / L_m < 0.06$
$p_c D_{50} / H_s L_m = -0.23 \left(H_s T_m g^{1/2} / D_{50}^{3/2} \right)^{-0.588}$	$0.01 < H_s / L_m < 0.06$
$h_c / H_s = 2.86 - 62.69 (H_s / L_m) + 443.29 (H_s / L_m)^2$	$0.01 < H_s / L_m < 0.06$

The model provides an estimate of the dynamic equilibrium beach profile that will form for any given combination of conditions, within the range of validity. The model has been developed for clearly defined combinations of conditions and ranges of validity and confidence limits are stated for each. Importantly, the model is not intended to describe the upper part of a barrier profile under overtopping conditions.

The model is designed to run on a beach profile of defined geometry. It is assumed that there is no net loss of material from the profile. Calculations are made of the profile descriptors relative to static water level at the storm peak. The application of this model, which was designed for use on restrained beaches, is discussed further in Section 4.2

Bradbury (1998, 2000) formulated the dimensionless barrier inertia parameter to identify threshold conditions for overwashing of barrier beaches, based on physical model tests of a wide range of barrier cross section geometry. The model builds upon the empirical framework developed by Powell (1990) and is the first attempt to quantify the onset of overwashing on a shingle barrier beach, but defined with limited variables. The model was developed following a site-specific study into crest evolution of barrier beach profiles and the conditions that promote overwashing, overtopping and crest cut-back; this investigation was subsequently expanded to examine a broad range of beach geometry and hydrodynamic conditions, but was confined to examination of a single size grading of beach and approach bathymetry. Investigations included both laboratory experiments, using a 3-dimensional mobile bed random wave physical model (Bradbury, 1998), and field verification at Hurst Spit (Bradbury et al 2005).

Observations made during the development of the parametric framework, of relevance to barrier evolution, are highlighted below.

A restrained beach responds to lengthening wave period, through increased wave run-up and the raising of the elevation of the foreshore crest. On exceeding the overwashing threshold, lengthening T_m causes more rapid changes to the barrier crest elevation. The transition zone (containing both overwashing and overtopping) reflects, perhaps, the influence of solitary waves or groups of long period waves on the crest. The large volumes of water contained within a single wave can result in significant changes at the crest. Elevation changes were generally <0.5m when overtopping resulted, whilst overwashing could result in crest elevation reductions of several metres.

The influence of changing water level on a barrier beach is complex. For a barrier of particular geometry, the freeboard reduces between the water surface elevation and barrier crest as the water level increases. This effect impacts on the barrier, by reducing the surface emergent CSA and the span, at SWL; it affects nearshore depth-limited wave conditions. For similar offshore waves, the higher waves impinge higher up the profile. Tidal surges, in conjunction with storm wave conditions, are likely to be important in controlling sediment and profile response.

Changes in crest elevation of <0.5m resulted from overtopping; the response was less well-ordered, when overwashing and crest reduction occurred. This pattern suggests that crest evolution is sensitive to freeboard, CSA and spatial variation in the barrier geometry. Overtopping was generally confined to a narrow range of conditions; the maximum crest elevation increase observed, in response to this process, was 1m. However, most of the elevation changes were within the range 0.1-0.6m. In contrast, overwashing resulted in a wide variety of crestal changes; these ranged from minor reductions, to the lowering of the crest below SWL under extreme conditions.

Whilst crest evolution is clearly linked with freeboard, the cross section of the barrier is also significant. The influence of the barrier cross section has been considered only rarely in studies of barrier evolution, although it has been suggested that sub-decadal evolution may be a function of CSA and barrier height (Orford *et al*, 1995). Barrier freeboard is often, although not necessarily, linked with barrier cross section; hence, these two variables must be considered separately, to analyse crest response. The effects of increasing surface emergent CSA, examined for common crest elevation, have indicated that a larger barrier is more likely to result in crest accretion than a narrow barrier, (under similar hydrodynamic conditions): smaller barriers are likely to undergo overwashing. CSA is an important variable, which must form part of a dimensionless parameter group; it can be used alone, but this provides no indication of the relative height or width of the barrier.

The model observations have identified the apparent influence of wave grouping on crest evolution, for conditions close to the overtopping and overwashing thresholds. The instrumentation used and test procedures adopted did not permit the impacts of solitary waves, or groups of waves, to be monitored within context of profile response; however, this was noted qualitatively. The absence of such data may explain some of the scatter of results close to the overtopping and overwashing thresholds.

The influence of storm duration may be more significant in the formation of profiles on barrier beaches. Waves exceeding the pre-storm beach crest may result in overwashing, crest reduction and lee slope deformation. A second stage of profile development follows, during which the overwashed roll-back profile evolves to reach a new dynamic equilibrium; this is influenced by storm duration at a defined water level. The inclusion of a time-dependent variable into the analysis of profile development is complicated by the initial beach geometry. Similarly, a test duration of 3 hr may be insufficient to allow a dynamic equilibrium overwash profile to form under sluicing overwash conditions; however, it should be adequate to describe a restrained or overtopped profile, when water level conditions result in small quantities of overwashing. The tests were designed to examine the profile response, based upon a SWL coinciding with a tidal stand over 3 hr. An equilibrium profile was not achieved within the test duration for some of the overwashing conditions; this occurred usually when the barrier CSA was small, relative to the prevailing hydrodynamic conditions.

A barrier with a small CSA is found to be more likely to be subject to crest lowering by overwashing than one with a larger CSA but with the same freeboard. Overwashing events do not necessarily result in crest lowering; this is essentially a function of CSA. The effects of a large CSA, but low freeboard, can result in overwashing and the formation of the crest at a higher level than the initial profile. However, for this to occur, the volume of the barrier must be sufficiently large to permit the dynamic equilibrium profile to form within the barrier. If the barrier is high and narrow, it will perform differently (in response to hydrodynamic forcing) than if it is wide and low. Freeboard can be used to determine the overtopping threshold, but CSA is found to be a better variable for the determination of overwashing. When combined, the two variables provide a barrier inertia grouping; this is a function of barrier freeboard and mass, which can describe evolution of the barrier in systematic manner. This relationship can be non-dimensionalised by wave height to provide a dimensionless barrier inertia parameter:

$$B_i = R_c B_a / H_s^3$$

This relationship is similar to the dimensioned barrier height and CSA relationship proposed for decadal evolution (Orford *et al*, 1995); it differs in terms of the absence of any reference to hydrodynamic variables; the use of mean sea level (as opposed to storm peak freeboard); and barrier height (as opposed to freeboard).

Bradbury (1998) presented a summary of a barrier response to storm wave conditions, identifying the range of geometric configurations that were subject to crest elevation change, during model investigations. Barrier crest evolution responses are shown, without reference to hydrodynamic conditions, and for various combinations of freeboard, CSA, and barrier width (Figure 3.6). Each point denotes the response of an individual profile. All events shown have resulted in crest modification. Although the hydrodynamic conditions are not shown in the Figure, crest evolution data are concentrated where the barrier is low, and has a small volume. The overwashing threshold, for the most severe test conditions, occurred approx. when CSA<80m^2 and R_c<4m.

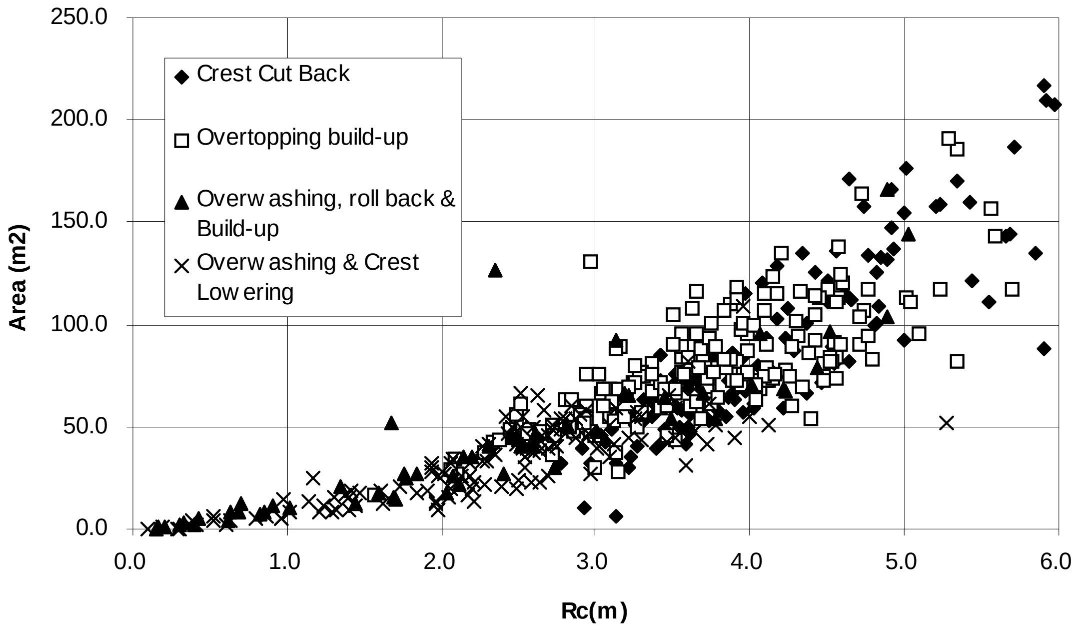


Figure 3.6 Barrier crest response as a function of freeboard and area: crest modification data only (Bradbury, 1998)

The influence of any spatial variation in barrier geometry cannot be ignored, although the data do not permit a clear relationship to be determined between such variability and other controlling variables.

On the basis of the model results, wave height, period and water level have been identified as the most significant of the controlling variables.

A summary of the range of model test conditions is given below.

Environmental variables		Range
Wave height, at beach toe	H_s	1.1-4.1m
Wave period	T_m	7.4-10.9s
Wave length	L_m	85.5-185.5m

Number of waves	N	990-1460
Angle of wave approach	ψ	0-20^{O}
Water depth, at beach toe	h	7.9-9.3m

Constants used in the model test programme		
Spectral shape		JONSWAP
Mass density of water	ρ	1
Acceleration of gravity	g	9.81ms^{-1}
Structural variables		
Barrier crest freeboard	R_c	-0.37 to 7.8m
Supra-tidal CSA	CSA	0.1 - 433m^2
SWL span	SWL_s	2.9 - 110m
Nominal shingle grain diameter	D_{50}	0.015m
Foreshore slope angle	Cot α	5-20
Mass density of shingle	ρ_a	2.65

Data was collected for a range of geomorphological and hydrodynamic scenarios, and dimensionless groupings determined. These are defined as the barrier inertia parameter (R_cB_a/H_s^3) and the wave steepness parameter (Hs/Lm). Threshold curves have been determined by analysing conditions for the onset of overwashing. Overwashing is predicted when the critical barrier inertia threshold is exceeded (Bradbury, 1998). Predictive curves identifying the threshold conditions are given below:

If

$$\frac{R_c B_a}{H_s^{\ 3}} < 0.0005\left(\frac{H_s}{L_m}\right)^{-2.545}$$

Overwashing is likely to occur.
The upper confidence limits for this threshold are presented below

$$\frac{R_c B_a}{H_s^{\ 3}} = 0.0006\left(\frac{H_s}{L_m}\right)^{-2.5375}$$

Rc = crest freeboard, level of crest relative to still water level.
Ba = supra-tidal barrier cross sectional area.
Hs = significant wave height (the average of highest one-third wave heights)
Lm = wave length of mean Tm period.
Tm = mean wave period.

The relationship determined effectively demonstrates that a barrier with low elevation and cross section above static water level at the storm peak is more vulnerable to overwashing than a larger barrier.

The field and model data used to develop the empirical model of Bradbury (2000) related only to the shingle barrier at Hurst Spit. Bradbury et al (2005) found that the model was generally not applicable at other sites and concluded that use of the model outside the valid predictive range would result in the under prediction of overwashing. The parametric model of Powell (1990) is used to predict the dynamic equilibrium profile that will develop for any given combination of wave conditions assuming that there is sufficient time and sediment available for the profile to form. This limitation means that the model is not valid for the prediction of overwashing and breaching of shingle barrier beaches. Further data obtained by HR Wallingford as part of the FLOODsite research programme (Obhrai and Powell, pers. comm.) has been used to test and extend the range of validity of these models to predict the threshold for breaching of shingle barrier beaches.

The definition of breaching used within the FLOODsite investigation was the short-term lowering of the barrier crest resulting from wave induced overwashing (Bradbury, 2000). The dimensionless barrier inertia parameter (Bradbury, 2000) was used to estimate the threshold of breaching. The model is valid in the range 0.015 < H_s/L_m < 0.032. Figure 3.7 shows a comparison between the threshold curve and the field data used to derive the curve combined with the new physical model data. Being below the curve implies that breaching will occur. It is clear from the latest physical model data that extrapolation of the original empirical model is not valid and that the predictive curve needs to be modified. Three different types of regressions were investigated (linear, exponential and logarithmic) and the results show that all three represent an improvement on the Bradbury threshold in view of the new data. However the simple linear fit provides the best description of the upper limit for the threshold for breaching and can be described as follows:

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

and is valid for the range 0.01< *Hs/Lm<0.06*.

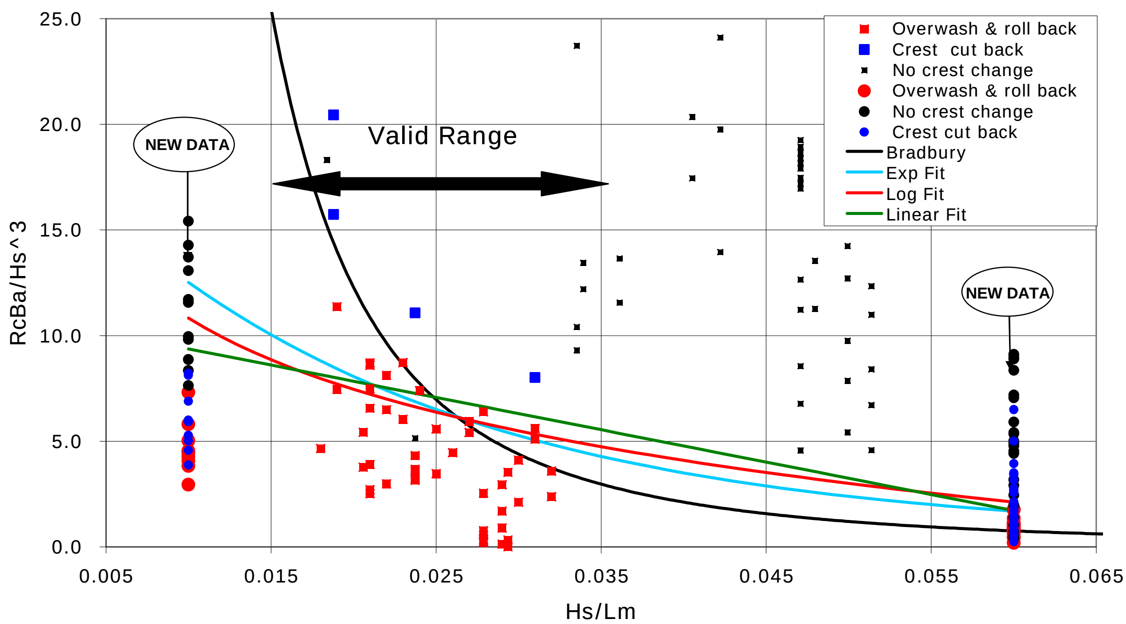


Figure 3.7 A comparison between the empirical approach of Bradbury (2000) and the combined field and model data (Obhrai and Powell, pers. comm.)

Using the results from the hydraulic model tests it was possible to asses whether the parametric of Powell could be used to predict the failure of shingle barrier beaches. The SHINGLE model appeared to perform well under the storm wave conditions particularly for the less-coarse sediment. Figure 3.8 shows an example of the measured profile, with the initial profile input to the model, the equilibrium profile for H_s of 2m and the observed failed profile. The position of the crest for the failed H_s is close to the rear of the crest which suggests that SHINGLE would have predicted failure at the correct threshold H_s in this case.

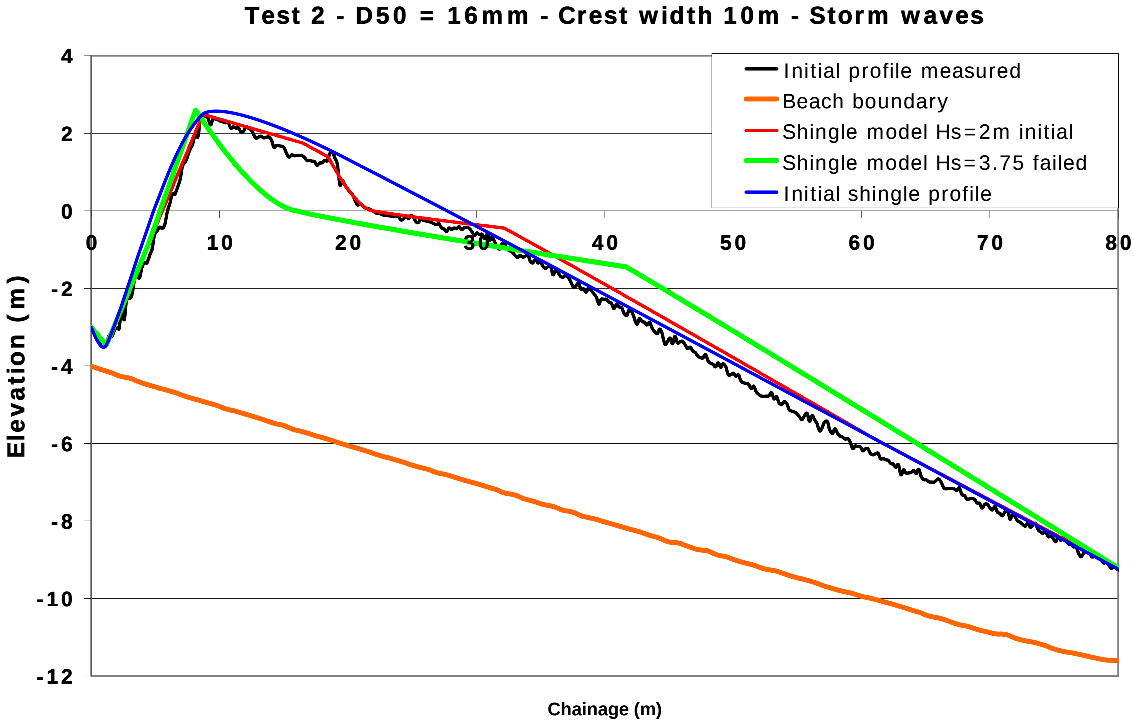


Figure 3.8 Predictions of SHINGLE compared to the results of Test 2 with the less-coarse sediment (Obhrai and Powell, pers. comm.)

The original hydraulic tests used to derive the SHINGLE model were based on sediment of a similar size to the less-coarse material that was used (d_{50}=16mm). However SHINGLE was not calibrated to work with much coarser sediment similar to that used in Bay 4 (d_{50}=57mm). It is therefore not surprising that it did not work so well for the coarser sediment used. Figure 3.9 shows the same test as the previous figure but shows the results for the coarser sediment. SHINGLE predicts 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.

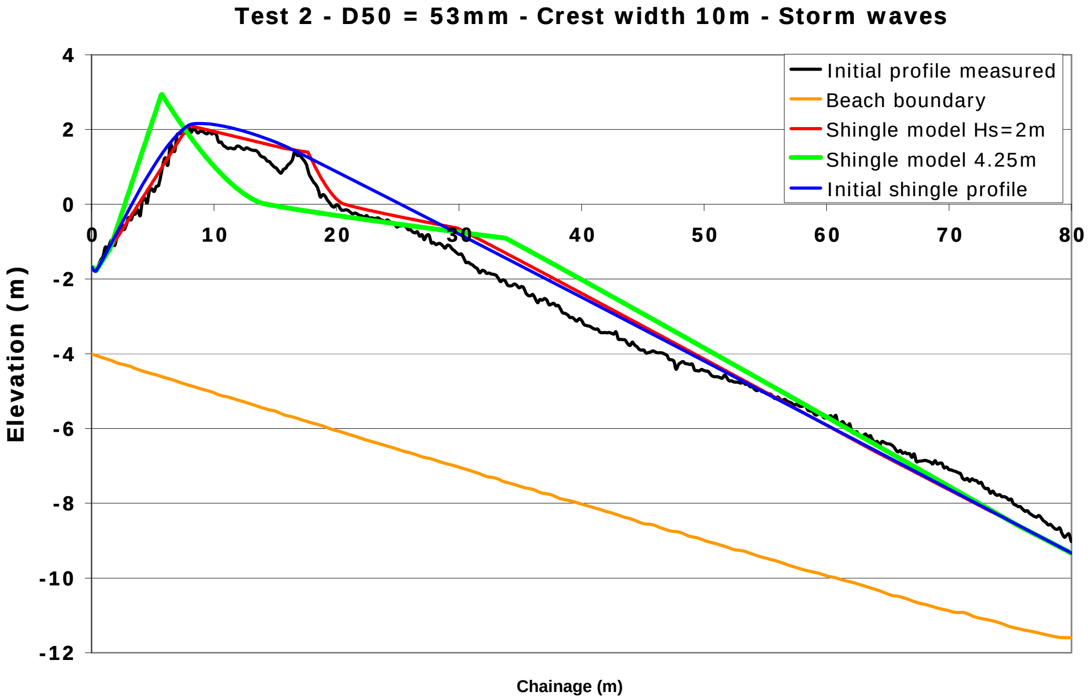


Figure 3.9 Predictions of SHINGLE compared to results of Test 2 for the coarser sediment (Obhrai and Powell, pers. comm.)

SHINGLE did not perform well for the swell wave conditions due to the fact that the original tests from which the model was derived did not include many swell wave conditions. Figure 3.10 shows an example from one of the swell wave conditions which illustrates that SHINGLE is unable to predict the correct position or elevation of the crest for the breached or even the initial profile. In general SHINGLE predicted a much higher crest elevation than was actually measured for the swell wave conditions.

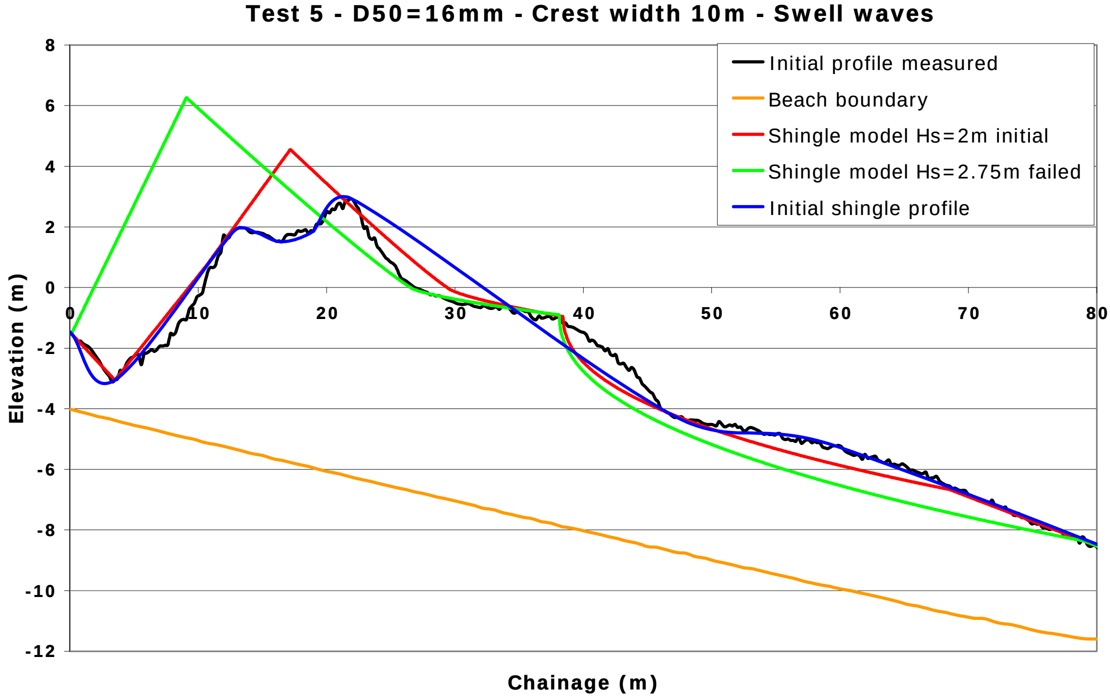


Figure 3.10 Predictions of SHINGLE compared to the results of Test 5 with the swell wave conditions (Obhrai and Powell, pers. comm.)

This latest research has identified uncertainties in the influence of a series of processes on the morphodynamic response of a coarse-clastic barrier beach. These processes include:

1) Permeability
2) Hinterland levels
3) Internal structure of the beach
4) The total volume of sediment in the system not just the volume above the SWL as in the Bradbury model. This was of particular importance for the swell wave conditions where sediment was removed from far offshore and pushed up the beach.
5) Run up level

The Bradbury Barrier Inertia model has the advantage that is a relatively simple method to apply to a limit state equation which is needed to generate a fragility curve. However it does not take account for the effect of beach permeability or hinterland levels on the failure process. These processes were observed to be important factors during these recent experiments.

The revised 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 a conservative upper limit for the failure threshold as it does not include all of the processes involved in barrier beach morphology.

The SHINGLE approach does appear to work well under storm conditions particularly for the less-coarse sediment. However it does not work well under the swell wave conditions and for the coarser sediment. 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.

Following the advances made in understanding the limitations of current predictive methods, and extending the range of validity of Bradbury's Barrier Inertia model through a relatively limited series of physical model experiments, it would seem intuitive that greater progress could be made from a series of more thorough and extensive testing programmes.

3.3.3 Numerical approaches

For the time-being, numerical modelling approaches are by necessity divided into those models which represent short-term storm-response morphodynamics (days), models which represent medium-term morphodynamics (~10 years), and models which represent long-term morphodynamics (~1000 years).

In terms of short-term storm-response modelling, no references have been found in the literature purporting to deal with the morphodynamics of coarse-clastic barriers. Numerical models which deal with the short-term response of fine-clastic beaches do exist (e.g. Southgate and Nairn, 1993), but these models are generally only representative of some of the processes governing the behaviour of the seaward face of a barrier, and are not applicable to barriers as a whole.

More recently, Donnelly *et al.* (in press) derived an overwash algorithm which, when coupled to the beach profile response model SBEACH (Larson and Kraus, 1989) was designed to simulate the evolution of an overwashing fan. The algorithm takes sediment transport rates at the surf/ swash-zone boundary (as supplied by SBEACH, but could be provided from other beach profile models) as input. Sediment transport over the beach or dune crest is considered to be a function of the local depth, the overwash regime (i.e. whether it is caused by run-up or inundations), and flow velocity at the beach crest.

The flow velocity at the crest is translated into a volume, and the volume of sediment transport over the barrier crest is considered to be proportional to the overtopping flow rate. Due to the force of gravity on the overtopping volume, represented as a block, it accelerates down the back of the barrier. The action of friction induces a consequent steady flow state, which in this model is assumed to be immediate. As the block of overtopping water travels down the back of the barrier, it is deemed to spread and becomes shallower due to infiltration. Sediment transport down the back barrier was assumed to be proportional to the velocity of the "block" of water cubed.

The algorithm was calibrated against field data from the sandy east coast of America, which consists of pre- and post-storm profile surveys and measured shallow water wave conditions, and a range of calibration parameters were obtained. Model verification was reasonably successful, and this was partly due to the extent of the available observations. This work represents an emerging science, and is therefore relatively untried and tested, but could be extended in it's application in the event that sufficient datasets were available. The applicability of the method to coarse-clastic barrier beaches would also have to be assessed.

Other work which has investigated the short-term behaviour of coarse- and mixed-grained beaches was described in the Defra Mixed Beaches research programme; commission FD1901 (HR Wallingford (2002), Lopez de San Román-Blanco (2003) and Lopez de San Román-Blanco *et al.* (2003)). One of the objectives of this research was to consolidate previous MAFF (now Defra) funded work on surf zone hydrodynamic over porous beaches (for example the OTTP – ANEMONE (Dodd, 1998; Dodd et al., 2000)) and Shingle Beach Research Programmes (Coates *et al.*, 1999). This involved further development and validation of the OTTP-1D (Peet and Dodd, 2000) cross-shore beach model.

The OTTP-1D model is an extension of the OTT-1D model (Dodd, 1998). OTT-1D simulates wave transformation on a natural beach from the inner surf-zone, wherein most waves have already broken, through to the moving shoreline and beyond (i.e. overtopping and flooding). It can also be used to simulate wave action on a sloping, impermeable structure. Its main use, therefore, is in simulating overtopping in terms of instantaneous rates, volumes and velocities and numbers of events, as well as all mean quantities. An example of a situation where OTT-1D would provide effective modelling is shown in Plate 3.2. The situation depicted in this Plate of an alongshore uniform seawall, so that under normal or near-normal wave incidence the 1D model OTT-1D is appropriate. Figure 3.10 shows a snapshot of an OTT-1D run and analysis.

Plate 3.2 Wave run-up and overtopping of a sloping seawall, Prestatyn (Dodd *et al.*, 2000)

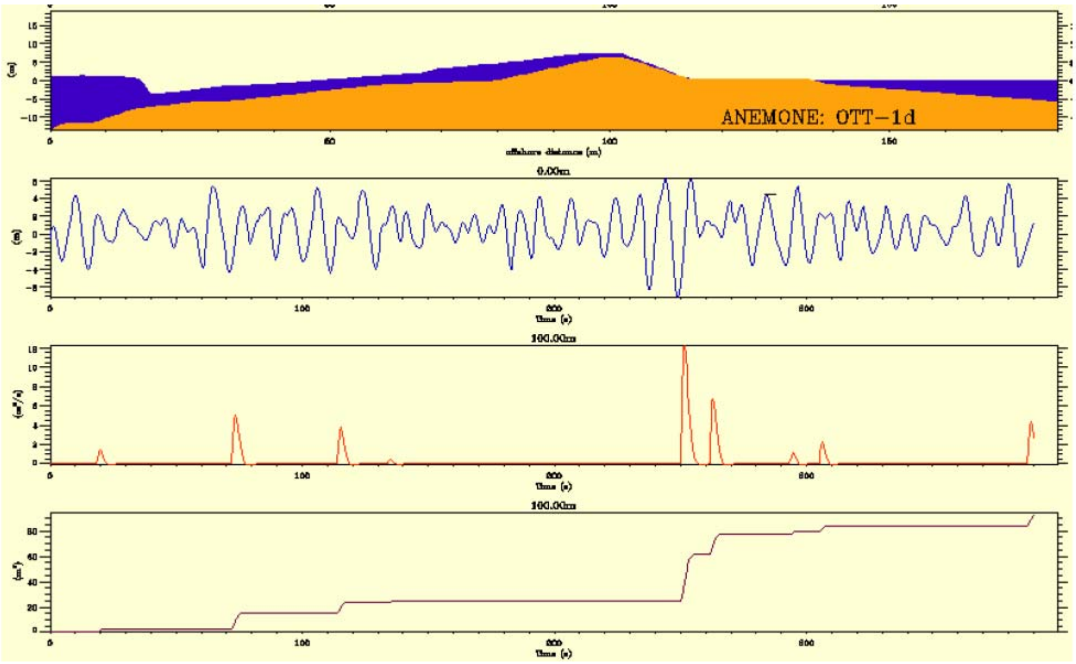


Figure 3.10 Example of an OTT-1D run and overtopping analysis

OTTP-1D is similar to OTT-1D in its limitation to very nearshore motions, but also has the capability of simulating water movement on and within a porous beach or structure. It includes the capability o, therefore, of simulating overtopping in much the same circumstances as OTT-1D, but including water passing through a permeable seawall or rock revetment. It can also be used to simulate overtopping of and transmission through a breakwater, and is useful for simulating run-up and run-down on a shingle beach. To date, the authors have not stated whether they view the applicability of the OTTP-1D model to barrier beaches, although it is likely that this is the case. A snapshot of an animation is presented in Figure 3.11 where a gravel beach is seen above an impermeable layer, which itself is overtopped.

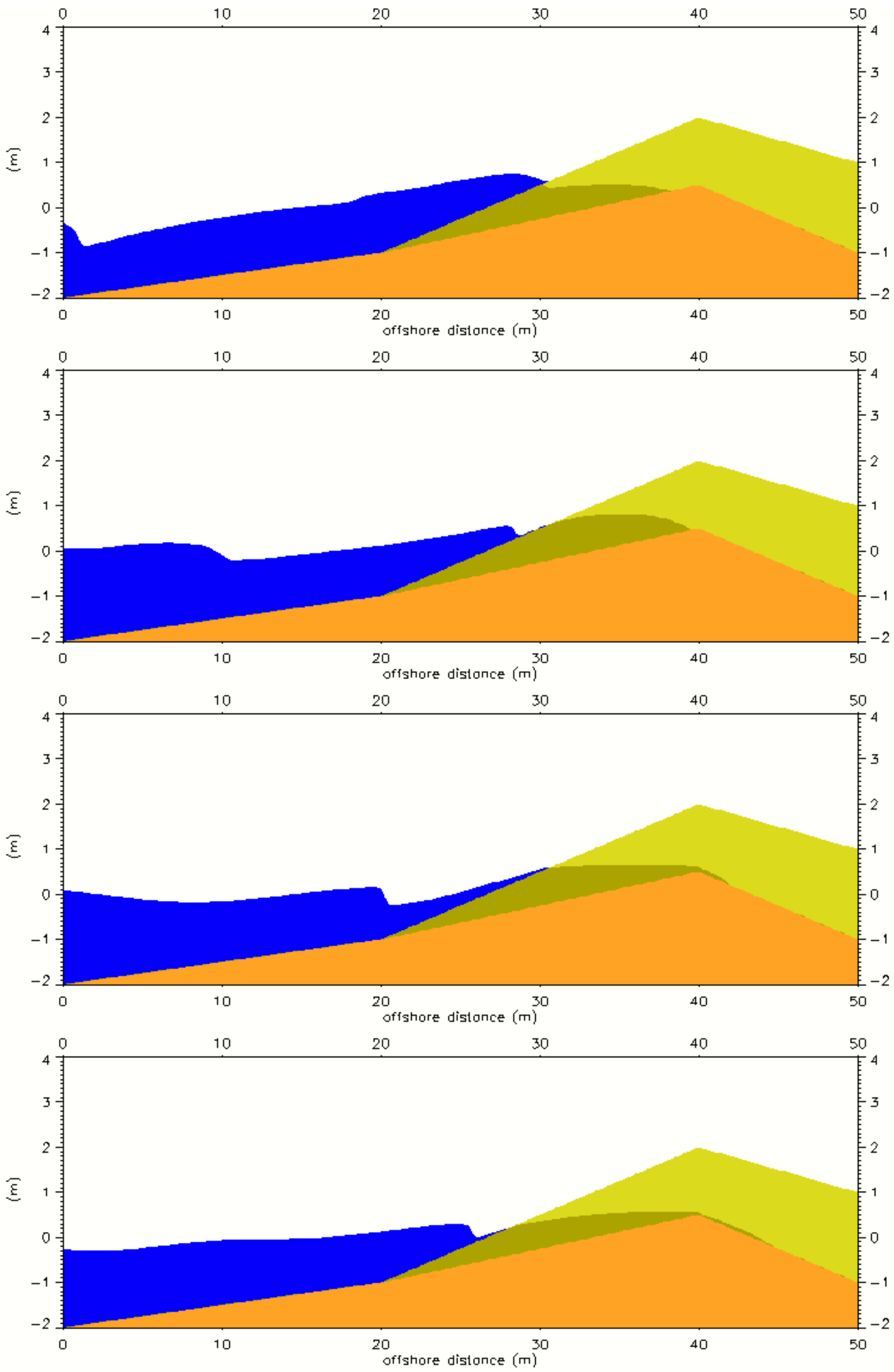


Figure 3.11 Water table response to incident surface waves for a gravel beach with an impermeable core

The Defra commission FD1901 investigated the possibility of extending the OTTP-1D (one-dimensional swash zone model with a porous layer) towards a morphological capability (Clarke *et al.*, 2004). The research concluded that further work was required since the task of representing the complex physical behaviour was more onerous than initially thought, but that some progress was made. Nevertheless, the model was considered suitable for predicting wave processes, velocities, overtopping of the beach crest and flow over an impermeable beach core. Early validation results for the morphological model of gravel beaches were encouraging.

In the medium term, it is common for a 1-line longshore transport model (e.g. Brampton, 1980) to be applied to the seaward face of a barrier, although again, such models are not strictly applicable given the processes governing the evolution of barriers. Nevertheless, these models have the advantage of being able to represent the dynamics of coarse-clastic sediment, and may be used to examine barrier shoreface behaviour to some degree.

An interesting development in this regard has been offered by Jiminez and Sanchez-Arcilla (2004). Their model of the Trabucador-La Banya barrier, Ebro delta, Spain, is based on system behaviour, and therefore requires observation made over several decades. The evolution model is divided in to two modules – the littoral drift model, on the seaward face of the barrier, and an over-washing model that is dependent upon barrier width. The littoral drift model is a curvilinear 1-line model. The deterministic aspect of the metocean model is heavily parameterised, and most transport derivation aspects are eliminated.

However, the results are very promising, with the method offering distinct advantages with regard calibration effort. Whilst the results are hopeful, the model was successfully tuned to observed behaviour spanning 30 years – a period during which there was no breaching of the barrier. As a result there is no attempt to deal with breaching and the difficult subject of tidal-inlets. Straightforward application of this model is likely to be prohibited by the requirement for detailed long-term observations. Nevertheless, this tool appears useful for investigating uncertainty in future forcing conditions. In addition, there may be methods used here which, on further investigation, may be found to be transposable to coarse-clastic barrier beaches.

With regard the long-term evolution of barrier beaches, models appear largely to be simplistic and behaviour-oriented. There are currently no deterministic process models able to represent the processes of roll-back and breach, for example. Some techniques, e.g. Cowell et al. 1995, allow the “injecting of qualitative experience” during execution. Such a method allows the modeller to correct any wayward hindcasting – in effect allowing an ongoing calibration of the model to be carried out.

Other developers, such as McBride et al. (1995), have attempted to assign geomorphic response-types to shorelines given historical changes in shoreline position over >100years. Extensive datasets are critical in assigning the behaviour characteristic of the barriers. All processes are synthesised in the model to produce a response-trend, which is evaluated from long time-period datasets.

Essentially, long-term modelling effort is based around the “accommodation-space” concept, whereby the beach moves landward in response to rising sea-levels, provided that there is enough space landward of the barrier for it to transgress. Entire coastal cells are often represented by a single aggregated shore-normal profile, and no short-term morphology of this profile is examined.

3.4 Recommendations for further work

This Chapter has demonstrated the emphasis that researches have placed on developing parametric models for studying barrier beach morpohology, and it also highlights the limited options available for practitioners who are trying to assess the performance of barrier beaches or the effect of management practice. As a result, the tools available are often stretched in their applicability.

Nevertheless, recent research has demonstrated the value added to the study methods by even a limited set of physical model tests. This may purely be a facet of the narrow set of observations that the parametric models were built upon in the first instance, and that as datasets increase in size so added value decreases. It also demonstrates the relative infancy of barrier beach science and application.

The following bullet points provide an indication of the types of future research suggested following this review of study methods.

- Some detailed measurements of internal flow through barrier beaches have been found to provide useful information. It is recommended that such approaches are investigated in greater detail, and the methods and exploitation of outcomes subject to refinement.

- Descriptors exist which enable shingle beach face morphology to be estimated, and effort has been made to include crest and back barrier morphology. However, these methods are restricted in their ability to represent barriers of different type and varying forcing conditions. It is recommended that such descriptors are further investigated and refined to enable wider applicability. This includes ensuring that field and laboratory observations are accommodated.

- A beach with impermeable cores have been shown in the laboratory to behave differently when compared to a fully permeable beach. None of the empirical methods for studying barrier beaches account for this fact. As many shingle barriers around the UK have a relatively impermeable core any reliable application would be need to include this process, and because of this, it is recommended that further investigations be carried out.

- Studies to date have shown 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. However results from recent physical model tests have shown that the land levels in the hinterland were also an important factor in the resilience of the shingle barrier. In particular it affected the ability of the shingle barrier to reform once it had been overtopped. In a case where there were raised hinterland levels, once the barrier was overtopped sediment was washed into the hinterland and effectively removed from the system and the barrier was therefore unable to reform. This aspect has not been fully investigated and it is recommended that further work be needed to asses the effect of hinterland levels on the failure process.

- It is noted that effort has been made to relate emergent CSA to over-topping and overwashing events, but with limited success and/or applicability. It is recommended that further research be conducted which should aim to develop understanding of how one might relate to the other and to produce a descriptor. It would seem appropriate to incorporate such a descriptor within the RASP-type fragility assessment. The finding from recent research that longer wave periods might alter the definition of CSA should be investigated further.

- Whilst there is a general lack of numerical models capable of representing the processes on coarse-grained beaches and barrier beaches, some guidance could be sought from "sandy" studies such as those described by Jiminez and Sanchez-Arcilla (2004) and Donnelly *et al.* (in press). It is recommended that such methods are investigated in some detail, and an assessment of whether some of the ideas and principles presented could be related to coarse-grained barriers. Such methods may constitute the foundations for the development of a morphological process-based predictive tool. It is vital that any model development should maintain strong links to data sources, and ideally be capable of simulating longshore connectivity – perhaps on a broad-scale.

- A further possibility for applied research would be to develop the numerical model, or the principles and science at the core of, ANEMONE OTT1P (Clarke *et al.*, 2004). This model represents groundwater levels and flows within a permeable beach, but this would need improvement and validation before it could be offered as a reliable tool. Part of this process would be to represent the permeability of the sand/ gravel mix in a shingle beach in a realistic manner. A full morphodynamic capability should be sought, and a methodology developed to describe how such a model could be utilised in association with the long-term numerical model developments described above.

A nearshore morphodynamic process-based deterministic model should enable:

a) quantification of the modification of the unconfined barrier crest by wave overtopping;

b) investigation of a range of hydrodynamic and geometric controls on the development of the beach profile;

c) the identification and quantification of the first-order hydrodynamic and geometric threshold conditions which give rise to crest level raising by wave run-up and crest level lowering by overtopping;

d) consideration of an advancement in the understanding of percolation bought about by monitoring of permeability and the Beach Permeability research programme conducted under Defra commission FD1923;

e) the influence of falling water level on an overwashing barrier to be examined.

Any future research projects should aim to ensure that further developments are valid for the full range of conditions in the UK.

4. Application of state of the art to coarse-clastic barriers

This Chapter provides an indication of how several of the modelling methods for studying barrier beaches described in the previous Chapter can be applied in support of barrier beach management. Whilst various methods for studying barrier beaches have been described (Chapter 3), relatively few are routinely applied. This is indicative of the general absence of definitive methods which are appropriate to the barrier beaches around the UK coastline. Most notably, the research effort to date has concentrated on sandy beaches (e.g. Jimenez and Sanchez-Arcilla (2004), and where coarser material has been considered (e.g. Clarke *et al.*, 2004), the complex behaviour has proven difficult to describe and deterministic models are themselves correspondingly complex.

As a consequence, simpler parametric approaches (which generally cost less to apply, and appear to offer greater value for money) are generally adopted. In England and Wales, this generally means that the practitioner is restricted to a conceptual-type model when investigating long-term morphology and how it might be influenced by sea level rise, for example, whereas when investigating short-term behaviour, the industry is restricted to the application of models such as those described by Powell (1990) and Bradbury (2000). These methods are workable, but how applicable they really are is uncertain.

The following sections describe the application of various common methods, describing strengths and weaknesses where appropriate. Of note is the limited options available to practitioners when attempting to qualify and quantify coarse-clastic barrier beach morphology.

4.1 Conceptual models of morphodynamic forcing

The Coastal Engineering Manual, Part III, presents a discussion of the application of a modified ‘Bruun Rule’ for barrier island migration in response to sea level rise (Dean and Maurmeyer, 1983). Although Bruun (1962) suggests a relationship between sea level rise and shoreline migration which is applicable to all grain sizes, the validity of the ‘Bruun Rule’ is unproven for coarse-clastic barriers. Orford *et al* (1991a) have provided qualified evidence in support of the Bruun Rule, for coarse-grained systems, recognising the importance of wave activity but questioning the representativeness of barrier type used in their investigation.

Carter and Orford (1993) discuss a conceptual model describing medium-term changes on coarse-clastic barriers, linking wave climate, morphology and sea level rise to crestal build up and barrier migration. As the relative width of the seaward slope increases, the slope becomes increasingly dissipative, even to the point where the crest ridge may become abandoned; this is especially significant if accompanied by a fall in sea level (Carter and Wilson, 1992). If the barrier progressively loses material, migration is likely to increase and it may ultimately breach (Carter, *et al* 1987).

Whilst sea level is a passive plane, it forms the basis for dynamic mechanisms such as wave activity. Evidence for the landward migration of gravel barriers in response to sea-level rise has been provided for Loe in Cornwall (Hardy, 1964), Chesil Beach (Carr and Blackley, 1974); and in Ireland (Carter and Orford, 1981). These hypotheses are developed by inference, but Orford *et al* (1993, 1993a) has presented evidence, at a decadal time scale, of relationships between sea level rise and barrier migration in Nova Scotia.

Much research on the evolution of shingle barrier beaches has focused upon the effects of sea level rise on the transgression of the beach (Orford *et al*, 1995). Whilst recognising that barriers are influenced by the effects of storm waves, few researchers have attempted to quantify these effects. Changes in mean sea level are seen as the primary driving mechanism for the evolution of barriers (Hardy, 1964; Carr and Hails, 1972); this must assume a non-varying wave climate over the period of sea level change. The effects of relative sea level rise on the geomorphological response of swash-aligned gravel barriers has been examined over a range of temporal scales by Orford *et al* (1995); McKenna *et al* (1993); and Carter *et al* (1993). Evidence is presented for a linear relationship between sea-level rise, barrier inertia, (specified in terms of height and cross section geometry) and barrier migration; this was based upon three gravel barriers over the period 1837-1986. The importance of barrier cross section and elevation is emphasised and it was suggested that the smaller the cross-sectional area, the more rapid the retreat regardless of sea-level rise (Figure 4.1). Importantly, reference was made to the assumption that wave climate remains constant over this period.

The studies discussed above provide evidence for the evolution of shingle barriers over periods ranging from several thousand years to the sub-decadal scale. Whilst the evolutionary processes have been discussed, by inference, no near-instantaneous measurements have been made of the response of shingle barriers to extreme storm-events with simultaneous measurement of wave, water level and geometric conditions.

These short-term processes may be particularly important where the barriers provide a coastal defence function. Barriers which provide flood defence have been discussed for a number of locations, including Chesil Beach (Babtie, 1997; Bray, 1997) and Hurst Spit (Nicholls, 1985; Bradbury and Powell, 1992; and Bradbury and Kidd, 1998). The balance between barrier crest build up due to overtopping and crest breakdown by overwashing dictates the rate of barrier migration (Orford et al, 1991). Differential response of the barrier crest and back barrier limit provides an indication of the evolutionary phase (Figure 2.13). If the seaward shoreline retreats faster than the back barrier the crest must be building. The opposite response suggests a falling crest elevation. These inferences suppose that net sediment transport is in balance and the CSA of the barrier is maintained.

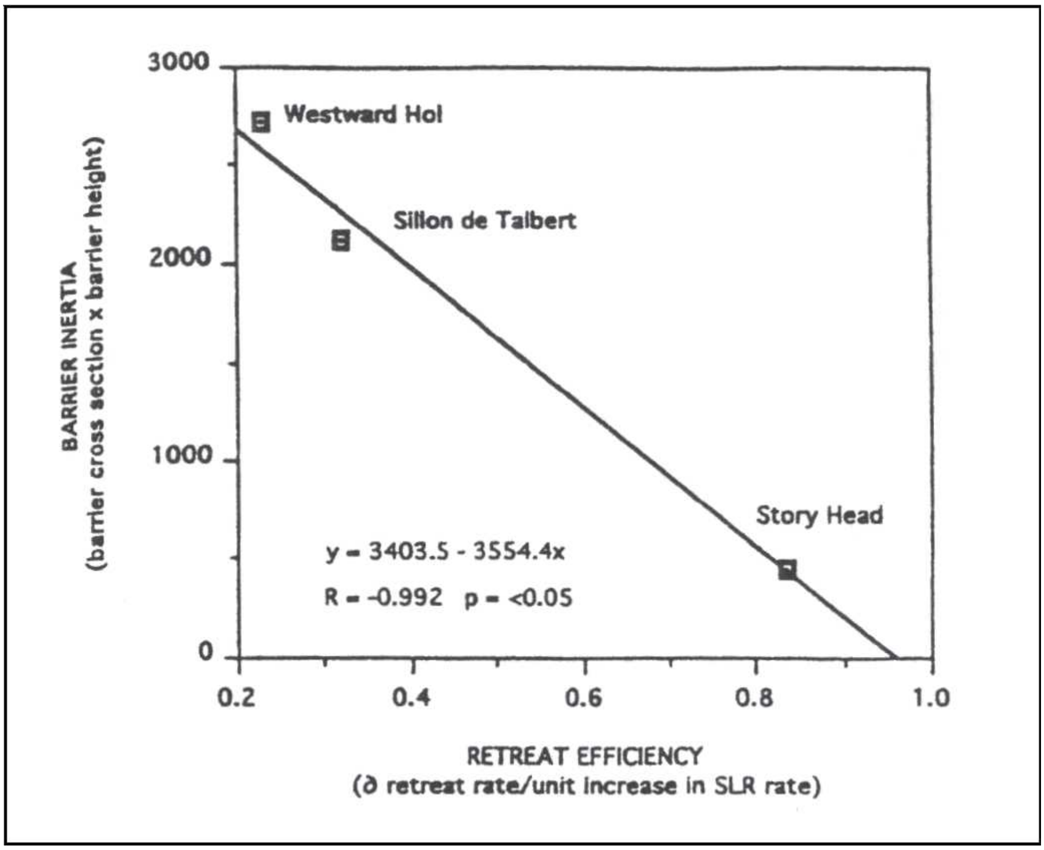


Figure 4.1 Barrier inertia against the efficiency of barrier retreat for Story Head, Sillon de Talbert and Westward Ho! barriers (Orford *et al*., 1995)

4.2 Application of the Shingle model to barrier beaches

The Shingle model (Powell, 1990) was developed with the intention of providing a predictive tool for the short-term profile response of shingle beaches, where an unrestricted volume of beach is available to landwards of the water level and beach intersection (i.e. not barrier beaches). The model operates on the basis of development of a dynamic equilibrium profile, for any given combination of wave and water level conditions, and for a defined grading of beach material; it assumes a mass balance across the profile and no longshore losses of material.

In the case of application to a barrier beach, the finite volume of available beach material may not be insufficiently large to enable the dynamic equilibrium profile to develop prior to initiation of overwashing. In the event that there is insufficient material within the beach cross section to develop a dynamic equilibrium profile for the stated hydrodynamic conditions, the SHINGLE model will fail to run, as a cross shore a mass balance cannot be achieved. In this case the formation of an overwashing event is implicit; this failure of the model actually provides a true representation of an overwash event and an assumption that overwashing will occur under these conditions is valid. Caution should be exercised however to ensure that the beach cross section is the reason for model failure!

In many instances where conditions are close to the overwashing threshold however, the model will provide a profile prediction which is not representative, in real terms, of the actual beach response. This is demonstrated in field and physical model tests by Bradbury (1998), based upon the geometry of Hurst Spit (Figure 4.2). The SHINGLE model suggests a dynamic equilibrium profile will form with an increased crest elevation arising from wave run-up, yet the actual response measured in the field demonstrates that the crest will be lowered. Bradbury (1998) suggests that overwashing and subsequent crest elevation reduction is likely to occur on beach cross sections with a much smaller cross section area than those that are likely to result in failure of the SHINGLE model.

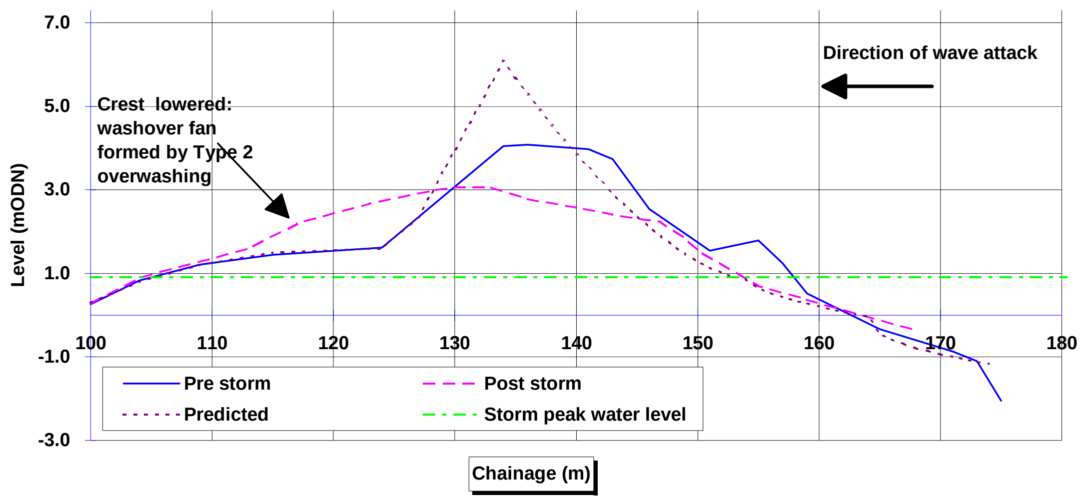


Figure 4.2 Measured pre- and post-storm profiles and predicted profile response for the storm of 29/10/89 for storm conditions: Hs =3.8m; Tm=10.1s; SWL=0.87mOD

Further analysis suggests that the SHINGLE model is not appropriate when analysing conditions that approach the overwashing threshold i.e. those conditions where an accurate representation of the response is most desired. The main reason for this appears to reflect the variability of individual waves within random wave trains which may contain isolated large and particularly long period waves that may result in destruction of the beach crest. There is a strong dependence upon the original geometry of the barrier cross section to the form of the post storm cross section. No further guidelines have been produced to assess the limits of applicability.

Although Powell's (1990) model of shingle profile response was never developed with the intention of application to barrier beaches, numerous applications of the SHINGLE model have subsequently been made to shingle barrier sites, often in an attempt to predict overtopping or overwashing. Regrettably, these inappropriate applications of the model are likely to present misleading results (generally under-predicting the onset of overwashing) which should be viewed with some caution.

The model is shown to work reasonably well at sites where the barrier is wide and high relative to the wave and water level conditions and where overtopping of the crest does not occur. The SHINGLE model can be used to provide a useful a guideline to determine the maximum crest elevation and active beach width that might form in the event of defined conditions on a wide beach. It is suggested that the maximum theoretical run-up elevation can be determined by applying the model to a cross section of enlarged width; this provides a useful guide for the determination of beach recharge design if overtopping is to be limited.

A further limitation arises with regards to applicability of the model to sites which are characterised by widely graded beach material, or where there is a significant sand content within the beach. As the model takes no account of longshore transport, this too has a significant influence on the final profile.

4.2.1 Determining Standard of Protection with SHINGLE (Powell, 1990)

Although the SHINGLE model of Powell (1990) was developed as a tool to predict the evolution of a shingle beach with sufficient volume for the profile response to be realised, the failure of the modelled beach to adjust to incident storm conditions on the grounds that insufficient beach material exists has been used as an indicator that overwashing is likely to occur.

In 2005/6 West Dorset District Council commissioned a study to address the standard of protection (SoP) provided by East Beach. In this case the SoP was defined as the return period at which, on average, flooding of any depth might occur in the hinterland as a result of excessive overtopping, or due to a breach of East Beach.

The method applied required a filtering process to remove a large proportion of non-critical cases (i.e. wave-tide combinations) such that the modelling procedure became manageable. The risk of breach was then defined by deriving a fragility curve. Fragility is the name given to the relationship between the probability of failure (i.e. a breach causing flooding) and the position of maximum erosion or incursion of the beach due to wave action. The probability of a breach or significant overtopping is effectively nil until wave incursion penetrates the shingle barrier. Figure 4.3 depicts two distinct fragility curves used for East Beach.

In Curve 1 (Figure 4.3), the risk of a breach was taken as zero up to and including zones a, b and c. In zone d, the risk increases to a probability of 0.3. In zone e the risk continued to increase to certain failure (probability of 1.0) within about 3m of incursion.

Curve 2 differed in respect of zone e by supposing that the risk of flooding does not increase with increasing penetration of the shingle ridge. This may be reasoned from the work of Bradbury and Powell (1992) which considered that the beach may respond by crest lowering or crest accretion; i.e. failure is still uncertain (probability of failure < 1).

Different probabilities of occurrence cannot be ascribed to the two fragility curves as they represent alternative views of the same probabilistic event. It was considered that Curve 1 represented a more robust alternative. Any landward regression of the beach would be accompanied by a lowering of the beach (resulting from the topography of the back slope) and, therefore, an increasing probability of failure. In view of this, and other unknowns, Curve 2 was considered optimistic. However, it was included to provide an upper bound solution.

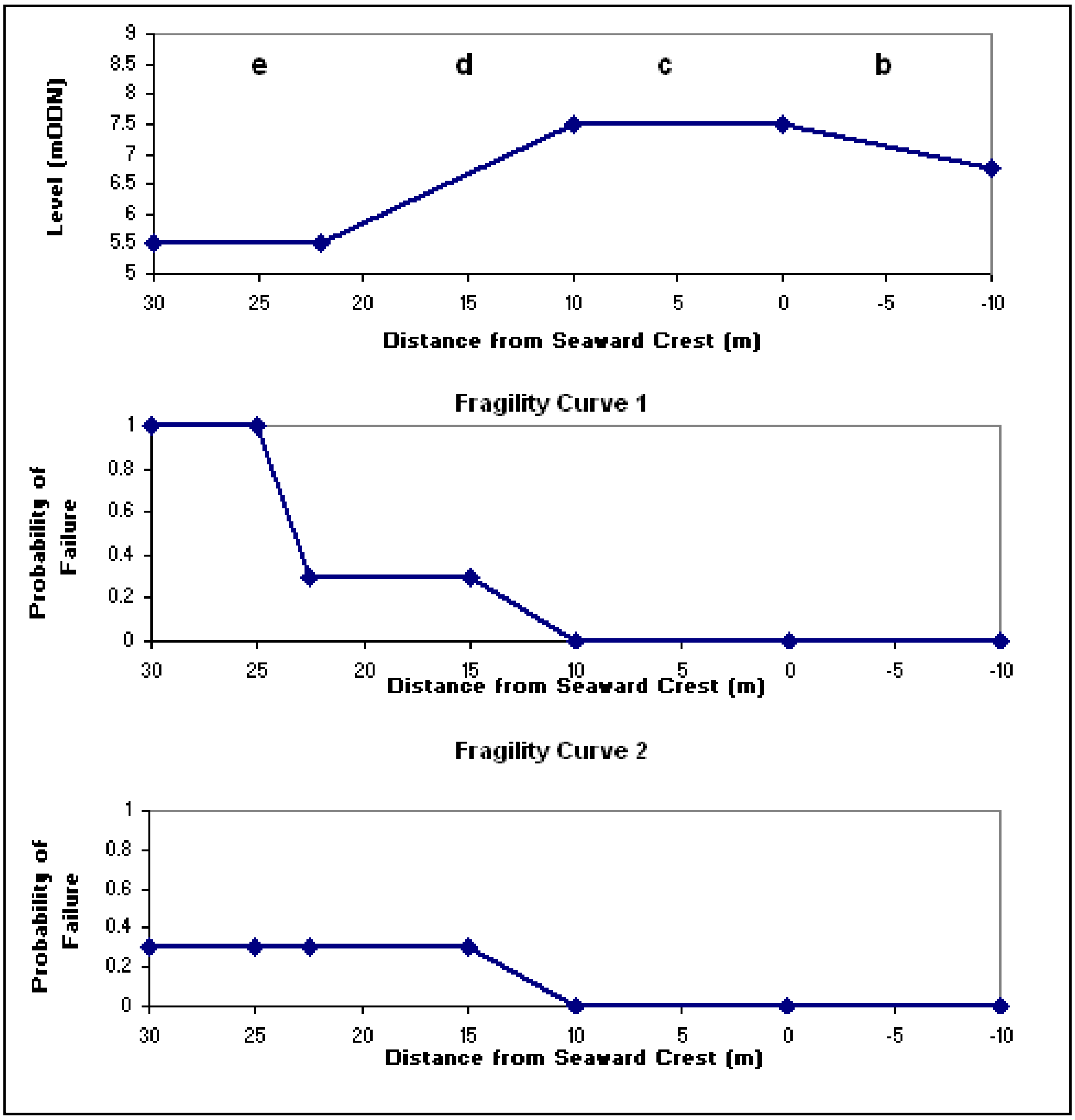


Figure 4.3 Fragility curves for East Beach

The SoP is expressed as a return period, measured in years. The relationship between SoP and the probability of failure is:

$$SoP = 1/(1 - (1-p)^n) \text{ years}$$

where n is the number of high tides in a year (=707) and p is the probability of failure, related to high tide events.

In order to arrive at the probability p, the following fundamental steps were carried out:

(1) The generation of a large database of high water/wave events based on the source data.

This was achieved using Monte Carlo simulations to synthesize event data corresponding to an equivalent of 10,000 (years) times 707 (tides). The process essentially involves sampling from each of the two datasets, one for waves and the other for sea level.

(2) For each of the event data created in step (1), incursion into the shingle ridge by the erosive combination of tide height and wave attack was calculated.

Firstly, all those events which, at the outset, would not create a threat (i.e. all those that would result in incursions in or seaward of beach zones c were removed from the data set; this accounted for about 99.9% of the data set. This is explained in more detail below:

The study process entailed the computation of the behaviour of the shingle beach to wave attack. This was evaluated using the SHINGLE (Powell, 1990) model. The analysis involved a vast number of wave height/water level combinations (over 7 million). The modelling of every one of these input cases was considered to be an impractical computational task if some simplifying measures were not taken.

The method acknowledged that the input data base contained a large number of cases that would not yield threatening conditions. These are filtered out from the data set using the critical freeboard parameter factor (C_f) expressed by Bradbury and Powell (1992) as:

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

where:

- C_H : The freeboard from static water level to the beach crest
- H_{sb} : The shallow water breaking wave height
- L_{ms} : The shallow water wave length

Bradbury and Powell (1992) stated that if C_f is more than 0.7, no crest lowering will occur. Having applied this criterion, less than 0.1% of the original data remained. As a result, it becomes practical to run the SHINGLE model on each of the remaining cases.

Having shortlisted the conditions under which significant incursion of the shingle ridge could occur, the depth of ridge erosion, i.e. the incursion, was computed using SHINGLE (Powell, 1990). The end product of this step (2) was a statistical distribution of the numbers of events that would cause incursion into the various beach zones a to e (including those that would be of no consequence to a breach scenario).

Not mentioned in the above methodology (for the sake of clarity) is the fact that the total beach width w varies with time, although it should remain between 120m and 160m for compliance with the Beach Management Plan for East Head. This was taken into account in Step (2) by considering the likelihood of the beach being at a given width between the two extremes, and allowing for this in the process of computing shingle erosion (SHINGLE). A wider beach (i.e. wider "zone a" and hence width w) resulted in less erosion of the ridge, as defined by zones b to e.

(3) The annual probability of inundation given the statistical distribution of the incursions from Step (2) was calculated.

In principal, this amounted to factoring the distribution (effectively the number of "hits") according to the failure probability given by each of the fragility curves (Figure 4.3). The result, which corresponds to 10,000 years' of high tides, was normalised to yield the equivalent 1 year probability.

Two sets of results were arrived at, corresponding to the present day and to the predicted situation in 50 years time (accounting for anticipated sea level rise). For the latter case it was supposed that the beach management practice continues as at present into the future.

The results are also differentiated according to the fragility curve used in the analysis. The case relating to Curve1 represents a conservative approach to the risk of a breach; the case relating to Curve 2 supposes that the risk of a breach does not increase beyond a certain depth of beach scour incursion. It was suggested that the latter result should be regarded as a "no better than" result, whilst the case associated with Curve 1 should be regarded as conservative but realistic. In this manner, a range of standards of protection (conservative through to optimistic) for the present day and 50 years in the future were derived.

Whilst the methodology applied here may be state-of-the-art, and beneficial in terms of supporting the management of East Beach, there still remains the uncertainty in how appropriate the SHINGLE (Powell, 1990) model is for defining overwash events for barrier beaches.

4.3 Application of Dimensionless Barrier Inertia model

Bradbury (1998, 2000) formulated the dimensionless barrier inertia parameter to identify threshold conditions for shingle barrier overwashing, based on physical model tests of a wide range of barrier cross section geometry. The model builds upon the empirical framework developed by Powell (1990). The empirical framework was developed following a site-specific study into crest evolution of barrier beach profiles and the conditions that promote overwashing, overtopping and crest cut-back; this investigation was subsequently expanded to examine a broad range of beach geometry and hydrodynamic conditions, but was confined to examination of a single size grading of beach and approach bathymetry. Investigations included both laboratory experiments, using a 3-

dimensional mobile bed random wave physical model (Bradbury, 1998), and field verification at Hurst Spit (Bradbury et al 2005).

Application of the parametric framework requires simple substitution of geometric and forcing variables for a given storm event into a formula. The dimensionless barrier inertia values (R_cB_a/H_s^3) for each profile are plotted against the wave steepness (H_s/L_m) parameter. Overwashing is predicted when the critical barrier inertia threshold is exceeded (Bradbury, 1998):

$$\frac{R_c B_a}{H_s^{\ 3}} < 0.0005\left(\frac{H_s}{L_m}\right)^{-2.5457}$$

4.3.1 Reported limitations

A series of observations regarding the applicability and limitations have previously been made (Bradbury, 2000). These are highlighted below.

The location of the wave measurement point (selected for subsequent use in the parametric analyses of beach profile response) is of importance. Conditions measured in deep water are unsuitable for use in the analyses, since potentially large changes can result from shallow water wave transformation processes. Waves measured above the active section of the beach are unsuitable for use, since they can be modified subsequently by the beach evolution process. The most suitable measurement point for waves is at the toe of the mobile beach, close to the point of beach profile closure. The wave conditions used in formulation were measured in 7-9m of water, and with a fixed approach bathymetry. The influence of the foreshore slope angle, between the wave measurement point and the barrier, has not been examined; however, this may have a significant effect on the applicability of the formulae to some sites.

Tidal height measurements should be undertaken at, or very close to, the site: the effects of local wave set-up, or surges, can be missed when the tide gauge is located remotely from the site. Small errors in water level measurement can have significant effects on the outcome of the results, in terms of: freeboard; surface emergent CSA; point of (wave) attack on the beach profile; and, finally, the breaking wave height.

The pre-storm foreshore geometry may affect the rate of profile evolution. If the initial beach profile is similar to the dynamic equilibrium profile (for a particular storm), less profile evolution will be required to achieve the dynamic equilibrium profile. Most of the model tests were run on the ‘as surveyed’ profiles, or were stepped through a sequence (from low-to-high) water levels; in this respect, the profiles were not dissimilar to the natural profiles. An artificially-graded beach, with a plain slope, may respond initially quite differently to a natural system. In such circumstances, the empirical framework proposed for the present investigation may not predict correctly, the barrier crest evolution.

The model results, which identify overwashing or roll-back, do not differentiate between initial overwashing resulting either from profile widening, or from run-up exceeding the crest. Initial overwashing of the barrier results in many instances from foreshore widening and cut-back of the supra-tidal barrier, as opposed to run-up. It is not reasonable to assume that overwashing cannot occur, because the predicted run-up limit is below the barrier crest. In this case, the pre-storm profile and CSA is important. A large number of profiles have been measured where there has been crest reduction due to widening of the profile - but no overtopping. This particular process does not seem to be recognised elsewhere in the established literature; however, it is considered to be one of the most important processes leading to overwashing, especially on managed sites where the crest is artificially high.

Beach profile measurements should identify all slope break points on the profile; they should include typical profiles and topographic lows, in order to examine the possible influence of spatial variability in abrupt changes in crest elevation.

Variability in the response function is dependant upon spatial variations in the pre-storm barrier geometry; the barrier geometry adjacent to the measured profile is also important in this respect. Analysis based upon profiles which lie within a zone of either irregular cross section or freeboard, may not represent the typical barrier response accurately. Instead, the processes are more likely to reflect the geometry of topographic lows, which may be subject to outflanking when overwashing occurs. If the barrier geometry is consistent spatially, with longshore transport in equilibrium along the whole of the barrier, spatial variability of the barrier crest response is less likely to occur. Further research could examine, usefully, strictly controlled 2-dimensional barrier response; this would refine the empirical framework.

Scatter is more evident over the lower range of the wave steepness parameter (<0.018); this may be indicative of the effects of wave grouping, or a series of relatively long-period waves. Further data collection is necessary within this range, to provide more confidence in the response of barrier beaches to swell conditions. Such conditions have been suggested to be the cause of overwashing on Chesil Beach (Babtie, 1997).

The wave energy spectrum can be described by reference to characteristic periods defined by the peak period of the spectrum (T_p) and the average period of the zero crossings (T_m). The relationship is constant for a defined spectral shape, varying only with spectral type (Goda, 1976). The development programme used waves with a constant spectral shape (JONSWAP). Provided that a consistent spectral shape is used, it is not necessary to use each of the values of T_p, T_m in any analytical work; the relationships are linear and can be determined from a single value of wave period. The average zero up-crossing wave period (T_m) used is consistent with that applied in other related research (van der Meer, 1988; and Powell, 1990).

Previous studies undertaken on shingle beaches have suggested that the influence of spectral shape of the incoming waves is minimal (van der Meer, 1988). However, the results obtained from Bradbury’s (1998) investigation have been confined to an examination of waves described by a JONSWAP spectrum. Field data has identified the occurrence of bi-model spectra, with a secondary spectral peak resulting from a swell wave component. Such spectral variations have not been considered within previous investigations in the context of shingle beach profile response. The influence of two clearly defined spectral peaks results in waves of two distinct period groupings, which are unlikely to be modelled well by the empirical relationships discussed. Insufficient data is currently available to examine the effect of such spectral variability. This particular variability of spectral shape is worthy of further investigation, and is considered in more detail later. A long-period wave component, in a bi-modal sea, may have a significant influence on the profile response. Analysis of wave data collected in the field, which is often confined to the determination of H_s and T_m, may be insufficient to describe the profile response of shingle barriers under such conditions. The frequency and magnitude of long-period waves within a storm is, perhaps, the most significant variable: observations obtained from model tests have demonstrated that individual long-period waves are able to modify the barrier crest.

4.3.2 Application

Attempts to apply and validate the model have met with mixed success. Where measured site conditions have been between the valid limits of the original model development the model has performed well. Researchers have also applied the model beyond the valid limits and have reported some success (Chadwick *et al*., 2005). Other investigators (Box, 2005; Cope, 2005; Bradbury *et al*., 2005) have had more mixed success; the underlying theme is that the model has not been developed or validated for a sufficiently large range of conditions to allow generic applicability.

The first attempted application of the empirical framework was at Hurst Spit (Bradbury, 2000). As the bed geometry and sediment grading used for establishment of the model was based on this site there should be a reasonable expectation of a good representation by the model.

Field data for pre-recharge overwashing events at Hurst Spit are scattered largely about the predicted overwashing threshold regression curve (within the predicted confidence limits). Regrettably, the valid limits of the model are limited to within the range $H_s/L_m<0.032$; the regression curve has been extrapolated (to 0.055), to cover the full range of measured field data. The extrapolated curve may be defined inadequately over this range: predictions may be in error due to the lack of data. Notwithstanding this potential problem, the field data are totally consistent with the predicted response over the range of the extrapolated curve. Additional data are required to improve the validity of the framework; likewise, to define the overwashing threshold more precisely for $H_s/L_m>0.032$. Field data (for a steepness of 0.031) lie outside the confidence limits of the suggested overwashing threshold conditions (but are reasonably close to the threshold).

Events, which were analysed upon completion of the (1996) Hurst Spit recharge scheme, have shown no crest evolution (Figure 4.4). All the data lay comfortably within the overwashing threshold limits (due to the relatively large recharge volume). The data presented are confined to barrier inertia parameter values of less than 50. Additional data were also recorded (to values as high as approx. 140); none of these exceeded the threshold conditions.

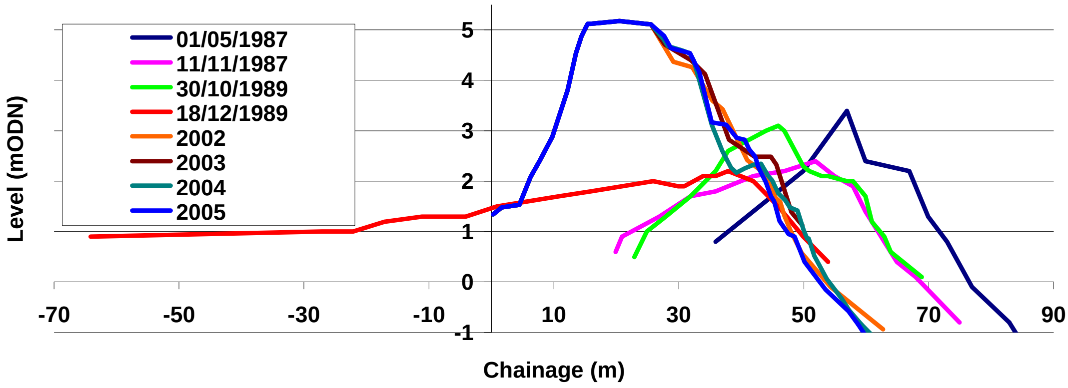


Figure 4.4 Profile evolution prior to (1987-1989) and following beach recharge (2002-2005) at Hurst Spit (Bradbury *et al*., 2005)

Applications of the model have been made to provide prediction of the evolution of the barrier at Gull Island within the western Solent (Bradbury *et al*., 2005). This has proven to be largely successful, although it should be noted that this site lies within a low energy environment, where water level is the predominant driving mechanism. This site is reasonably simple with fetch limited wave conditions and no prospect of swell waves occurring. Predictions of the overwashing threshold have been made for a range of extreme wave and water level combinations (Figure 4.5). These have been validated against measured profile response.

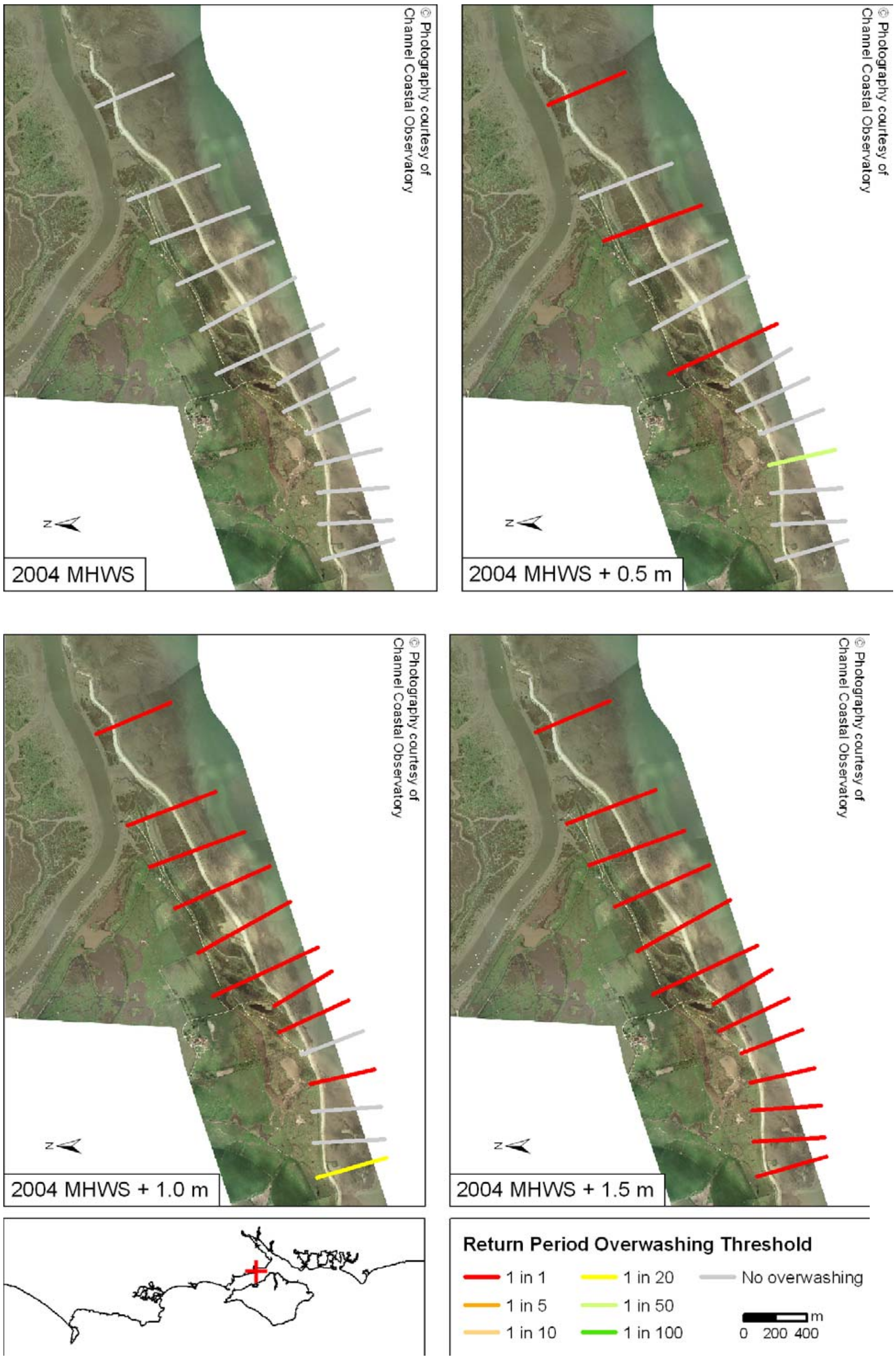


Figure 4.5 Overwashing return period thresholds for Gull Island based on 2005 survey, for tidal elevations in 0.5m steps (Bradbury *et al*., 2005)

A typical application of the predictive framework is illustrated in Figure 4.6 which shows predictions for the shingle barrier at Reculver (Kent). The data presented are based upon measured profiles and analysis of extremes for waves and water levels provided by Canterbury City Council (Table 4.1). Two barrier beach profiles (with variable CSA) are examined in the analysis. The profile and hydrodynamic data show an increasing likelihood of overwashing, with increasing severity of the prevailing storm conditions. The framework of the hydrodynamic conditions examined indicates that both the barrier profiles may be vulnerable to overwashing within the context of 1:100* year design storm. The profile with smaller cross section (BLS59) might be expected to undergo overwashing during an event with a return period of 1 in 50 years. Qualitative assessment of the predictions, based upon the analysis of beach profile records (McFarland, *pers com*), suggest that the results are representative.

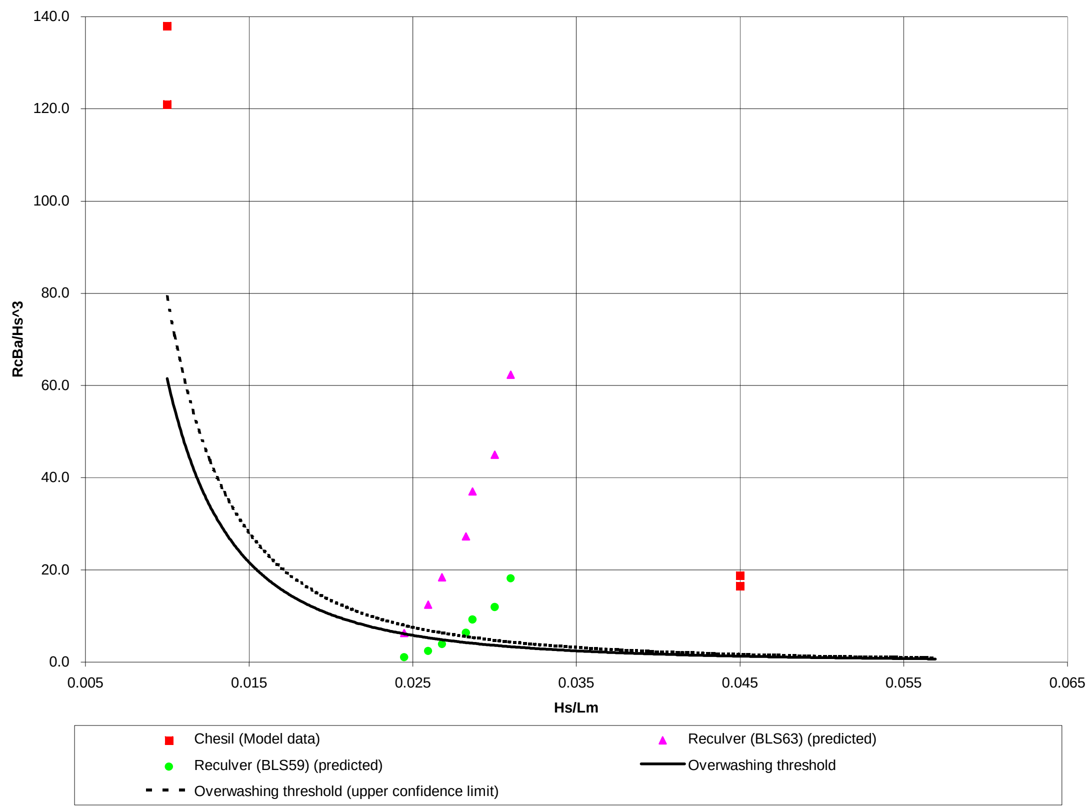


Figure 4.6 Application of threshold prediction, to Chesil Beach and Reculver (Northern Sea Wall) Shingle Bank

Table 4.1 Design storm data for Reculver (data provided by Canterbury City Council)

Return period (Yr)	SWL (mODN)	H_s (m)	T_m (s)
1	3.20	1.80	6.1
5	3.47	1.86	6.3
10	3.63	1.89	6.5
20	3.80	1.98	6.7
50	4.07	2.05	7.0
100	4.33	2.10	7.2
100*	4.74	2.15	7.5

Note: 100* refers to design conditions allowing for a factor of safety above the 100 yr return period event

A further application of the parametric framework, based upon data relating to Chesil Beach (Dorset) is also shown in Figure 4.6. The data presented are derived from 2-dimensional physical model studies, designed to model water percolation through Chesil Beach (Hydraulics Research, 1984). Waves were measured in a water depth of approximately 14m. The beach profiles were based upon surveys of Chesil Beach.

The conditions, for a wave steepness of 0.01 (H_s =3.6m; T_m=15.5s), resulted in occasional waves reaching the crest of the barrier (at a level of 14.7mOD), but no overwashing. This suggests that conditions were close to the overwashing threshold, but did not exceed it. Extrapolation of the predictive curve to a steepness of 0.01 suggests that the proposed relationship is also reasonable for the above conditions.

The curve becomes very steep over the lower range of wave steepness (<0.015). Any inherent measurement errors may be accentuated by this trend. No overwashing occurred for the other data set, shown for a wave steepness of 0.045 (H_s=7.0m; T_m=10.0s); this is consistent with the predicted response.

A hindcast application of the empirical framework has been applied by Cope (2005) to assess its validity at Medmerry. Synthetic wave data modelled to 5mCD depth and tidal elevations measured at Portsmouth have provided the basis of hydrodynamic inputs. Profiles were typically measured in summer, not immediately prior to storm events. Management records have formed the basis of response evaluation.

The majority of wave steepness events determined for Medmerry are defined by conditions where H_s/L_m>0.033, which is beyond the valid range of the empirical framework. The wave steepness range provided by the model was found to be too narrow for commonly occurring storm conditions at the site, particularly steep wave conditions. Predictive curves were extrapolated outside of the original range.

The original framework was based upon measurements determined using T_m to define wave period and with a constant spectral shape (T_m=0.78T_p). Observations have often been made in circumstances where T_m is significantly less than 0.78T_p, suggesting that the empirical framework would under-predict the likelihood of overwashing under these circumstances.

Eight events were analysed for the period between 1993 and 2002. All of these were classified as crest cut-back, overwashing or breaching. Results were mixed, with the model successfully predicting overwashing on some occasions, but failing to do so on others. Those events which the model failed to predict were characterised generally by wave conditions where T_p>1.3T_m.

Results of the associated field tests highlight a combination of accurate prediction and under-prediction of barrier response. Suggestions for conditions where the model failed to predict correctly are outlined (Cope, 2005).

Summer profiles may not be representative of pre-storm profiles, which are characterised by lower cross section areas.
The majority of predictions are outside of the valid wave steepness range.
The model was defined using T_z conditions using the JONSWAP spectra.
Medmerry sometimes has a sea state characterised by a bi-modal spectrum when swell waves are present (Figure 4.7). This is not considered by the model.
Subjective terminology has been used to define documented events.
Portsmouth was the nearest real-time recording station.

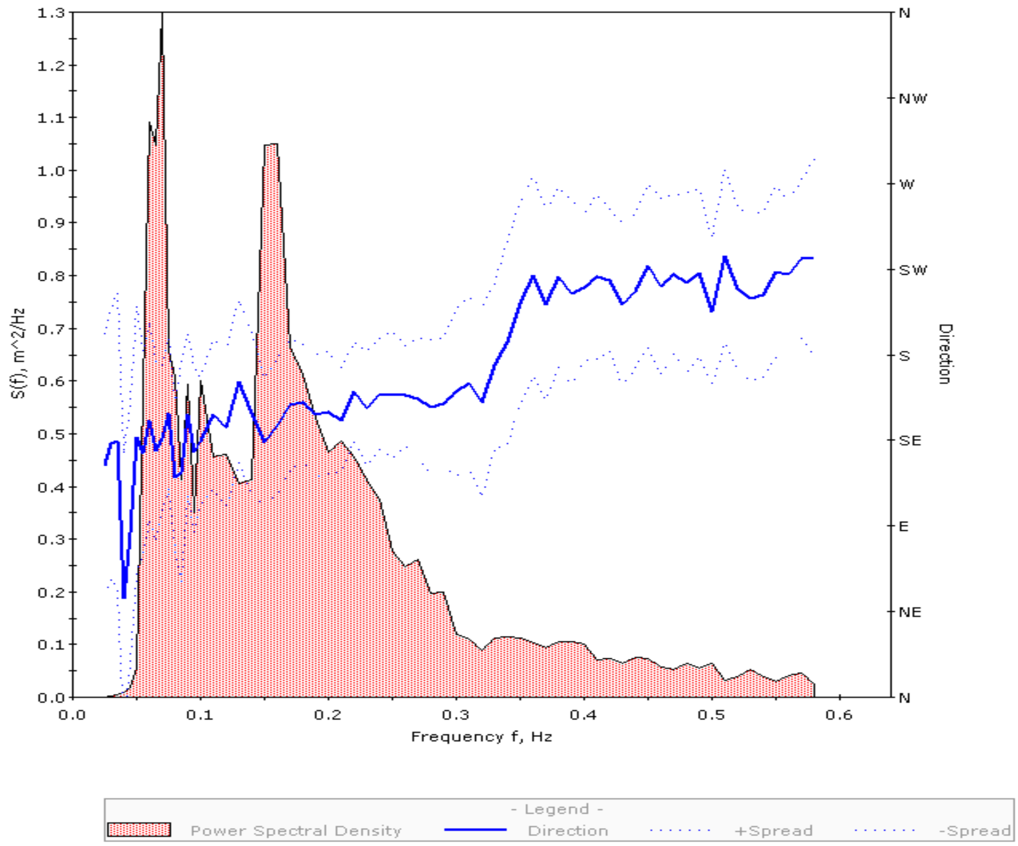


Figure 4.7 Bi-modal wave spectrum at Hayling Island

Additional storm events would be required to further validate the model. Data accuracy and availability is being improved through the South-east Strategic Regional Monitoring programme. There is also a need for documentation of barrier and spit response to storm attack and emergency management works in order to further validate the model and better understand these systems for future coastal management.

Chadwick et at. (2005) report a successful application of the technique at Slapton. The model application presents analysis of a 4-year time series (1999-2002) of many conditions, which confirm that breaching did not take place during this period, in accordance with predictions. A single event was identified very close to the specified threshold that caused overwashing and this was predicted by the model. It should be noted that many conditions tested were outside of the valid range of the predictive curves, however, with H_s/L_m=0.04-0.05 typically.

Application of the model at Cley, Norfolk (Box, 2005) has identified similar problems. The model failed to predict overwashing at any location, although a number of overwashing events were known to have occurred. Wave conditions used were in approximately 5m water and could be expected, therefore, to be somewhat lower than for the original model calibration. Sensitivity tests were conducted to assess the use of T_p as opposed to T_z, which gave generally better results. It should be noted that the model was calibrated using T_m, with a constant relationship T_m=0.78T_p. The problem at this site is compounded by the fact that engineering works are conducted regularly and these management activities have not been recorded in conjunction with the beach profiles.

4.3.3 Summary

Barrier evolution processes have been observed and described in physical model studies (Bradbury, 1998); such evolution has been examined previously (Orford *et al*, 1991a), primarily by inference of the processes (on the basis of examination of the change in sedimentary structure). The implications of overwashing, resulting from foreshore widening, are identified as an important process in barrier crest development; this is in addition to the run-up exceeding the barrier crest.

An empirical framework has identified threshold conditions for overtopping and overwashing of barrier beaches under extreme conditions. A barrier crest evolution categorisation framework has been defined, and a framework of governing variables has been examined, for shingle barrier crest evolution: overwashing of a shingle barrier is controlled primarily by wave height, wave steepness, freeboard and barrier CSA. A dimensionless barrier inertia parameter has been defined, which identifies the threshold conditions for overwashing of a shingle barrier (Bradbury, 1998). A predictive equation has been presented, together with confidence limits and defined range of validity. Overwashing will occur if the critical barrier inertia threshold is exceeded. Over-topping, or containment of the barrier crest, will occur if the critical barrier threshold is not exceeded. Empirical calibration of the physical modelling methodology has demonstrated that such techniques are able to reproduce the response of a shingle barrier to extreme conditions, at an appropriate level, by direct comparison with measurements of full-scale storm events, for some conditions. Limitations to the modelling methodology have been identified, and recommendations for improvements to both the modelling methods and field validation techniques have been made.

A number of common themes appear within the validation observations.

The range of commonly occurring wave steepness is often outside of the valid range of the parametric framework, particularly steep wave conditions. There is a requirement for the valid range to be extended to cover the range H_s/L_m to 0.03-0.055

Extrapolation of the formulation may result in misleading results.

The original model was based upon measurements determined using T_m to define wave period and with a constant spectral shape (T_m=0.78T_p). Observations have often been made in circumstances where T_m is significantly less than 0.78T_p. Observations suggest that the empirical framework under-predicts the likelihood of overwashing under these circumstances.

The relationship between T_m and T_p is seen to vary widely at many sites. Situations where T_p>1.5T_m are common (Bradbury and Mason, 2006)

Applications of the model framework have been conducted in widely ranging, and often undefined, water depth.

The spatial variability of the barrier cross section may vary widely. Outflanking of otherwise sound profiles may occur at locations where the CSA is small.

4.4 Recommendations for further work

As this Chapter highlights, there is a clear need for predictive tools to be developed such that practitioners can apply reliable, appropriate methods with confidence. The previous Chapter allowed recommendations for the development of existing methods to be made, and these recommendations should be driven by the need to apply such models as a routine element of barrier beach management practice.

- In the first instance, it would seem appropriate that the range of applicability of existing parametric tools be extended with genericity as an aim. Recent experimental results (Obhrai and Powell, pers. comm.) have been used to extend the range of conditions over which Bradbury's dimensionless barrier inertia model can be applied, and research such as this should be consolidated and pursued further.

- It would also be worthwhile investigating the application of the concept of fragility as reported in this Chapter with a view to establish robust guidelines for methodology. The effects of assignation of fragility are not fully understood when used to derive standards of protection, and experimental research could be carried out, for example, to further this understanding.

- The lack of guidelines for managers often renders learning from experience difficult. There is a need for documentation of barrier and spit response to storm attack and emergency management works in order to further validate models and better understand these systems for future coastal management. Difficulties are compounded by the fact that engineering works are sometimes carried out but not recorded in a way conducive to furthering research. It is recommended that a procedural document be produced which allows managers a reference point to observe best practice in monitoring.

- One aspect of model application that is absent from the discussion presented in this Chapter is the issue of connectivity of longshore processes, and how these interact with cross-shore processes. For example, where is a breach likely to occur, and how resilient (i.e. able to recover) is the barrier? The ability to perform this type of broader-scale modelling assessment is relatively remote from the current state-of-the-art. Nevertheless, it may be possible to establish some of the required outputs through methods currently being developed and adopted as part other research programmes investigating broad-scale modelling procedures (e.g. FLOODsite, FRMRC work packages, PAMS).

Coastal managers responsible for barrier beaches are justified to ask, for example, if the barrier breaches, where will that breach occur, and when is it likely to happen? What effect will my management practice have on that outcome? How will the barrier behave following a breach? What risk is involved? It is apparent that, to date, the answers to such questions are not easily forthcoming given the range of tools available for analyses and application. Nevertheless, it is vital that any developments to/ of tools are made in such a way that the ultimate practical application is foremost in the researchers approach.

5. Case histories

5.1 Introduction

Barrier beaches around the UK tend to perform an important role as flood defences. The dynamic nature of these beaches, and the sometimes sudden failure in their role as defences makes management difficult and necessary. Often, the consequences of defence failure are dramatic and shocking imagery is readily exploited by the press. Where assets are at risk, management difficulties are compounded by the uncertainties in determining risk. These uncertainties, with regard barrier beaches, stem from incompleteness in our understanding of the governing processes, and the resultant limitations of our predictive tools.

This situation, coupled with a lack of definitive guidelines, means that managers are often left with inherited practices, or are left to find a way through issues by trial and error. In addition EC directive on coastal and flood defence works and the majority of barrier beaches being assigned statutory protective status (such as SSSI, SPA, SAC etc.) means that environmental impact assessments must be carried out. It sometimes evolves that different stakeholders have different remits or requirements and as such the coastal manager can find that making headway through the management process is a struggle.

As part of this scoping study, a dedicated website www.barrierbeaches.org.uk was established. The website offers a focal point for publicising the research, and amongst other elements, contains an interactive mapping feature that allows visitors to locate and access summary database information relating to barrier beaches around England and Wales. The Links page enables visitors to readily discover detailed information (such as statutory designations) relating to many of our barrier beaches, and is a valuable starting point for many studies.

The website was publicised by mailing information to approximately 25 of the country's leading figures in barrier beach processes and management, including staff at Natural England and the Environment Agency as well as prominent academics and Local Authority coastal engineers. There was no restriction imparted to forward distribution, which indeed was encouraged. The website was also publicised in The Surveyor, and in Defra's own research newsletter. Each individual consultee was granted a password which enabled access to a private area of the website.

The private area of the website contained a proforma for completion. The proforma was a consultation document that asked for information regarding management of barrier beaches, and allowed for any other comments to be made. The option for the proforma to be completed anonymously was available, and completed forms could be accessed by all those who were able to login to the private area (if acceptable to the consultee) thus providing a means through which experience could be shared efficiently and effectively.

In the event, response to the consultation online was poor. However, the well-publicised website and study did invoke direct consultation with several individuals who were keen to discuss their experiences – providing valuable assistance with some of the case histories, and also raising some points which highlight the low level of understanding and development of tools to guide management.

One aspect of study that was to be furthered through the consultation document was the selection of a series of case histories to present in this research report. It was expected that these case histories would be selected partly based on the level of interest and current information made available by the consultees. The general absence of response meant this was not possible. Instead, a series of case histories were selected which were considered to represent a variety of barrier types, management practices and degrees of study or knowledge. Information relating to several sites was augmented by project staff making personal visits to various coastal managers.

The following sections in this Chapter present information about various barrier beach sites around England and Wales. Each barrier is located on a map, and a brief overview of the site starts each section. The barrier beaches are varied in character, experience differing forcing conditions, have formed through different processes and, perhaps most importantly, offer different levels of flood protection – clearly dependent upon anthropogenic use of the environment surrounding the barriers. The information presented for each site ranges from the geomorphological context, through physical processes to conservation, defence role and beach management.

Readers are urged to treat the text as background information which is intended to indicate the range of sites characteristics and depth of knowledge across the sample. The case histories are not intended to be exhaustive, but serve to give a greater insight in to the behaviour and issues surrounding barrier beach processes and management. Whilst every effort was made to ensure the validity of the presentations, it should be borne in mind that there may be inconsistencies in the contemporary nature of the data, and some comments may not be representative of current thinking.

5.2 Porlock

Location	Porlock Bay, Somerset (see Figure 5.1)
Designations	SSSI
Length	5km
Width	Approximately 40m.
Crest	Approximately 7m at the partly sheltered western end of the shingle ridge, increasing in height to 9mOD at the more exposed, eastern part of the ridge (Bray and Duane, 2001).
Sediment	Shingle
Tides	Spring tidal range 9.3 m
Waves	Dominant westerly waves refracted to a north-westerly direction of approach.
Drift	1110 m^3 per annum, longshore drift in a west to east direction on the west spit.
Classification	Swash-aligned barrier spits west of the New Works outlet and drift aligned barrier beach east of this outlet.
Intervention	Timber groynes, re-nourishment, re-profiling

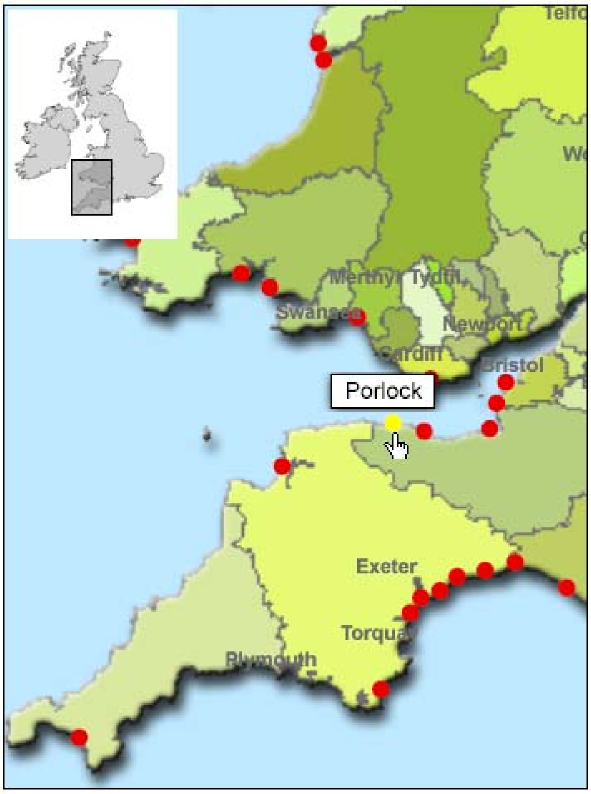


Figure 5.1 Location of Porlock

5.2.1 Introduction

The barrier beach spits at Porlock Bay are 5 km in total length, situated in a bay that is enclosed by headlands, fronting low-lying farmland and Porlock marsh (Plate 5.1). The barrier is dissected at Porlock Weir harbour, where a man made channel is maintained (Plate 5.2).

Plate 5.1 Looking west towards the breach at Porlock, January 2005 (Cope)

Plate 5.2 Porlock Weir harbour (Motyka)

Further east there is further obstruction at the New Works outlet from a land drainage channel. The barrier is also dissected some distance to the west of the New Works outlet, as a result of a breach that occurred in 1996. As a consequence, the backbarrier area adjacent to the breach is flooded with the tide. The breach has shown no signs of self-healing, apart from slow movement of the western spit with longshore drift. Saltmarsh development is now taking place.

5.2.2 Geomorphological context

The current geomorphological situation at Porlock has very much been influenced by the Holocene inter-glacial. The coastal slopes in this area were covered by Devensian sollifluction sediments, which extended onto the foreshore as fans. The finer material from these fans was subsequently reworked during the Holocene transgression. As sea levels rose, so the developing shingle ridge moved landwards over these fans, leaving a “lag” of large boulders to the seaward of the ridge. The lag deposits are important in modifying wave action, which would otherwise have made the ridge more vulnerable to wave damage and higher rates of shoreward retreat than it already is.

Cope (2004) notes that, “despite a large longshore sediment supply, the rapid horizontal shift in sea level position prior to 8,000 years B.P caused Porlock barrier to migrate landward. This allowed continuous breaching and back barrier flooding from marine sources (Orford and Jennings, 1998). However, 6,000-8,500 years B.P. rapid post-glacial sea level rise decreased to between 8.5mmyr^{-1} and 2.0mmyr^{-1}, permitting barrier stability and self-organisation (Jennings *et al.,* 1998). Even though longshore sediment supply was also reduced, it was still sufficient to seal existing barrier breaches and enclose a freshwater lagoon. This environment was similar to the artificial situation at Porlock maintained by groynes and sediment replenishment, prior to the breach of October 1996. Still, the macro-scale process of decelerating sea level rise beginning around 7,500-7000 years B.P caused a reduction in coarse clastic sediment supply (Jennings *et al.,* 1998) which has promoted barrier instability ever since. It is thought that some 4000 years B.P. the barrier started to become more swash-aligned and broke up into major sub-cells due to cannibalisation (Orford and Jennings, 1998).”

Being pinned between Porlock Weir (Plate 5.2) and the New Works outlet, and having little contemporary supply of material, the barrier beach has tended to retreat landwards and become thinner as its length has tended to increase. The barrier has become more arcuate and in doing so has become increasingly prone to breaching. The net volume of sediment supply has also been found to be decreasing with time, despite the input of material due to landslips west of the bay (May and Hansom, 2003).

Cope (2004) notes that, “the barrier has previously been studied by Pethick (1998), reporting to the National Trust and Environment Agency respectively, Bray and Duane (2001), reporting to the Environment Agency, Jennings and

Orford (1996, 1999) Jennings *et al.,* (1998) and Orford and Jennings (1998), who used stratigraphic data to reconstruct Holocene evolution and most recently, Orford *et al.,* (2003) who studied barrier response to storm conditions. In addition to this, the ecology has been studied by Jarmen (1986)."

5.2.3 Sediment size and grading

The barrier spits are composed of large, rounded boulders and cobbles overlying a flat boulder foreshore (see Plates 5.3 and 5.4).

Plate 5.3 Boulders on foreshore (looking east, Jan 2005, Cope)

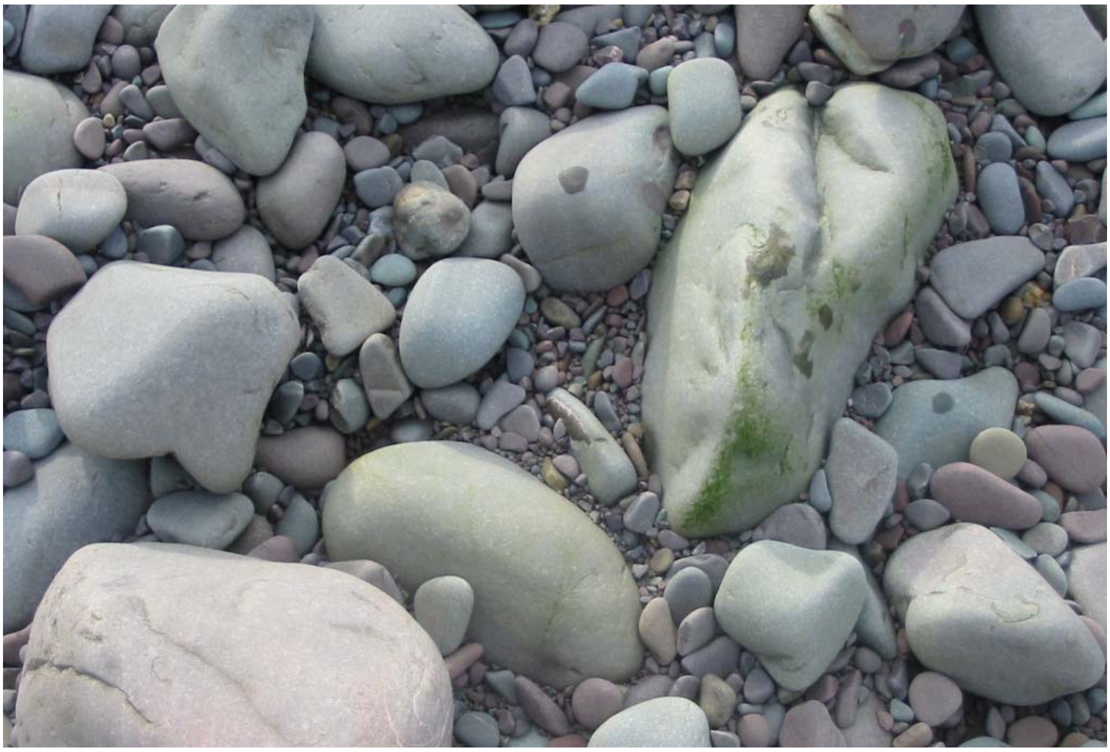


Plate 5.4 Mixed boulders and cobbles (Jan 2005, Cope)

The harbour at Porlock Weir constitutes a major interruption to littoral transport, which is predominantly from west to east on this frontage. The ridge in front of Porlockford Cliffs has therefore been retreating landwards faster than it might otherwise do, as a result of the lack of sediment supply. This retreat helped the arcuate, swash aligned plan-shape of the ridge to develop. There is negligible drift from west to east due to the predominant swash-alignment of the beach and the fact that groynes trap any limited input of new material from Gore Point (Bray and Duane, 2001). Crude estimates indicate that some 40,000 to 50,000m^3 of shingle have been trapped within the harbour area since the eighteenth century, suggesting that the net drift is only about 200 to 250m^3 per year (Carter, personal communication 1990).

Cope (2004) calculated longshore drift for the west barrier spit from topographic profiles collected by Bray and Duane (2001). An estimate was deduced from longshore volume changes on the west spit (see cell 1 – Western Spit on Figure 5.2). In 22 months, there was approximately 2,034m^3 sediment accretion on the spit. When averaged, this equated to 1110m^3 per year. However, in 22 months, there was a decline of 4,385m^3 in sediment volume on the west beach feeding the spit, indicating that 2,034m^3 of this was transported as longshore drift, and the remaining 2,351m^3 was transported offshore. This estimated longshore drift rate of 1110m^3 is within the 250 - 2000m^3 annual range estimated by Pethick (1992).

It has been suggested that shingle could be recycled from east to west along the Porlock Bay frontage. However, it could be argued that the removal of material from one cell could lead to a deficit and hence instability in the other cell.

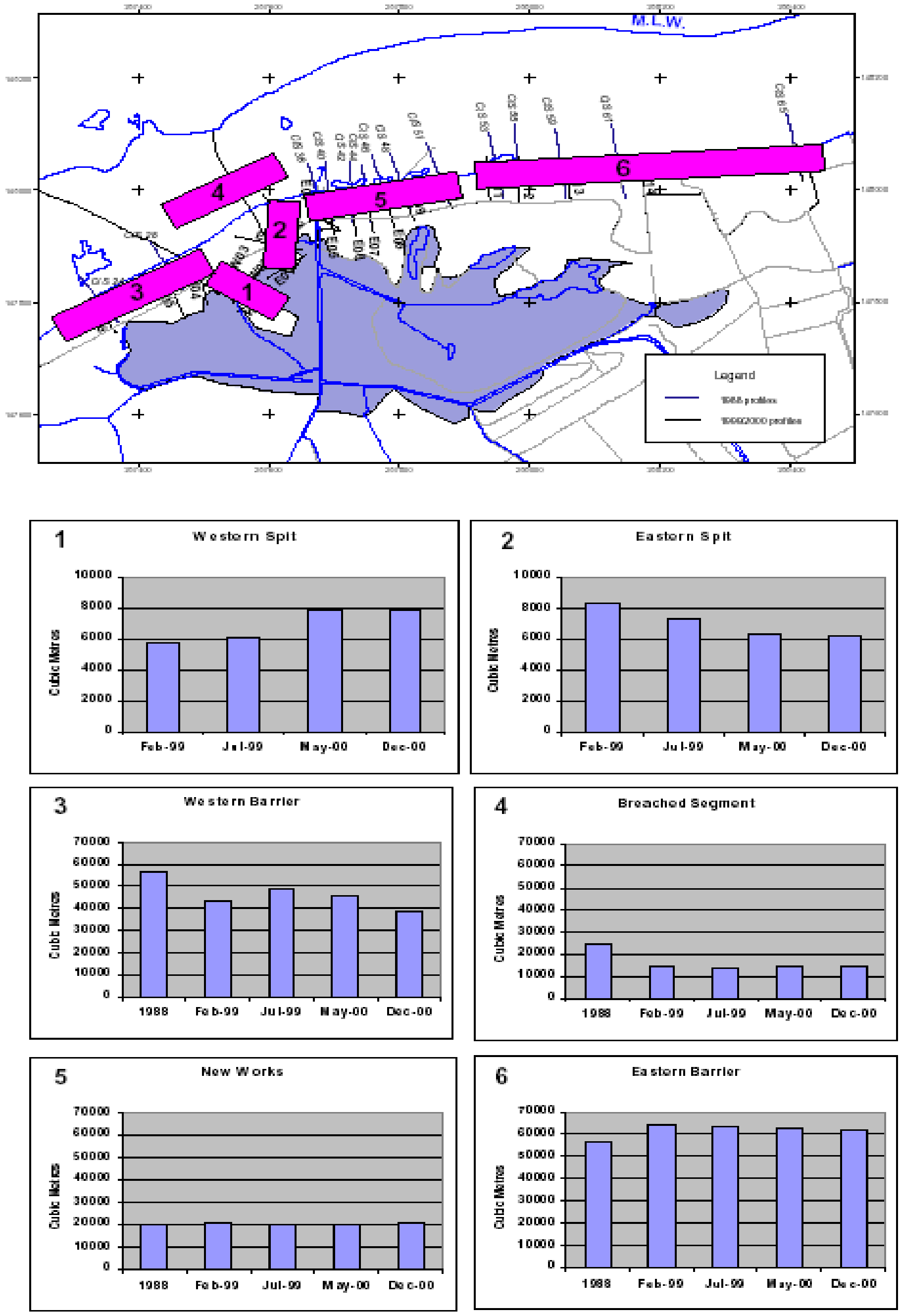


Figure 5.2 Barrier volume changes within defined behavioural cells (Bray and Duane, 2001).

5.2.4 Geometry

The crest of the barrier to the west of the breach is relatively low (6 to 7mOD) because of the effects of overwashing and partial shelter against wave action. Rollover has resulted in emergence of Holocene sedimentary deposits at the beachface, which are reworked by wave action. In addition, the barrier to the east of the breach is more exposed to wave activity and has a slightly higher crest height of 9mOD.

Bray and Duane (2001) note that the barrier has increased its length by 6% since the breach, due to formation of the two spits. This has resulted in a reduced barrier cross-sectional area and increased vulnerability to landward rollover (Cope, 2004).

Figure 5.3 shows the plan shape and a typical cross-section of the barrier and the deposits that are found within the bay (May and Hansom, 2003). Note the distinct change in character/classification of the barrier, east and west of the New Works outfall. Note also the change in cross-section of the ridge (flattening and crest lowering) between 1994 and 1998, i.e. following the 1996 storm.

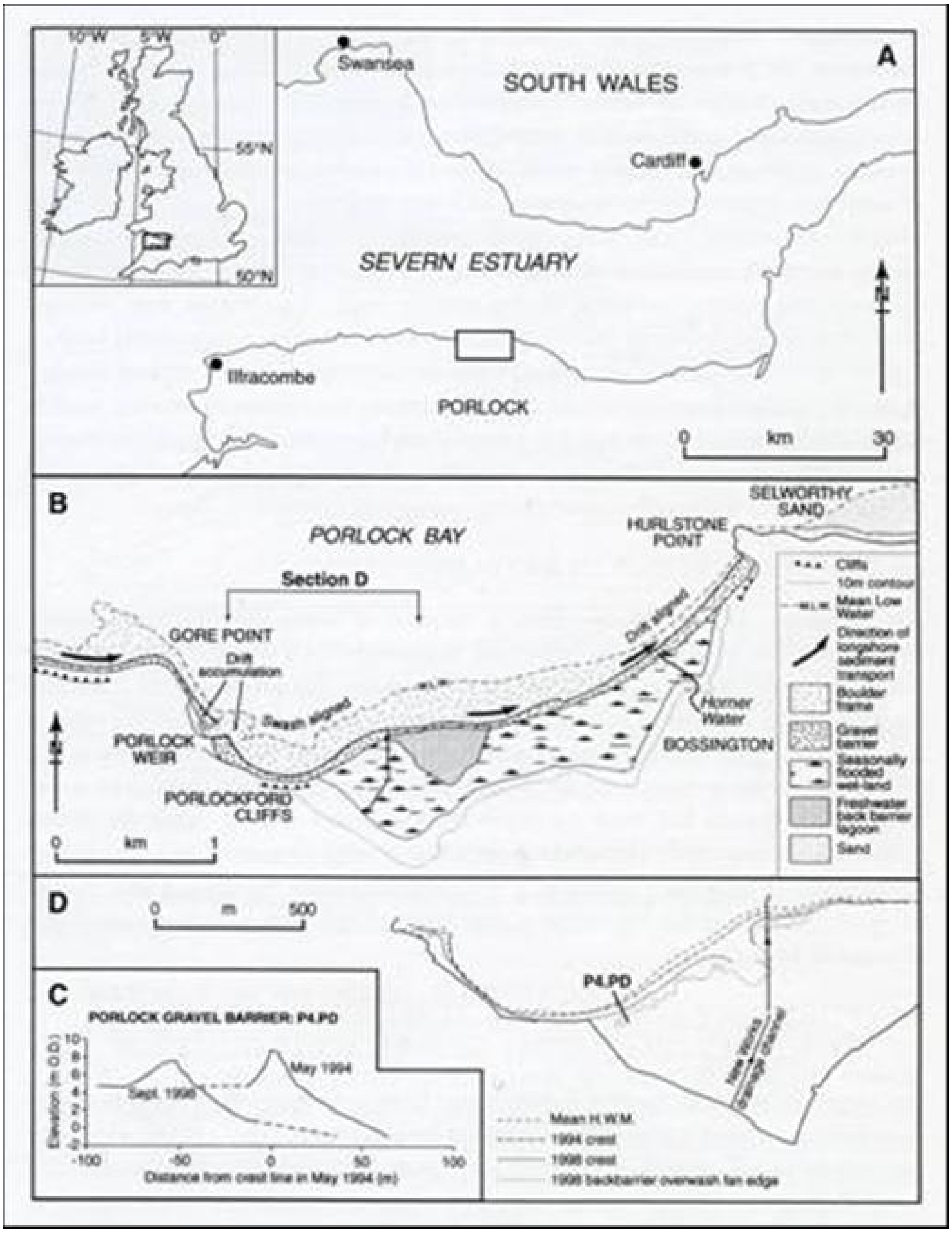


Figure 5.3 Plan-shape and typical cross-section of Porlock barrier (May and Hansom, 2003)

5.2.5 Contemporary processes

The reduced cross-section of the shingle barrier and the variability of wave exposure has made it vulnerable to hydrodynamic forces. This and the land drainage works have promoted a “non homogeneous” form in an alongshore direction, leading to the creation of “cells” (Orford and Jennings, 1998). Carter (personal communication 1990) recognised one such major cell to the west of the New Works, where the barrier is swash aligned, and another one to the east, where it is drift aligned. However, barrier/lagoon interaction must also play a part in this fragmentation.

Individual extreme events are responsible for episodic over-topping, crest cut-back, overwashing, seepage canns and breaching. Studies of the backshore have revealed presence of old lagoons and also creeks, supporting the fact that the marsh behind the ridge has been both saline and freshwater at different times in the past. Thus, the natural cycle of evolution before attempts were made to stabilise the ridge, was one of sporadic breaching, followed by self-healing. Washover fans are evident along the rear face of the barrier for much of its length but particularly on the west spit.

The most recent breaching event, which converted the continuous barrier (Plate 5.5) into double barrier spits, occurred on the 28th October 1996. Cope (2004) notes that, “The barrier has been managed since the mid-nineteenth century to reduce the risk of flooding to the grazing land behind. In the early 1990’s a policy of non-intervention was introduced. The relatively recent, meso-scale processes such as breakdown from a drift-aligned into a steeper, more swash-aligned barrier (Orford and Jennings, 1998), accompanied by previous inappropriate management, increased its vulnerability to storm events. Therefore, the storm on the 28th October 1996 was sufficient to trigger barrier over-washing and breaching (Jennings and Orford, 1999).”

Plate 5.5 Porlock barrier, January, 1985, looking south-west (Bray and Duane, 2001)

The barrier retreated landwards by up to 40m, following which a "permanent" breach was allowed to develop. Prior to 1996, the beach was repaired following breaches and washover events by a combination of recycling and re-grading of the seaward face of the barrier.

Site inspection suggests that coastal defences around Porlock Weir harbour may be exacerbating the barrier instability by cutting off the supply of shingle from the west. The beach to the west of the harbour is considerably healthier.

Cope (2004) notes that, "Since 1996, the breach has been migrating eastwards due to longshore drift, with an accumulation of material on the western spit. Still, the breach is currently stable as the tidal prism is large enough to produce strong ebb tidal velocities that flush away any incoming material that may block the breach. Bray and Duane (2001) note that the breach is enlarging and extending by downcutting, cliffing and headward recession through the clay Holocene deposits."

Following vegetation surveys at Porlock in 2001 and 2002, it is evident that the flooded hinterland is continuing to mature in terms of salt marsh colonization

5.2.6 Wave and water level climate

Cope (2004) notes that, "The Porlock Bay tidal range is large (9.3m for mean spring tides and 3.9m for mean neap tides) and does not enter Porlock lagoon until it has risen above 2.2mO.D (in 2003). This is because the lagoon is set upon a clay platform and therefore experiences a very small tidal range (2.8m for mean spring tides and 0.2m for mean neap tides). Prior to recession of the clay lip within the inlet (Plate 5.6), the lagoon tidal regime was different. When water on the seaward side of the lagoon had reached the level of the clay lip it would start infilling the lagoon on the flood tide. The water would then be held in the lagoon during high tide and for most of the ebb as it slowly drained over the clay lip. Recession of the clay lip persisted following the breaching of the barrier in October 1996. This process continued until the inlet joined with the main channel within the lagoon between May 2000 and July 2001 (Plate 5.7)."

Plate 5.6 Receding clay lip, 5th May 2000 (Bray and Duane, 2001)

Plate 5.7 Tidal inlet joined with the main channel, 7th January 2005 (Cope)

5.2.7 Scientific significance and designations

The conservation significance of Porlock Lagoon is reported by JNCC at http://www.english-nature.org.uk/livingwiththesea/project_details/good_practice_guide/shingleCRR/shingleguide/Annexes/Annex01Porlock/Index.htm. This is summarised below.

"Originally scheduled as Porlock Marsh Site of Special Scientific Interest (SSSI) in 1990 for its shingle ridge, saltmarsh and coastal grazing marsh the interest today centres on its geomorphological interest, shingle ridge vegetation and developing saltmarsh."

"A large part of this site is lower saltmarsh dominated by glasswort *Salicornia europaea* and annual sea-blite *Suaeda maritima.* Other plant species associated with this habitat include sea aster *Aster tripolium,* sea purslane *Atriplex portulacoides,* common saltmarsh-grass *Puccinellia maritima* and spear-leaved orache *Atriplex hastata.* On areas of slightly higher saltmarsh, sea plantain *Plantago maritima,* sea arrowgrass *Triglochin maritima* and sea milkwort *Glaux maritima* also occur. On shingle areas which are not inundated by salt water a variety of vegetation communities have established. Where the shingle ridge itself is most stable, saxicolous lichens cover the pebbles. Amongst the species which occur here are *Rinodina aspersa* which is nationally rare and three other species which are nationally scarce: *Buellia subdikcifonnis, Caloplaca arnoldii and Lecanora subcarnea.* On the back face of the ridge and on shingle deposited to the landward side of it communities of higher plants are found. These include swards with coastal species such as upright chickweed *Moenchia erecta,* sea storksbill *Erodium maritimum,* bird's-foot clover *Trifolium ornithopodioides* and subterranean clover *Trifolium subterraneum.* Also found here is the nationally scarce Babington's leek *Allium ampeloprasum ssp. Babingtonfi* and *Geranium robertianum.*"

"The site is visited regularly by grey heron *Ardea cinerea,* little egret *Egretta garzetta* and shelduck *Tadorna tadorna.* Small winter flocks of lapwing *Vanellus vanellus,* curlew *Numenius arquata,* teal *Anas crecca* and shelduck occur on the site as a whole. The site is also visited by a very wide range of migratory species."

The Porlock Bay barrier beach system is thus of great scientific and environmental interest in that it provides a "working example" of how a barrier beach continues to develop following a major breach. It is also a good illustration of how past maintenance activities (profile regrading) were detrimental to the health of the beach, and may in fact accelerate barrier beach degeneration.

Cope (2004) notes that, "Many changes have taken place at Porlock since the breach, in terms of barrier morphology, inlet stability and development of the flooded hinterland. This is because the Environment Agency and landowner have let the barrier respond naturally, thereby providing the only recent (1-10 years) example of an open coast permanent breach in southern England."

5.2.8 Flood defence / coast protection role

The shingle ridge used to protect low-lying land from being flooded, but that role has reduced with the present policy of allowing the ridge to develop "naturally". The ridge continues to be managed around Porlock Weir harbour so as to protect this asset.

5.2.9 Management

5.2.10 Intervention

"The apparent lack of new source material to 'feed' the Porlock shingle ridge has resulted in attempts to 'protect' the ridge and site from erosion and flooding since 1824 when the first groynes were built. Up to 1985 a series of coastal protection measures were undertaken to maintain the ridge in situ with varying degrees of success." (May and Hansom, 2003)

Continued retreat was threatening backshore development (the coast road to Porlock Weir harbour). For this reason the section between Porlock Weir and Porlockford Cliffs has been groyned in the past, using timber piles driven into the beach. Plate 5.8, taken in 1988, shows these piles were never efficient, being flimsy, porous and too short to be very effective. Nevertheless, such groyning probably made the upper part of the shingle ridge immediately to the east (i.e. between Pockford Cliffs and New Works) more susceptible to breaching.

Plate 5.8 Shingle ridge east of Porlock Harbour (1988, Motyka)

Further east the ridge is backed by low-lying agricultural land and marsh. Between Porlockford Cliffs and the New Works drainage channel outlet the ridge has been especially prone to breaching. It has been maintained by a combination of the following:

Reshaping of the ridge cross-section, making it higher and thinner.
Placing washover material onto the ridge crest
Since the 1950s, occasional re-nourishment with gravel, sand and mud, dredged from the entrance to Porlock Weir harbour (Attempts to recycle shingle from the eastern part of the bay have been resisted by the National Trust, landowners of that area).

“Because problems of erosion and instability remained Halcrow (consulting engineers) were commissioned to produce a report on how these might be overcome. This report recommended four interventionist options of which the last - "a beach nourishment programme" was accepted. Although proposals were put in hand the owners of the site where the source shingle was to be removed refused permission. In 1992 a further study (by Posford Duvivier) suggested a further four alternatives including: "do nothing"; "managed retreat"; "sustaining existing standard of defence" and providing an "improved standard of defence". In the event, in 1993 the National Rivers Authority, which had previously maintained the ridge, indicated that they would no longer do so.” (May and Hansom, 2003)

“In 1994 because of this decision the National Park Authority chaired a working group (the Porlock Bay and Marsh Working Group) to look at the issues and produce a management plan for the area. The group, after long deliberation, recommended that a ‘do-nothing or managed retreat’ option was adopted following a 1990 breach. This recommendation was rejected by the National Park Authority who suggested that the owner concerned should be allowed to maintain a sea defence if he wished and offered money towards his costs effectively adopting Option 3 of the Posford Duvivier report. (The report itself did not recommend this option). Funding for this was agreed by the local councils and one of the owners, but not by the National Trust or Natural England.” (May and Hansom, 2003)

“Any protection would have been contrary to the "Management Statement" issued by English Nature as part of the SSSI notification package under the Wildlife & Countryside Act 1981 Section 28(4) as amended. A scheme of working was, however, agreed with English Nature and shingle was moved from the harbour to the weak point of the ridge at Porlockford for about 2 years. Then the 1996 event occurred following which nothing has been done, though the owner concerned tried to persuade the National Park Committee that he should be allowed to dump tons of spoil from a new sewage work construction, in the breach.” (May and Hansom, 2003)

Further east, between New Works outlet and Hurlstone Point a coastal “sub-cell” has formed. Along this frontage littoral drift has produced a tendency for the western part to erode and the eastern part to accrete. The most vulnerable part of the ridge in the eastern cell is therefore just east of New Works.

Monitoring

Little information has been found about the monitoring of the barrier ridge, but the Wessex Water Authority may hold such data.

Visual monitoring since the 1996 breach has shown that by 2001 a tidal creek was forming, and that the lagoon, which had been a feature of the marsh, was disappearing. The high turbidity within the estuary results in large volumes of silt entering through the breach on most tides, helping to infill the lagoon. (Annual deposition rates of 10mm per annum were measured during 1999-2000, suggesting the potential for saltmarsh development (May and Hansom, 2003)).

Plans to implement a cohesive monitoring programme, in conjunction with the South-west Strategic Coastal Monitoring Programme are now in place; this programme has recently commenced and will provide the following;

Annual 1m resolution LiDAR surveys of the entire beach?
Permanent control locations, surveyed by static GPS observations
Production of 10cm resolution digital orthophotos based on low level aerial surveys, every two years

Baseline mapping of ecology to a level suitable to inform the extent of designated habitats for biodiversity actions plans
Bi-annual beach profiles
Post-storm beach profiles
Directional waverider buoy in shallow water

Issues

In this instance ownership has had a bearing on the management of the ridge. The landowners within Porlock Bay include the National Trust, which owns the eastern part of the bay and Porlock Manor Estate, which owns the more developed western part.

Management issues are dealt with by a Working Group, which consists of statutory authorities and the riparian owners. When the 1996 breach was formed, the majority view was that no further maintenance of the shingle ridge should be carried out, but that monitoring should be used to determine the need for possible future intervention.

(It is worth noting that, at that time, it might have been possible to enter the marsh into the Saltmarsh Option of MAFF's agri-environment Habitats Scheme. The landowner would have then received payments to allow salt-marsh to form after a deliberate breach. This possibility was not pursued).

Washover fans have spread shingle landwards, leading to a net reduction in the volume of the ridge. The spits have also been migrating alongshore, so there is the possibility that the new creek system may become blocked yet again in the future, a process that has occurred in the past.

5.3 Medmerry/ Selsey

Location	Medmerry, west of Selsey Bill, West Sussex (see Figure 5.4)
Designations	SSSI
Length	4.5km
Width	30m (2006)
Crest	5.25mOD (2006)
Sediment	Shingle
Tides	Spring tidal range 4.5m
Waves	Dominant westerly waves diffracted around the Isle of Wight and refracted inshore onto a south-westerly direction of approach.
Classification	Swash aligned shingle barrier beach, with a tendency for net east to west littoral drift.
Intervention	Groynes, recharge, re-grading, recycling, crest-lowering

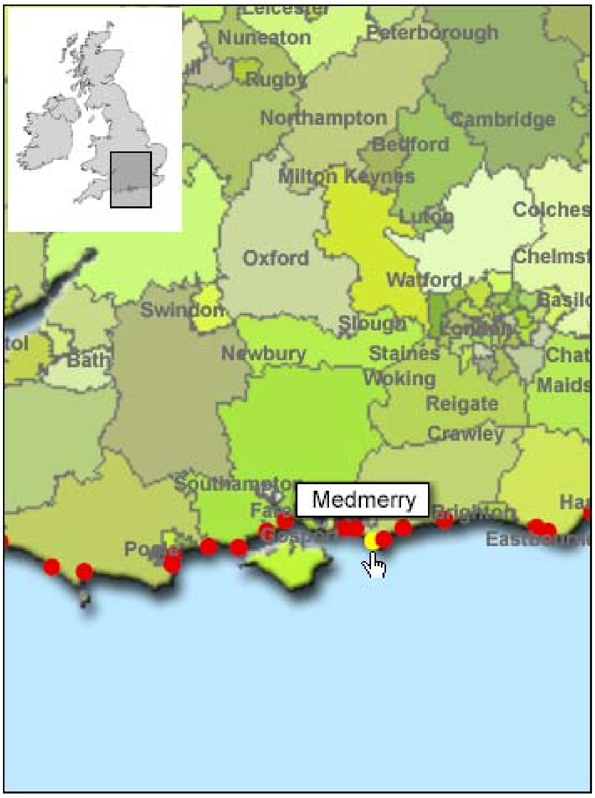


Figure 5.4 Location of Medmerry barrier

5.3.1 Introduction

Medmerry is a barrier beach that extends 4.5 km from Selsey Bill in the southeast to Bracklesham village in the northwest (Figure 5.4). The barrier is backed by medium and high-grade farmland, which lies within the floodplain of the Broad Rife river (Plate 5.9). The area has been described as the, “jewel in the crown of the Solent” as it is the largest potential mudflat and saltmarsh habitat creation site between Hurst Spit, Hampshire and Pagham Harbour, West Sussex. The backshore at the southeastern end of the barrier is developed as a holiday camp and is frequently flooded when the beach is breached.

Plate 5.9 Medmerry barrier, 2001 - S.E (Cope, 2004)

Due to the low lying hinterland, the flood plain extends northwards and then eastwards to Pagham Harbour, thus encircling the higher ground on which the village of Selsey is situated (Figure 5.5).

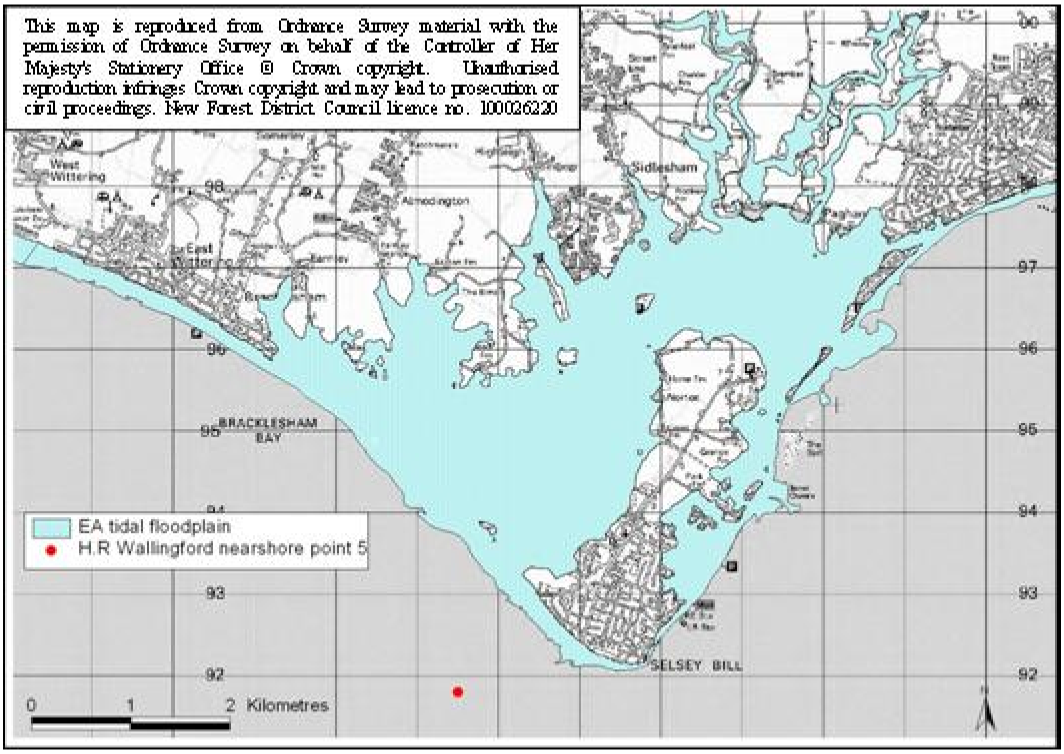


Figure 5.5 EA tidal floodplain

Should a permanent breach develop, there is a possibility of flooding almost encircling Selsey Bill reverting it back to an island (Figure 5.6). At the western end, the beach ties into slightly higher ground at Bracklesham village (Figure 5.5). However, there is a (weak) possibility that floodwater could also affect Bracklesham, by dispersing laterally from the Broad Rife floodplain.

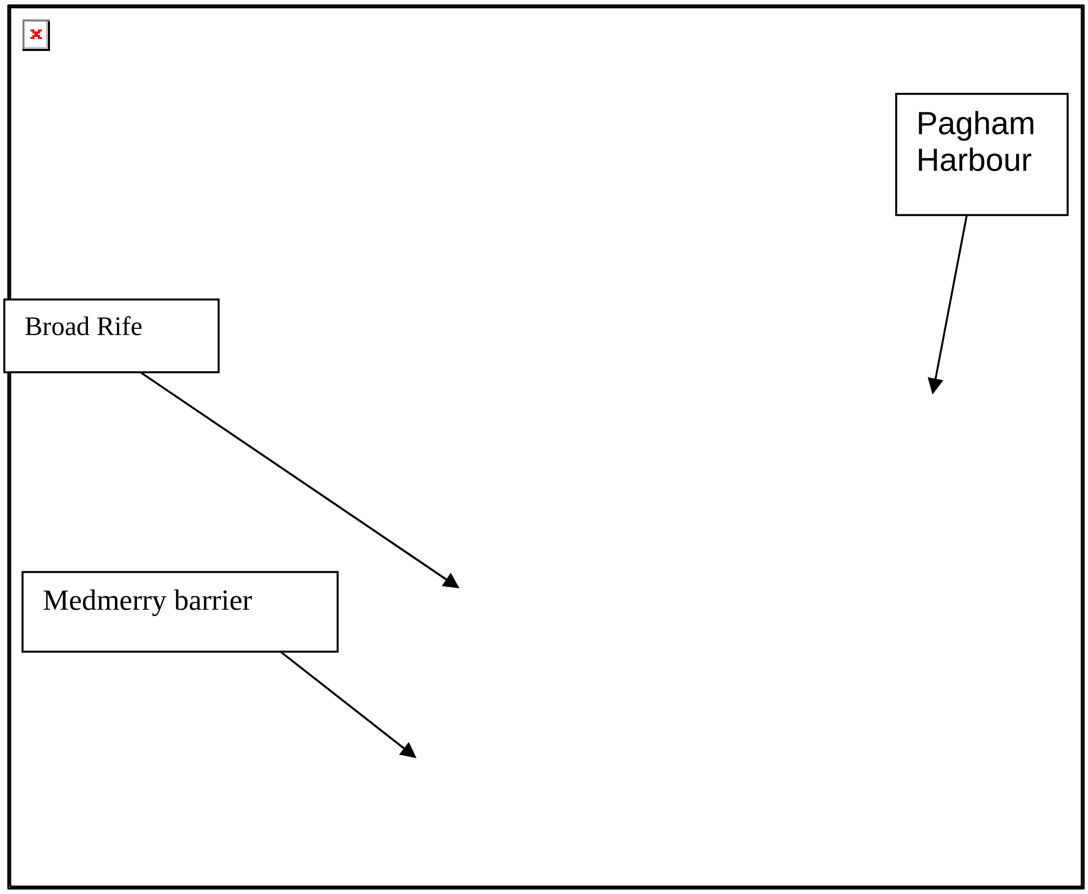


Figure 5.6 Selsey Island,1587 (Clifford Fidler, 1987)

5.3.2 Geomorphological context

The East Solent is the drowned channel and flood plain of the ancient river Solent. Rising sea levels during the Holocene transgression caused the river valley to become drowned and infilled by fluvially deposited gravels. It is hypothesised that Medmerry was once part of a “super” barrier extending from Selsey Bill to Portsmouth that was formed throughout the early Holocene transgression. The super barrier would have been relatively stable as it rolled onshore due to an abundance of sediment supply in relation to sea level rise. Relict barrier islands named the Mixon Shoal and Owers, situated offshore from Selsey Bill indicate that the barrier continued to migrate inland 2,000 to 3,000 years B.P. (Wallace, 1996) but in doing so would have undergone sediment depletion due to reworking of the Holocene finite source (Jennings and Orford, 1999) (see Figure 5.7). A reduction in cross-sectional area would have resulted in increased vulnerability to storm attack (Cope, 2005).

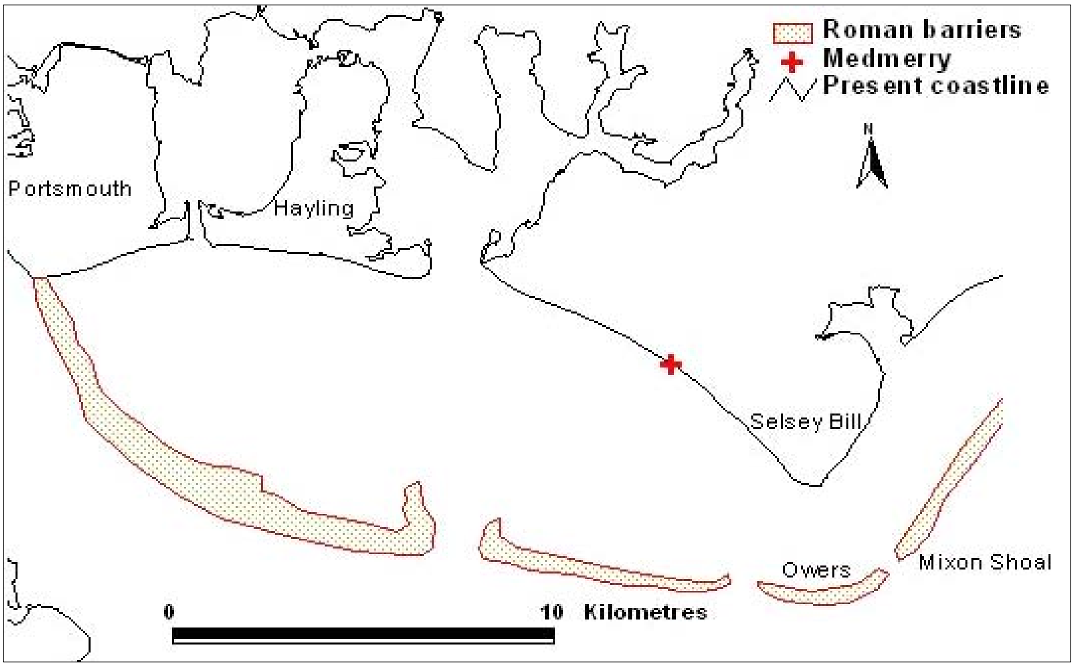


Figure 5.7 Location of Medmerry Roman barrier-not map accurate (Cope, 2005, after Wallace, 1996)

These barriers would have protected the coastline 2,000 yrs B.P. when Selsey was part of an island separated from the mainland by the Broad Rife tidal channel running from Pagham Harbour to the Medmerry frontage (Posford Duvivier, 1999). Figure 5.6 shows a similar scenario in 1587.

Geodata Institute (1994) note that throughout the 7th century AD, the south-western part of Selsey was infilled by sediments and Selsey Harbour was thus formed. Even from 1587 - 1644, Selsey Bill remained an island due to the presence of Broad Rife. The Medmerry barrier was the only link to the mainland (Carter and Bray, 2004). With time, these rifes were reclaimed (Geodata Institute, 1994) and Broad Rife slowly filled with alluvium, thereby forming the contemporary low-lying hinterland.

5.3.3 Sediment size and grading

The Medmerry barrier is composed of a shingle storm ridge (clast diameter between 2mm and 75mm) that overlies a flat sandy foreshore (H.R Wallingford, 1997). The main source of shingle (5,000m^3) comes from offshore at Selsey Bill. 90% (4,500m^3) travels north-east towards Pagham and Bognor, whilst 10% (500m^3) travels north-west towards Medmerry and East Head Spit. Additionally, 1,400m^3 per annum is eroded from the low-lying cliffs at Selsey Bill (Posford Duvivier, 1999). Minor onshore transport occurs by kelp rafting.

There is greater potential for shingle transport than can be met by natural supply (Posford Duvivier, 1999). As a consequence, the main source of sediment supply has been from nourishment schemes since 1974 (H.R Wallingford, 1997; Posford Duvivier, 1999) thereby creating a mixed sand and shingle barrier. There is longshore sediment sorting, with the beach widening to form a more protective barrier at the north-west end. Shingle moving through the Medmerry frontage also feeds the downdrift beaches and, ultimately reaches East Head spit (see separate case history). In the future, Medmerry is likely to lose volume and, in turn, so too will the downdrift frontages, as far west as East Head. Cross-shore sediment sorting is not prominent, due to the mixed sediment composition that follows bulldozing.

5.3.4 Geometry

The shingle ridge that forms the upper beach, and hence the main line of defence against flooding had a crest elevation of approximately 6m, width of approximately 25m and an artificially steep slope of 1:8 - 1:10 (2001 data) for decades. These dimensions have now changed to 5.25m crest height and 30m width, following beach lowering and widening (see Management Practice). The shingle overlies a flat (1:50), approximately 300m wide, sandy foreshore.

The artificially steep backslope is relatively free of washover features due to periodic bulldozing. However, the barrier does undergo seepage as sea water percolates from the seaward to the landward side.

5.3.5 Contemporary processes

The shingle ridge at Medmerry has been eroding and retreating landwards at a similar rate, or higher, than the cliffed adjacent frontages, which eroded rapidly until defences were constructed to protect them in the 1950s. The shingle ridge has been maintained on a "stable" alignment for more than half a century; hence the recession rate has been relatively low.

Because of the tendency for landward retreat, the lower foreshore is being eroded also, although there is little documented data on the rate of lowering of the sand levels, or of the clay substratum (HR Wallingford, 1997).

Due to a lack of sediment supply, resulting in a reduced cross-sectional area, the barrier is vulnerable. The most vulnerable section of the Medmerry barrier is highlighted in Figure 5.8. The majority of overwashing and breaching events occur in the areas highlighted "Breach" and "Windmill".

The barrier undergoes overtopping on an annual basis. Crest-cut back and crest rollback and lowering are dominant processes during low magnitude storm events, whilst breaching occurs on slightly higher magnitude storm. However, machinery is on standby to quickly reform the beach crest. There is evidence of previous breaching and re-sealing along the Medmerry frontage since the 8th century A.D, particularly during "super" storms, recorded for Selsey and Hayling Island between 1014 and 1490 (Carter and Bray, 2004). Hydraulics Research (1996) note that Medmerry was breached in 1910 (along with Pagham spit barrier), temporarily reverting Selsey into an island.

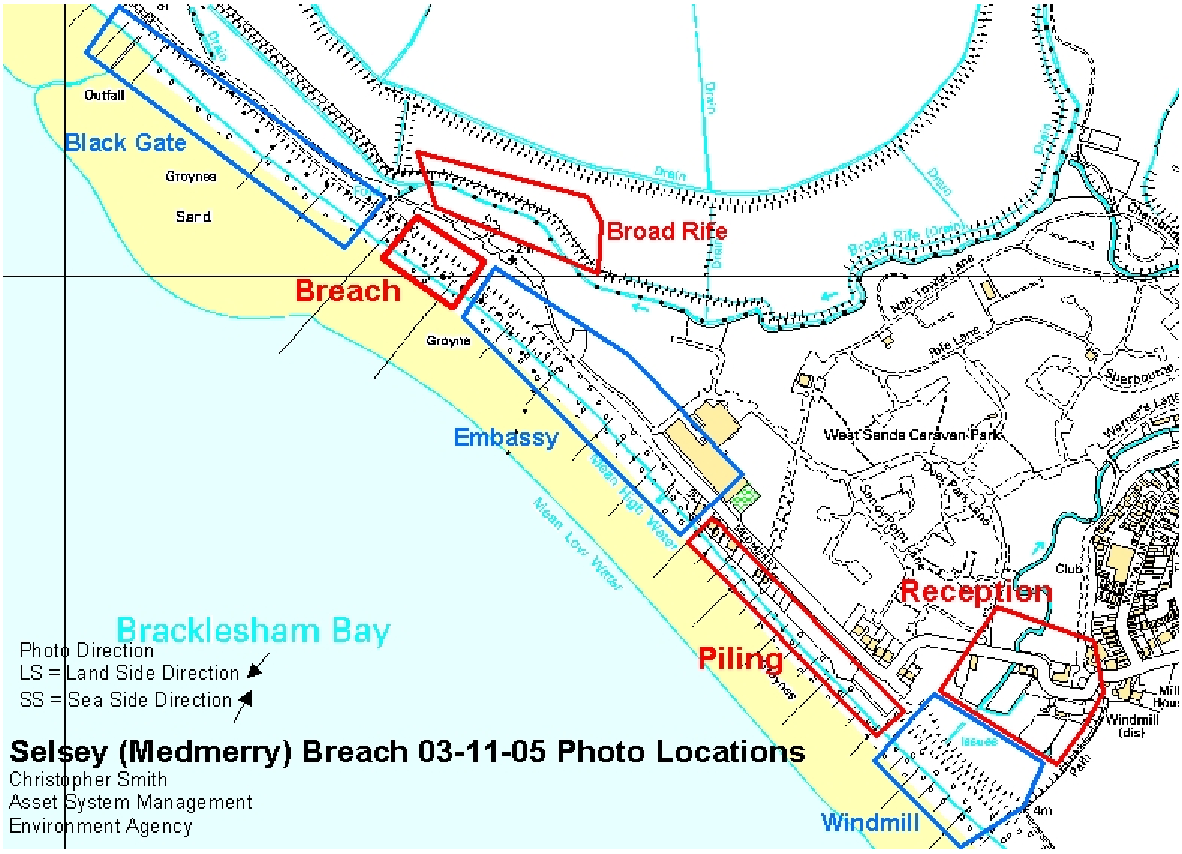


Figure 5.8 Location of management practices (produced by Chris Smith, EA)

5.3.6 Wave and water level climate

Medmerry barrier is not only prone to overwashing and breaching due to a reduced cross-sectional area but because of swash-alignment at the most vulnerable section, with predominant waves approaching from the south-west 40% of the time and secondary waves approaching from the south 30% of the time (H.R Wallingford, 1997). Still, the Boulder, Pullar, Medmerry Banks and Hounds Rock which are relict barrier beaches can act as breakwaters by reducing incident wave energy (Posford Duvivier, 1999).

Jelliman *et al.* (1991) suggests that mean wave approach has altered in the past 20 years, thereby affecting the dominant wave direction and littoral drift rates. They also note that there has been an increase in mean wave height (H.R Wallingford, 1997).

Tables 5.1 and 5.2 demonstrate storm and swell water levels, wave height, wave period and wave length data (H.R Wallingford, 1995; Posford Duvivier, 1999).

Table 5.1 Nearshore storm data for point 5 (SWL (Posford Duvivier, 1999), H_s (m) and T_m (s) (HR Wallingford, 1995)

Return period	*SWL* (mOD)	Wave height (H_s) , m	Wave period (T_m), s	Wave length (L_m), m
1:5	**3.2**	**2.97**	**5.21**	**42.40**
1:50	**3.6**	**3.69**	**5.87**	**53.83**
1:200	**4.0**	**4.06**	**6.09**	**57.94**

Table 5.2 Nearshore swell data for point 5, wave periods 13.4 (s) and 17.3 (s)

Return period	*SWL* (mOD)	Wave height (H_s), m	Wave period (T_m), s	Wave length (L_m), m
1:1	**3.0**	**2.4**	**13.4**	**280.49**
			17.3	**467.52**
1:10	**3.3**	**3.0**	**13.4**	**280.49**
			17.3	**467.52**
1:100	**3.8**	**3.4**	**13.4**	**280.49**
			17.3	**467.52**

Tidal levels and predicted extreme water levels are presented in Table 5.3 below.

Table 5.3 Tidal levels and predicted extreme water levels for Broad Rife (mODN) (HR Wallingford, 1995b)

	Tidal levels for Broad Rife (mOD)
MHWS	2.40
MLWS	-2.10
Return interval (yrs)	**Extreme still water level (mOD)**
1	2.78
5	2.91
10	3.21
50	3.28
200	3.43
Estimated 50 yr rise (m)	0.3

Surges of 1m at high water are relatively common (Table 5.4). Table 5.4 presents key storm events between 1993 – 2002 resulting in overtopping, crest cut-back/scouring, overwashing and breaching between profiles 54-57 (Figure 5.9).

Table 5.4 Hydrodynamic input data for storm events between 1993 – 2002 (Cope, 2005)

Date	SWL (ODN)	Portsmouth surge (m)	Wave height H_s (m)	Wave period T_z (s)	Peak wave period T_p (s)	Documented response between profiles 54-57
10.1.93	2.6	0.85	2.40	6.5	9.6	Overtopping
11.1.93	2.6	0.95	2.16	6.3	9.2	Scour
3.12.94	2.6	1.13	1.72	5.9	8.0	Overwash or Breach
4.1.98	2.1	0.47	3.04	6.7	10.3	Overwash or Breach
4.3.98	2.0	0.70	2.04	5.8	8.6	Overwash or Breach
24.12.99	2.6	1.00	2.45	6.4	9.3	Overwash or Breach
25.10.00	2.2	0.90	1.82	5.9	8.6	Overwash or Breach
1.2.02	2.6	0.60	2.10	6.1	9.1	Crest cut-back

5.3.7 Scientific significance and designations

The foreshore and backshore around Broad Rife are designated SSSI sites. The foreshore is also a designated Geological Conservation Review Site due to its geological interest and Crablands Farm Meadow and Bracklesham Balls are Sites of Nature Conservation Interest (H.R. Wallingford, 1997).

5.3.8 Flood defence / coast protection role

The shingle ridge protects low-lying land from being flooded. This includes the Broad Rife floodplain, within which there is grazing pasture, West Sands Caravan Park (Figure 5.9) and residential and commercial property. The western end of the barrier protects the village of Bracklesham from flooding.

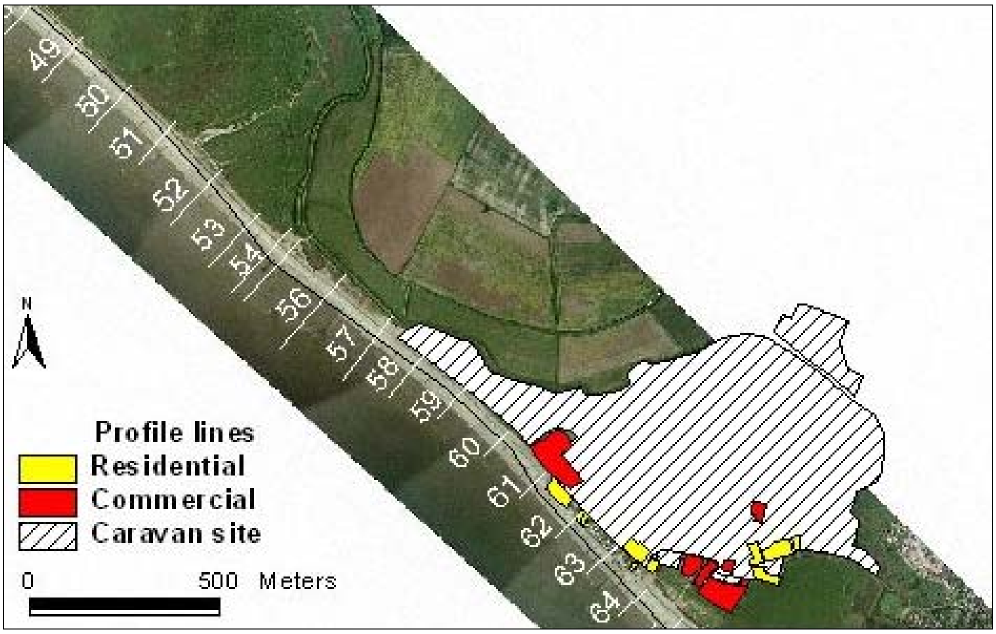


Figure 5.9 Location of residential and commercial properties vulnerable to immediate flooding (Cope, 2005)

5.3.9 Management

Intervention

Groynes were implemented along the Medmerry barrier and West Wittering frontage in the late 1930's, whilst the fagotting that is found at the eastern end of the frontage may be of similar date; it has been preserved by being covered up with shingle, only to be exposed now that the shingle beach is seriously eroded. There was further groyne construction along the frontage in 1964 (Hydraulics Research, 1996).

However, the groynes have never been efficient in trapping shingle as they are covered by sediment replenishment, require continuous maintenance (due to rapid abrasion) and are currently 4 main and 5 stub groynes short (Phil Pett, E.A, *personal comment*, November 2001; H.R Wallingford, 1995). In addition, the steepness of the beach results in beach drawn down onto the lower foreshore during storms, where it is transported uninterruptedly alongshore. There have always been problems of erosion downdrift of Broad Rife as the groynes are generally better maintained around the outfall, thereby leading to downdrift sediment starvation (H.R Wallingford, 1997). The groynes require ongoing maintenance; otherwise the barrier will become increasingly mobile and vulnerable to breaching (H.R Wallingford, 1997; Bray, 1999).

Successive recharges and recycling operations have also promoted overwashing. Prior to 1975, the reduced volume of the shingle ridge made it increasingly difficult to maintain a stable alignment; it was therefore first nourished with 225,000m^3 of land-based shingle material between 1975-80. It has since been managed by smaller recharges, and continuing recycling and re-grading operations (by dragging up beach material on the front face).

Between September and April, it has become the norm for there to be two bulldozers on site.

In 1997, H.R Wallingford (1997) estimated that, with maintenance, the shingle ridge and groynes provide a 1:20 year standard of defence but without maintenance they only provide a 1:1 year standard of defence. In 1999, Posford Duvivier (1999) predicted that in;

0-5 years there will be more frequent breaching and increased wave attack to the sheet pile sea wall at the south-eastern end.
5-10 years the barrier will generally be ineffective as a coastal defence, the sheet pile wall will collapse and the groynes will deteriorate.
10-30 years there will be a total loss of effective defences.

The 0-5 year prediction is correct. However, there has been a slight change in management regime since 2002. Between circa 1985 – 2002, management practices tried to achieve a crest height of 6.5mOD and maintain a clay core. This is no longer the case, since the Environment Agency (EA) has decided that the barrier was too high and narrow. EA Annual Beach Monitoring Surveys (ABMS) profiles showed the barrier crest had been slowly steepening over time. Therefore, in 2003/04 7,500 tonnes of shingle (40 - 70mm) was imported at the Black Gate end of the barrier (Figure 5.8).

The shingle was placed on the back of the defence and was also partly won by lowering the crest from 6.5m to approximately 5.25m. This resulted in a less steep profile on the back and front and a much wider crest. The profile underwent crest cut-back during storms, due to poor sediment sorting, but there were no major breaching events for a year. Encouraged by the success, the EA imported another 7,500 tonnes of shingle at the Windmill end in 2004/05 (Figure 5.8) and lowered the crest height. There was a 10 metre gap between the shingle barrier and a bank behind the barrier, which was infilled to increase the cross-sectional area. In addition, any remaining shingle was deposited near the car park at the Black Gate end (Figure 5.8), inefficient groynes were removed and the barrier plan-shape was straightened. However, comparison of 2001 and 2005 photography does not show a major difference in plan-shape (Figure 5.10).

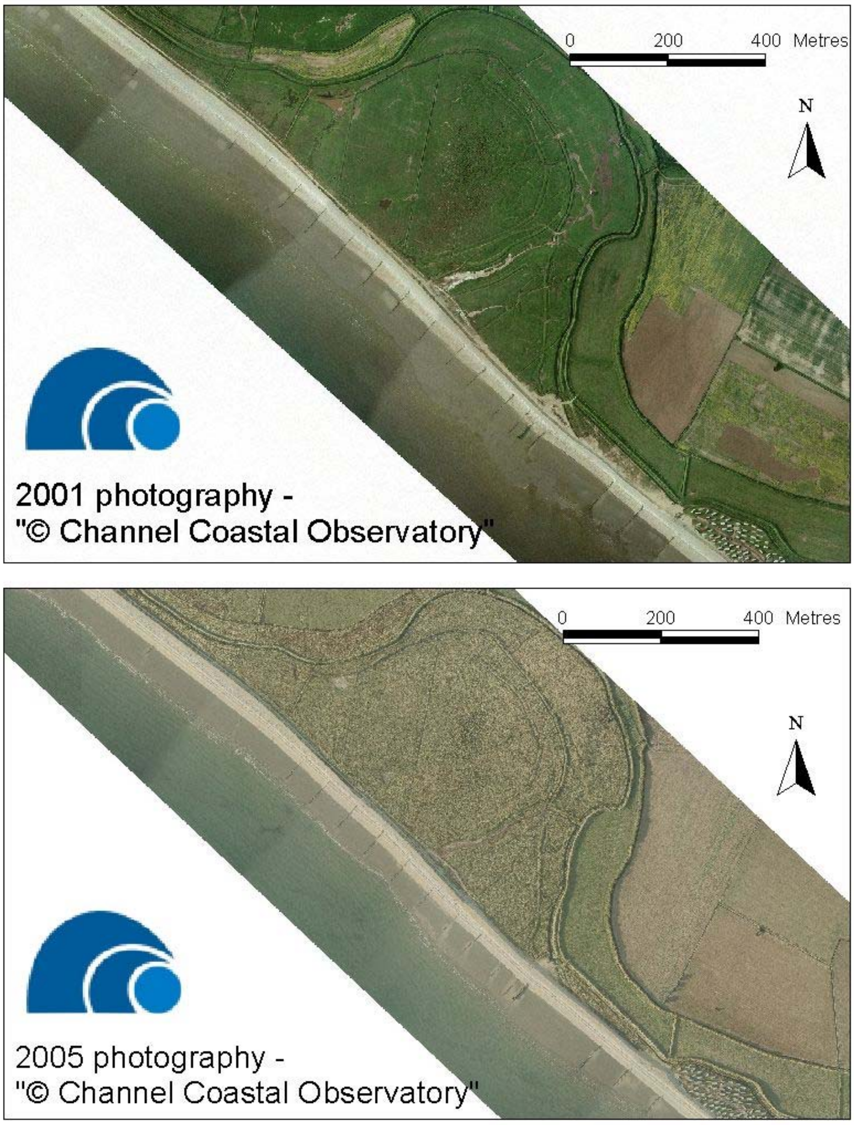


Figure 5.10 Comparison of 2001 and 2005 plan-shape at Medmerry

The work carried out at the Black Gate and the Windmill end, was put to the test in November 2004. The barrier was relatively stable where additional works had been carried out but breached between the Black gate and the Embassy club (Figure 5.8 and Plate 5.10).

Plate 5.10 Breach at Medmerry on 3rd November 2005. Looking NW. (Cope)

The beach breached because the additional works were inhibited at this location by the proximity of caravans directly behind the barrier (Plate 5.10). The barrier was therefore not wide enough to withstand breaching and had an over-steep backslope.

Monitoring

Historically, the Medmerry barrier was monitored through EA ABMS surveys. These were taken from photogrammetric aerial photography interpretation. Since the South-east Strategic Coastal-process Monitoring Programme, the following data is now collected;

Permanent control locations, surveyed by static GPS observations
Production of 10cm resolution digital orthophotos based on low level aerial surveys, every two years
Baseline mapping of ecology to a level suitable to inform the extent of designated habitats for biodiversity actions plans
Annual beach profiles
Post-storm beach profiles
Directional waverider buoy in shallow water off Hayling Island

Issues

The management of the ridge is now increasingly less able to conserve the “status quo”. There is a short length of timber/sheet pile breastwork at the southeastern end of the frontage that protects a line of shorefront properties. As the line of cliffs southeast of the ridge continue to retreat, so it is becoming increasingly difficult to maintain the shingle ridge in front of the housing, due to it becoming out of line with the naturally retreating coastal plan shape. As the breastwork is relatively unsubstantial, this structure is likely to become degraded by exposure to wave action.

From visual inspection, erosion during the last two to three years appears to have accelerated, resulting in the exposure of the underlying clay substratum, not only on the lower foreshore, but almost to the toe of the defences (old lines of fagotting have been exposed during this process, being the remnants of former management activities). This recession is now threatening the shorefront properties.

The general standard of defence provided by the shingle ridge is now relatively low and variable. As already mentioned, the area in front of Broad Rife, named “breach” in Figure 5.8, is regularly breached and serious damage would result if the Environment Agency were unable to respond quickly during emergencies. Plate 5.11 shows the ridge in its managed condition.

Plate 5.11 Management of the shingle ridge at Medmerry

The barrier hinterland is within the 1 in 200 year tidal flood plain, although the caravan site, residential and commercial properties are vulnerable to flooding under approximately a 1 in 5 year event.

When considering the robustness of the management approach it is worthwhile considering the effect of po4ssible changes in management policy. The various policy options, together with the likely outcomes, are documented in the Shoreline Management Plan (HR Wallingford, 1997) and briefly are as follows:

A “Do Nothing” policy would result in the shingle ridge being overtopped and breached, leading to the development of new salt-marsh. There would be significant loss of farmland and the holiday camp would be regularly flooded and possibly become unusable. Flooding might also extend into Pagham Harbour, through a flood route on the alignment of Broad Rife.

A “Hold the Line” policy would result in continued erosion of the foreshore and increased wave energy reaching the ridge. The geological interest of the foreshore would be reduced. The present management policy could not be sustained indefinitely and it is considered that hard defences would eventually be needed along the full frontage. It was determined that maintaining the existing shoreline would become increasingly expensive, with the main benefit being to the holiday developments. Maintaining an appropriate standard of defence under these conditions was considered to be unsustainable in the longer term.

A “Managed realignment” policy would result in reduced wave attack and ultimately a more stable shoreline position. This would result in the loss of several shorefront houses and would require the re-siting of some holiday facilities. There would be minor loss of wetland but improved geological interest. The new line of defence would require less management, but the embayed nature of this frontage might lead to reduced beach stability in adjacent areas, as these would now be more exposed to wave action and have less beach supply. Location of a flood embankment to the landward of the existing shingle ridge, would allow an improved standard of defence to be established.

5.4 Chesil

Location	*Lyme Bay, Dorset* (see Figure 5.11)
Designation	SSSI, SAC, SPA, Ramsar site.
Length	28km
Width	150m-200m opposite the Fleet (where it is unconstrained) but reducing in width to 40 to 50m at eastern end.
Crest	6m to 14.7mOD at eastern end.
Sediment	Shingle; d_{50} = 4mm to 100mm increasing from west to east
Tides	Spring range 2m at Portland.
Waves	Atlantic fetch with SW dominant. Waves with a height of up to 6.5m have a return period of about 1 in 5 years, while 9m waves have a 1 in 50 year return period. (May and Hansom, 2003).
Littoral drift	Weak and variable and no net drift at eastern end because of the presence of the Isle of Portland. Some material was able to bypass West Bay harbour, but the recent harbour extension means that any such losses are now virtually zero (material entering the harbour rather than being transported westwards).
Classification	Swash aligned barrier beach that is pinned at both ends but free to roll back landwards over most of its length.
Intervention	Mostly unmanaged, but; gabion mattresses, interceptor drain, seawall, road level raised

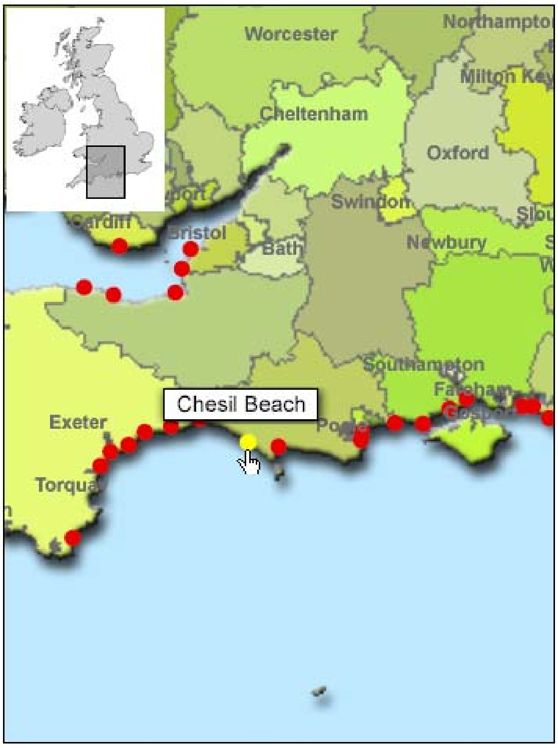


Figure 5.11 Location of Chesil Beach

5.4.1 Introduction

Chesil Beach is a 28km long shingle barrier beach that extends from the Chiswell at its east end to West Bay harbour at its west end, and is Europe's longest barrier beach. For much of its length (13km) it is backed by a shallow tidal lagoon, called The Fleet (Plate 5.12). The Fleet is the largest of its type in England. Water percolates into The Fleet through Chesil Beach, but most of the tidal exchange is through a narrow channel that enters Portland Harbour below Ferry Bridge (towards the eastern end of Chesil Beach). Low freshwater input produces high saline conditions for most of the length of the Fleet, but with reducing salinity towards the western end at Abbotsbury.

Plate 5.12 Chesil Beach and The Fleet lagoon

The lagoon supports two species of eelgrass and three species of tasselweed, together with a diverse fauna (including a number of nationally scarce species. Chesil Beach itself supports drift line vegetation, although much of the length is subject to washover, percolation etc. and is therefore sparsely vegetated. The Fleet is recognised as a marine Special Area of Conservation (SAC). Chesil

Beach and The Fleet combined are part of the Dorset and East Devon Coast World Heritage Site.

Chesil Beach is composed primarily of flint and chert pebbles, which become more angular with depth. The barrier is unique for its longshore pebble size grading. The pebble size also reduces from about 100mm diameter at its eastern end to about 4mm diameter towards the western end.

Estimates of the volume of material within Chesil Beach are between 15 and 60 million cubic metres. The uncertainty in the exact volume is due to the difficulties in assessing where the actual “horizon” of Chesil lies. However, considerable volumes have been lost through mineral extraction in the last century. It has been estimated, for example, that some 1 million tonnes of material have been removed between the mid 1930s and 1977 (Hydraulics Research Station, 1979).

Chesil Beach is thought to be now in a fragile state (May and Hansom, 2003). It is self-evident that there is very little material now being added to it, while there is material being lost through attrition and possibly offshore transport (with considerable losses in the past due to mineral extraction). It is undoubtedly, like most barrier beaches, moving shoreward under sea level rise and storm impacts. It is thought that, in doing so, The Fleet will become “pinched in” and may eventually fragment into individual lagoons (May and Hansom, 2003).

Chesil beach has been studied extensively in recent years, and the volume of information thus obtained is considerable. The information presented here is by necessity a précis. For further information, it is recommended that a search on the internet for Chesil Beach is carried out as a minimum.

5.4.2 Geomorphological context

The beach origins and development are widely debated. The consensus is that the Chesil barrier formed as an offshore bar approximately 80,000 years B.P (Carr, 1978). The classical transgression concept (Carr, 1973) is added to by Bray (1997) who suggests that the contribution of landslide gravels is of significance to beach evolution. It is suggested that the bar migrated onshore throughout the rapid Holocene transgression, thereby incorporating gravel deposits into the system from Lyme Bay sea floor (Carr, 1978; Nicholls, 1985; Posford Duvivier, 1998). Cliff erosion from sites to the west, including, Black Ven, Stonebarrow, Broom Cliff, Golden Cap, Ridge Cliff and Thornecombe Beacon, would have provided a longshore sediment source from which the beach prograded and became a sediment sink (Bray, 1996). As sea level rise continued to slow in the late-Holocene, offshore deposits were no longer prominent as a sediment source and the beach became more dependent on the longshore sediment supply. The mass of shingle is coarsely estimated at 25 to 100 million tonnes - based on borehole data.

Shingle transport from the west, to maintain Chesil Beach, would have been possible in the past. However, after the mid 1800s the construction of the harbour at West Bay has severed the link between the coast west of the harbour and Chesil Bach. The beach immediately to the west of the harbour is very fragmented and unable to provide any significant quantities of material to bypass the harbour in an easterly direction. There is no transport of shingle at the eastern end of the frontage, where the beach abuts the Isle of Portland. The effects of beach mining and natural attrition make Chesil Beach a relict feature that will continue to decline in volume.

5.4.3 Sediment size and grading

The beach composition is 98.5% chert and flint with the remainder composed of pink Triassic (Budleigh) Quartzites; Portland Limestone and Chert, occasional “exotics.” The sediment becomes increasingly sandy with depth into the beach and mobile sediment extends seaward at least out to -10 to -20m water depth. Size grading varies significantly along the beach with “Pea”-sized shingle at West Bay to Cobbles at Chiswell.

5.4.4 Flood defence / coast protection role

Chesil Beach provides a natural line of protection against coastal flooding to several developed areas. Gradual recession of the beach towards the settlement at Chiswell has made this area increasingly vulnerable to flooding. Flooding arises as both the result of overtopping and overwashing and also percolation through the beach.

Percolation through the beach arises as a function of a combination of the beach permeability, long period waves and high water levels. Wave period is considered to be the first order hydrodynamic variable, whilst tidal elevation is of secondary significance. This conclusion is based on internal flow monitoring derived from instrumented boreholes through the cross section of the beach (EA). The highest flow values monitored are always found in association with long period (18-20s) swell events. The highest level recorded (November 2005) was coincident with an event characterised by a storm event with a bimodal wave spectrum at several wave buoys further to the east. The characteristic wave periods of the two spectral modes for this event were 18s and 8s. Observations of the highest seepage rates have generally not been over high water periods, but they are always associated with long wave period events. Regrettably these wave events have not been quantified. This should be possible in the future when wave rider buoys are installed as part of the monitoring programme. There is some considerable merit in developing the internal flow monitoring system at this site.

Similarly, wave period is considered to be the first order variable in connection with wave - run-up and overtopping, whilst water level is considered to be of 2nd order significance at this site. Wave overtopping is unlikely to occur even at extreme water levels unless the wave conditions are characterised by long period waves.

5.4.5 Geometry

The beach crest elevation increases eastwards from West Bay to Chesilton, with the maximum crest height being 14.7mOD at Chesilton (near Portland) (Carr and Blackley, 1974). The progression in beach crest elevation from west to east is summarized below.

6.0mOD (West Bay),
10.5m (Abbotsbury),
12.5m (Wyke narrows)
13.5m (Chiswell)
14.7m (Chesilton)

The dramatic variability of the beach crest elevation reflects the variability of longshore wave climate, the beach cross section and the back barrier geometry. The beach is 150 – 200m wide opposite the Fleet Lagoon but is narrower towards its western and eastern extremities (Carr and Blackley, 1974). Its width is 155m at Abbotsbury, 182m at Portland.

The back barrier geometry varies along the length, with the Fleet lagoon backing some 13km of the beach. The barrier fringes against cliffs in other areas. On the seaward side offshore the beach drops at a broadly similar gradient to that of the seaward face above low water mark. The shingle extends to a depth below low-water mark of 11m at West Bexington (some 270m offshore) and also at Abbotsbury, to about 18m at Wyke Regis (also at some 270m offshore) and to 15m at Portland. On the landward side, however, the shingle rests on a bed of clay 1 to 1.2m below low water-mark.

5.4.6 Wave and water level climate

The following section is based upon a review by Bray (2005). Hindcast wave data were analysed to determine an offshore extreme wave height of 7.25m for a 1 in 100 year event with corresponding inshore values (1 in 100yr) of 6.5 and 6.65m at West Bexington and Wyke Regis respectively. An offshore extreme wave height of 4.5m for a 1 in 100 year event was calculated for swell waves. A numerical model (Babtie Group, 1997) was applied to further investigate response of swell waves on approach to the beach.

It was found that these waves were extremely sensitive to the bathymetry of Lyme Bay where features in water depths of up to 50m could affect transformation processes as the waves travel inshore. A depression and mound located well offshore on the bed of Lyme Bay appear to focus the waves upon specific sections of the coastline at Portland Bill, Wyke Regis and Abbotsbury. Results suggested that swell waves were only a significant phenomenon on Chesil to the east of West Bexington, whereas the shore to the west was sheltered from them. The analyses were undertaken using significant wave heights of 2m and 4m with periods of 15 to 25 seconds.

Offshore wave climates for West Bay have been determined by hindcasting based on Portland wind data covering 1974-84 (Hydraulics Research 1985) and 1974-90 (Hydraulics Research 1991d). These studies showed that prevailing wave direction was from the south-west, but that directional distribution was subject to significant change after 1982 with markedly fewer south-easterly storms and a higher proportion of west and south-west waves.

It must be concluded that with existing information it has proved difficult to define a reliable wave climate, because of this variability and this site is thus understood to be highly sensitive to any future changes in wave direction (Brampton, 1993). Extreme wave heights of 5.4m and 6.4m were calculated for the 1 in 10 and 1 in 100 year return periods, respectively (HR Wallingford, 2000a; 2000b). Analysis of local wave heights has suggested an increase of some $4mma^{-1}$in recent decades, although there is much scatter in the data (HR Wallingford 1998a).

Chesil was one of the locations for which wave modelling exercises were undertaken as part of the DEFRA Futurecoast Project (Halcrow, 2002). An offshore wave climate was synthesised based on 1991-2000 data from the Met Office Wave Model and then transformed inshore to a prediction point off Chiswell at –4.1m O.D. Potential sensitivities to likely climate change scenarios were then tested by examining the extent to which the total and net longshore energy for each scenario varied with respect to the present situation. Results suggested that a one to two degree variation in wave climate direction could result in a 3-7% variation in longshore energy and confirmed that the beach was significantly more sensitive to this factor than most other south coast locations, as might be expected of a swash aligned coastline.

Wave energy was also found to be especially sensitive to sea-level rise. These results accord with those of Brampton (1993) who noted that net drift at West Bay was extremely variable in direction and was highly sensitive to small changes in the directional wave climate. This phenomenon was studied in further detail by Halcrow Maritime et al (2001) who undertook modelling of likely future wind speeds for a climate change scenario representing 2080 using the Met Office Hadley Centre Regional Climate Model. The wind speeds output by the model were used to derive offshore extreme wave conditions in Lyme Bay and results demonstrated a potential for significant increases in wave energy e.g. 1 in 50 year wave height of 8.1m could increase to 11.3m by 2080s.

The hindcast waves have also been used to study the potential changes in alongshore sediment transport. Results for the east end of Chesil Beach indicate a potential for a dramatic shift in the sediment transport regime due to a small two-degree shift in the mean wave approach direction. The current net westward drift potential of $900m^3a^{-1}$ would under this scenario alter to a net westward drift of up to $15,000m^3a^{-1}$. A similar type of study was undertaken by Sutherland and Wolf (2002) who simulated drift on Chesil Beach up to the year 2075. Results suggested that net drift could in future increase by up to 30% due to the potential effects of climate change. Results of the two studies differed due to use of different climate model outputs and application of alternative hydrodynamic and sediment transport models and calibrations;

however, they both suggest that (i) waves in Lyme Bay are likely to vary with future climate change and (ii) Chesil Beach is likely to be sensitive to these changes.

The shoreline is exposed to modest storm surges that travel up the English Channel. Analyses of historical tide gauge records for Portland and Devonport with appropriate adjustments for local tidal levels reveal extreme 1 in 100 year sea-levels of 2.72mOD for Chesil at Chiswell (Babtie Group 1997; Posford Duvivier 1998) and 3.08mOD for West Bay (HR Wallingford, 2000a). Table 5.5 presents water level data in tabular form.

Table 5.5 Tidal levels (Admiralty Tide Tables, 1998 from Posford Duvivier, June, 1998) and extreme still water levels for Chesil Beach (mOD) (Dixon and Tawn, 1995 from Posford Duvivier, June 1998)

	Tidal level (mOD)
MHWS	+1.8
MHWN	+0.8
MLWN	-0.4
MLWS	-1.3

Return interval (yrs)	Extreme still water level (mOD)
1	**2.1**
10	**2.21**
25	**2.26**
50	**2.29**
100	**2.35**
200	**2.41**

5.4.7 Evolution

A synthesis of measurements to fixed objects, beach surveys by Coode (1953), Carr (1969; 1990) and Babtie Group (1993), has been compiled by Bray (2005) to determine the decadal scale evolution of Chesil. This is illustrated in Figure 5.12. The average long term (decadal-scale) recession rate is 0.1-0.2 per year, although this varies along the length of the beach.

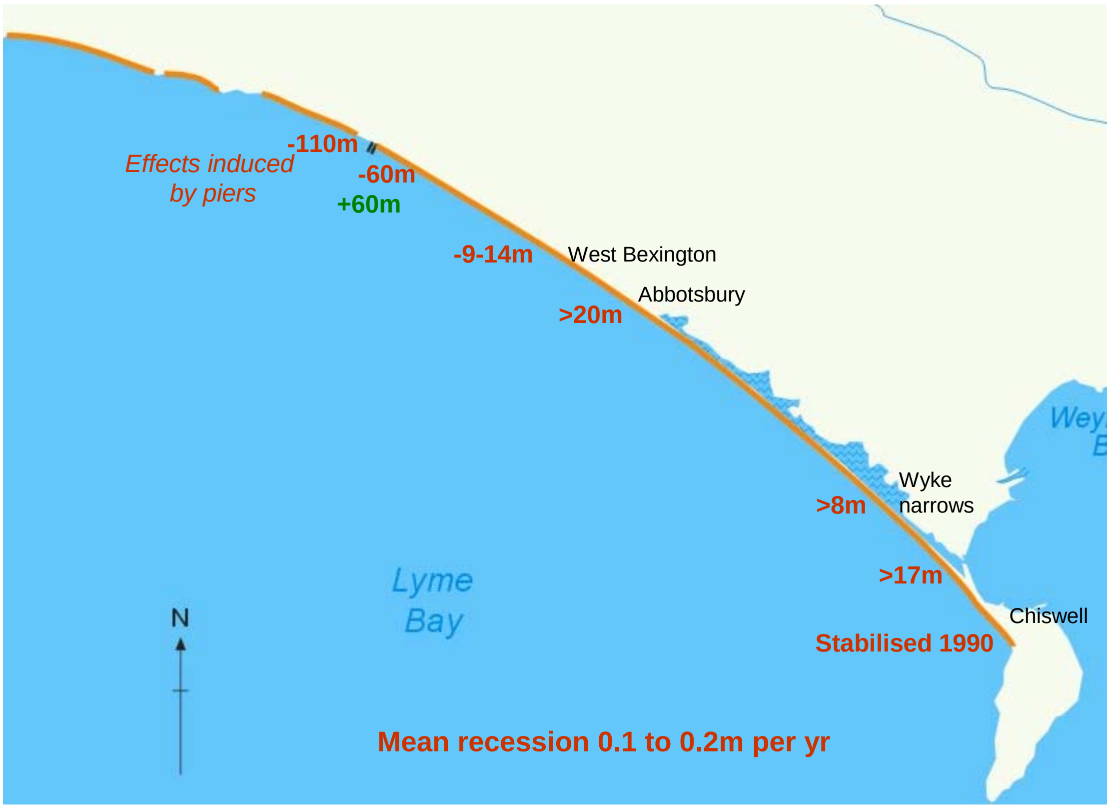


Figure 5.12 Typical recession rates from 1900-1990 (Bray, 2005)

Recession rates are variable along the length of the beach and are discussed by Carr and Gleason (1972) and Carr (1981). The narrowest stretch of the beach, opposite Portland Harbour, shows 17m recession from the 1850s to 1968/9. The retreats elsewhere are too small to be determined because the surveys are not sufficiently accurate.

Carr (1981) suggests that localised retreat of the crest from opposite Portland Harbour to Chiswell in the storm of February 1979 was of the same magnitude as the long-term recession opposite Portland Harbour. The implication of this is that at the present rate of recession the Chesil Beach and the west cliffs of Portland would only have been 1.5 km offshore from the present coastline at the beginning of the Flandrian Transgression, 10,000 years ago. This does not seem to agree with the evidence that the whole English Channel was dry at the start of this rise in sea-level.

If the Isle of Portland was a hill not extending far to the west prior to the transgression then it is probable that the early transgression over the low ground was far more rapid. If Portland is, instead, the remains of a large limestone upland previously extending far to the west it is difficult to explain the situation. The beach and the limestone hill to which it was tied could not have retreated fast enough. All this is good evidence for the pre-Flandrian shaping.

5.4.8 Contemporary processes

Chesil beach appears to have been in dynamic equilibrium with sediment supply from the west until the two piers at West Bay were built in 1825. By 1866, the eastward drift had declined (Posford Duvivier, 1998). This, along with over 1.1 million m^3 of shingle extraction from Chesil beach since 1900, promoted a closed unstable system which became more vulnerable to wave attack (Bray, 1996; Posford Duvivier, 1998).

Crest lowering and landward migration are dominant processes operating on the barrier, with numerous flooding events, particularly along the eastern end (Jolliffe, 1979; Carr and Seaward, 1990, 1991; Posford Duvivier, 1998). Posford Duvivier (1998) note that prior to construction of the Portland Bill breakwaters Chesil beach was regularly breached by easterly waves from Weymouth Bay and repaired by south-westerly waves from Lyme Bay. Overtopping and overwashing events are infrequent but occur sufficiently regularly for overwash fans and other deposits to be modified.

There are several records of “unusual” swell waves that have affected eastern parts of the beach (Bray, 2005). An event of this type overtopped the beach crest in February 1979. Local wave data are not available, but an offshore significant wave height of 7m and a period of 18 seconds was recorded 120 miles off the Isles of Scilly (Draper and Bownass 1982). The crest of Chesil was overtopped and it can be postulated that this type of event may be a major factor in beach recession. A less extreme event of this type occurred on 8th March 2003 which reprofiled the seaward crest face, exposing consolidated substrata and bedrock clay (Moxom 2003). Other events affecting this coast are described by Dawson et al (2000) and may represent tsunami generated by seabed earthquakes or submarine landslides in the Atlantic. Several events are described in which high waves apparently arose out of an otherwise calm sea achieving heights of 2m to 9m with periods of up to 10minutes. Eastern parts of Chesil directly facing the northeast Atlantic are especially exposed to such waves.

Recent evidence of episodic overtopping events is provided by the presence of peat blocks thrown onto the crest of the beach; these have been eroded from the toe of the beach from underlying layers of the Fleet lagoon sediments (Plate 5.13).

Plate 5.13 Peat blocks on the crest eroded from the underlying solid geology (Bray)

Washover fans are in evidence along the rear face of Chesil Beach, spilling into Fleet. Parts of the beach are characterised by a series of seepage “Canns” (Plate 5.14). These features are driven primarily by a differential head of water across the beach cross section. Water infiltrates into and through the beach under pressure, as a result of surges or high differential water levels between the Fleet lagoon and the open sea. The head differential can result in sufficient pressure to form springs on the landward side; this seepage can be a major cause of flooding and reflects the highly permeable nature of the beach. In some cases the head of water can result in a geotechnical failure of the rear slope, flow of the sediment and formation of *“canns”* or seepage amphitheatres in the back of beach. There is some uncertainty why the Canns form in only selected parts of the beach, but this may be due to a combination of factors including the internal structure of the beach and the beach cross section.

Plate 5.14 Active seepage Cann 5 November 2005 (Bray)

5.4.9 Management

Intervention

Management practice varies over the length of the barrier, although a high proportion of the beach is essentially unmanaged, with the exception of nature conservation and recreational based wardening activities. Flood defence management activities are focused at the eastern end of the barrier, where the crest is highest and where the beach fringes on developed areas. Several management schemes have been implemented between Chiswell and the eastern extent of the barrier.

The recommended management strategy for Chesil beach is to selectively hold the line in the short and medium term, as it is economically viable to protect Chiswell from flooding (Posford Duvivier, 1998).

The low-lying land behind the eastern end of Chesil Beach contains the settlement of Chiswell, which has a history of being flooded. The first recorded incident occurred in November 1824 when 26 people were drowned and some 80 properties were destroyed.

In 1852 during a heavy southwesterly gale the face of Chesil was severely eroded and John Coode estimated volume removed to be of the order of 4,500,000 tons of shingle (Minikin, 1952). Within a few days the loss was almost fully restored.

On 13 December 1978 the eastern end of Chesil was hit by 12s period waves, which eroded so much of the front face that the crest was also lowered and the back face also eroded. The lowered crest lead to further overtopping and crest lowering, leading to 1.2m depth of flood-water at Chiswell. In January 1979, 18s swell waves, in combination with strong onshore winds, gave rise to surge conditions. This led to severe overtopping. As on the previous occasion some 30 properties were flooded and the Isle of Portland was cut off from the mainland. These events led to the implementation of the Chesil Sea Defence Scheme in 1981/2.

Following the flooding in 1978 and 1979 a scheme was implemented to prevent further such incidents. The scheme consisted of:

Gabion mattresses to protect the crest of Chesil Beach
Modifications to the adjoining seawall to increase its height and to extend the toe to prevent undermining
An interceptor drain to carry flood-water from behind the beach to empty into Portland Harbour
Raising the crest of the road to 3mOD, so as to prevent it from being flooded.

Because of the environmental importance of Chesil Beach, the Institute of Oceanographic Sciences reported on the environmental impact of the proposed works. This report included the observations by the Nature Conservancy Council. It was argued that although the scientific interests would be best maintained by an option of “Least Intervention”, it could be argued that some defence work would be justifiable on the grounds of safeguarding Chesil Beach itself. It was accepted that the interceptor drain would have some impact, but that this could be limited if the drain were located as far from the beach crest as possible.

The intention of the three layered gabion mattress design is to maintain an adequately high beach crest for the most vulnerable, eastern end of the frontage, where the beach adjoins the seawall. By using a flexible mattress design some beach fluctuations can be accommodated.

Monitoring showed that up to 1987 no significant movement of the mattresses had occurred, but there had been some corrosion of the basket wires.

On 16 December 1989 this end of Chesil Beach was overtopped by 18s period waves coinciding with a 1m surge. This storm caused a lowering of the crest of the beach to the north of the gabion mattresses, as well as considerable damage to the toes of the mattresses themselves. Despite the overtopping only 8 properties were damaged, the interceptor drain flowed full, and it was considered that all components of the scheme successfully performed their function. Since then the Chesil Sea Defence Scheme has continued to perform satisfactorily, although the bank has not been subject to the intensity of the earlier storms.

Monitoring

Historically, monitoring of Chesil Beach has been on an uncoordinated and irregular basis. Limited data is available to describe beach evolutionary processes. Beach profiles have been captured at a few locations photogrammetric profiling was undertaken several times during the 1990s and a LIDAR survey was conducted in 1996. The most significant measurement programme however, is the internal flow monitoring conducted by the EA. A series of instrumented boreholes across the beach are used to provide data to assist with the prediction of percolation flooding. There is recognition by the EA that the picture is incomplete however, since there is a lack of complementary data on forcing conditions such as wave climate and tidal elevations.

The lack of a cohesive monitoring programme probably reflects the practical difficulties associated with monitoring a large-scale site such as Chesil Beach, which has poor access along most of its length. Plans to implement a cohesive monitoring programme, in conjunction with the Southwest Regional Coastal Monitoring programme are now in place; this programme has recently commenced and will provide the following:

Annual 1m resolution LIDAR surveys of the entire beach
Permanent control locations, surveyed by static GPS observations
Production of 10cm resolution digital orthophotos based on low level aerial surveys, every two years.
Baseline mapping of ecology to a level suitable to inform the extent of designated habitats for biodiversity action plans
Biannual beach profiles at key locations, where access is reasonable.
Post storm beach profiles at several key locations (to be taken approximately once per year).
Directional waverider buoys in shallow water, off Chiswell and West Bay
Tide gauge at West Bay

Issues

The barrier provides a natural defence line to protection of properties from flooding at several locations, most notably at Chiswell. Storm damage is well documented here, notably during storm events of 1978 and 1979 (Plate 5.15), when severe overwashing, percolation and flooding occurred.

Plate 5.15 Overwashing at Chiswell during 1979 storm

Chesil Beach is considered by the EA as the highest risk site for flooding in the South Wessex region in terms of both severity of flooding and speed of flood development. It is also noted in this context that the highest risk occurs where there is least confidence in the defences. Management concerns are based upon decision making that is currently based on non-proven scientific models. In particular, the risk time window is uncertain; therefore management is cautious and possibly not efficient or cost effective. Flooding can occur very rapidly with little warning and this lack of warning is of particular concern. The risks are compounded by the fact that maintaining a defence of a constantly varying defence standard is complex and presents considerable uncertainty.

A key risk is of water percolation through the beach, sometimes in combination with overtopping or overwashing, which has resulted in flooding of property. Overtopping and overwashing events are infrequent, but their impacts are serious.

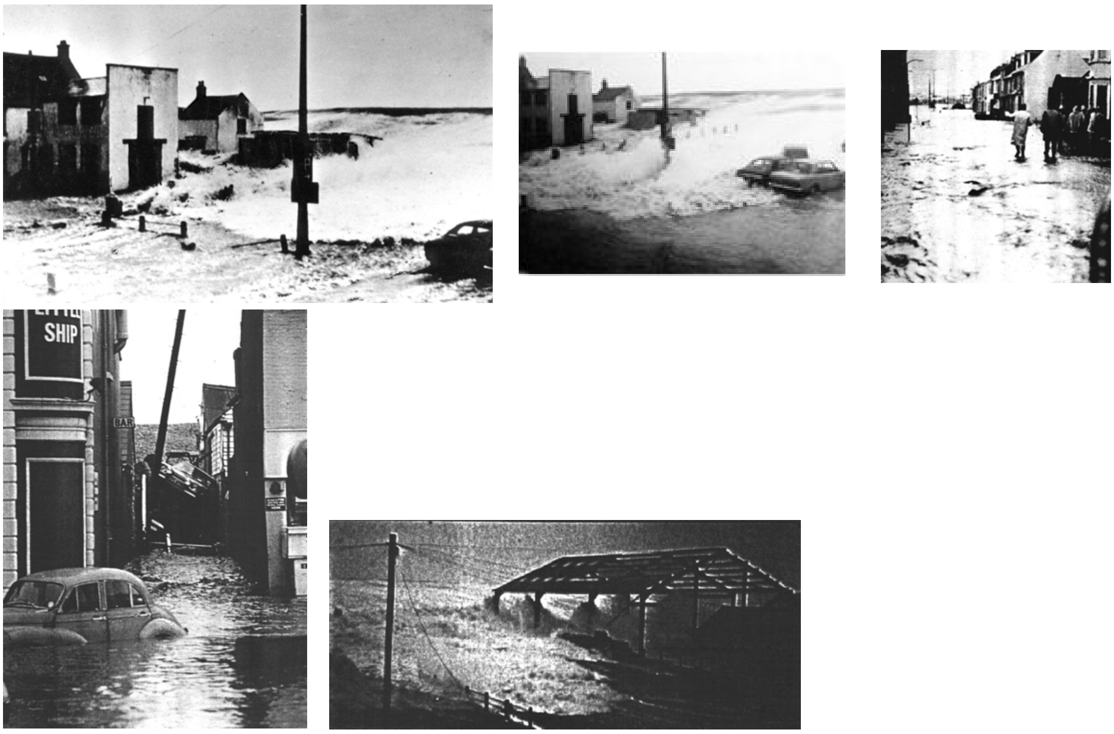


Plate 5.16 Inundation caused by percolation and over-topping of Chesil Beach

The 1978 event (Plate 5.16), which induced percolation through the Beach and overtopping, has an estimated return period of five years (storm surge flood). The February 1979 event, which caused seepage and overtopping from long-period swell waves, has an estimated return period of 50 years.

Long period waves are considered to provide the most damaging conditions, with respect to both overtopping and percolation events. There is no measured evidence of the magnitude of these events although there are several consistent sources of anecdotal evidence that suggest that long period swell waves are highly significant in these events. No guidance appears to be available to assist with the prediction of beach response, overtopping or percolation volumes in combination with these events.

A low key and largely ineffective flood forecasting system is in place, based upon the storm tide forecasting service. This is considered by the EA asset management team to be inadequate, as it lacks reliable input of local wave and weather conditions, using only data in deep water and distant from the site. No guidance is provided on current or predicted severity of wave conditions (period, height and direction) close to the site. There is similarly no tool in place to enable morphodynamics responses of the beach or flooding potential, either by overtopping or percolation, to be predicted.

The current limitations of the existing flood warning system are highlighted:

Improved management and knowledge of the flood risk window is required

Poor wave forecasting is available –no local forecasts, measured data or modelling are integrated. STFS is inadequate as there are no local wave-conditions provided
There is a need for real time measured and predicted wave data
There is a need to maintain and manage beach groundwater records to develop an empirical response framework linked to forcing conditions
There is no flow monitoring from the interceptor drain
The monitoring system is segmented and the various elements are not linked together

There is an immediate need to improve flood warning at this site since there is currently no programme that enables linkage of wave and water level conditions with beach response or flooding potential and there is also a high risk of flooding during extreme water level and wave events. The forecasting of nearshore wave and water level conditions can be achieved by implementation of existing technology (Tozer *et al.*, 2005), but prediction of overtopping or percolation volumes and the morphodynamic response of the beach requires development of new predictive techniques.

The barrier beach at East Beach, West Bay, which is effectively the western limit of Chesil Beach, presents similar management problems. Current management includes the following:

Reprofiling of the beach with mechanical plant and addition of material from stockpiles
Recycling from Freshwater to east beach

Intervention is currently determined using the following criteria

Maintenance of the crest height and width at 7.5mOD
Values are based upon Monte-Carlo simulations to determine fragility curves, based on model output\
Maintain variable standard of defence
Intervention cannot be undertaken during the storm event

Issues of concern at this site highlighted by the EA include the following.

What is the beach doing during the storm?
How can reference points be determined to identify when critical conditions have been reached and can these be captured using CCTV?

5.5 Slapton Sands

Location	Start Bay, Devon (see Figure 5.13)
Designation	SSSI, NNR, GCR site, AONB, Heritage Coast
Length	5.6km
Width	100m – 140m
Crest	3.5m to 4.0m above MHWS (6.0mOD ±0.5m)
Sediment	Shingle; d_{50} = 2mm – 9mm. Coarser in the south, reversible.
Tides	Neap range 2m; Spring range 5m
Waves	SW (modified by Skerries Bank) to NE.
Classification	Swash-aligned straight attached barrier beach.
Intervention	Drainage, seawall, armour-flex mattressing, recycled shingle "bastions"

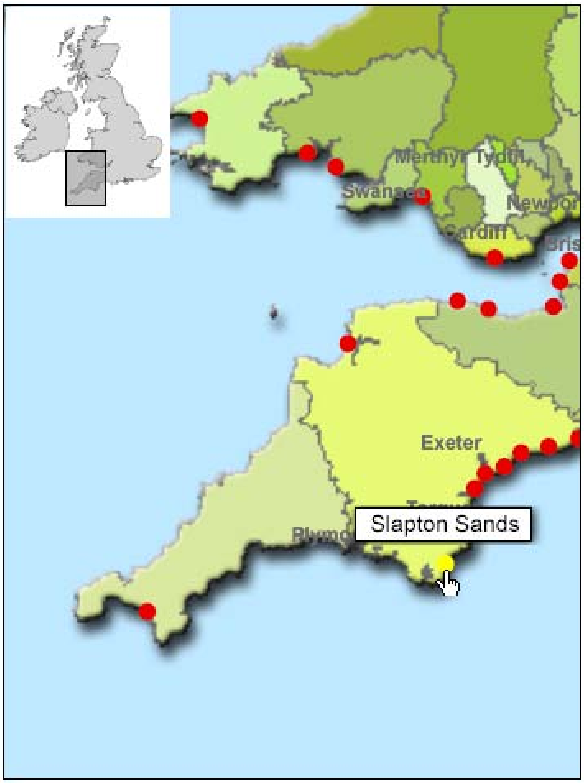


Figure 5.13 Location of Slapton Sands

5.5.1 Introduction

Slapton Sands is a 5.6km long shingle barrier beach stretching from Limpet Rocks (south of Torcross) in the south to Shiphill Rock at Strete in the north. The beach from Torcross to Strete Gate (Plate 5.17) is about 3.5km in length. It encloses an extensive freshwater lagoon called Slapton Ley. The lagoon is part of a nature reserve, has been designated an SSSI, and is an important winter roost for wildfowl.

On the ridge is a road, the A379, which connects the villages of Torcross and Strete. The northern part of the barrier beach is relatively undeveloped and has been able to respond freely to hydrodynamic changes, changing its alignment with changes in wave approach. In the recent past it has tended to accrete, due to northward littoral drift. The southern part has tended to erode. The shingle ridge at Torcross is backed by a seawall that was constructed in 1980 following the damage to earlier defences in the winter of 1978. Torcross and Beesands (a smaller, similar barrier beach to the south), have been provided with rock armouring to minimise overtopping.

Plate 5.17 Slapton Sands and Slapton Ley looking north towards Strete Gate from Torcross (Dave Mitchell)

5.5.2 Geomorphological context

Although named Slapton Sands, the ridge is composed of shingle. It is unlikely that much of the shingle, composed of flint, chert and quartz, is locally derived. It is thought that much of the material has been eroded from past coastlines which have been submerged as a result of rapid sea-level rise during the Holocene period. It is thought that sea-level has risen by 18m in the past 9000 years, with a notable deceleration of the rate of rise during the past 5000 years (Chadwick et al., 2005). During the early Holocene, Skerries Bank (a 6km nearshore feature sheltering Slapton Sands from south-westerly wave conditions) sheltered a transgressional shoreline where salt-marsh, estuaries and lagoons formed. Skerries Bank took up its present day position towards the late Holocene transgression as the rate of sea-level rise slowed (Hails, 1975a and 1975b).

During the period of reduced rate of sea-level rise (from 5000BP), it is probable that barriers have been formed, broken down and submerged in the lee of Skerries Bank, with only limited quantities of beach material transported landwards from Start Bay (Hails, 1975a). During this time, it is thought that the shoreline was close to its present day position (Morey, 1983). Hails (1975a) and FUTURECOAST (2002) consider Start Bay to have become a closed system, with no new material entering the system from alongshore or offshore.

The geology inland of Slapton Sands consists of grits and slates of the Meadfoot beds, with the bedrock being mainly of mid-Devonian age, aligned east-west.

At present the barrier is trying to roll-back, but is constrained by the presence of the A379 road. As a result, the beach profile is steepening, and the beach crest is rising more rapidly than it might otherwise.

5.5.3 Flood defence / coast protection role

Slapton Sands barrier beach protects the freshwater lagoon Slapton Ley, and forms part of the associated SSSI. Breaching of the barrier would not constitute a breach of the SSSI designation, and as a result, Natural England is unlikely to consider breaching a problem.

The village of Torcross, at the southern end of the Ley, is built partly on the barrier as is the A379 road. As such the barrier, together with some hard-defences, offers coastal protection and flood defence to various elements of infrastructure.

5.5.4 Geometry

Close to the barrier beach, the seabed geometry varies, but levels of about -15mOD are present by 600m offshore (May, 2003). Between about 500m and 3km offshore, according to Kelland and Hails (1972), a pronounced change of seabed (bedrock) slope at about -40mOD indicates the presence of an ancient coastline which would have been exposed during the Pleistocene Epoch when sea levels were lower. Offshore from the Slapton Ley, there is a second abrupt change to the seabed slope at about -28mOD.

Skerries Bank, aligned SW-NE, which is attached to Start Point (the southernmost headland in Start Bay), is approximately 6.5km long (Figure 5.14). The Bank offers protection from the majority of wave conditions apart from those propagating from north of east at Slapton Sands, with the crest of the bank being as high as -4.8mOD (corresponding to as little as 1.5m of water at some low tides). Skerries Bank strongly influences nearshore refraction patterns, and is thought to concentrate wave energy towards the southern portion of Start Bay under certain wave conditions.

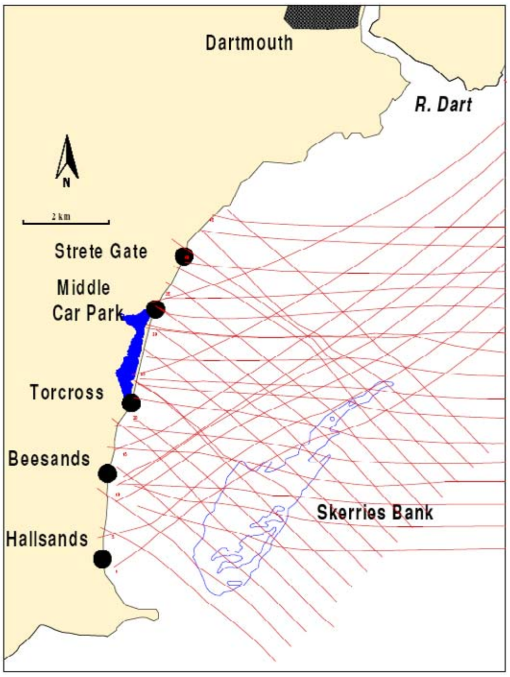


Figure 5.14 Influence of Skerries Bank on wave orthogonals (Ian Stevenson)

The crest elevation of Slapton Sands varies is approximately 6.0mOD ±0.5m. Lawrence (*pers. comm.*) states that in recent decades, the crest level has risen by 450mm.

Figure 5.15 below shows a typical cross-section of the barrier and the Ley which it contains. A 1:1yr wave condition is estimated to be of the order of H_s=3.0m (inferred from Chadwick et al. (2005)).

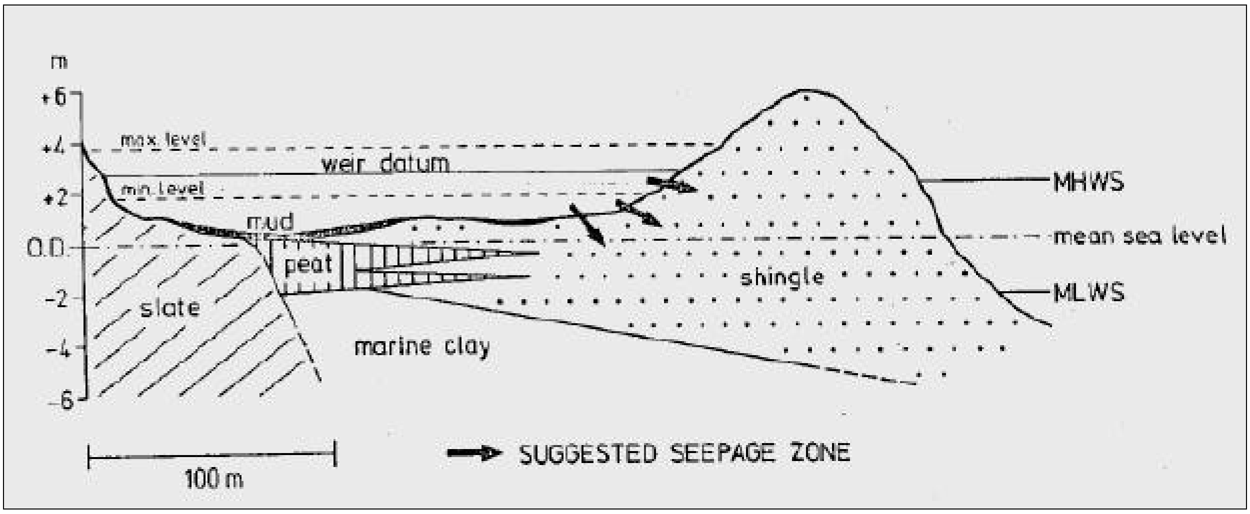


Figure 5.15 Typical cross-section of Slapton Sands and Slapton Ley (www.saveslaptoncoastroad.co.uk)

5.5.5 Evolution

Average recession rate

The long-term recession rate of the barrier beach face has been calculated by Pethick (2001) to be 1.15m/yr. This theoretical value is higher than that determined from measurements (1972-1995), which suggest that the recession rate over the entire length is 0.8m/yr, reaching 1.2m/yr towards the centre of the barrier (Pethick, 2001).

Back barrier

Washover fans are in evidence along the rear face of the barrier at Slapton sands, though less developed than others seen around the world (Pethick, 2001). The long-term transgression rate of the barrier has been estimated at 0.2m/yr (Pethick, 2001). This rate is markedly lower than the observed retreat of the seaward barrier face.

Short term (storm event response)

Individual extreme events (extreme combinations of wave height and water level) are responsible for episodic erosion of the barrier crest. Some defence works have been introduced as a result of storm events, which may induce several metres of beach crest erosion. It is possible (Lawrence, pers. comm.) that there is only weak grading of sediment size in the cross-shore direction. This might imply a more strongly reflective, steeper beach face which is prone to draw-down during storm events.

5.5.6 Management

Intervention

A culvert was built in 1856 at Torcross to drain the Ley (which also drains via seepage through the shingle barrier). This culvert has prevented breaching of the kind experienced in 1824, which is thought to have been caused by high winter freshwater levels in the Ley. Evidence from core samples taken in 1999 and 2001 (Chadwick et al., 2005) suggests that the barrier has breached from the sea in the past (Atkins Consultants Limited, 2003).

Following storm action in 1918, a seawall was constructed at Torcross. Later in the 20th Century, such as during the storm of January 4th 1979, significant overtopping of the wall and the barrier was observed (Plate 5.18). The present-day wave-return seawall was built in 1980 as a result of damage incurred by the original seawall during the 1979 storm.

Plate 5.18 Over-topping at Torcross during the 1979 storm

During the early-nineties, armour-flex concrete-block mattressing was installed approximately mid-way along the Sands to protect the WWII War Memorial (subsequently re-located during 1995 following undermining) and car park. By the late 1990s, the extent of the managing authority's management plan was simply to repair the damaged armour-flex units as they were considered unsafe.

Over the winter of 2000/2001, a series of storms caused localised erosion of the barrier leading to significant damage of a 250m length of the A379 road. As a standard emergency response by Devon County Council, 2000m^3 of rock armour was placed at the damaged crest. Following realignment and repair of the A379, the rock armour units were removed and relocated at Beesands to form part of the sea defences there.

At this time the Slapton Line Technical Group (The Slapton Line Partnership) was formed. This Group consisted: Devon County Council, South Hams District Council (SHDC) and Natural England with input from the Department for Environment, Food and Rural Affairs (DEFRA), the Environment Agency, the Whitley Wildlife Trust and the Slapton Ley Field Centre.

The Group together with their consultants, Scott Wilson, concluded that the placement of several shingle bastions, originally 12,000 tonnes (Plate 5.19), in the vicinity of the damaged crest would satisfy a SSSI status of the Ley, and would also serve to protect the infrastructure. Five such bastions have been constructed to date. These bastions were constructed from beach material mined from the upper foreshore at the Strete Gate end of the Sands. A five-year agreement to this management option has been made with Strete Estate (local land-owner), on condition of beach profile monitoring being conducted in order to identify any adverse impact which may not be anticipated.

Plate 5.19 Shingle bastion placement at Slapton Sands, 2002 (Ian Stevenson)

Monitoring

Monitoring of the barrier to some dedicated degree or another has been ongoing since about 1975. The Slapton Ley Field Centre has been taking beach profiles for about 30 years. The Field Centre is a tenant of the landowners, Whitley Wildlife Trust. As such, the Field Centre data is not part of the managing authority's (SHDC) dataset, although it is understood that access to the data is possible upon request.

In addition to the Field Centre data, SHDC has beach survey data dating back to 1995, with concentration of effort at the Strete end in recent years.

A report by Scott Wilson (2004), which was jointly funded by DEFRA and SHDC, is the first bank of knowledge that has become available specifically to the managing authority.

Wave monitoring was attempted during the 1990's, however the wave buoy was lost in a storm after a short period of recording and has not been replaced.

It is understood that SHDC has been consulted with regard monitoring needs as part of the Channel Coastal Observatory South West programme.

Issues

Until the storm of 2001, which damaged the A379, it appears that there have been no significant management issues relating to Slapton Sands in the recent past. The SSSI status of the Ley, together with the Outstanding Natural Beauty of the area, and the extensive beach facility has ensured a thriving eco-tourism industry in the area.

During the 1990's, it became apparent that the long-term health of the beach was deteriorating, as the WWII War Memorial became undermined and erosion events were gradually narrowing the crest.

Towards the end of the 1990's, a Technical Group (The Slapton Line Partnership (http://www.slaptonlinepartnership.co.uk) was established. The aim of the Group, which incorporated key stakeholders, was to address the issue of a sustainable management strategy for the gradually deteriorating barrier. At the time of the Group's formation, the ongoing management strategy was no more than repairs to block mattressing to bring their condition in to line with Health & Safety requirements.

In late 2000 and early 2001, a series of storms caused significant damage to the A379. Emergency response by Devon County Council was to protect the crest with rock armour. This action contravened the SSSI designation, and the rock was subsequently removed following repair and realignment of the damaged road.

Consultants were engaged to scope the situation, and the option of abandoning the road in favour of alternative routes further inland, amongst others, was considered. A consultation of stakeholders was conducted, and a campaign was started by local businesses and residents to "Save Slapton Coast Road (http://www.saveslaptoncoastroad.co.uk)" through fear of losing the road entirely.

Following the scoping study, a main study was commissioned. An outcome of the main study was that it might be beneficial to place some shingle bastions (sacrificial groynes) along the beach where the erosion was occurring. There was no guidance on this method available to the managing authority, and the exercise was treated as purely experimental. Natural England, as key stakeholders, conceded to this level of intervention as it was not considered to impact on the SSSI status.

The entire investigation and negotiation process, conducted in the main through the Council's coastal engineer was arduous and, at times, heated. There is still tension and uncertainty in the minds of the local business men and women and amongst the residents, and hence amongst the managing authority, with regard the future of the coastal road. This is more than 5 years after the damage occurred.

It seems fortunate that the formation of the Technical Group occurred prior to the storm of early 2001, as this placed all the key stakeholders on the same panel in a situation where the aim was to work together to achieve a result. When the damage to the road occurred, the Group was already in communication and on good terms. This could only have served to ease the negotiating process.

In July 2006, SHDC submitted an application to DEFRA for funding to install a project manager who would deal solely with the needs and concerns of local businesses. The project manager's task would be centred on adaptation plans for local businesses when the coastal road ultimately closes.

5.6 Hurst Spit

Location	Christchurch Bay, Hampshire (see Figure 5.16)
Designation	SSSI, Ramsar site, SAC
Length	3.5km
Width	150-200m
Crest	4m-6.5mOD
Sediment	Mixed sand and shingle d_{50} 12mm-16mm
Tides	2.2m range on spring tides
Waves	See Figure 5.21
Classification	Spit
Intervention	Crest reformation, rock armour, groyne, recharge, recycling, breakwater

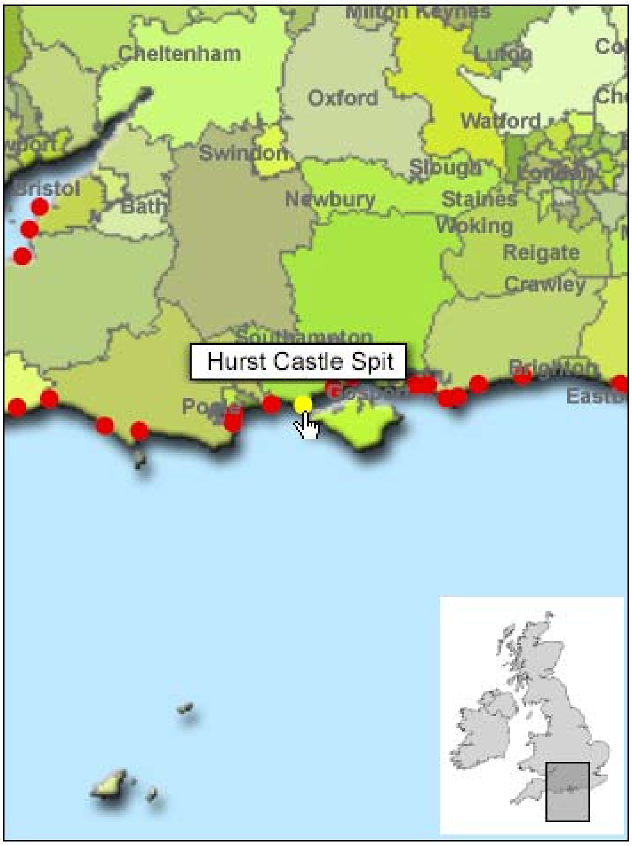


Figure 5.16 Location of Hurst Spit

5.6.1 Introduction

Hurst Spit (Plate 5.20) is located in Christchurch Bay, a shallow coastal embayment bounded by Hengistbury head to the west and by the Needles promontory to the east. The bathymetry of the embayment is dominated by the bedrock outcrop of the Christchurch ledge to the west and by the offshore sand and shingle banks system, which extends across the whole of the embayment. Hurst Spit has evolved as a barrier spit since the formation of the embayment, at the entrance to the Western Solent, and provides protection from wave attack to an extensive area of low lying land in the western Solent. The Spit is composed largely of shingle and is approximately 3.5km long.

Plate 5.20 Hurst Spit

5.6.2 Geomorphological context

Hurst Spit has probably receded relatively uniformly over the past 4000-5000 years since sea-level approached its present position (Nicholls and Webber 1987a). Due to the complexity of controlling factors (and the fact that it is not a sink), it is particularly sensitive to change. Its response has been to vary its rate of recession, with periods of more rapid retreat being associated with phases of diminished sediment supply and high magnitude, low frequency storm surges (Nicholls 1985; Bradbury 1998).

As Christchurch Bay was opened out during the mid Holocene due to sea-level transgression, dominant south-westerly waves drove sediment both onshore and alongshore in a west to east direction. A substantial proportion was gravel, derived from the erosion of Pleistocene river terrace deposits originally deposited by the Solent River (West, 1980; Nicholls, 1987).

An ancestral form of Hurst Spit developed following the creation of the entrance to the Western Solent circa 7000 to 6,500 years BP (Nicholls and Webber, 1987a; Nicholls, 1987; Velegrakis, et al, 1999). With continuing sea-level rise, both updrift cliff recession and offshore sea bed erosion released large quantities of coarse sediment that created the Shingles Bank. This provided a large store of material which, together with a rate of littoral drift up to five to seven times what it is at present (Nicholls, 1985) created a substantial barrier spit. Low wave energy conditions to its lee promoted mudflat and salt-marsh accretion. Nicholls and Clarke (1986) have described truncated sequence of estuarine muds and peat deposits that outcrops seawards of the modern beach face, indicating that spit recession was in progress at least by approximately 4,500 years BP. This process is presumed to have been continuous (in a time-averaged sense) since then.

The progressive south-eastwards growth of Hurst Spit appears to have been episodic, or phased, as indicated by the three main "fossil" recurves (Bray, 2005). None have yet been precisely dated, but each must represent a stage of temporary equilibrium between sediment supply and loss. As now, loss of sediment at the distal end would have been due to a combination of wave action and tidal currents; waves propagating into the Western Solent, and refracted by the Shingles Bank, would have caused distal curvature and powered south to north littoral drift.

Tidal currents would have increased in velocity and capacity to transport sand and fine to medium gravel, due to narrowing of the entrance channel between the Isle of Wight coastline and the Spit terminus. The dominant pathway of gravel transport then, as now, would have been towards the Shingles Bank, thereby establishing a sediment circulation that sustained growth. (King and McCullough, 1971; Nicholls and Webber, 1987a; Velegrakis, 1994).

The ultimate position of the distal point - Hurst Point - may not have been achieved until the historical period and it was determined by the presence of a steep, tidally-scoured slope at the beach toe that prevented any further accretion. This stability is evident form the fact that the modern distal recurve is substantially larger than its predecessors, and that the latter increase in size with decreasing age. There may, however, be additional explanations for earlier phases of distal recurvature, such as short-term sea-level standstills; "pulses" of gravel supply from submerged sources or differences in wave climate. Alternatively, major storms might have caused barrier breakdown over the proximal sector, introducing large quantities of sediment into both the longshore and onshore transport pathways feeding the distal end. This would imply that forcing conditions and morphodynamic response were similar to those prevailing today (Bradbury, 1998).

5.6.3 Sediment Composition

The dominant constituent is sub-angular to sub-rounded flint gravel, with a mean diameter of 15mm (Bradbury, 1998). Median clast size diminishes slightly from west to east in the direction of wave energy reduction (Nicholls, 1985). Contemporary sediment character has been modified by several replenishments in the 1980s using gravel from inland sources that had different size and shape characteristics than the indigenous material. The 1996 stabilisation scheme introduced some 280,000m^3 of gravel dredged from the Shingles Bank. Limited evidence (Bradbury, 1998) suggests that the proportion of sand to gravel increases slightly with depth, but does not prevent infiltration and cross-barrier percolation (Nicholls, 1985). Both cross-shore and longshore grading are relatively poorly developed.

The structure of Hurst Spit is complicated by the varied sorting of the beach sediments which can vary from openwork shingle with no sand, through sandy openwork shingle with up to 20% sand and finally to sandy shingle which leaves only pore space within the sand component. The nature of overwash deposits often results in the deposition of a mixture of the finer and coarser sediments; consequently the permeability of the beach is reduced due to this layering effect. Similarly, beach recharge operations may influence the sorting of the beach.

5.6.4 Evolution

Human activity has drastically changed the natural coastal processes around Poole and Christchurch Bays since the late eighteenth century. In particular, the construction of coast protection and flood defence structures over the last 70 years has stopped the erosion of sand and gravel from the soft cliffs along much of this stretch of coastline. Consequently, the volume of shingle moving onto the Spit in the littoral drift has declined and, as a result, the Spit has decreased in size; a process, which has accelerated markedly since the 1940s when large scale groyne construction began at Bournemouth and Christchurch.

A synthesis of measurements of Ordnance survey mapping, has been compiled by Nicholls and Webber (1987) to determine the decadal scale evolution of Hurst beach, prior to intervention. This is illustrated in Figure 5.17. The spit is a transgressive feature, moving landwards, due to the processes of overtopping and overwashing. The rate of transgression increased from approximately 1.5m per year (1867-1968) to 3.5m per year (1968-1982) (Nicholls,1986). It is probable that recession occurred intermittently during this period, with phases of stability (egg 1890-1910) alternating with short-term retreat of several metres under occasional high magnitude wave conditions.

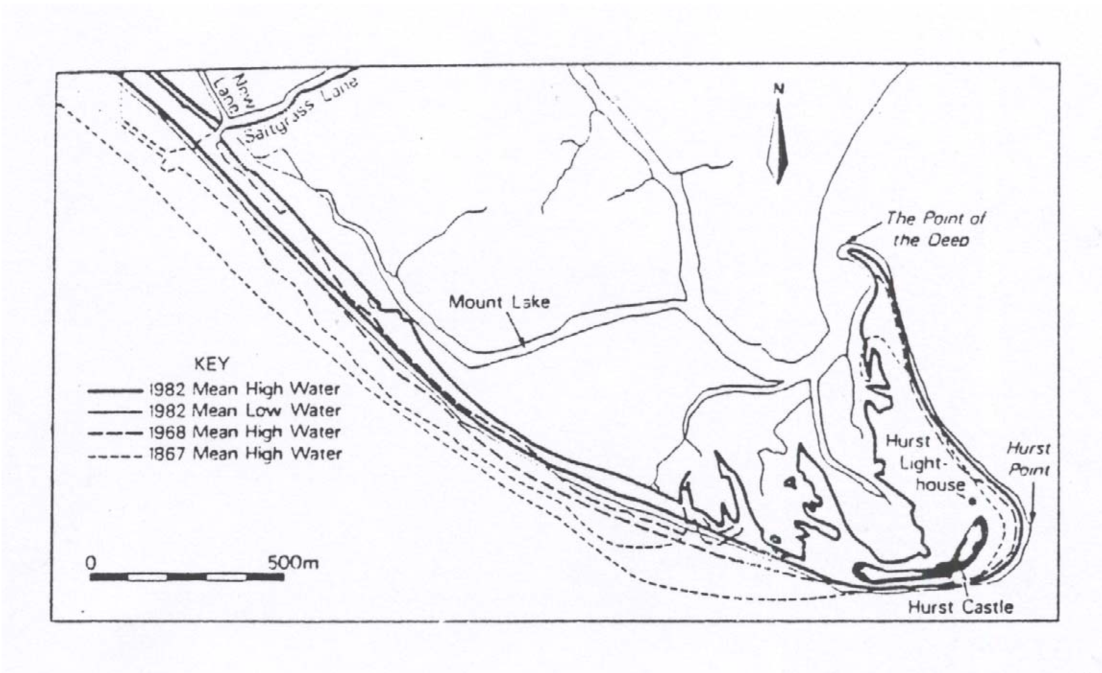


Figure 5.17 Barrier crest roll back between 1987-1993 (Nicholls and Webber, 1987)

Hurst Spit has been declining in volume, and its foreshore receding, since at least the late nineteenth century (Hooke and Riley, 1987). Steady volume loss is evident over at least the past 140 years. Volume loss was calculated by Nicholls (1985) to have been 1-2,000 m^3a^{-1}, 1965-1982; and 14,000 m^3a^{-1}, 1980-1982.

5.6.5 Geometry

The natural crest elevation prior to the 1996 beach recharge scheme was typically 2.4m-4.2mOD (Figure 5.18). The variability of the beach crest elevation reflects the variability of longshore wave climate and the influence of the offshore Shingles Banks, which has a marked impact on wave climate.

The back barrier geometry varies along the length of Hurst Spit, with the Mount Lake channel backing some 800m of the beach (Plate 5.20). The barrier is partially backed by salt-marsh at an elevation of about 0.5mOD and the shingle rests on a bed of salt-marsh deposits at an elevation of 0.2-0.5mOD. The beach drops into a 10-12m deep channel on the seaward side at a broadly similar gradient to that of the seaward face above low water mark. Seawards of the channel the seabed elevation rises again towards the North Head bank. The mobile bed shingle extends to a depth below low-water mark of 8-10m along most of the length.

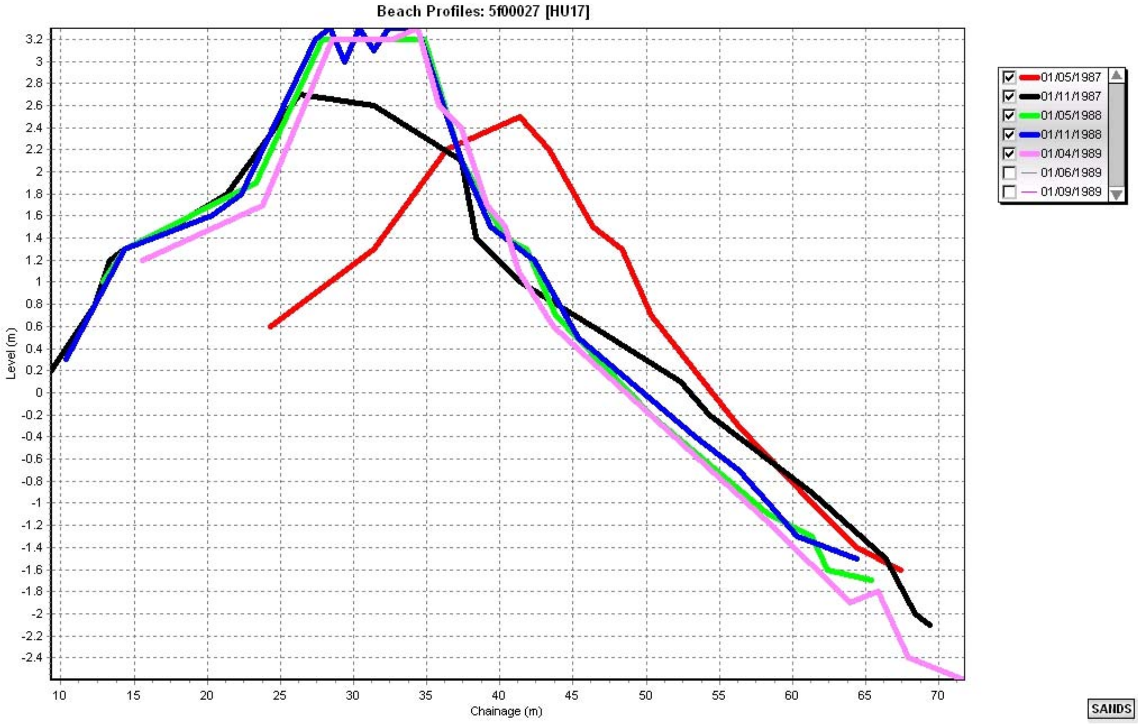


Figure 5.18 Typical pre recharge profile evolution

The beach crest elevation (Figure 5.19) is currently maintained artificially reducing from west to east. The maximum crest height is 6.3mOD and falling to 3.8mOD. The recharged beach is 33 –60m wide at MHWS, narrowing from west to east.

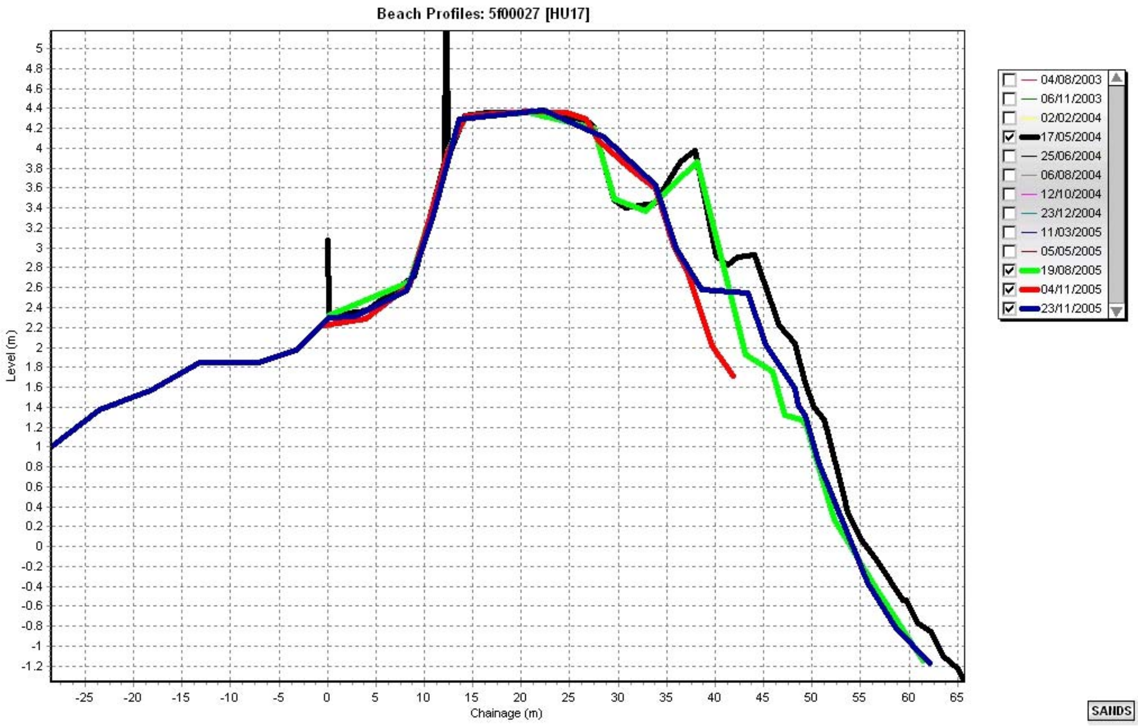


Figure 5.19 Typical post-recharge profile evolution

5.6.6 Storm event response – pre beach recharge

The main processes that have controlled the cross-profile form, and steady landward recession, are berm formation, over-washing and overtopping (Bradbury and Powell, 1992; Bradbury, 1998; 2000). The last two are associated with surge conditions and other factors creating high water levels and high energy breaking waves at or above crest level (Plate 5.21). Overwashing involves swash passing over the crest and then running down the backslope towards the back barrier salt-marsh. Crestal breaching occurs under conditions of overwash sluicing, which creates low crestal points that facilitate further overwash. It is almost certain that there have been numerous overtopping and overwashing events over the past 3-4,000 years of the history of Hurst Spit, but the first to be fully documented occurred in 1954.

(1a) Run-up

(1b) Throat confined overwashing

(1c) Cut-back breach

(1d) Sluicing overwash

Plate 5.21 Phases of shingle barrier evolution at Hurst Spit (Bradbury *et al.,* 2005)

The crest of Hurst Spit has been breached many times since 1954, notably in January 1962 when the recently constructed timber groynes were outflanked as the Spit rolled back during storms which caused widespread flooding in Milford and Keyhaven. The increasing frequency of storm damage and sharply rising maintenance costs since the 1970s indicate that a threshold of stability has been passed at this stage, and that the spit is no longer able to withstand even moderate storms and tidal surges without suffering severe damage. Sediment transport calculations confirm this, and indicate that the potential rate of loss from the Spit, is more than double the rate of shingle movement onto the Spit. A sudden increase in the frequency and magnitude of overtopping, overwashing, breaching and breakdown events occurred in the early 1980s. Crest lowering, breaching and rollback occurred in 1981/2; 1984/5; 1989/90 and 1994 (Dobbie and Partners, 1984; Wright and Bradbury, 1994; Wright, in Bray and Hooke, 1998; Mackintosh and Rainbow, 1995; Bradbury, 1998).

The storms of October and December 1989 caused dramatic crest lowering and roll back across the salt marshes, and outflanking of the rock armouring. Landward recession of 10-25m took place over a length of 2,300m on 29 October 1989. Crest lowering in excess of 2.5 metres, roll back of the seaward toe by up to 60 metres, and roll back of the lee toe by up to 80 metres resulted in displacement of more than 100,000 tonnes of shingle overnight in December 1989 (Plate 5.22). Tidal breaches formed for the first recorded time through the Spit in the December 1989 event, allowing water to flow through at all states of the tide and resulting in rapid erosion of the salt marsh in its lee. This recession exposed some 600m^2 of foreshore to erosion and exposed underlying saltmarsh deposits on the foreshore (Plate 5.23). The profile response during these events is shown in Figure 5.20.

Plate 5.22 Sluicing overwash December 1989

Plate 5.23 **Exposure of saltmarsh on the seaward side of Hurst Spit (December 1989)**

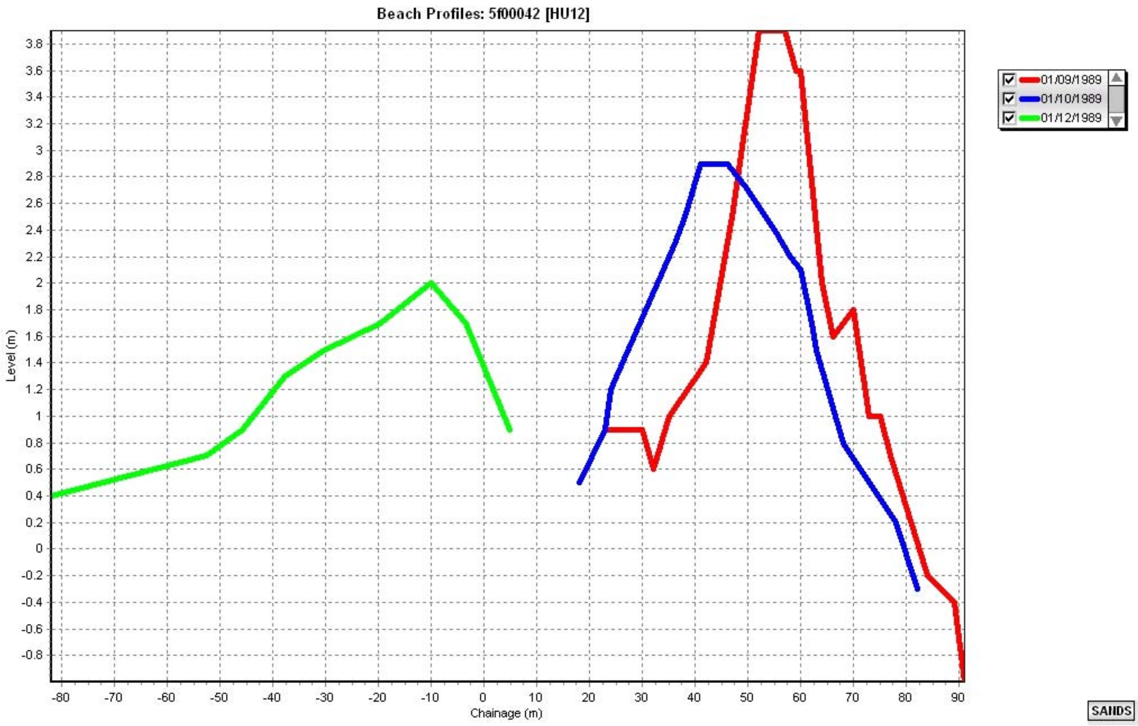


Figure 5.20 **Profile response during storms of October and December 1989**

As a consequence of its declining volume, Hurst Spit became subject to frequent over washing and crest lowering under storm action. Extensive throat and over wash fan systems have formed and the volume of the bank above low water has declined further due to the displacement of the shingle into the Mount Lake river channel, in the lee of the Spit (Plate 5.24). The overwash events which were once localised and were sufficiently small to permit the crest to reform grew in frequency and size to such an extent that the natural recovery was virtually impossible.

Plate 5.24 Displacement of overwash deposits into Mount Lake channel

A major storm was experienced on 1 April 1994, causing overwashing, elevation lowering, crest cut-back and fans extending up to 26m across the back-barrier slope (Bradbury, 1998). This event emphasised the urgent need for a comprehensive scheme of barrier management.

5.6.7 Wave and water level climate

An offshore wave climate was derived for Christchurch Bay using a hindcasting technique based on 15 years of Portland wind data and direct measurement from a buoy at Milford-on-Sea (Hydraulics Research 1989a and b). Prevailing direction was southwesterly and waves exceeding 1.0m were predicted for 31% of the time, and those exceeding 3.0m for 2.6% of the time. Extreme value analysis for waves of longest fetch (225-255°N) revealed offshore maximum significant wave height of 5.9m for a 1 year return period, and 7.2m for a 20 year return period.

The comprehensive scheme of protection and stabilisation of Hurst Spit, completed in 1996, generated specific studies of the variation of wave climate along its proximal to distal ends. These are set out, and analysed, in Bradbury (1998), who proposes a 1 in 100 year mean offshore significant wave height of 4.14m, approaching from 240°, for the proximal 800-1000m sector. For inshore waves, with an annual frequency, maximum significant wave height varies between 3.57m (240°) and 2.89m (210°).

Halcrow (1999) used models previously employed by HR Wallingford (1989a; 1993) but applied them to UK Meteorological Office synthetic wave data. For the distal end of the spit (Hurst Point and Castle), Bradbury (1998) determines that the 1 in 100 year mean significant wave height (240°) is 3.10m. For inshore conditions, extreme wave heights with a probability of recurring once a year are 2.10m (210°) and 2.68m (240°). There is a progressive north-west to south-east reduction in nearshore wave energy along Hurst Spit (Figure 5.21).

Taking into account all previous studies, Bradbury (1998) suggests that mean maximum nearshore wave height declines from 4.1m to 3.1m. The main reason for this is the attenuating/dissipating influence of Shingles Bank and North Bank, setting up complex refraction and wave train "crossover". Wave shoaling and breaking (at low water states) induced by the complex bathymetry of the banks and channels seawards of the distal sector reduces the height of offshore waves by almost precisely one third (Bradbury, 1998).

Contemporary monitoring (New Forest District Council, 1992; 1997-2001) reveals the impact of high magnitude, low frequency storm and tidal surges on water levels, and therefore water depths, on shoaling and refraction. This is apparent from analysis of the 1 in 100 year recurrence storms of October and December 1989 (Bradbury, 1998). The latter generated a higher water level, but prevailing inshore wave heights (2.90m) at the neck of Hurst Spit were lower than two months previously (3.60m). Monitoring of the frequency of maximum nearshore wave heights since 1996 (New Forest District Council, 1997-2001) will allow further analysis.

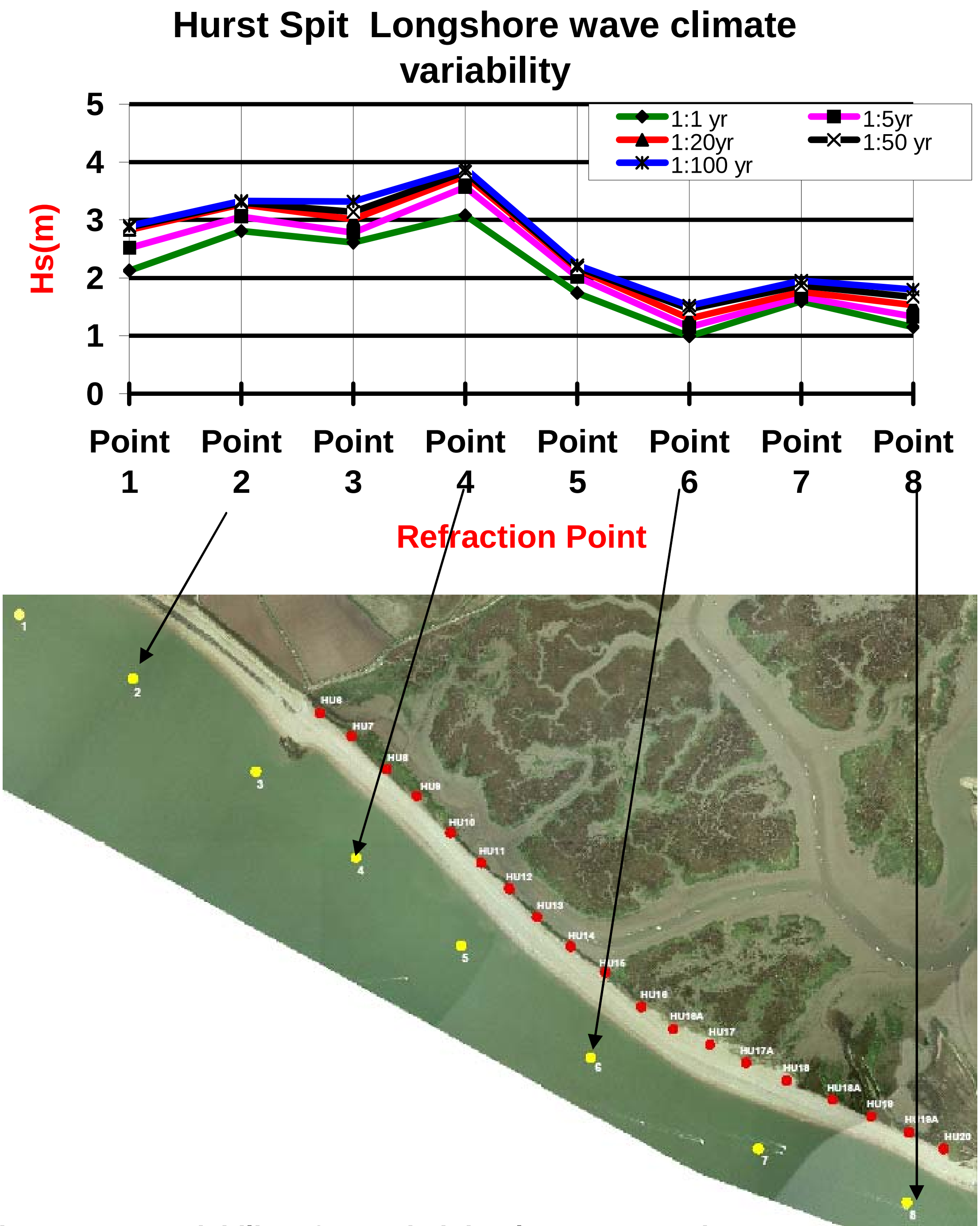


Figure 5.21 Variability of wave height along Hurst Spit

The tidal range of Christchurch Bay is the lowest along the south-central Channel coast. Tidal currents are rapid at Hurst Narrows (up to a maximum $3 ms^{-1}$ at the surface and $2.5\ ms^{-1}$ close to the seabed) and are capable of transporting coarse sediment.

5.6.8 Scientific significance / designations

Hurst Spit is of national and international importance for geomorphological features. It is designated as a SSSI for its morphological importance and is also within an SAC and Ramsar site. The beach management scheme (Bradbury, 1998; Bradbury and Kidd, 1998; Wright, in Bray and Hooke, 1998b) has maintained much of its morphodynamic character as a barrier structure. Vegetated shingle is associated with the 'fossil' recurves of the distal part of Hurst Spit. The historical recession of the backbarrier slope of the western and central sectors of Hurst beach has historically inhibited the establishment of vegetation, but this has begun to develop at some locations since the beach recharge in 1996.

The maintenance of Hurst spit is crucial to the continuing survival of the rich variety of habitats in the North-West Solent SAC (Keyhaven and Pennington Marshes) (Bray, 2005); this is one of several objectives justifying the Hurst Spit beach management project. There will, however, be some loss of both intertidal and terrestrial habitats if this barrier structure continues to evolve by transgressing landwards.

The SAC is part of the only estuary cluster site. The Solent and its inlets are unique in Britain and Europe for their hydrographic regime of four tides each day, and for the complexity of the marine and estuarine habitats present within the area. Sediment habitats within the estuaries include extensive estuarine flats, often with intertidal areas supporting eelgrass, algae, shingle spits, and natural shoreline transitions. Solent Maritime is the only site for smooth cord-grass Spartina alterniflora in the UK and is one of only two sites where significant amounts of small cord-grass S. maritima are found. It is also one of the few remaining sites for Townsend’s cord-grass S. x townsendii and holds extensive areas of common cord-grass Spartina anglica, Other features of significance fall in to the categories of annual vegetation of drift lines and perennial vegetation of stony banks.

5.6.9 Management

Intervention

New Forest District Council (NFDC) and its predecessor authorities have carried out maintenance work on Hurst Spit since the early 1960s. Management of the beach from 1963-1989 comprised bulldozing and re-working of the overwash deposits to reform a new crest ridge along the pre-1989 storm alignment. In general the crest was pushed seawards to reform the beach crest. Beach material was also recovered from the Mount Lake channel with a dragline. The bulldozing was supplemented with import of gravel rejects, which were much coarser than the indigenous material.

To prevent breaching at the root, or proximal end, a 600m length of rock armour was emplaced between 1963 and 1968. Renourishment at an annual average of 1000m^3 was undertaken between 1980 and 1985. Storm erosion in 1984 resulted in widening, re-profiling and recharge over a 450m length of the spit beyond the earlier rock armour (Dobbie and Partners, 1984). Although effective as a protection measure, it reduced the rate of longshore drift and provoked erosional outflanking (Mackintosh and Rainbow, 1995). Typically 5000-8000 tonnes of material were added per year from 1985. A number of attempts were made to stabilise the proximal end of the Spit by armouring with rock, and by the construction of a groyne at the junction of the rock armour and the shingle beach. These schemes have not been entirely successful, serving only to transfer the problem further to the east.

Severe storms in October and December 1989, with a 1:50 year recurrence, caused overwashing, crest flattening over some 800m of the spit, up to 80m of landward migration, and the displacement of some 50,000 tonnes onto the leeward salt-marsh of Mount Lake. Unquantifiable offshore losses also occurred. The immediate response was to recover some 25,000 tonnes of gravel from Mount Lake, and import 36,000 tonnes from inland sources, to rebuild the beach, set back from its original location (at a cost of £440,000). Some recharge was also carried out following major storm-induced erosion in February 1991 and April 1994.

The practice of bulldozing the crest following storm conditions was abandoned following a management review of maintenance practice after the December 1989 storm event. This practice was considered to result in:

Damage to the underlying soft clay substrate
Mixing of the naturally graded gravel and sand material with clay
Be a primary cause of beach cliffing
Prevention of the natural equilibrium slope and plan shape from developing

In response to the impact of the 1989 storms, and because imported recharge sediments from inland sources proved to be too small to be retained over the longer-term, New Forest District Council undertook a series of field-based and model investigations, to determine a long-term, optimum scheme of protection (HR Wallingford, 1993; Mackintosh and Rainbow, 1995; New Forest District Council, 1990, 1996; Wimpey Environmental Ltd, 1994; Bradbury and Kidd, 1998).

Various mathematical and physical modelling investigations into forcing conditions, and evaluation of the relative merits and probable performance of alternative protection measures, were initiated in 1990. Physical model testing demonstrated that, in 1990, Hurst Spit was very sensitive to water level, and that a south westerly storm with a return period of once a year, combined with a tidal surge of 0.5m above mean high water springs, would generate similar damage as the 1989/90 event. A sensitivity analysis at the time of the scheme design (1990-91), indicates that this combination of storm return period and tidal surge level is, on average, likely to occur once every five years, which corresponds well with records of major damage to the Spit.

Research by Bradbury (1998 and 2000) into the relations between gravel barrier morphodynamics and hydraulic conditions was fundamentally based on extensive field and model tests carried out on Hurst Spit. This work has demonstrated the critical importance of antecedent barrier geometry in controlling behaviour under extreme forcing conditions (Bray, 2005); it has also identified a "dimensionless predictive inertia parameter" for defining threshold conditions leading to barrier breakdown.

The main measures that were implemented by 1996 (New Forest District Council, 1996; Bradbury and Kidd, 1998; Bradbury, 1998) were:

- Recharge with 280,000m^3 of gravel obtained from The Shingles Bank immediately offshore; this closely matched the indigenous sediment. The effect of this recharge was to almost double the previous volume of the spit.
- Crest level and width were both increased, declining eastwards in conformity to the reduction in wave climate severity from proximal to distal ends. The finished profile was to a crest elevation of 7mOD at the western end, reducing to 5mOD at Hurst Castle; this was designed to resist overwashing under surge conditions occurring up to a 1 in 200 year return frequency.
- Recharge allowed for ground settlement due to compaction and possible shearing affecting the basement sediments.
- Construction of a short, obliquely-aligned, detached and armoured rock breakwater, with a crest height of 2mOD at the proximal end of the spit. The purpose was to act as a headland structure and to "smooth" the former transport discontinuity between the terminal point of the rock armoured sector and the adjacent unprotected beach. The breakwater creates localised wave diffraction and promotes realignment of the beach by sand and gravel deposition in its lee rapidly producing a tombolo.
- Design parameters ensured that Hurst Spit would continue to function as a dynamic barrier system, but with an enhanced standard of defence (Bradbury, 1998; Bradbury and Kidd, 1998).
- Periodic recycling of gravel and topping up formed part of the initial beach management plan.

The capital cost of the scheme was £5.2m of which approximately £1.2m was for beach recharge.

The current beach management plan relies on a comprehensive monitoring programme (now being undertaken through the Southeast Strategic Regional Monitoring Programme) to inform the maintenance programme; this is used in conjunction with empirical predictive models (Bradbury, 2000) to provide a decision support system for the maintenance programme. The scheme has a design life of 50 years, during which it will be necessary to recycle or top up the recharge and to maintain the associated rock beach control structures. This programme will be revised in conjunction with the results of the planned monitoring programme and coastal strategy to a 100-year strategy, at strategic intervals.

Since scheme completion in 1996, Hurst Spit has successfully resisted over 20 storms that would otherwise have caused overtopping or overwashing. However, crest cut-back has occurred, with crest cliffing also a feature before fine sediment in recharge material was winnowed out or moved down-profile (Bradbury, 1998). Accretion behind the breakwater has occurred as predicted. Overall, the morphodynamic behaviour of the spit has been close to model predictions. Although the beach response has been largely as predicted in most storm events (Bradbury, 2000) a number of events have resulted in much more damage than predicted.

Data collected to date indicates that the beach recharge scheme is performing as predicted, and that no major changes to the 50-year strategy are required, at this stage, provided that the capital recharge is maintained. Emergency action should not be necessary although a contingency plan has been outlined.

Key maintenance activities include the following:

Recycling of beach material from the active recurve at North Point to areas of the main body of the spit that are thinning (Figure 5.22).
Trimming of the beach crest elevation to reflect local variations in compression of the underlying salt-marsh deposits, to maintain a crest elevation that will permit some overtopping.
Bypassing of the headland breakwater structure
Recycling from the lee slope of zones of the main body of the spit where accretion is the predominant process (Figure 5.22)
The first planned interim recharge is now scheduled for 2007/08 when an estimated 100,000m^3 of shingle will be required. This will be followed by recharges of 100,000m^3 at 15-year intervals until year 40. The volumes are based upon historical rates of loss and have been reassessed using the results of the first ten years of the beach management plan.

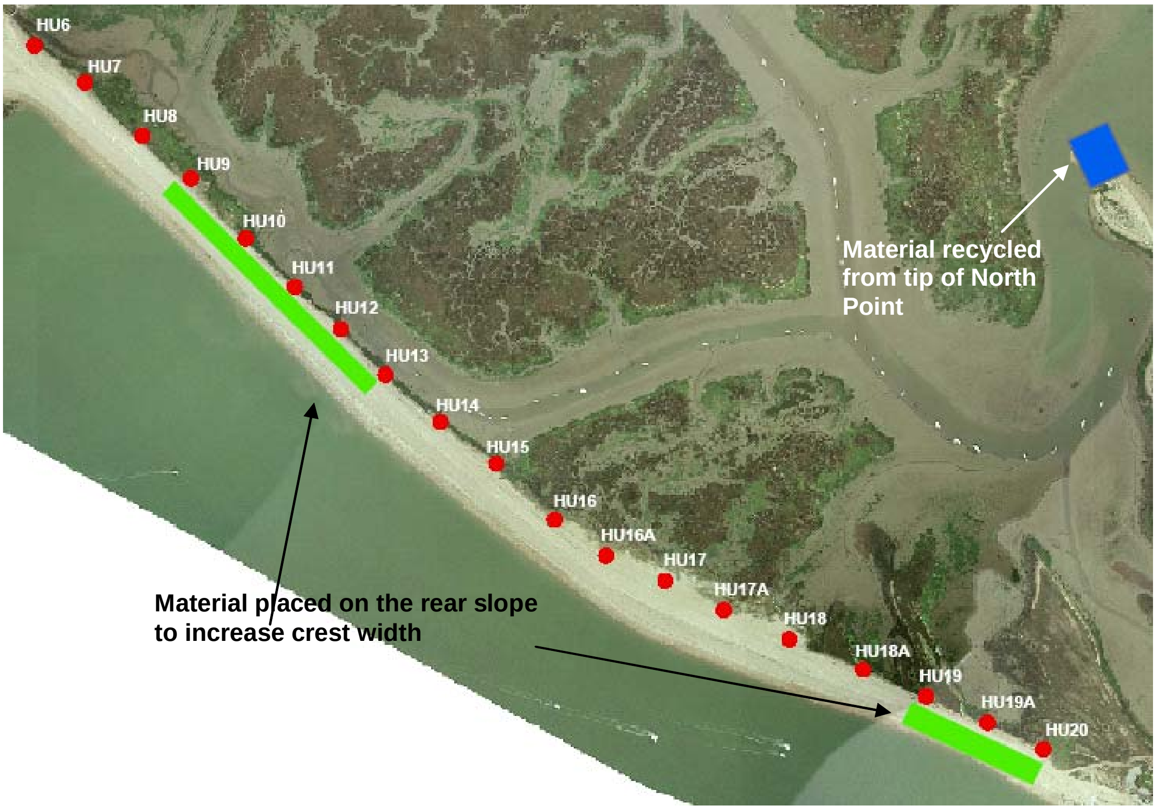


Figure 5.22 Typical beach recycling and barrier maintenance layout

The beach management plan is reviewed annually and minor revisions are made to the maintenance and monitoring programmes to reflect changes to beach performance. The expenditure profile has also been reviewed and modified to reflect the changes required. The results of the monitoring programme have been tested against new developments in best practice for design and management methods.

Monitoring

A comprehensive monitoring programme has been developed at Hurst Spit since 1987 (Bradbury, 1998). This has drawn on data derived for earlier investigations (Nichols. 1985). Excellent data is available to describe evolution during the past 20 years**.**

The cohesive monitoring programme reflects the complex issues and need to develop a large scale beach recharge and beach management programme. The programme has been further refined, in conjunction with the Southeast Regional Coastal Monitoring programme, which has been running since 2002. The current programme comprises:

Annual baseline spot height surveys and generation of ground models
Annual bathymetric profiling at 100m spacing
Annual aerial photo survey
Permanent control locations, surveyed by static GPS observations
Production of 10cm resolution digital orthophotos based on low level aerial surveys, every two years.
Baseline mapping of ecology to a level suitable to inform the extent of designated habitats for biodiversity action plans
Biannual beach profiles at 100m spacing.
Post storm beach profiles (taken approximately once per year).
LIDAR surveys of the entire beach at 1m resolution every 3 years.
Directional wave rider buoy in shallow water, off Milford on sea
Tide gauge and weather station at Lymington
Refraction model prediction points at several locations driven by the UK Met Office UK waters model
Georectified imagery has been conducted for approximately 10 year epochs since 1946
Data are managed within a SANDS database

Applications of the monitoring programme have included the following:

Design and validation data for development of a 3d physical model
Storm event performance evaluation pre- and post- recharge
Validation of empirical predictive model of barrier overwashing (Bradbury, 2000)
Performance review of beach recharge and beach management scheme (Bradbury, 2001)
Determination of operational maintenance activities following storm events (Bradbury, 2001)

Issues

The main problems and management issues affecting the evolution and stability of Hurst Spit are:

Longshore sediment transport - more shingle is being lost off Hurst Point than is being supplied from the Christchurch Bay beaches. This is the direct result of the construction of defence structures within Christchurch Bay, which have cut off the sediment supply. The balance of sediment supply could be maintained by removal of existing defence systems that protect developments in the coastal zone to the west of Hurst Spit. The prospect of this occurring is extremely unlikely, given the value of properties protected. Alternatively, the balance could be redressed by the introduction of beach recharge to the west of Hurst Spit; this is currently being examined as a management option within the coastal strategy.

Damage from over-washing - as the spit becomes smaller in cross section, over-washing occurs more frequently causing greater damage to habitat and accelerating the rate of roll back or landward transgression.

Hurst Spit is considered to be a strategic feature that provides protection to the Western Solent from severe wave action. It is also an important control on the tidal currents within the Solent. Large scale changes in the geomorphology of Hurst Spit could have a profound influence on the tidal regime of the Western Solent.

The plan shape of the spit is held in place artificially, by a headland structure and beach management. This requires long term intervention and a commitment to maintain.

The spit provides protection to extensive areas of internationally important wetlands, which would erode rapidly if Hurst Spit were unmanaged. The current management strategy has avoided the loss of habitat loss (primarily mudflat and saltmarsh at an estimated rate of 7500m^2 per year since 1996 (Bradbury, 2001).

It is estimated that the Spit is now at risk of overwashing by a 1:1 year return period storms. The loss of the protection provided by the spit will eventually expose an extensive length of coastline to wave attack. This will put the existing Environment Agency sea defences at considerable risk of overtopping and breaching The amenity value of the Spit will be destroyed and Hurst Castle will be subject to wave attack from all sides.

Despite the excellent quality of field data available for this site, it is still not possible to predict the onset of overwashing or the rate of rollback of the beach crest or back barrier limit. Further development of empirical or process based models is required to support understanding of this process. It is suggested that bimodal (wave period) conditions may cause significantly more damage, than conditions with a single spectral peak. Existing predictive approaches are unable to deal well with long period wave conditions (>10s). The implications of this are that:

The beach recharge scheme may be under-designed and will allow overwashing more frequently than was intended.

There is a need to identify the return period of such events with bimodal wave periods or with waves of long period (>16s), in order to determine the standard of service.

Improved predictive modelling techniques are required to assess overwashing potential

There is a lack of certainty in nearshore wave climate definition. Improvements are needed in the definition of the variability of longshore wave climate, to reflect the influence of friction affects arising from the offshore Shingles Banks and also to provide a more reliable indication of nearshore wave direction. The model predictions for direction typical vary by 15° relative to observations made at the shoreline. Similarly, the bimodal direction of the wave climate is poorly defined by modelling at the eastern end of the spit. This site is subject to an extremely complex wave climate characterised by:

Rapidly changing intensity of conditions longshore
Occasional wave energy spectra characterised by bimodal wave period
Regular nearshore wave energy spectra characterised by a bimodal direction (at the eastern end of the site)

Cliffing of the beach following beach recharge operations (Plate 5.25) remains a poorly understood process, and results in less than optimal performance during storm events. There is evidence however, that reworking of material will result in re-grading of the beach after a period of time (Plate 5.26).

Plate 5.25 Post storm response of beach showing cliffing, in January 1998 (Bradbury, 2001)

Plate 5.26 Post storm response of beach showing well graded foreshore, October 2000 (Bradbury, 2001)

Management of beach recharge schemes requires an appropriate procedure for dealing with geotechnical issues, such as loading of partially consolidated salt-marsh deposits with beach sediment. Further information is required to refine prediction of rates of loss of beach material arising from compression of partially consolidated sediments.

5.7 East Head

Location	East Head, West Sussex (see Figure 5.23)
Designations	SPA, Ramsar site, cSAC, SSSI, GCR.
Length	1km
Width	50-80m at neck, but varying as the spit develops.
Crest	Variable (generally well above the high water line except at the neck).
Sediment	Mixture of sand and shingle, but mainly sand.
Tides	4.2m on spring tides and 2.2m on neap tides. MHWS is 2.16mOD.
Waves	SW for 40% of time and S for about 30% of time
Drift	Net east to west shingle transport of 1-2000m^3yr^{-1}. Sand unknown.
Tidal currents	1.5m/s on ebb and 1m/s on flood.
Classification	Drift aligned sand and shingle barrier (formerly swash aligned).
Management	Sand fencing, timber groynes, rock berm, nourishment,

Figure 5.23 Location of East Head spit

5.7.1 Introduction

East Head spit (Plate 5.27) is the remnant of a barrier ridge that once stretched westwards across the entrance to Chichester Harbour from the mainland connection near West Wittering. As the supply of sediment from updrift sources dramatically reduced, so East Head has evolved, changing from a narrow shingle ridge to a broad, sandy, re-curved spit.

Plate 5.27 East Head c.2004 - prior to breaching

The eroding cliffs of Selsey Bill once provided large volumes of sediment that were then transported alongshore to the mouth of Chichester Harbour, there being redistributed by waves and tidal currents (some material even reaching Hayling Island, via the ebb delta off the Chichester Harbour entrance). However, due to the construction of sea defences the drift declined from 70,000m^3yr^{-1} in the 1850s, to 35,000m^3yr^{-1} in 1900 and to 7000m^3yr^{-1} by the 1970s (Webber, 1979). Now, the drift is thought to be no more than 1-2000m^3yr^{-1} of shingle. The acute change in alignment at the neck results in most of this drift being transported onto the sand flats in the harbour mouth, rather than reaching East Head. The neck of East Head is thus deficient in sediment, even if the proximal end of the spit continues to accrete.

Plate 5.28 Dune recession from line of timber piles, at neck of spit

East Head is volatile and there has been a history of 'soft' intervention on the spit to prevent it disappearing. This dates to the 1960's, when local volunteers reinstated the dune system by means of low cost fencing materials that successfully captured the wind blown sand drifting from the inter-tidal sand flats (Searle, 1975). Despite these efforts man-made defences immediately to the west have continued to cut off the longshore supply of sediments to the neck of the spit, causing significant landward retreat there and threatening to cause a breach. Concerns therefore began to grow, particularly amongst the users of Chichester Harbour that a breach could lead to the development of a new tidal channel into the harbour, diverting flows there and possibly leading to siltation in the main channel.

To prevent a breach forming, a 200m rock berm was built on the landward (east) face of East Head in 1999/2000. However, a storm on 27th October 2004 resulted in the erosion of the dunes (Plate 5.28), overwashing of the neck of the spit (without a tidal breach occurring) and exposure of the rock berm on the top of the foreshore. Natural England became concerned that exposure of the rock berm would disrupt the coastal processes that sustain the conservation interest of the site.

The neck of the spit has therefore been recently reinforced by nourishing it with sand (see Plate 5.29).

Plate 5.29 Sand nourishment berm at neck of East Head

5.7.2 Geomorphological context

The East Solent is characterised by drowned river valleys, which form natural harbours. Rising sea levels caused the river valleys to become drowned and for sand and shingle to be combed up and driven landwards, forming a series of barrier islands, spits and offshore shoals, creating spits that narrowed the inlet mouths, thus creating the "harbours".

East Head spit developed from what was once a linear shingle barrier that extended from the Selsey frontage, westwards across the entrance to Chichester Harbour (leaving a narrow entrance channel adjacent to the east side of Hayling Island). When the spit extended across the Harbour entrance the spit was essentially swash aligned. However, as the supply of sediments for maintaining the spit has reduced, East Head has rotated clockwise and has become drift aligned.

The spit is thus interesting geomorphologically, because it is continually evolving and changing shape.

5.7.3 Flood defence / coast protection role

East Head protects low-lying land on the eastern shore of Chichester Harbour from wave action and from being flooded by wave overtopping. It is considered that the spit may also be important in maintaining the tidal regime of the harbour, preventing the opening up of new tidal channels.

5.7.4 Evolution

Average recession rate

The historic recession rate of recession was governed by the recession of the adjacent coastline, being as high as 3m per year in the 1800s. (This only refers to the barrier when it was a linear feature extending across the mouth of Chichester Harbour). Figure 5.24 shows the historical evolution of the spit from 1842 (Webber, 1979).

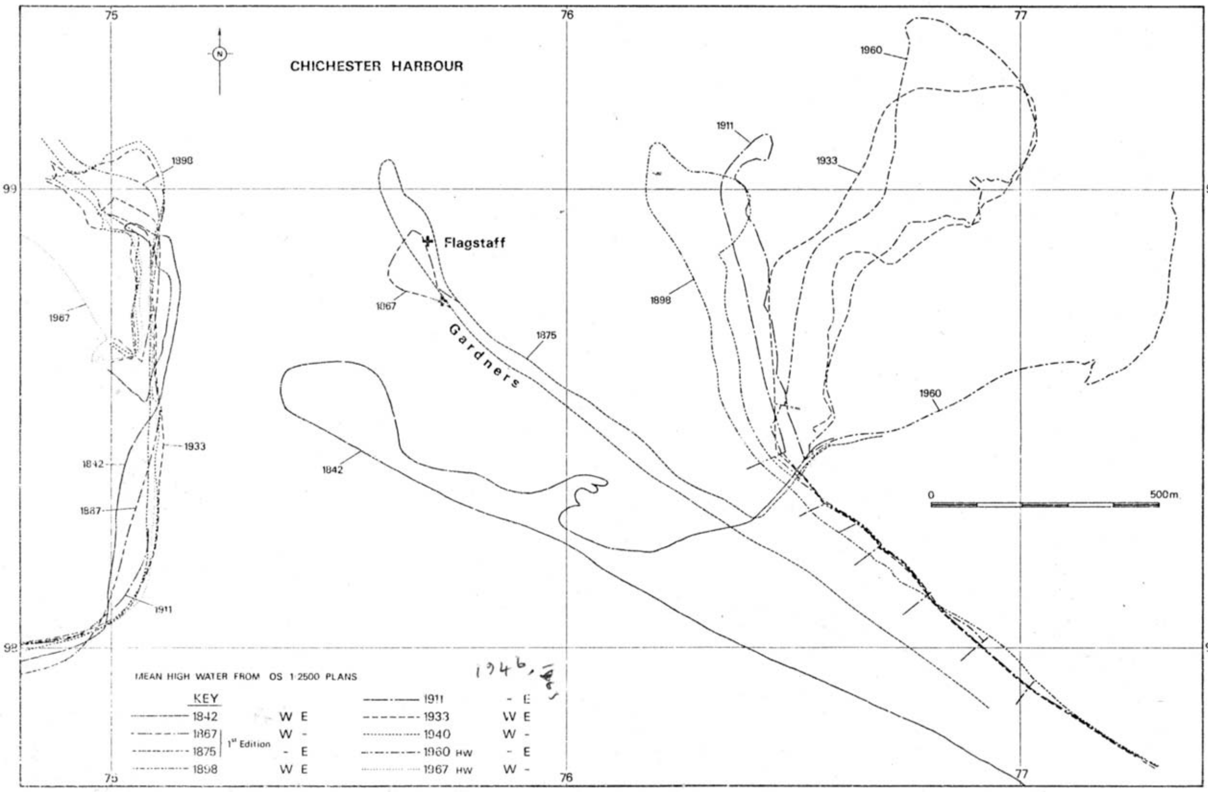


Figure 5.24 Realignment of the spit since 1842 (Webber, 1979)

Back barrier

The barrier is backed by dunes but has been washed over at the neck on several occasions, notably in 1963 and most recently in 2004, sand being washed over into the marsh to the landward and severing the spit from the mainland.

Short term (storm event response)

East Head has been rotating in an anti-clockwise direction since about in response to the reduction in sediment supply due to declining littoral drift rates (see Figure 5.24). In doing so the neck has become vulnerable to breaching.

The dunes at the neck have now been eroded and this area is prone to overwashing and possible breaching during severe events.

5.7.5 Management

Intervention

Only the proximal end of East Head spit is defended and it is uncertain whether the defences there have benefited the remainder of the spit.

The defences on the mainland coast itself, east of the Hinge, consist of timber breastworks and long timber groynes. There is a complex system of breastworks, gabions and radiating groynes around The Hinge itself. These defences seriously restrict the amount of sediment reaching East Head.

On East Head itself sand fencing has traditionally been used to stabilise the toe of the dunes. This has been possible because of the vast quantities of sand that are found on the flat, inter-tidal foreshore seaward of the East Head and the exposure to strong westerly and southerly winds.

Despite these measures East Head is vulnerable to erosion during surge tides, when breaching is possible. The rear face of the neck of the spit was therefore reinforced by riprap armour in 1999/2000. Following serious overwashing in 2004 the neck was nourished with sand and the crest planted with marram grass.

Monitoring

Although the area is now being monitored by aerial surveys, the most historically useful information has been collated through the analysis of old maps. Coastline changes at East Head have been documented by Webber (1979). Figure 2 shows a narrow shingle ridge extending almost the full width of the entrance of Chichester Harbour. By 1875 the spit is shown to have migrated northwards, keeping pace with the retreat of the coast (i.e. at about 3m per year).

By 1898 the spit had further retreated landwards, but had also rotated clockwise and become wider and shorter, losing shingle but gaining sand. Recession of the coastline to the east was still averaging at 2 to 3m per year.

By 1911 the spit had rotated still further, at a time when coastline recession had reduced to about 1m per year. The coastline immediately east of the spit is shown to be groyned by that time.

By 1933 the spit had taken on an alignment that was almost at right angles to the general coastal alignment to the east. The spit had by then developed a spatulate plan shape due to a dwindling shingle supply keeping the neck (Hinge) narrow, but extensive sand build up and subsequent dune growth making the distal end wider. The foreshore deposits left behind as the spit changed shape became stranded on the lower foreshore in the area known as the Winner bank, a large expanse of sand and gravel located between the spit and the main entrance channel to the west.

By the 1960s much of the coastline to the east had been protected by sea defences and groyned and the drift was considerably reduced (Webber, 1979). By then the coastline recession had slowed down to about 0.5m per year.

In November 1963 East Head was breached and the dune system inundated and almost totally destroyed. Following this, a programme of dune building was started. By the mid 1970s National Trust volunteers had succeeded in restoring the dunes, using sand fencing, substantially increasing the area of the spit. Old photographs show the dunes being on the lee side of a shingle beach, which had developed due to the groynes at the Hinge becoming ineffective at that time.

However, rates of spit retreat increased from the early 1980s onwards (Pontee et al, 2002). In 2004 a breach was formed at the neck, but has subsequently been partially healed. An extensive photo library exists of the neck of the spit and the erosion of the dunes and subsequent overwashing in storms.

Issues

The foreshore is in, or adjacent to, Chichester Harbour SSSI and West Wittering Beach SNCI. The area is also within the Solent Maritime possible candidate SAC. The area is designated fro its biological, geological and geomorphological interests. Shoreline management operations must therefore comply with statutory procedures including the Habitats Directive. Operations therefore have to consider the sand dune/shingle systems of East Head, as well as the salt-marsh/wetlands in its lee. The area is also within the Chichester Harbour AONB. It is designated as Countryside and is protected from significant development.

The public has raised several important possible consequences of the breaching of the East Head spit. It is thought that breaching could lead to changes in the tidal flows into and out of the Harbour, potentially leading to siltation, navigation problems in the main entrance channel and alteration to mooring conditions inside the harbour. Concerns have also been felt about the possibility of an increased flood risk on the east shore of the harbour. From the viewpoint of conservation interests Natural England has viewed changes to the habitats and species associated with overwashing, or even breaching of the spit as not necessarily damaging.

East head was studied during the development of the Shoreline Management Plan for the area (HR Wallingford, 1997). In assessing the various management options it was concluded that the impacts of the “Do Nothing” option might be the breaching of the neck of East Head and wave and current erosion in the lee. Thus, in the extreme, East Head could become an island at high tide. It was also predicted that existing sheltered habitat in the lee of East Head may be lost and the harbour shoreline of West Wittering would be subject to increased wave attack. Therefore, “Maintain the Line” was the preferred option, which would allow East Head and the harbour habitat/regime to be maintained (HR Wallingford, 1997).

More recently, the evolution of a breach at the neck of East Head was studied using computational modelling of tide, wave and sediment transport conditions (Pontee et al, 2002). The studies upheld the view that the initial formation of the linear spit and its extension across the harbour entrance before 1842 was due to relatively high rates of drift, with the subsequent clockwise rotation and landward movement due to the decreasing supply of material. The modelling suggested that East Head spit is in a state of “barrier breakdown” under depleted sediment supply. As part of the study the tidal prism versus area relationships were examined. These indicated that the harbour mouth is constrained (i.e. it would naturally have a larger cross-section if left to its own resources). This implies that changes in its configuration will be necessary before the harbour mouth can achieve a stable state. These could be one or more of the following, any of which increase the tidal cross-section:

retreat of East Head
creation of new channels
deepening of existing channels.

With sea defences to the east (updrift) causing a continued lack of supply then East Head spit is likely to continue to erode, leading to breach formation at the Hinge.

Modelling suggests that a breach can be maintained by tidal and wave action, possibly leading to siltation within the harbour mouth, due to redirection of currents away from the mouth. It is thought that modification of the channels within the harbour might lead to the need for dredging in order to maintain navigable depths (Pontee et al, 2002).

More recently HR Wallingford has advised Natural England about how best to manage the spit. In the short term it was recommended that the spit should be strengthened by means of sand nourishment. In the longer term one should consider realigning the spit and moving it further landward, in line with the natural tendency for coastal retreat.

5.8 Hayling Island

Location	Hayling Island, Hants (see Figure 5.25)
Designation	SSSI, LNR, SPA, Ramsar site, AONB (Eastoke Point, a shingle barrier ridge at the east end of the island, lies within the Chichester Harbour AONB. It is also within the Chichester Harbour SSSI (also SPA and Ramsar site) and the undeveloped land behind the ridge is within the Sandy Point LNR.
Length	7km
Width	Varies
Crest	Varies
Sediment	Shingle upper, sand lower.
Tides	Spring range 4.2m. Tidal flows into and out of the tidal inlets at either end of the island are rapid, reaching 3m/s on the ebb tide and 1.5m/s on the flood tide, hence capable of transporting coarse sediment a considerable distance offshore, enabling ebb delta bars to form.
Waves	South-westerly waves diffracted around the Isle of Wight. Nearshore significant wave heights of 2.8m in combination with a maximum water level of 2.7mOD have a return period of about 1:1 year.
Littoral drift	Littoral drift is both eastward and westward from a drift divide in the eastern part of the island at Eastoke.
Classification	Drift aligned barrier beaches.
Intervention	Seawalls, timber groynes, pioneering nourishment, recharge

Figure 5.25 Location of Hayling Island

5.8.1 Introduction

Hayling Island is a low barrier island that that separates Langstone and Chichester harbours, except for a narrow link at its northern end. While historically it may have been able to retreat landwards, that process has been interrupted by man-made as well as by natural processes. At its eastern end it has been prevented from rolling back by the presence of sea defences. Where it is backed by seawalls, the lower foreshore levels have tended to fall. The island has also been extending out at the ends, as a result of littoral drift, so that the ends no longer display any strong tendency for retreat.

Plate 5.30 shows the barrier beach as a narrow strip, with a low-lying backshore that is prone to flooding (from both landward and seaward).

Plate 5.30 Hayling Island from the air

Gunner Point, at the western end, consists of a series of shingle ridges that have accumulated due to the strong east to west littoral drift. Eastoke Point, at the eastern end, has a smaller accumulation due to a lower and more fluctuating west to east drift. Eastoke Point encompasses a nature reserve and, if a breach were to occur, it would cause flooding of this low-lying area and the houses surrounding the reserve.

In the 19th century a long thin spit called East Head spit extended from the Selsey peninsula almost the full width of the tidal inlet called Chichester Harbour (immediate foreground of the Plate 5.30). At that time there was a considerable feed of material to Hayling Island via an ebb delta bar that was relatively close inshore. Plate 1 shows a seaward trending spit called the West Pole at the eastern end of the island. Observations have shown that shingle is driven onshore under the right weather conditions. A reduction in alongshore supply of shingle, due to coast protection works along the Selsey to East Head frontage, caused the spit to decline and to rotate clockwise. This, in turn, reduced the supply of material available to be transported across the harbour entrance to Hayling Island. It is thought that, at one time, an average annual feed of some 20,000m^3 of sand and shingle, may have come onshore at the eastern end of the island (Clare, 1988). Any such onshore feed is now greatly reduced.

The westward drift from Eastoke is about 20,00m^3/yr, while the eastward drift is about 10,000m^3/yr. With the earlier onshore transport of about 20,000m^3, this still left a net deficit of some 10,000m^3/yr due to “end losses” (Clare, 1988). This deficit has been much larger in recent years, leading to the problems at the eastern end of the island.

5.8.2 Geomorphological context

Hayling Island is a barrier island that was formed from sediments deposited in the (then) river Solent and that have been move landwards in the post-glacial era. The conditions for its development were initiated when the chalk ridge that joined the Purbeck hills to the Isle of Wight was breached and the Solent was developed on its present course. A series of barrier islands then developed on what were the shallow margins of the former river plain of the Solent. These are separated by shallow lagoons (Chichester Harbour, Langstone Harbour etc.).

5.8.3 Flood defence / coast protection role

The shingle ridge and sea defences protect low-lying land against marine flooding. They also protect the shores of Langstone and Chichester harbours against offshore wave activity. Management of the Island defences helps to maintain inlet entrance widths, hence navigation into harbours.

5.8.4 Evolution

Seawalls delimit the back of the beach in low-lying areas at the eastern end of the frontage. Multiple ridges have protected the central and western parts of the island.

Individual extreme events (extreme combinations of wave height and water level) are responsible for episodic erosion of the barrier crest, over-washing and flooding of the hinterland.

5.8.5 Management

Intervention

Development at the eastern end of Hayling Island dates back to the 1930s, when chalets first began to be constructed on the backshore immediate. Plate 5.31 shows a wide sand and shingle beach at that time. However, the “cliffing” apparent at the back of the beach suggests that erosion was already occurring. By 1939 a short length of wall had been constructed at Beach Club, Eastoke, at the eastern end of the island. This may have triggered erosion adjacently; certainly the seawalls had to be extended eastwards and westwards from this point soon afterwards.

Because of differential times of seawall construction, the seawall at Beach Club is now slightly further seaward than the walls to the east and west, causing it to be more exposed to wave action than elsewhere. Continuing backshore erosion led to the seawalls being extended to a length of about 400m by 1947, and about 2km by 1954.

Timber groynes were also constructed as early as the 1930s, but despite this, beach levels in the eastern part of Hayling Island continued to fall and the seawalls were at risk of both undermining and overtopping.

Plate 5.31 The beach at Eastoke in the 1930s

By the late 1970s extensive underpinning and repairs were. Plate 5.32 shows the kind of conditions that the backshore was exposed to during severe storms.

Plate 5.32 Wave overtopping at Eastoke in the late 1970s

Major works were required to prevent further overtopping from occurring. Following extensive studies a scheme was adopted which tackled the fundamental problem of the reduced shingle "sediment budget". The beach nourishment scheme that was initiated was one of the first of its kind in the UK.

In December 1985 some 500,000m^3 of sand and shingle were won from a nearby offshore dredging area and placed over a frontage of some 1.5km at the eastern end of the island. The material was bulldozed upwards from where it was landed, to form a 35m wide berm in front of the seawall (grading seawards at a slope of 1 in 5). The crest of the berm was 5.5mOD, approximately 0.5m higher than the crest of the seawall.

It was decided to try and capitalise on the presence of coarse aggregate and the nourishment material was therefore sieved in situ and the coarser fraction of 30 to 115mm cobbles was placed as a surface covering on the finer fill. In the event this very permeable coarse armour layer was less stable than the "natural mixture" and it was found that it dispersed very rapidly both alongshore, as well as rolling down the beach face (Clare, 1988). The lessons learnt from this are that it is important to maintain as natural a grading as possible for optimum efficiency.

A new system of timber groynes was constructed 2 years after the beach nourishment scheme, by which time the nourished beach had achieved a more natural profile. At the same time, the beaches were topped up with 46,000m^3 of shingle. About a year after this the beaches were topped up with a further 25,000m^3 of shingle.

Monitoring

Soon after construction of the nourished beach rapid dispersal of the nourishment material was noted, probably aggravated by severe wave conditions. Between 24th March 1986 and 6th January 1988 wave heights in excess of 1.5m were observed on 9 occasions, and on two of these the wave height was as large as 2.5m. On four of these occasions the wave period exceeded 12s and on one occasion the maximum period observed was 17s. On any of these occasions serious overtopping of the seawall would have occurred had the beach nourishment scheme not been in place (Clare, 1988).

The rapid easterly transport of material led to accretion at Eastoke Point and, ultimately, led to losses in the “sediment budget”. As the ness at Eastoke Point grew, it impinged on the deep-water channel into Chichester Harbour. The rapid ebb flows then removed material from the ness and transported it offshore. A groyne was therefore constructed in 1990 to prevent further material reaching the ness. However, erosion then occurred east (down-drift) of the groyne, threatening to create a breach. Soon afterwards a rock revetment and short groynes were constructed east of the first groyne to reduce the threat of breaching. As Plate 5.30 shows, the ridge at the eastern end of the island remains quite narrow, and is therefore sensitive to any future changes in supply.

Issues

The nourishment scheme has successfully protected the eastern end of Hayling Island for the last twenty or so years. Firstly, the nourished beach has protected the seawall which was already seriously undermined by the 1970s. Secondly, there has been little flooding of the properties behind the wall. Thirdly, the scheme has provided an amenity beach at all stages of the tide (Clare, 1988).

Since 1991 annual recycling has been ongoing in order to maintain an adequate standard of flood defence. However, the overall beach volume at the eastern end of Hayling Island continues to diminish as material is lost into Chichester Harbour. Recent studies have shown that the annual recycling rates, over the period 1993 to 2004, have been as follows:

14,500m^3 from west to east (i.e. from west Hayling to Eastoke)
7,500m^3 from east to west (i.e. from Eastoke Point to Eastoke)

Whilst it has been possible to maintain the main frontage by sediment recycling, it has proved to be more difficult to secure the fluctuating eastern end of the frontage at Eastoke Point. The LNR is designated for its unusual sand/shingle habitat. Shoreline management operations at Eastoke Point therefore have to comply with statutory procedures including the Habitats Directive.

The various options for protecting this area are being examined. Under the "Do Nothing" option a breach would occur, damaging the habitat in the LNR. A breakthrough would also put many of the properties at Eastoke directly at risk of flooding. In addition, if recycling along the main frontage were terminated, wave overtopping would occur frequently over the Eastoke frontage also (as it did in the past), causing flood damage to residential areas.

It is self-evident that protecting a *developed* barrier island is likely to require a continuing maintenance commitment, since a non-intervention option is unlikely to be acceptable because of overriding public interest. It is also self-evident that management of the beaches on a barrier island is much more complex than on a relatively straight and continuous coastline, because of the necessity to consider "end effects" that are particularly complex at the mouths of tidal inlets.

5.9 Easton Broad

Location	Easton Broad, nr Wrentham, Suffolk (see Figure 5.26)
Designation	SPA, cSAC, Ramsar site, NNR
Length	1.6km
Width	60m – 80m
Crest	Maintained at 4.0m to 4.5mOD, with natural crest at about 2mOD.
Sediment	Sand and shingle: d_{50} of 1mm, range between 0.45mm and 2.5mm.
Tides	Spring range 1.9m
Waves	Waves approach from a wide sector, modified by offshore sandbanks
Classification	Swash-aligned attached barrier beach.
Intervention	Crest-reformation, re-grading,

Figure 5.26 Location of Easton Broad

5.9.1 Introduction

Easton Broad (Plate 5.33) is an in-filled river valley that is now effectively “landlocked” due to the presence of a barrier beach that has developed at its mouth. The river flow and water levels are controlled by discharge into the sea, via an outfall pipe. This pipe extends seawards beneath a sand and shingle ridge at the top of the beach.

The valley contains a large freshwater reed-bed, home to marsh harriers and bitterns, the latter depending upon the freshwater fish that the river supplies. At the downstream end of the river valley there is a saline lagoon, created as a result of the river mouth being blocked by the sand and shingle barrier beach. The brackish lagoon receives saline water through percolation, wave overtopping and wave spray; fresh water is provided by the river. However, the drainage outlet is prone to blockage, making management of the lagoon and reed-bed difficult.

Plate 5.33 Easton Broads (January 2006, Motyka)

The sand/shingle barrier beach that impounds the lagoon extends continuously across the river mouth and ties in to low sandy cliffs to the north and southward. Erosion of these cliffs does provide some sediment supply for the beach (which is sandy) but is too fine to support barrier crest levels. The natural crest of the beach would normally be regularly overtopped, but its crest is artificially maintained to reduce the frequency of overtopping.

It would appear that the barrier has tended to migrate landwards and in doing so it has been "pinching in" the saline lagoon, which has therefore now reduced in area to about 4ha. It has also become less saline. Prediction of future shoreline positions indicates that if allowed to roll back, by 2050 a substantial area of the lagoon downstream of Potter's Bridge will be lost through coastal retreat (Harvey *et al*., 2004).

The river valley has both brackish and freshwater conservation features. The reed-bed is designated a Special Protection Area for birds (SPA) and is also a Ramsar site. The saline lagoon is a candidate Special Area of Conservation (cSAC), by virtue of containing rare/unusual invertebrates. The salinity balance within it is maintained through percolation, spray and wave overtopping, with fresh water input from the river itself. The sand/shingle ridge itself is considered to be an important conservation feature, having the potential for supporting nesting little terns. At present, however, the crest is disturbed mechanically on a frequent basis and is relatively “sterile” (see Plate 5.33). The crest is also too narrow and the sides too steep to provide much nesting area.

5.9.2 Geomorphological context

As sea levels have risen since the last period of glaciation, so the river valleys on the East Anglian coastline have become flooded and partially silted. This has contributed to the formation of the marshes that characterise the hinterland. Sediment that forms the beaches and that encloses the heads of the valleys is derived from the reworking of glacial deposits and their shoreward transgression. Thus, the shingle beaches in this area are considered mainly “relict”. The low glaciogenic cliffs do produce beach building material, but this is mainly sand and the shingle content is low (Futurecoast, 2002).

The coastline, like other parts of the East Anglia coast, is believed to be still actively responding to sea level rise. The widespread foreshore steepening that is taking place in this region is considered to be a continuing response to the drowning of the North Sea basin, rather than the impact of sea defences on coastal regime (i.e not through interruption of the natural alongshore sediment supply). It is therefore likely that the tendency for foreshore steepening will limit the extent to which the beaches in this region can retain sediment (Futurecoast, 2002).

With the shingle component of the beaches being largely relict, together with the limited supply of shingle from cliff erosion, maintaining flood banks such as the one protecting Easton Broad may not be sustainable in the long term.

5.9.3 Flood defence / coast protection role

Flood protection is for the conservation of natural features of the area (a saline lagoon and reed-bed) rather than built assets (almost all of the town of Wrentham lies upstream of the limit of tidal incursion).

5.9.4 Evolution

Average recession rate
The barrier has tended to migrate landwards under sea level rise, but in recent years this has been slowed down by nourishment and mechanical maintenance of the berm.

Short term (storm event response)

The unnaturally steep profile is vulnerable to breaching. Overtopping in December 2003 caused an 800m breach and virtually flattened the ridge over this length.

5.9.5 Management

Intervention

The barrier beach is maintained by bulldozing it to a maximum height of about 4.5mOD. The barrier comprises a mixture of coarse sand and fine shingle, within the size range of 0.45 to 2.5mm and having a median diameter of about 1mm. The incoming littoral supply consists mainly of sand, so it is not easy to maintain a coarse beach ridge that resists scour. The height of the ridge is maintained by mechanical regarding, with the intention of controlling wave overtopping, as shown in Plate 1. This creates an artificially steep cross-sectional profile, which is more vulnerable to breaching than a naturally flatter and lower profile. In addition the re-grading probably interferes with the internal structure of the beach. This management practice is considered to be detrimental to the conservation value of the ridge, as well as being considered unsustainable in the long term (Harvey, 2004).

A further problem is wind blown sand that tends to infill the lagoon. Sand is not only blown into the lagoon from the sandy foreshore to the seaward, but some from the crest of the ridge itself (see Plate 5.34).

Plate 5.34 Sand blow removing fines from the crest and infilling saline lagoon (Motyka)

Monitoring

During the Easton Broad Flood Management Project (Harvey et al, 2004) various options for flood defence were considered through a combination of hydrodynamic and fluvial modelling.

As input the barrier was surveyed in August 2003 and the profile response to wave and high tidal levels assessed. Cross-shore transport numerical modelling indicates that erosion of the ridge can occur under conditions as frequent as a 1:5 year event. Modelling determined that the barrier provides a 1:5 year standard of protection against breaching, but only a 1:1 year standard against overtopping. Further, when sea level rise is taken into account the standard of defence is less than 1:1 year.

For the future management of the site the following options were considered:

The "Do-Nothing" option. This was determined to be unsustainable. A breach in the barrier beach (as occurred in December 2003) would cause saline intrusion into the reed-beds, affecting not only the reeds themselves, but also the associated freshwater fish populations and the nesting birds that depend on them. About 20 properties on the southern outskirts of Wrentham would also be at risk of flooding.

The "Maintain" option. Continuing the present maintenance of the barrier beach, through re-profiling the crest height is considered to be damaging. For this option also, the modelling predicted that significant flooding of the reed-beds would occur, even under relatively small surges. Again, about 20 properties on the southern outskirts of Wrentham would remain at risk of flooding.

Managed realignment. This option involves building a new "secondary embankment" some distance upstream, while allowing the barrier beach itself to evolve naturally in the future. The embankment would best be sited either at Potter's Bridge, or at two other possible locations nearby. Allowing the beach to adopt a natural cross-section by the action of waves and high water levels, would lead to more frequent overtopping and landward migration by overwash. Modelling has shown that in this option the flood embankment would not only protect the southern outskirts of Wrentham against flooding but would greatly reduce saline intrusion into the reed bed, even when only a 1:20 year standard of defence for the embankment is adopted (i.e. for an embankment with a crest of 3.7mOD).

Issues

At this site it appears unfeasible to maintain all three features (reed-bed, saline lagoon and barrier beach) in favourable condition simultaneously. Allowing the barrier beach to evolve naturally will inevitably result in the lagoon and the reed-bed shrinking in area. During early discussions, Natural England indicated the requirement for a replacement reed-bed to supersede that lost in the lower valley.

Hydrodynamic and fluvial numerical modelling has been used to show that the following solution provides a reasonable compromise between conserving freshwater (reed-bed) habitats and brackish (saline lagoon habitats):

Allowing the beach to evolve naturally. This means that the crest height is expected to fall to about 2mOD, to approximately the crest level of unmanaged ridges at Covehithe and Benacre Broads. The ridge will then be able to respond naturally to washover processes and will migrate landwards.

Provision of an embankment will reduce the penetration of saline water upstream and will also protect the southern outskirts of the town of Wrentham against flooding. The embankment is to be sited at a location that will provide sufficient area for the saline lagoon to migrate landwards as the coastline retreats, at least 50m years into the future.

Replacing the losses in reed-bed area by provision of the remainder at an alternative location (unspecified).

This case study illustrates how, in situations where new defences, or improvements to existing defences, are required, it is important that full consideration is given to nature and geological conservation in the concept, planning, design, implementation and maintenance stages. There is therefore a need for a strategic approach to shoreline management, which makes advance provision for habitats and communities to migrate as the shoreline evolves (e.g. conducting managed retreat to replace salt-marsh which has been eroded).

This case study illustrates an instance where a scheme has been put forward primarily to protect important conservations, rather than built assets. In this instance there was an absence of significant “traditional” assets requiring protection, with the area affected by marine inundation only just reaching the southern outskirts of Wrentham.

This will make an interesting case history for future assessment. Particularly it will be interesting to see how quickly the barrier retreats landwards, once it is no longer constrained in position. Also, it will be interesting to see how the saline lagoon fares, as sand is pushed landwards, both under wave action and wind blow.

5.10 Sand Bay

Location	Sand Bay, Avon (see Figure 5.27)
Designation	SSSI
Length	3km
Width	75m (after nourishment in 1983).
Crest	9.3mOD (3.3m above MHWS)
Sediment	Sand and shingle; d_{50} = 1.2mm.
Tides	Spring range 14m. Storm surges of up to 1.4m can occur at high tides.
Waves	Exposed to dominant westerly waves from the Atlantic Ocean. However, these are modified by the processes of refraction, diffraction and energy dissipation, as they propagate up the Bristol Channel, and into the Severn estuary. (Significant inshore wave heights of up to 2m have been observed).
Littoral drift	Negligible, due to enclosed nature of the bay.
Classification	Swash-aligned straight attached barrier beach.
Intervention	Seawall, renourishment

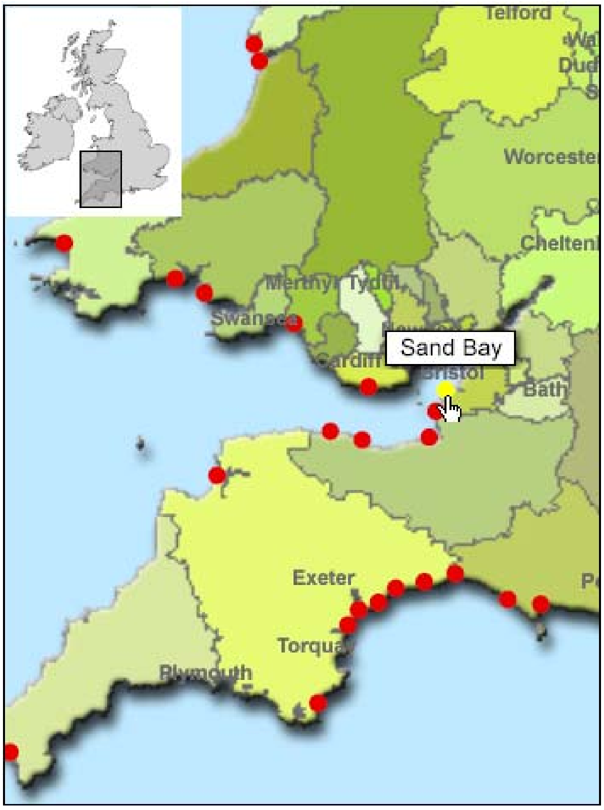


Figure 5.27 Location of Sand Bay

5.10.1 Introduction

Sand Bay (Plate 5.35) is a small, partly enclosed sandy embayment, situated near the upstream limit of the Bristol Channel, just to the north of Weston-super-Mare. At this location the bay is partly sheltered by the alignment of the shores of the Bristol Channel, thus having a relatively small wave "window" to the west from which Atlantic waves can approach directly. The bay is well sheltered from other wave directions by the large limestone headlands to the north and south. At the northern end of the frontage the shelter is so pronounced that a salt-marsh has developed in the immediate shelter of the Middle Hope headland. However, the central and southern parts of the frontage are more exposed and the sand beach had become quite narrow there by the early 1980s.

Before the 1980s the bay had a low shrub-covered ridge that separated caravan parks, low-density housing, and low-lying former marshland from the inter-tidal zone. The ridge was backed by a wall of relatively low crest height, in turn backed by a coastal road running parallel to the wall. Seaward of the ridge was a sandy beach and below this a wide, muddy lower foreshore (the beach width has substantially increased since nourishment in 1983 and the backshore is now dune covered).

Being located between two limestone headlands, Sand Bay experiences no losses of sediment in a longshore direction. The lower foreshore is muddy, so any offshore sand transport would result in material being permanently lost from the system. Also, when beach levels are high there can be some slight losses due to onshore sand transport by wind action.

The low beach levels in the central and southern part of Sand Bay made the seawall in that area vulnerable to wave overtopping. In the early 1980s Sand Bay experienced several severe storms, which caused flooding on at least two occasions. This occurred when high tidal levels, elevated by surges, coincided with severe westerly gales. During 13 December 1981 the storm surge elevated high water level by 1.4m. This, in combination with significant inshore wave heights of about 2m caused serious overtopping, structural damage to the wall and the flooding of 82 properties (Bown, 1987).

Plate 5.35 Sand Bay (2005, Motyka)

5.10.2 Flood defence / coast protection role

The sea defences protect some 250 properties, a holiday camp, and some 400 ha of land against flooding.

5.10.3 Geometry

Figure 5.28 below shows the cross-section of the nourished beach in 1983/4.

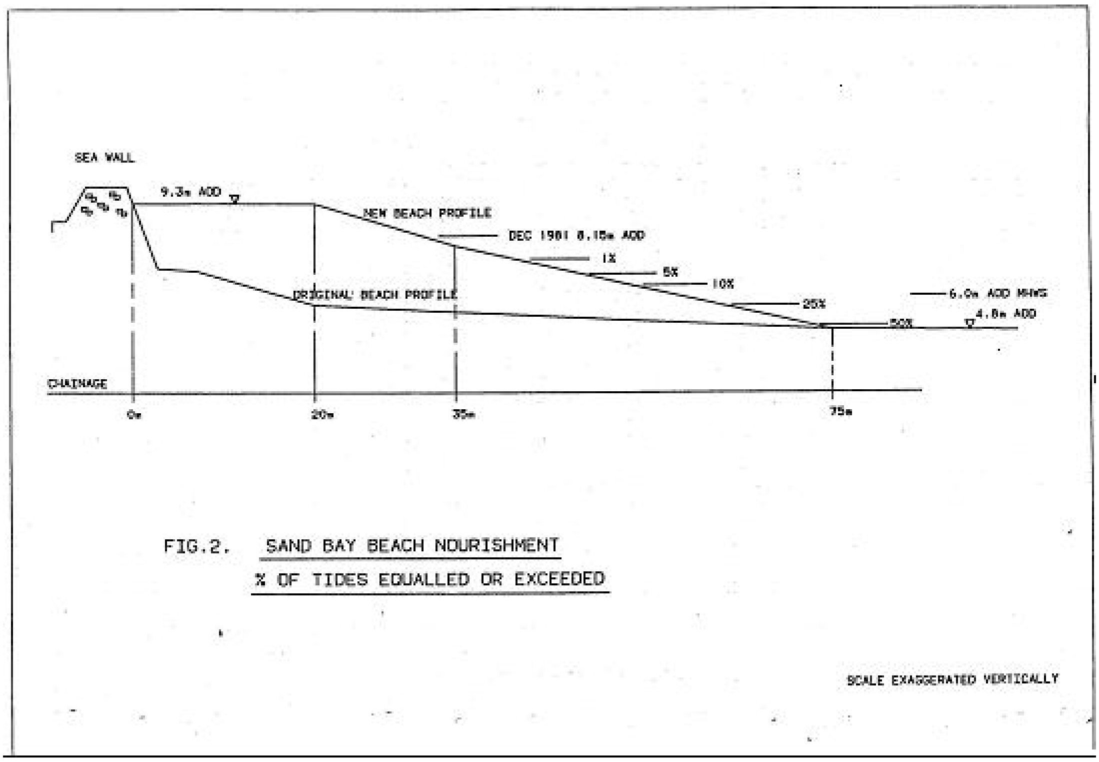


Figure 5.28 Cross-section of beach in 1983/4

5.10.4 Evolution

Average recession rate

Insignificant, with beach being backed by a seawall.

Back barrier

Sand dunes backed by a seawall.

Short term (storm event response)

Beach and dune erosion, but no significant overtopping since sand and shingle nourishment in 1983/4.

5.10.5 Management

Intervention

HR Wallingford advised the (then) Wessex Water Authority about the likely performance of a beach nourishment scheme. It was concluded that due to the embayed nature of the bay and the extensive muddy foreshore to the seaward (which showed little evidence of being covered over with sand from the beach) that, if relatively coarse sand was used as a nourishment material, then offshore losses would probably be insignificant.

Following consideration of various defence options, and taking into account the extensive damage to the wall by recent storms, a decision was made to improve the flood defence standard by raising the height and width of the sand beach,

Starting in June 1983 and completing by March 1984, some 300,000m^3 of coarse sand and fine shingle had been brought onshore over a frontage of some 2200m, to form a berm some 0.5m higher than the crest of the seawall to the rear. The material was first formed into a bund into which the dredgings were pumped. Bulldozers were then used to create the 20m wide berm, with a crest some 15.3m above Chart Datum, i.e. about 1.5m above the high water of spring tides.

The nourishment material was obtained from an existing licence area for dredging aggregate that was situated nearby in the estuary. The material had a minimum particle size of 0.2mm, thereby minimising likely offshore losses. The maximum particle size was 30mm, still small enough to be pumped the considerable distance from beyond the low water line to the upper beach (the foreshore is very wide and the tidal range is as high as 13 metres on spring tides).

The sand was graded to a relatively steep slope of 1 in 10, thus avoiding displacing the material onto the muddy lower foreshore.

Monitoring

Beach levels had been monitored by the Water Authority for some years before the nourishment scheme was implemented. Monitoring then continued by carrying out cross-sections at about 100m intervals and extending some 75m offshore. The profiles were twice yearly (in August and April). (Bown, 1987) reports that there had been no significant loss in volume of the upper beach since the nourishment. Details of any more recent monitoring/management operations (i.e since the 1987 report by Bown) are not yet available.

Issues

Within the Water Authority planning procedure there was a 30% allowance included in the design for topping up of the nourished beach at 10 years after construction. However, the availability of a relatively coarse nourishment material has meant that, in the event, there has actually been little loss of material.

Monitoring has shown that what losses there have been are primarily the result of strong onshore winds carrying sand onshore. The problem of onshore transport was solved by the use of sand fencing and the encouragement of dune grasses. Even with pedestrian usage the losses to the backshore sand deposits have been very small, while the beach itself has not significantly reduced in volume.

Even now, more than twenty years after nourishment, the beach and dune system continue to provide against wave overtopping to the area.

5.11 Pett Levels

Location Pett Levels, Rye Bay, East Sussex (see Figure 5.29)
Designations SPA, cSAC, Ramsar site, SSSI
Length 8km
Width Shingle beach 40m-60m wide.
Crest Shingle crest 6.5mOD. Embankment crest 7.6mOD.
Sediment Shingle
Tides 6m on spring tides
Waves Waves from the southwest predominate.
Drift c.19,000m^3yr^{-1} from west to east.
Classification Drift aligned shingle barrier beach.
Intervention Timber breastwork, groynes, clay embankment, concrete blocks, recycling

Figure 5.29 Location of Pett Levels

5.11.1 Introduction

The 5-mile long shingle ridge protects the Pett Levels against flooding. Behind it runs the coast road from Hastings to Rye (Plate 5.36). At its eastern end there are a series of shingle inland of the active shingle beach. Some of the shingle ridges have been covered over with soil and no longer support "stony habitat". Other ridges remain bare and support the perennial vegetation that is found on stony banks. The most seaward banks support annual vegetation that grows each summer on the high tide lines of sand and shingle beaches.

The shingle ridge has been declining and would be subject to both realignment and to landward retreat. Eventually it would take on a more acute plan shape and become swash aligned.

While there was a strong movement of shingle from west to east along this frontage the shingle ridges were relatively healthy. With a dwindling supply of shingle from the west the ridge is becoming increasingly more vulnerable to breaching. To the west the Hastings harbour forms a major barrier to littoral drift. Thus, while the beach west of Hastings has accreted that to the east is eroding. The eroding cliffs between Hastings and Cliff End are providing little input of coarse sediments into the system. Therefore, the Pett frontage east of Cliff End is starved of coarse sediment and is vulnerable to breaching.

The seaward projection of the cliff line at Cliff End, at the western end of the ridge, and the terminal groyne at Rye Harbour, at the eastern end, provide a "cell" that is ideally suited for beach recycling operations. Following serious beach erosion and deterioration of the flood defences, an initially high transfer of shingle from the eastern end (near the harbour arm) was made to the western end of the frontage near Cliff End. This has been followed by annual recycling at an average rate of some 19,000m^3yr^{-1}. The shingle has traditionally been removed from just updrift of the harbour entrance. Hence, the operations have not only helped maintain the beaches along the Pett frontage, but have also reduced the risk of the mouth of the river Rother from being blocked with shingle (though infill with sand does occur).

Plate 5.36 Pett Levels

5.11.2 Geomorphological context

The development of the shingle accumulation at Pett Levels is related to the development of the Dungeness foreland. However, although the shingle accumulations at Pett Levels and Dungeness are interrelated, they are now separated by the entrance to Rye Harbour and thus form separate physiographic units.

The shoreline was shaped by rising sea level during the Holocene. A large bay was formed by post-glacial sea level rise, which can be traced by the line of inland cliffs that extend from Cliffs End to Sandgate. These cliffs are now fossil features.

Reworking of seabed sediments and their onshore movement produced a series of sand and shingle bars that coalesced (possibly as they were fed by longshore transport) producing a long thin spit that was more or less continuous across Rye Bay (May and Hansom, 2003). Fine sediments would have been deposited in the lee of the bar, providing the impetus for marsh development.

In the 13th century a series of storms fragmented the barrier beach and the shingle ridges at Camber were destroyed. The ness at Dungeness was thus separated from the beaches to the west (May and Hansom, 2003). From that time the shingle spits were deflected into the mouth of the Rother by wave action. Thus, spits developed westwards from Broomhill (from the direction of Dungeness) and eastwards from Winchelsea, narrowing the mouth of the estuary of the river Rother.

Spit development was more rapid on the west side of the estuary mouth, due to the high littoral drift from the west. This accumulation produced a series of relic ridges that are present to the landward of the present “active” beach zone (see figure 1). A relic spit that is found well to the landward of the present coastline provided the foundations for Camber Castle, which was constructed in 1539.

When the river was canalised in the 19th century the area east of the entrance mouth became separated from that to the west. The shingle ridges west of the entrance continued to accumulate, but those to the east declined. The ridges east of the entrance are now covered by dunes, due to onshore sand transport from the wide, sandy beach at Camber Sands.

5.11.3 Flood defence / coast protection role

The shingle ridge protects low-lying land from being flooded, as well as providing protection to the western side of the entrance to Rye Harbour. Historic monuments, including a Martello tower that is located close to the harbour entrance, are also protected. The ridge also protects the Hastings to Rye coast road.

5.11.4 Evolution

Average recession rate

Information about the long-term recession rate of the barrier ridge is not readily available. The tendency for shoreline retreat is evidenced by the erosion of the lower foreshore (see Plate 5.37), even if the upper foreshore is maintained forward of the natural line of the coast by recycling shingle.

Plate 5.37 Erosion of the lower foreshore

Back barrier

For much of its length the barrier ridge is backed by a seawall, with little evidence of overtopping. At the eastern end of the frontage a series of ridges have developed as shingle has accumulated on the west side of the entrance to Rye Harbour.

Short term (storm event response)

Beach erosion in front of the embankment, but no significant overtopping as the ridge is backed by an embankment.

5.11.5 Management

Intervention

The sea defences at Pett Levels protect 390 houses and some 700 hectares of low-lying farmland. In the 1930s the shingle ridge was breached just east of

Cliff End. To prevent further breaching, two lines of timber breastwork were constructed, the landward one being impermeable and the seaward one being permeable (Minikin, 1952). Groynes were also placed along the frontage.

In the 1940s a clay embankment was constructed to prevent wave overtopping. This was faced in the 1950s with concrete blocks called Essex blocks, leaving a relatively thin shingle beach to the seaward.

Subsequently the frontage has been maintained by shingle recycling from the accretion on the western side of Rye Harbour. Maintaining the shingle beach is important as the seawall is of relatively lightweight construction and is likely to provide an insufficient standard of defence should the recycling operations cease.

Issues

The shingle beach at Pett Levels has been maintained by recycling now for about 50 years. A study of the existing defences in this area revealed that the current defences provide only a 1 in 10 year standard of protection. The EA has put forward a new plan to increase this standard, proposing to construct new groynes and to carry out renourishment with shingle.

The traditional donor area for shingle extraction is the shingle ridge west of the harbour entrance into Rye Harbour. The area has several European conservation designations (cSAC, SPA and Ramsar) for which the requirements of the Habitats Regulations apply. Natural England therefore has to be consulted about any plans that include disturbance of the shingle banks. EN considers that shingle recycling is likely to lead to loss of vegetation and likely to disturb nesting bird communities on the immediate backshore.

A recent scheme proposed by the Water Authority, involving shingle recycling, has therefore necessarily been amended to concentrate shingle extraction in a local area close to the western arm of Rye Harbour. Here the shingle is already disturbed and extraction less likely to affect conservation interests than extracting along the length of the foreshore, as was planned initially.

The Pett Sea Defence Scheme also includes the construction of a secondary flood embankment inland, which would fragment the agricultural holding of Rye Harbour farm. This farm has been purchased and the land is to be used to provide “compensatory habitat”.

Habitats Regulations require an Appropriate Assessment to be made when a proposed scheme is likely to have a significant impact on a site with a European conservation designation. The purpose is to determine whether the scheme would have an adverse impact on the designated site. An Appropriate Assessment must be made by a decision-making authority, which provides consents for the scheme proposed. Such an authority is termed a Competent Authority. In this case the Environment Agency, as the scheme proponent, had to provide the necessary information by which the Competent Authority could make the Appropriate Assessment. Therefore, EA commissioned various environmental studies, which had a significant effect on the scheme design. This included the control of vehicle movements and limiting extraction to September to February, thus minimising the disturbance to nesting birds). Compensation is to be provided to make up for damage to/loss of perennial vegetation of the pebble ridges. From various options the best solution was found to be the restoration of damaged habitat in Rye Harbour Farm. The farm consisted of salt-marsh and shingle ridges before being converted to agricultural land. Some of this land can be converted back into pebble ridges.

The Environment Agency has entered a 50-year agreement to manage the area in an appropriate manner in accordance with the Habitats Regulations. The objective is to restore the vegetation communities specific to shingle ridges.

The area has further potential for habitat creation and restoration and may therefore accommodate compensatory habitat for other Flood Defence schemes in the area (the Dungeness scheme, for example).

5.12 Westward Ho!

Location	Taw Torridge estuary, North Devon (see Figure 5.30)
Designation	SSSI
Length	3km
Width	Reducing from about 48m width in 1884 to about 25m by 1954.
Crest	7 to 7.5mOD
Sediment	Cobbles
Tides	Spring range 7.9m
Waves	Atlantic waves modified by attenuation.
Drift	South to north at a rate of about 3000 to 5000m^3yr^{-1}
Classification	Swash aligned cobble barrier beach.
Intervention	Groynes, masonry walls, recycling

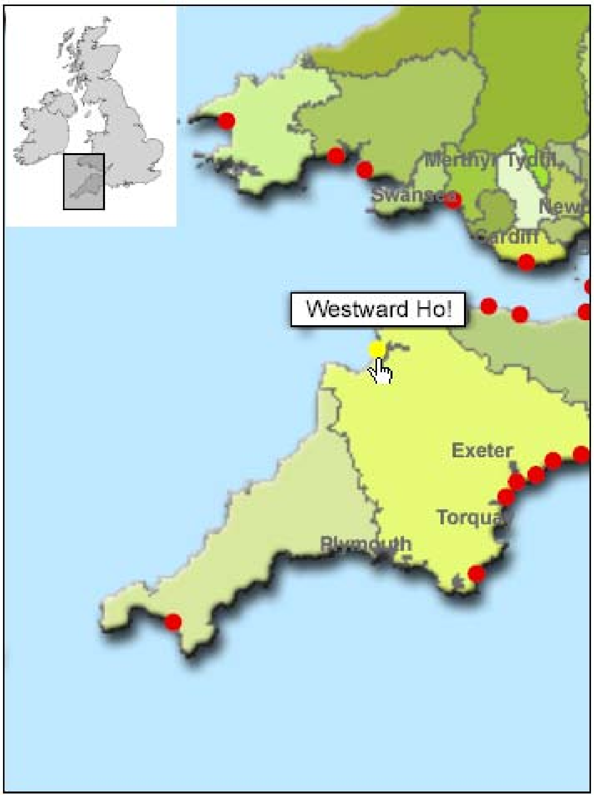


Figure 5.30 Location of Westward Ho!

5.12.1 Introduction

The massive cobble beach (Plate 5.38) extends north-eastwards into the Taw Torridge estuary from is mainland connection at the village of Westward Ho! This village was founded in 1863 and named after Charles Kingsley's classic novel of Elizabethan seafaring adventurers.

The cobble beach is backed by low sand dunes called the Northam Burrows. The cobbles are transported northwards by wave action. Near the distal end the ridge the cobble beach merges into sand dunes at Grey Sand Hills. Some cobble material is dispersed northwards into the low water channel of the Taw Torridge estuary.

As the cobbles are transported northwards so the accretion at the northern end at Grey Sand Hills is at the expense of erosion over much of the length of the cobble ridge.

The volume of the cobble ridge is reducing with time, with the net loss being estimated as of the order of 1500 to 5000m^3yr^{-1} on the eroding part of the ridge (May and Hansom, 2003)..

Plate 5.38 Cobble ridge at Westward Ho! (Natural England)

5.12.2 Geomorphological context

The cobble ridge is considered to be a fossil feature, having very little contemporary supply of material. The origin of the ridge is not known (Hansom and May, 2003), but its formation would have been completed by the time of the period of reduced sea level rise after the latest period of glaciation. The ridge has been supplemented with material derived from the cliffs westward, which extend to Hartland Point. (The erosion of cliffs and rock platforms, together with the transport of Pleistocene pebble deposits provided a source of material). Any such supply is far less than the volume required to make up the losses due to longshore transport along the cobble ridge itself.

5.12.3 Flood defence / coast protection role

Protects the low dunes of Northam Burrows. Protects the south side of the estuary against wave action and helps to preventing the low water of the Taw Torridge from migrating southwards.

Figure 5.31 below shows the historic movement of the cobble ridge and a simplified cross section of the ridge.

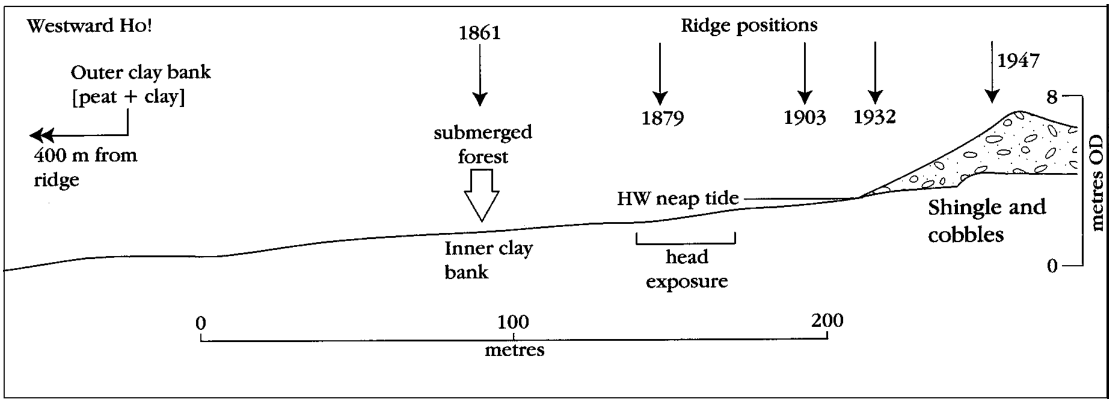


Figure 5.31 Historical movement of the cobble ridge (from Hansom and May, 2003)

5.12.4 Evolution

The average recession has been about 1.5m per year, but appears to be accelerating (see Figure 5.31).

Figure 5.32 below shows the possible evolution of the ridge as estimated by Natural England.

Figure 5.32 Possible future evolution of Westward Ho! (English Nature)

5.12.5 Management

Intervention

Groynes have been used to reduce the rate of longshore transport, although this has failed to maintain the cobble ridge in front of the village of Westward Ho! Here the narrowing beach is now backed by masonry walls.

Since the early 1980s recycling has been carried out from the distal to the proximal end of the cobble ridge. The moving of some 3000 to 5000m^3 of cobbles has been insufficient to maintain the alignment of the ridge, and it has continued to retreat.

In recent years the formerly accreting distal end has also begun to retreat.

5.13 Dawlish Warren

Location	Dawlish Warren, Exe estuary ,South Devon (see Figure 5.33)
Designation	NNR, SAC, SPA, Ramsar site*,* SSSI, LNR
Length	2.5km
Width	About 500m
Crest	Backed by sand dunes up to 6m high.
Sediment	Sand, size 0.19 to 0.28mm
Tides	Spring range 4.1m
Waves	South-easterly gales on spring tides most likely to cause breaching, whilst south-westerly and westerly waves tend to heal the breach by producing high longshore transport.
Classification	Drift aligned sand spit.
Intervention	Groynes, gabion baskets,

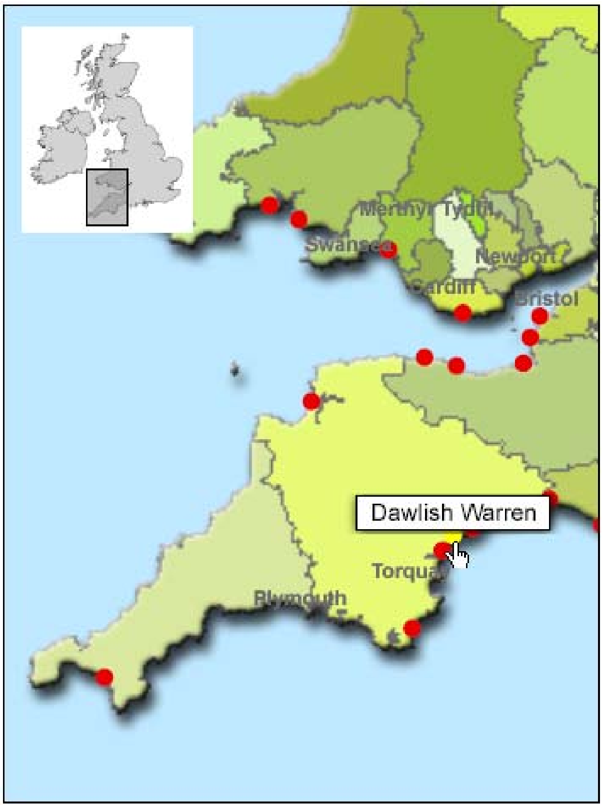


Figure 5.33 Location of Dawlish Warren

5.13.1 Introduction

Dawlish Warren (Plate 5.39) is a sand spit and dune system that has developed on the west side of the mouth of the Exe estuary. It is connected to the mainland near Dawlish, while its distal end is opposite Exmouth. A smaller spit also developed from the eastern shore of the estuary at Exmouth, at the same time as the Warren was formed. However, this smaller spit has been developed, so that it is no longer a natural feature, no longer being able to respond to regime changes. Plate 1 shows an aerial view of the Warren in

1960. The smaller spit, mirroring the Warren, can also be seen, on the extreme left hand side of Plate 5.39.

Plate 5.39 Dawlish Warren c1960

The Exe estuary mouth has a large triangular, asymmetrical, ebb tidal sand delta, whose complexity is well illustrated in Plate 5.39. This has a large store of sand, which can easily be transported onshore in the right wave conditions, being almost connected to the shoreline.

Tidal flows in the estuary mouth are ebb dominant. In the narrows off Warren Point the flood stream can reach 3 to 4kn, while on the ebb speeds can exceed 4.5kn as the banks uncover.

Landward of the estuary mouth there is a large flood delta composed of shingle over which there is sand cover (Futurecoast, 2002). Within the estuary itself there are interconnecting sand banks and channels, which are mobile, so that the movement of sediments there is extremely complex. On the margins of the estuary there are extensive inter-tidal flats and salt-marsh. Those to the landward of the Warren are particularly well sheltered from wave action and unaffected by the strong tidal currents that are present in the low water channel. These mudflats are accreting.

The Warren itself is a complex feature, containing two parallel sand spits, with dune slack between. Most of the Warren is a nature reserve, which is jointly owned and managed by Teignbridge DC and the Devon Wildlife Trust. It contains a wide range of coastal habitats, from mudflats to sand dunes. The NNR has more than 600 plant species, including the rare Warren crocus. Humid dune slacks support the petalwort and liverwort. The area around Warren Point attracts a large number of wading birds at high tide. More than 180 species of birds are recorded here every year, including waders such as black-tailed godwits, curlews, greenshanks and sandpipers. Wintering birds include Brent geese. The mudflats of the Exe estuary contain beds of eelgrass and host an abundance of invertebrates, including mussels, providing rich feeding grounds for wintering waders and wildfowl.

5.13.2 Geomorphological context

It is considered that the principal sources of sediment for Dawlish Warren were the eroding sandstone cliffs to the west of the Exe estuary (Kidson, 1950). Erosion of the cliffs at the end of the Holocene probably supplied the sediments for spit formation, though some question marks remain about its provenance. Much of this supply has been cut off by the construction of the seawall that protects the railway line, which runs at the foot of the cliffs between Dawlish Warren and Teignmouth. The offshore seabed has only a thin (averaging about 5cm deep) covering of mobile sediment, mainly of sand, so the amount of sediment that could be transported shoreward is limited also.

Historically the Warren has been retreating landwards and a measure of the more recent retreat is seen in the “set back” of the beach line east of the sea defences adjoining the railway line near Langstone Rock (bottom right hand corner of Plate 5.39). The greatest net, i.e. long-term, losses of sand from the seaward side of the Warren are probably into the estuary itself. This can take by wind blowing sand onshore, or waves transporting it alongshore, eastwards along the face of the spit and around its distal end at Warren Point, and then into the estuary on the flood tide (the estuary is a sink for both sand and mud).

In addition to any net losses or gains there are also significant (shorter-term) circulations of sand at the mouth of the Exe estuary. Sand reaching the distal end of the spit can either be deposited there, or carried into the main, ebb-dominant tidal channel. The majority of the sand entering the channel is carried onto the ebb delta, from where it can be driven landwards by wave action. By this mechanism the longshore drift eastwards along the face of Dawlish Warren can be supplied by sand coming ashore from the ebb-tide shoal. (A similarly complex sediment circulation system is present at Teignmouth).

5.13.3 Flood defence / coast protection role

Dawlish Warren protects the western shore of the Exe estuary and the coastal railway line against wave action and flooding.

5.13.4 Management

Intervention

Dawlish Warren consists of two parallel spits, the Outer and Inner Warren, possibly developing separately as a result of changes in the sediment supply. Prior to 1932 Outer and Inner Warren spits were separated by a tidal inlet, whose mouth was opposite Exmouth. This was called Greenland Lake. Subsequently longshore sand transport carried material around Warren Point (the distal end of the Outer Warren) to create a narrow neck of land joining the Outer to the Inner Warren. The neck remained vulnerable to breaching and in March 1962 the low-lying area between the Inner and Outer Warren was flooded. In the autumn of 1963, the Outer Warren was breached again. Emergency works included filling the breaches with sandbags.

Studies were subsequently undertaken into possible ways of strengthening the Outer Warren (HR Wallingford, 1965). As a result, existing timber groynes were lengthened and additional ones were constructed. Gabion baskets filled with stone were also constructed at the dune foot at the eastern part of the spit (Plate 5.40).

Following these works the rate of erosion at Dawlish Warren appears to have diminished until quite recently. Longshore transport has in fact caused the distal end of the Warren to grow, so that the threat of breaching at the link between the Inner and Outer Warren has actually decreased with time.

The Warren has been unaffected by serious erosion for nearly 40 years. The recent erosion of the dune face in October 2004 took place under a quite exceptional combination of waves and high tide levels. This particular storm also caused erosion at South Coast locations, including East Head (see case history). As a result of this erosion some of the gabion baskets near the westward end of the Warren (constructed some 40 years ago) were damaged.

However, soon afterwards the beach began to recover and immediately downdrift of the seawall at the proximal end of the spit beach levels had increased by an estimated 1m by March 2005.

Thus, the sand spit appears to be capable of self-healing in the short term, despite the tendency for longer term landward retreat.

Plate 5.40 Old gabion baskets at western end of Dawlish Warren – 2005

Issues

The role of Dawlish Warren as a coast protection feature was examined by HR Wallingford for Natural England in 2005. Since the spit extends much of the way across the mouth of the Exe estuary, it and the adjoining, intertidal ebb delta shoal prevent waves penetrating very far upstream. This not only allows the mudflats to continue to accrete behind it, but also protects the coastal railway line, which runs along both the eastern and western shores of the estuary.

The model studies carried out by HR Wallingford (1965) indicated that the Warren also affects the propagation of the tide into and out from the estuary. A shorter route for the rising tide could conceivably raise tidal levels upstream and also increase the upstream penetration of the tidal wave. This could have an impact on the upstream flood defences. Fragmentation of the spit could also lead to changes in the sand banks and channels. A shortened spit would make it possible for the main channel into the estuary to migrate away from the eastern shore at Exmouth. The replacement of the one channel by a more complex system of channels would result in shallow and tortuous entrance conditions, affecting navigation.

The history of changes at Dawlish Warren indicates that while there is a tendency for landward retreat the Warren has not been affected by serious erosion for about 40 years and even after a major storm in October 2004 it has quickly healed. Despite the lack of any obvious contemporary supply of material the Warren does not show any tendency to fragment. Backshore defences that were constructed in the 1960s are still largely intact. This is a candidate site for monitoring coastline changes, as it shows a certain “robustness”. Earlier predictions that it may fragment have not been borne out by events. The role of the ebb delta in providing sediment to the spit, in particular, is worth study.

More recently, beach lowering and seawall undermining at Exmouth has led to the formation of a Management Group consisting of Natural England, Environment Agency, East Devon Council and Teignbridge District Council. This Group is currently seeking funding to examine the role of coastal defences in the sediment regime associated with Dawlish Warren and Exmouth. In the short-term, works to maintain groynes on Dawlish Warren will be progressed. The Management Group has also embarked on a public awareness programme.

5.14 Recommendations for further research

A series of case histories has been presented which demonstrates the level of interest that there is in the management of barrier beaches in the context of flood defences. Common management practices include re-profiling (using bulldozing plant), recharge and recycling of beach sediments. However, it appears that the impacts of these management activities on the governing physical processes are not fully understood, and positive outcomes from management technique application seem to be attained by trial and error rather than through any genuine scientific reasoning. It is suggested that further analyses of barrier beach management methods be carried out. Such analyses might attempt to link barrier types/ categories to practical methods, and the foundations laid for working towards best-practice guidelines.

It has been demonstrated that monitoring is a recognised form of management activity, although the extent and form of monitoring tends to vary from site to site. It may be that it is not possible to define monitoring requirements in a generic manner, with each individual location demanding site-specific measurements, but monitoring requirements, and the subsequent analyses of observations should contribute to a best-practice document.

6. Review of current management methods

6.1 Introduction

In many parts of England and Wales, barrier beaches form either the only defence against flooding of the hinterland, or a vital component of such defences. In these circumstances, and where the coastal defence management strategy for the frontage has been defined in the relevant Shoreline Management Plan as either ‘Hold the Line’ or ‘Advance the Line’, consideration will need to be given to assessing, and if necessary, reducing the risks of flooding by managing the barrier beach so that it provides a satisfactory standard of defence. This Chapter considers this requirement within the context of the PAMS Operational Framework.

At present, there is no established “Good Practice” guidance manual specifically providing guidance on managing barrier beaches. Where such management has been undertaken around the UK coastline, it has often been developed on an *ad hoc* basis. In this scoping study we have reviewed a number of mitigation techniques, but have not generally found much information on the choice of the management measures applied or on their design, costs and effectiveness.

There are many factors that need to be taken into account when considering intervention on barrier beaches to reduce flooding risks, for example:

- Level of expenditure warranted;
- Coastline length over which intervention is needed;
- The structural condition of any existing defence structure(s);
- Environmental sensitivities, particularly environmental conservation, amenity and aesthetic concerns;
- Strengths of longshore currents and drift rates; and
- Required lifetime of any measures undertaken.

It is clear from this that the choice of an appropriate management method will depend considerably on local conditions. Consequently, different methods might be appropriate for two barrier beaches along which the waves, tides and coastal morphology are similar.

In considering how best to manage a barrier beach, it is logical to start with an assessment of the existing situation, first defining the **condition** of the beach, and any existing structure(s) such as groynes or a seawall on its crest, as joint components of the coastal defence.

This should be followed by an assessment of the expected **performance** of this defence during storm events (i.e. a combination of high tidal levels and large waves), before deciding whether or not to intervene.

The following section addresses the monitoring, specifically, of barrier beaches themselves. Where the protection that the beach provides has been improved by the addition of structures such a groynes, seawalls and the like, then there will also be a need to monitor and establish the condition of these other components of the resulting “coastal defence”. This would involve consideration, for example, of the structural integrity of “built” defences, which in turn would need to relate to the changing morphology and levels of the barrier beach itself. Prior to its recharge in 1987, for example, the lowering of the shingle barrier beach at Seaford in East Sussex was leading to undermining of the seawall at its rear, hence dramatically affecting the condition (i.e. the structural integrity) and the expected performance of that seawall. The issue of beach lowering in front of coastal defence structures is discussed in a report produced under a separate Defra/ EA research project (Sutherland *et al*, 2008).

6.2 Monitoring methods

Little published guidance exists relating specifically to monitoring of barrier beaches. Some site-specific studies of barriers have been undertaken, and these have included a wide range of techniques. This review highlights features derived from some of the site-specific programmes and outlines the approach adopted for design of monitoring of barrier beaches for the south east and south west regional coastal monitoring programmes as a generic approach to programme design. Bradbury (2002) suggests a risk-based approach to design of coastal monitoring programmes in general and highlights barrier beaches as high-risk coastal features that require a high spatial resolution of coverage in order to provide adequate information for effective management.

In general, barrier beaches are characterised by large spatial variations in geometry, with features such as overwash fans and throats providing evidence of key performance characteristics. Similarly, the width and height of the barrier often varies, for example Chesil beach varies in width by 150-200m and by 6-14mOD in elevation (Bray, 2005). Barriers are also characterised by large-scale changes following episodic storm events.

Table 6.1 provides some examples of monitoring programmes for barrier beach sites.

Table 6.1 Example monitoring programmes for barrier beach sites

Site	Typical Spatial frequency of profiles	Temporal frequency of profiles	Objectives of monitoring	Other observations	Hydrodynamic measurements	Analysis
Chesil beach	Sample areas close to access points with groupings of	Biannual and post storm (from 2006)		Annual LIDAR (0.5m resolution from 2006) Photogrammetry 1993, 1994	Internal flow monitoring of standpipes. Real time Wave buoy from 2006	Annual report related to beach monitoring Risk analysis from 2006
Slapton	200m	Biannual	Inform operational beach management	Annual aerial photography. Orthophotos every 3 years. LIDAR (every 2 years)	Real time Wave buoy from 2006	

Medmerry		Biannual and post storm (from 2003)	Inform operational beach management plan development of beach management plan			Annual report related to beach monitoring Risk analysis from 2006
Hurst Spit	100m	Biannual (since 1987) plus post storm (since 1989)	Inform operational beach management plan development of beach management plan	Annual aerial photography. Orthophotos every 3 years. LIDAR (every 2 years) Habitat mapping (6 years) Spot heights (annual)	Real time Wave buoy since 1996, tide gauge-, wave models	Annual report related to beach monitoring Risk analysis
Cley-Salthouse	1000m	Biannual	Long term trends			Advisory group review
Porlock	200m	Biannual and post storm (from 2006)	Post breach evolution study		Real time Wave buoy from 2006 (Minehead)	

6.2.1 Baseline surveys

In general, a detailed baseline survey using techniques such as LIDAR should be conducted to identify weaknesses (e.g. locations with low crest levels or atypically small cross-sectional areas) and the variability in geometry of barrier beaches. The plan resolution should be at 1m or better. Even at this scale, steep features will not be well defined; this can present a problem when examining steep and narrow beach crests. Such an approach is excellent for identification and mapping the evolution and the geometry of geomorphological features such as throats, fans and canns. LIDAR surveys can provide a vertical survey accuracy of typically ±0.1m.

Alternatively, a more precise baseline survey can be conducted by using standard topographic survey methods such as RTK GPS to collect sufficient spot height data for subsequent creation of a ground model. However, these surveys are likely to be more expensive. Subsequent sampling of beach profiles can be determined on the basis of the baseline variability. Successful management programmes have typically included regular measurement of profiles at spatial intervals of 50-200m.

A baseline topographic survey of the barrier should ideally be accompanied by a geomorphological description of the site; this should include the following.

Description of plan shape of overwash features e.g. fans, throats
Delineation of the intersection of the beach with solid geology, e.g. shore-platform

Identify the plan location of the beach crest
Identify the plan location of run up berm crests
Description of the solid geology (e.g. substrate, shore-platform, outcrops)
Description of beach sediment grain size distribution (surface and at depth)
Identify plan location of MHWS, MLWS and ideally seaward toe of the beach
Strand line – visual
Ground model of site

Aerial photography can provide a useful backdrop to this since the morphological features and profiles can be placed in context. Ideally, map-accurate ortho-rectified imagery should form the basis of this. Such imagery is now widely available within England and Wales.

Regular aerial photography surveys can be of considerable value in determining the long-term evolution of barrier beaches. The minimum contact scale recommended for analogue data capture is 1:5000, although several well established programmes collect data at scales of 1:2500-1:3000. A number of studies have successfully tracked evolution of barrier sites using geo-rectified aerial surveys at epochs of about 10 years (Box, 2005). Data is available for many sites dating back to about 1940. The well-established southeast regional monitoring programme provides orthophotos every three years; data production is at 10cm, which provides extremely detailed coverage.

Grain-size data can be of general use in analysing barrier beach performance, although sampling is not generally required on a regular basis. Applications may include:

Input to beach plan-shape and cross-shore process numerical models
Design of physical models, and
Design of beach recharge schemes.

6.2.2 Beach profiles

Since the barrier response under storm conditions is linked with its cross-sectional area and crest elevation (e.g. Bradbury, 1998), regular monitoring of widely spaced survey profiles (1km) are unlikely to provide adequate information about the response of the barrier, unless it is highly uniform in cross section, which would be extremely unusual. However, a number of monitoring programmes have collected beach profiles at this coarse spacing. The main problem in the contexts of flood management and morphological evolution with this level of resolution is in gathering sufficient representative detail to identify key weaknesses that may result in overwashing or overtopping. The precise spacing of profiles needs to be refined on a site-by-site basis and considerations should be given to the local variability of the cross-section defined on the baseline survey. Weak points are often evident, for example where there is a hard feature that interrupts the barrier, or where the wave climate varies alongshore sufficiently to result in differential rates of longshore transport and potential formation of tears or build up.

The most frequently observed problem with historical barrier beach profile data sets is that the initial survey invariably tends to extend insufficiently far landwards to allow closure of the beach profiles on a solid base when rollback occurs. This means that the volume of mobile material within the beach cannot be assessed accurately. This is highly significant when the back barrier geometry varies rapidly, for example when the beach fringes a lagoon, saltmarsh or rising land (Plate 6.1).

Plate 6.1 Variation of the back barrier geometry at Hurst Spit (Bradbury, 1998)

The baseline survey should ideally allow for anticipated evolution during the course of the projected programme. Rollback rates averaging 0.1-3m per year have been identified at sites investigated, but single episodic events resulting in displacements as great as 80m have been measured (Bradbury and Powell, 1992). Ideally, the baseline survey should allow for such gradual evolution and episodic events. This may mean extending the baseline at least a hundred metres landwards of the current beach position into the area on the lee side of the barrier, including the bathymetry of any lagoons fringing the barrier and establishing the underlying the solid geology. This will provide the base geometry of the leeward area over which the barrier is likely to migrate when over-washing occurs. Procedural guidance is provided by Pert et al (2004).

Location and relocation of survey profile lines has historically presented a problem on barrier beaches, which are comprised wholly of mobile features. Fixed markers have a tendency to disappear. Repeatability of profiles is a key issue. Kinematic GPS technology now allows profile lines to be defined and relocated precisely within the navigation and setting out software. The recommended procedure is to prepare data-logger line files for each profile, defining lines using start and end coordinates. Survey profiles are subsequently relocated in the field with the aid of navigation software. Fixed markers should be used at appropriate locations to provide checks. Profiles should typically be measured at visible breaks of slope and with interim spot height intervals of about 5m. Feature coding of the sediment type is also valuable, especially for determination of the landward limit of the barrier and its boundary with the back barrier zone.

Barriers are often located on partially compressed subsoil, such as salt-marsh or lagoon deposits. This provides further complications to the establishment of robust control marks, but also provides a further morphological variable resulting from compression of the solid geology by weight of the beach sediments. The effective cross-sectional area of the barrier is reduced as a result. This issue has been highlighted previously by Nicholls (1985), who suggests that compression of underlying saltmarsh deposits under the weight of beach deposits may exceed 1m over a period of ten years. This problem was addressed by Bradbury (2001), where settlement beacons were installed within a beach recharge on a barrier beach. Subsequent monitoring of the beacons has demonstrated between 0.1m – 0.6m settlement over a period of 4 years. Variability in this rate is largely dependent upon the underlying geology.

6.2.3 Bathymetry

The approach bathymetry is often the most dynamic zone of the beach profile, and is often where the largest cross-section changes occur. This zone has historically been ignored at many sites despite its importance. Ideally, bathymetric data should be collected as far seawards as the depth of closure of sediment movement. At some sites this may be as deep as 10-15m below low water. It should be possible to link topographic profiles with bathymetric profiles. Ideally these should be captured at the same time, but in practice weather conditions are rarely sufficiently benign to allow this. The need for bathymetric profiles varies. Those sites with a small tidal range and mobile sediments at the beach toe will benefit from regular (annual surveys). Other sites where there is solid bedrock within the inter-tidal zone and limited sediment below this will only require infrequent bathymetric surveying.

The temporal frequency of topographic and bathymetric surveys should be determined by reference to risk of flooding, wave and water level climate and the existing state of the barrier. Considerations should include wave climate and history of change. Most sites merit topographic surveys at least once per year. Well-established programmes that have made good use of the data in site management typically allow for biannual profiles to be collected, together with post storm surveys. Some programmes have allowed, historically, for profiles to be measured quarterly. Following review of data usage however, this has been reduced. There may be some merit in conducting surveys more frequently following significant management activities, or where sites have become susceptible to rapid changes.

6.2.4 Post-storm surveys

Perhaps the most valuable, yet the least frequently executed survey is that carried our immediately after a storm event. Regular use is made of the post-storm data at south coast sites (Bradbury, 2005) to aid decision-making regarding the need for maintenance.

Both the Southeast and Southwest regional coastal monitoring programmes have made provision for post-storm surveys, which will typically be conducted once per year, with surveys triggered by an event of 1:1 year magnitude. These surveys have proven to be extremely valuable for the management of several sites (Cope, 2005; Bradbury and Cope, 2005; Box, 2005). Surveys should ideally be completed on the first low tide following the storm event, although this is often not practical in safety terms. To be of maximum value, these surveys should be used in conjunction with measured wave and tidal data and perhaps by reference to empirical cross-shore models (Powell, 1990, Bradbury, 1998). The topographic survey should be supplemented by a descriptive account of the storm event and beach response during that event (e.g. Picksley and Bradbury, 2006). This should include a summary of the following:

Observations of evidence of overwashing and overtopping
General site description
Photographs of features developed during the storm
Account of wave and water level conditions
Comparison of beach response with predictive empirical cross-shore models
Analysis of profile response by reference to earlier surveys
Evidence of exposure or erosion of solid geology
Plan shape change
Variability of cross section along the beach
Run-up limits
Evidence of percolation through the beach

6.2.5 Hydrodynamic data

Measurement of hydrodynamic variables is necessary in order to develop an understanding of forcing conditions and in order to support or validate any numerical modelling techniques used to determine the design conditions and standards of service of the site. Such measurements should typically include

Tidal elevations
Wave conditions
Nearshore currents

A local tide gauge is extremely valuable. The gauge should be located typically on the seaward side of the barrier, since significant differential heads may occur from the seaward side to the lagoon (if present). On occasion a second gauge may be of benefit on the lagoon side of the beach. Tidal elevations should be recorded over the full tidal cycle and real time data should ideally be provided, to enable performance to be monitored during storm surges. The benefits of real time systems also enable faulty equipment to be diagnosed in a timely manner. Data should be used to determine extreme water levels, after an appropriate period of time. Similarly, harmonic analysis should be conducted after an appropriate duration (typically one year) to enable storm surges to be identified and quantified. Where percolation is an issue tide gauges may provide a very useful boundary condition for evaluation of flow through the beach.

Wave data should ideally be measured at a local site in shallow water conditions, typically in a water depth of 8-10m; this is usually appropriate to identify wave conditions close to the depth of closure of sediment movement and provides appropriate input to empirical models of profile performance (Powell, 1990; Bradbury, 2000). Real-time systems are regularly deployed and are already present at a number of existing or proposed sites that are well suited to the analysis of barriers in southern England and on the east coast (Table 6.2).

Table 6.2 Locations of wave measurement sites in close proximity to barrier beaches

Location	Barrier	Instrument	Water depth	Record length
Milford-on-sea	Hurst spit	Datawell directional wave rider	10mCD	1996-present
Start bay	Slapton	Datawell directional wave rider	13mCD	Autumn 2006-present
Lymington	Gull Island	Pressure recorder	5mCD	2003-present
Chiswell	Chesil beach	Datawell directional wave rider	14mCD	2003-present
West Bay	East beach	Datawell directional wave rider	11mCD	Autumn 2006-present
Minehead	Porlock	Datawell directional wave rider	10mCD	Autumn 2006-present
Hayling Island	Medmerry	Datawell directional wave rider	10mCD	2003-present
Pevensey Bay	Pevensey	Datawell directional wave rider	10mCD	2003-present
Southwold	Walberswick to Dunwich	Datawell directional wave rider	20mCD	Dec. 2007-present

The most useful application of the wave data at barrier beach sites is for storm event analysis, in conjunction with beach response data. As a guide, hydrodynamic measurements and analysis should provide the following:

Measured tidal elevations and surge component during storm events
Diary of storm events above defined threshold conditions
Description of storm characteristics including:

- Wave conditions at depth of closure (approximately 10m water depth)
- Event wave statistics: H_s, T_z, T_p, direction
- Storm duration (event over threshold method (Mason, 2006)
- Spectral characteristics of storm events

Long-term wave climate

- Wave statistics: H_s, T_z, T_p. direction
  - Seasonal distributions
  - Annual
- Probability distributions of
  - H_s against direction
  - H_s against T_z or T_p
  - Period against direction
- Extreme events
  - By direction sector
  - For all sectors

Offshore wave hindcast
Nearshore wave transformation

For the purposes of storm event analysis and wave climate derivation, Mason (2005) has developed guidance on procedures for definition and monitoring and analysis of wave climate data which relate well to barrier beach process monitoring.

Hydrodynamic characteristics of particular interest to barriers include spectral characteristics, for instance bimodal wave periods or directions, relative magnitude of swell and wind wave components, relationship between T_p and T_z.

6.2.6 Analysis of monitoring data

Suggested analysis of barrier beach profile survey data should include the following:

Barrier width at interface with solid geology
Measurement of barrier width at MHWS and extreme water level
Cross section area above MHWS and extreme water level
Foreshore approach slope and changes in slope
Change in cross section of profiles relative to master profile
Crest elevation
Comparison of profiles with predictive empirical models
Estimation of critical cross section required to prevent overwashing within defined storm events
Definition of alarm and crisis levels for beach performance and monitoring against these values

Trend analysis should typically be conducted for each of the above variables.

Suggested analysis of barrier beach plan shape survey data, such as aerial surveys and ground models should include the following:

Erosion of foreshore solid geology
Formation and evolution of features such as throats and overwash fans
Plan shape variability
Plan shape evolution
Rate of crest rollback
Rate of back barrier transgression
Loss of area of habitat arising from rollback

Where management of the barrier site involves introduction of beach recharge, beach recycling or beach re-profiling management logs are extremely useful. The beach management logs should contain the following information:

Recycling log
Location of material sources and deposition sites
Quantities of material moved
Grading of new material
Description of method of maintenance
Pre and post maintenance beach profiles
Photographic record of works
Dates of activities

Interpretation of monitoring data at sites where such practice is not routine may lead to misleading conclusions. Difficulties have been experienced in interpretation at sites where records are not kept (Box, 2005).

6.2.7 Flood event consequences

In the specific context of flood defence management, it is also very helpful to gather information on the consequences for the hinterland of the changes in barrier beaches, particularly during storm-events. Such information is relevant to the assessment of both the performance of such beaches during exceptionally severe conditions, and the efficacy of mitigation measures undertaken previously or close to the time of a storm event. It will assist in the calibration and refinement of predictive techniques, for example of beach crest lowering, breach formation, through-flow, overtopping and over-washing. In addition, it will help inform the choice and application of future management and mitigation measures, for example flood warning or beach recharge schemes.

While it is often not possible to provide quantitative information on all of the following, there are clear opportunities to better record the consequences of such events, thus improving both operational procedures and the design of schemes aimed at reducing flood risks. It is suggested that the following information be sought and collated as soon after storm events, whether or not any significant problems as a result of flooding were caused.

- The start and end times of any observed (groundwater) flows through the beach
- The start and end times of any observed over-topping (or over-washing) of the beach
- The location and morphology of any breaches through the beach crest
- The spatial extent and depth of any flooding
- A description of other significant problems caused (e.g. debris or sediment deposited of road, disruption to transport/ access, injuries, damage to property)
- The actions taken before, during and after the event to reduce risks to the public and damage to property;
- The staff time and costs involved in such "near event" management

6.3 Assessment of barrier beach performance

A normal pre-requisite for contemplating intervention in the natural evolution of beaches to reduce flooding risks is an assessment of the likely performance of that beach during extreme events. As discussed earlier, however, the evolution of barrier beaches during severe storm events is very difficult to predict. It follows therefore that the prediction of the amount of flooding of the area behind such a beach that might occur in such an event is even more complicated. There are three ways in which this might happen, namely by the percolation of seawater through the beach, wave overtopping of its crest and by the creation of a partial breach it its crest allowing the tide to flow over it.

This research project has revealed little in the way of standardised quantitative methods for carrying out such flood risk predictions. This is despite the substantial number of locations where economically significant flooding by one of these mechanisms has already occurred and is likely to occur again in the future. Some models can indicate the likelihood of wave overtopping, crest retreat or breaching in extreme events. However, it is concluded that, at present, any calculations of the rates and volumes of inundation need to be based on rather substantial simplifications of the actual morphological events that might occur as a barrier beach responds to such extreme events.

The case for undertaking measures to reduce flood risks in such circumstances has therefore been justified on the basis of past difficulties, perhaps extrapolated to other locations and return periods by use of rather basic modelling techniques (see Chapter 4). This is clearly an area where further review of current methods, some targeted short-term research, prototype data gathering and authoritative guidance on performance assessment is required, to both improve the accuracy and achieve consistency in the evaluation of flood risks behind barrier beaches.

Despite this weakness in the methods available for predicting flooding problems behind such beaches, it is clear that guidance will be needed on mitigating the proven risks in many situations. The following section provides a review of the mitigation measures that have been adopted to provide a starting point for future development of "good practice" and its dissemination to operating authorities.

6.4 Review of mitigation measures

6.4.1 Accommodating overtopping and flooding

If a barrier beach, or a seawall situated on or behind it, is likely to be permanently breached, with a danger of large areas of the hinterland being flooded, then substantial intervention works will be needed to reduce this threat. Various possible remedial options for this type of situation are described later in this Chapter.

However, if the result of continued evolution of a barrier beach combined with gradually increasing sea levels and perhaps larger waves will first be to decrease the performance of the coastal defence (for example, causing an increase in the frequency and rates of wave overtopping) then it may be possible to "accommodate" these effects without altering either the structure, or improving beach levels. The options available can be grouped into "emergency", "short-term" and "long term" categories.

In the "immediate" or "emergency" category, the options include

Storm warning systems to anticipate overtopping events and evacuate areas at risk (Sayers et al., 1999); and
Preventing access to areas immediately to landward, e.g. closing roads.

In many areas, the greatest threats from overtopping at high tide are to people or vehicles attracted to the sea-front to "wave watch". Deploying operational staff in good time to ensure that risks to people and properties are minimised during an event can be effective, but can also be expensive. Such actions may include closing roads, promenades, paths etc that are at risk of flooding, warning local residents and businesses, removing cars etc from areas at risk of flooding and perhaps deploying temporary demountable flood defences (from sand bags upwards).

The prediction of coastal flooding events caused by wave overtopping (or breaching of defences) is both less easily and more rarely undertaken than for fluvial flooding. In part, this is because of the rapid changes in weather that can alter wave conditions and even tidal levels within a few hours leading up to a high tide. It is also more difficult to predict the extent to which defences will protect the hinterland during such events, even if the waves and tides can be predicted to a high degree of accuracy. Severe storms will typically take several days to develop, and beaches will alter their morphology during this period. The beach levels and profile approaching the high tide during a severe storm will, in all probability, be very different from those measured during a routine beach survey even if this was carried out a month previously. The issuing of warnings and deployment of operational staff is therefore also difficult. It seems likely that remote sensing methods, such as real-time video camera observations and perhaps, as on Chesil Beach, the monitoring of groundwater levels in barrier beaches will have a role to play in “emergency response” mitigation methods.

It is likely that such “emergency responses” will also involve some “clean up” operations after an event, for example removing debris and beach sediment carried over the crest of the beach. Recording the times and dates of such events, and evaluating the costs of responding, would provide valuable quantitative information on this approach to managing coastal flood risks, to be used in deciding on and designing more permanent mitigation measures.

In the “short-term” category, the mitigation options include:

Installing secondary flood defences to limit the extent of flooding; and
Improving the drainage or storage of overtopped water.

Where wave overtopping is both frequent and substantial, it may be worthwhile making provision for managing the resulting flooding, for example by installing secondary flood defences and/or making arrangements for the safe detention or drainage of the seawater that overtops the main defences. Any secondary flood defences might be demountable and temporary or installed seasonally, perhaps in the same fashion as for fluvial flood defences.

In the longer-term category, the options available include:

Increasing the capacity of structures, surfaces, and properties behind the structure to withstand greater flows; and
Relocating major assets at risk from flooding and restricting future development.

In some cases, particularly where a defence structure or what is landward of them is easily erodible, e.g. a clay embankment, glacial till cliffs or dunes, then the damage caused by overtopping waves could be reduced by strengthening them.

All of the above methods of accommodating the effects of changes to barrier beaches, however, can only be implemented and sustained in some situations. Where these cannot provide sufficient protection against erosion of flooding risks, alternative methods may need to be taken to mitigate the reduced standard of flood defence offered by barrier beaches. Some of these methods are described the following sections. However, it is likely that for at least some of these methods, there will be advantages in also using some of the “accommodation” methods described immediately above.

6.4.2 Re-grading or profiling with mechanical plant

Re-grading or profiling of the beach with mechanical plant following storm events is one of the most frequently used management methods. The practice is now becoming less common, in the light of subsequent monitoring of beach performance (see case studies for Hurst, Selsey and Cley for example). The profiling process may be carried out from the seaward face, combing material back towards the crest, or pushing the lee face back towards the crest, or both.

The practice adopted at Medmerry and Cley has until recently focused on re-profiling of the seaward face of the beach (Plate 6.2). This approach effectively narrows the dissipative zone of foreshore between low water and the beach crest, and increases the resistance of the crest to wave activity by increasing its cross-sectional area. The depth of water at the toe of the beach is also increased, enabling larger waves to attack the beach crest.

Plate 6.2 Post-storm reprofiling of Medmerry barrier, November 1998 (Pett, Environment Agency)

Natural cross-shore sorting of a mixed sand and gravel beach generally results in the coarser fraction of the beach migrating towards the beach crest, whilst the sandy fraction migrates to the toe or the core of the beach. The combing activity of the bulldozers used to reshape the beach inevitably results in artificial mixing of the beach. The sediment mixing arising from the use of bulldozers reduces the effective beach permeability, with the fine fraction of beach material filling the voids between the larger particles. Reports of mixing of clay form the underlying geology have exacerbated this process at sites including Medmerry, Hurst Spit and Cley. This results in a more reflective beach than might form naturally.

The profile shape which develops as a result of re-grading results also in a more brittle profile that is somewhat different to the natural dynamic equilibrium profile (Plate 6.3). There is therefore a natural tendency for the profile to develop more quickly under wave attack, in accordance with the geometry suggested by Powell (1990), towards a more natural dynamic equilibrium shape.

Plate 6.3 Brittle cross shore profile developed as result of re-grading with mechanical plant (Bradbury)

The crest elevation has at many sites been reformed at a much higher elevation than the natural beach crest would form at e.g. Cley-Salthouse; this affects the way that the beach responds to wave forcing. Because the crest is much higher than would form under natural wave run-up, much of the energy that would normally be dissipated in overtopping is reflected from the beach face. This often results in erosion of the face of the beach crest and thinning of the crest ridge. Under exceptional circumstances the waves may overtop, but this often occurs in a form more commonly associated with a vertical seawall than a beach. The symptoms of such a response are:

Formation of near vertical faces of the crest ridge
Thinning of the crest ridge, and
Formation of deep runnels on the lee side of the beach occurring as a result of downward jets of water that arise from overtopping.

Effects such as the formation of lee-side runnels are also symptomatic of impact by long-period waves. Evidence of this behaviour is shown at Cley (Plate 6.4).

Plate 6.4 Lee slope runnels formed by overtopping waves at Cley-Salthouse (Bradbury)

It is, in summary, increasingly doubtful whether the simple re-profiling of the seaward face of a shingle barrier beach will provide any increase in the standard of protection it can provide in a subsequent storm event. Even if such an event followed very closely after such re-profiling, the empirical prediction method, SHINGLE (Powell, 1990), described in Chapter 3 above, suggests little or no relationship between the initial and final beach profile shape after a few hours.

The practice adopted at Hurst Spit until about 1990 focused on reformation of the lee face of the beach by pushing the beach back to seawards following an overwashing event; this effectively reverses the rollback. The implication of this action is that the reformed profile will be much steeper than the natural profile. The active profile width is similarly shortened. The problems observed are similar to those arising by re-grading from the seaward side of the beach. This practice is less frequent, although it can add to the amount of shingle and hence the cross-sectional area of the barrier, a change that will improve the capacity of the beach to withstand future storm events. In many places, however, this practice is disliked by conservationists, because it damages the natural processes and resulting geomorphological interest of the beach. Such operations would be particularly strongly opposed within designated conservation areas (SSSI, SAC etc) because of the damage to both the geomorphology and the unusual habitats for plants and animals that such wash-over deposits create.

Re-grading operations often seek to achieve defined crest elevations and crest widths. Curiously, there is rarely any evidence of a decision making process leading to the determined values. Discussions with operations engineers have failed generally to identify how the crest elevation or width is selected for these operations.

6.4.3 Beach recharge

Beach recharge is used as a management technique at many sites, including Pevensey, Medmerry and Hurst Spit (Plate 6.5).

The largest single recharge operation to be undertaken on a barrier beach was in 1996 when 300,000m^3 was placed on Hurst Spit. The design was based upon extensive physical model tests (Bradbury and Kidd, 1998). Materials used were derived from a nearby sediment sink and provided a mean beach grading close to that of the indigenous beach material. Material was pumped ashore using hydraulic discharge from a pipeline and then trimmed to design profiles using mechanical plant.

The beach crest was set at an elevation approximately 1m higher than that required to withstand a design extreme event, without overtopping. The additional beach material placed allowed for settlement of the beach into the underlying solid geology, which comprises saltmarsh deposits. Settlement beacons, which have subsequently been monitored have identified that approximately 0.75m of settlement has occurred in that time; this compares with original estimates of 1m.

Plate 6.5 Placement of beach recharge materials at Hurst Spit (1996, David Bowie Photography)

During the early stages of post-contract monitoring clear evidence of the artificial mixing of the recharge sediment became apparent with crest cliffing evident as the new material was reworked (see Plate 5.25 Hurst Spit case study). Natural reworking, without further intervention on the seawards face has resulted in the formation of well sorted foreshore fronting the engineered beach crest (see Plate 5.26 Hurst Spit case study). Cliffing still does occur on a periodic basis, following major storm events, when sections of previously un-worked recharge sediments are exposed. By contrast segments of beach where the placement of material was by rainbow discharge on to the lower foreshore, and which have not been reworked with mechanical plant, have never experienced cliffing.

Design procedures for the recharge were based on site specific physical model tests that enabled comparison of pre-recharge profiles with a range of alternative beach recharge schemes. These were each tested with a range of wave and water level conditions to indicate the anticipated profile and longshore beach response. The 3-dimensional physical model test replicated most of the processes observed in the field, including overwashing and overtopping conditions (Plate 6.6a, b).


(a)


(b)

Plate 6.6 Physical model reproduction of (a) overtopping and crest build up and (b) overwashing (Bradbury)

Hydraulic model studies have shown that the response of barrier beaches is very sensitive to small changes in crest elevation and width (Bradbury, 1998). Small changes in crest detail may result in very different response under similar hydrodynamic conditions. Construction of the crest at an appropriate level and width may therefore require some adjustment in conjunction with monitoring. Post recharge trimming of the crest presents one management option. This enables overtopping to occur at a site which has become reflective due to its elevation. Management of the crest at an appropriate elevation is particularly challenging, particularly when it is desirable to allow some overtopping, but the recession rate is to be controlled. This is currently best refined in parallel with detailed monitoring programme, since there are no techniques available that enable the rate of recession to be predicted by relation to the barrier geometry. The beach slopes should be based on the natural slopes of the indigenous beach material, except when the source material varies significantly in grading from the indigenous material.

Placement of beach material may be on either the seaward or leeward side of the barrier. Both options have advantages. Placement of beach material low on the seaward side of the barrier (with the crest below mean high water) enables the material to be reworked quickly by wave activity, thereby improving the grading, sorting and hence permeability, but losses of material are likely to occur quickly. Placement of material on the landward side of the barrier enables the pre-recharge dynamic equilibrium slope to be maintained and also reduces the initial rate of loss of beach materials from the recharge. When the barrier has receded sufficiently far landward to reach the recharged section of beach the initial response may often result in cliffing of the sediments until the material has been reworked sufficiently.

It is generally desirable to allow some overtopping of the beach to occur under extreme events since this process will assist with natural sorting, encouraging the development of a more permeable beach. Although run-up elevations can be modelled (predicted) there is a tendency for Bradbury's (1998) model to over-estimate the actual crest elevations: the most reliable guide to the local natural crest-elevation, for the time-being, is to determine this from monitoring of a nearby healthy section of beach, which has similar exposure and foreshore characteristics.

In the event that recharged materials are placed high above the natural crest elevation, wave run-up will not reach the crest and re-sort materials and wave reflections will occur, often resulting in formation of a steep upper beach face and draw down of material onto the lower section of beach (Plate 6.7)

Plate 6.7 Steep upper beach face and draw-down of material onto the lower section of beach at Cley (Bradbury)

The design process essentially steps through the following sequence:

Identify design wave and water level conditions (numerical models)
Identify threshold conditions resulting in overwashing for the pre-recharge beach
Examine alternative beach cross section designs under various combinations of extreme wave and water level conditions
Identify maximum anticipated crest elevation based on extreme conditions
Determine rate of loss of sediments by longshore transport
Determine alarm and crisis cross-sections for the design storm conditions
Determine the necessary capital volume of recharge material required to last for the design life
Identify planned interim maintenance recycling and recharge activities
Conduct ground investigations to examine instantaneous and long term loading impacts on subsoil (CPT and odometer)
Allow for beach settlement into underlying solid geology
Identify critical conditions for beach cross-section
Fine-tune profile crest-elevations and beach cross-sections during monitoring programme
Monitor beach performance

Many barrier sites may require smaller-scale recharge. In situations where the total quantity is less than about 200,000m^3 other delivery options may be more economic than pipeline discharge; these include bottom dumping, rainbow and delivery by lorry for relatively small quantities (typically less than 100,000 tonnes).

Beach recharge is often used in combination with other techniques, particularly in conjunction with re-profiling and recycling.

6.4.4 Beach recycling

Beach recycling may be a sensible option at sites where longshore transport rates are relatively high. The longshore transport rates are typically very low on swash aligned barriers relative to drift aligned barriers. In some instances a sediment sink may occur at the distal end of a drift aligned spit where accumulations are useful for recycling. Recycling may often be fraught with environmental difficulties arising typically from the presence of vegetated shingle, which restricts haul routes from excavation to deposition sites.

Recycling presents similar difficulties to beach recharge in terms of placement of materials (Section 6.4.3). Suitable sources for recycling can often be identified in conjunction with a good monitoring programme which would highlight rates of accumulation.

6.4.5 Beach control structures

Structures are occasionally constructed to manage specific elements of the barrier. Examples for some of these are presented in Section 5. Such structures include:

Headland structure	Hurst Spit
Groynes	Selsey, Pevensey
Offshore breakwaters	(Elmer, West Sussex)
Crest road	Slapton
Crest gabion cage	Chesil beach
Beach drainage	Chesil beach

The use and design of such beach control structures on barrier beaches is not fundamentally different to that for any other beach, although there may be greater dangers in producing uneven beach widths, for example in groyne bays, on a barrier beach that is already narrow. In addition, such structures may affect access along a barrier beach much more than on other types of beach.

6.4.6 Environmental management

The management target for some barrier beaches includes or is mainly to maintain the geomorphological interest and ecology of the site. Management often warrants minimal intervention. This is the case for example over most of the length of Chesil Beach and the Fleet (the lagoon to landward of it). The lagoon system that is protected by the shingle barrier beach between Torcross and Strete Gate (Slapton) in Devon has similar conservation status. Natural evolution is desirable under such circumstances such that the designated conservation status of the area can be maintained or improved.

The volatile nature of the response of barrier beaches to episodic storm events results, however, in considerable loss of areas of ecological interest during such events. Estimated losses of habitat arising from rollback at Hurst Spit, prior to beach recharge, were 7,500m^2 per year. These have now been significantly reduced, but at a cost of managed intervention, which has reduced the geomorphological value of the site.

As there are currently pressures to maintain the extent of certain habitats at many sites, additional pressures are placed on site management. In such circumstances there may often be a conflict between allowing natural evolution and preservation of features with significant conservation status. The loss of certain key habitats will occur if barrier sites remain unmanaged, particularly saline lagoons and rare shingle vegetation. In the event that the barrier beach is managed, the geomorphological significance is likely to be reduced.

In order to better manage these sites and to predict the potential losses of habitat, as a result of natural evolution, there is a need to conduct detailed monitoring programmes; these must focus on the prediction of beach recession rates across the habitats in relation to storm events, and also over longer time scales in conjunction with sea level changes. Such geomorphological evolution must be monitored in parallel with detailed and regularly updated habitat mapping. Further predictive tools are required to develop this capability. Assessments of predictive techniques to evaluate the quantity of recession that might be expected to occur in a single storm event and over extended periods of time are also required.

Recent developments in management of the Cley-Salthouse shingle ridge, for example, are seeking to move away from a managed beach scenario; with an aspiration to revert to a natural system. There is currently some uncertainty relating to how the management changes that have occurred will manifest over a period of time.

6.5 Recommendations for further research

The management of barrier beaches in order to maintain or improve their reliability and resilience, as well as the standard of flood defence they offer, is a complicated subject area. The main difficulties relate to assessing the likely performance of the barrier during severe storms and then, if appropriate, choosing and implementing management methods that are acceptable from both economic and environmental viewpoints.

In regard to the assessment of barrier beach performance, particularly during severe events, there is a particular need for field data collection to assist in the development and validation of predictive modelling methods. Some existing models can indicate the likelihood of wave overtopping, of crest retreat or of the breaching of barrier beaches in extreme events. These models can therefore provide an indication of the condition of such beaches, thus allowing attention to be focused on those where intervention is likely to be most urgent. Even so, there are no models available, as far as we know, that can predict the likelihood and approximate intensity of the groundwater through-flow, and this may be sufficient at some locations to cause significant flooding of the hinterland.

There are much greater difficulties in accurately estimating the rates and volumes of flooding that might occur if a barrier beach is overtopped or partially breached. Present methods make substantial simplifications of the actual morphological events that might occur as a barrier beach responds to such extreme events, and of the processes of overtopping or flow through partial breaches. A review and assessment is therefore needed of the existing methods that have been and are used to calculate the risks of flooding landwards of barrier beaches. The future improvement to such methods will be greatly assisted by validation using information from the monitoring of the changes in barrier beaches during storm events and of the consequences such as the extent and depths of flooding.

There are numerous sites around the coastline of England and Wales where the problems of flooding behind barrier beaches are managed on an “emergency” basis. It is recommended that information on these schemes is also sought, in the first instance to assess the effectiveness and costs of coastal flood warning systems and emergency responses e.g. closing roads, installing demountable secondary defences and evacuation. The “post-event” actions needed at such locations, for example the pumping out of flooded areas and clearing up debris and sediments washed over the beach crest would also be worth reviewing at the same time.

The first benefit of collecting this information would be to collate, assess and disseminate good practice. However, it is also recommended that guidance is drawn up for the monitoring of future similar events, so that better information on the performance of barrier beaches can be accumulated, e.g. times of recorded through-flow, overtopping or breaching to assist in validating warning systems and extents and amounts of flooding to assess the severity of the risks associated with such events.

7. Conclusions and research needs

7.1 Conclusions

The observations made at the case study sites, the analyses of existing techniques, and the review of process understanding, have highlighted some major deficiencies in the predictive tools available for barrier beaches.

Tools for management are generally weak, primarily as a result of uncertainty with regard to process understanding. In particular there is limited capability associated with flood-forecasting arising from either overwashing or breaching. Environment Agency asset management teams have expressed concerns that high-risk sites are vulnerable, but that there are no flood-forecasting tools available to predict whether breaching of the crest will occur, and whether flooding is likely.

A specific request by the Environment Agency has been made that any proposed further research suggested here be centred upon current Environment Agency requirements. Whilst the scoping study itself has been centred on process understanding in accordance with the tender specification issued by Defra, the proposed research, although still firmly and unavoidably centred upon improved process understanding, is placed in to the context of flood-defence performance-based management (summarised in the following Section).

7.1.1 Barrier beaches in the context of flood-defence

The Risk Assessment of Flood & Coastal Defence for Strategic Planning (RASP) and Performance-based Assets Management Systems (PAMS) research programmes, funded by Defra and the Environment Agency, have pioneered the concept of addressing flood-risk in a performance-based manner. PAMS is specifically designed for the identification and prioritising of works needed to manage existing flood defences.

Recently, significant advances have been made in understanding the concepts underpinning a risk-based approach to flood management, for example the DEFRA / EA R&D Report, SR587, entitled Risk, Performance and Uncertainty in Flood and Coastal Defence – A Review (2002) www.environment-agency.gov.uk/commondata/105385/fd2302_c1.pdf.

This has built on the Government's standard "Source / Pathway / Receptor" approach to risk management (see Figure 7.1), establishing the concept of a tiered approach to risk-based decision-making.

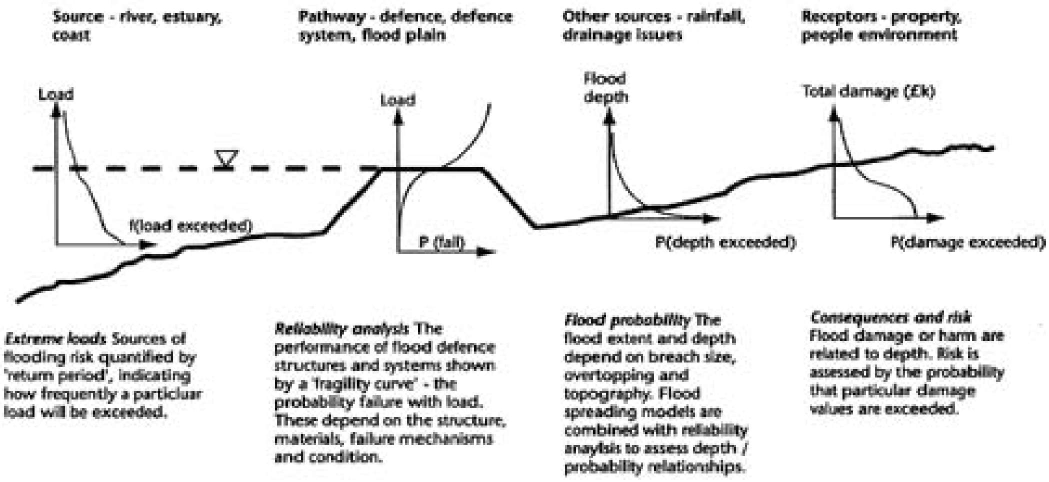


Figure 7.1 Source / Pathway / Receptor / Consequence model for flood risk

Barrier beaches can often act as flood defences, and as a consequence have to be managed. It is therefore pertinent to express the proposed research topics in terms of PAMS so that the whole-life costs of flood-defences can be better estimated by the Environment Agency.

The Environment Agency's strategy for Sustainable Asset Management in Flood Risk Management is underpinned by the need to understand how materials (such as barrier beach sediments) and components (such as barrier beaches as Pathways) within assets might change with time, or how they perform.

Performance can also be considered in terms of structure (e.g. a risk of breaching) and geometry (e.g. cross-sectional area or through-flow), and where management is required, understanding that performance is an aim. How barrier beaches as pathway components in the flood-risk model might change, or rather how they perform, can be further categorised according to descriptors such as reliability (fragility), resilience, and deterioration:

- Reliability is a measure of the ability of the flood-defence to perform as required, and is expressed in terms of fragility. In the case of barrier beaches, this might refer, for example, to the likelihood of a barrier breaching under a specific loading.

- Resilience is a measure of the flood-defences ability to self-heal. In the case of barrier beaches, this might refer to the natural repair of a breach in the barrier following a severe storm, rather than the ability of the barrier to resist breaching.

- Deterioration is a term allocated in flood-risk assessment to the long-term change in effectiveness of a flood-defence. Since these flood-risk assessment concepts were developed to express the state of <u>static</u> defences (such as embankments and seawalls), the assumption is that the older a defence becomes, the less able it is to provide the defence standard that it was designed for. The result is therefore a long-term reduction in the reliability of a defence, hence deterioration. In the case of barrier beaches, which in this context are dynamic flood-defences, it is considered perhaps a little presumptuous to assume that long-term changes in barrier beaches will automatically result in a deterioration of flood-defence standard (Chesil Beach has existed for millennia, for example, and still provides a flood protection role). It may even evolve that a changing barrier beach could actually increase its reliability, or perhaps become more resilient. Deterioration is used here, therefore, as a means to purvey long-term change, rather than to imply a reduction in performance.

The further research suggested here relies upon the notion that the performance of barrier beaches in terms of flood risk assessment can be divided in to short-term response (reliability and resilience) and long-term deterioration. For example, a short-term response might be a lowering of the crest height under storm conditions. A long-term process resulting in deterioration might be exemplified by the gradual roll-back of a barrier beach causing thinning and a reduction in the barrier's cross-sectional area in response to sea-level rise.

The following section seeks to place the perceived uncertainties and weaknesses in our knowledge and understanding, as realised through this scoping study, under the key headings of Reliability, Resilience and Deterioration. It is not entirely possible to make such a rigid division of processes since short-term performance is often dependent upon long-term performance in a dynamic system. Nevertheless, an attempt has been made to draw some distinction.

7.2 Perceived weaknesses

This section is intended to highlight the gaps in our understanding that have become apparent through the review and consultation carried out in the scoping study. They reflect the current poor understanding of barrier beach processes and are all issues which affect efficient flood-risk assessment. As a result, effective management strategies are difficult to define.

7.2.1 Uncertainties relating to Reliability

The following points are raised in relation to the short-term response of barrier beaches, and many of these can be viewed in the light of fragility (i.e. what is the probability of failure under a particular loading).

What conditions will cause a barrier beach to overtop?
How much overtopping will occur?
What conditions will cause a barrier beach crest to be lowered?
How is a breach likely to form?
How does the beach material (and its grading) affect the profile change?
How might permeability change as a result of management?

7.2.2 Uncertainties relating to Resilience

The following points are raised in relation to the short-term response of barrier beaches, and many can be viewed in the light of resilience (i.e. the ability of the barrier to “self-heal”, or otherwise).

How is a breach likely to form and be sustained?
Will a breach remain open if no active management is taken?
If the crest of the beach is breached will it reform and how quickly will that be?

7.2.3 Uncertainties relating to Deterioration

The following points are related to the long-term behaviour of barrier beaches.

How quickly will a barrier beach migrate?
What factors affect the rate of barrier beach migration?
What factors will cause the barrier beach to migrate more quickly?
How does the underlying geology affect the beach evolution?
What is the impact of rising/falling land on the migration rate?
How does longshore sediment supply/rate impact on beach performance?
How will anticipated sea level rise affect the barrier evolution?
How will the anticipated increases in the height and/ or frequency of extreme wave events affect the barrier evolution?
How might anticipated changes in mean wave direction affect the barrier evolution?

7.2.4 Pathway management tools

The following issues all relate to tools that may or may not be at the disposal of managers. They highlight the need for improved understanding and communication.

Predictive tools: what are available, are they robust, and are they appropriately applied?
Can breaching or through-flow events be forecast?
How does engineering work affect beach performance?
What will happen if breach/crest lowering is not repaired?
What management options are available?
Is beach scraping or reprofiling a sound management practice?
Why do beaches form steep seaward faces following re-grading (or re-profiling)?
How can water be removed quickly if a breach occurs?
Monitoring (see below).

7.2.5 Receptors

Although not examined in great detail by this scoping study, the report touches upon several issues which are of relevance in the source-pathway-receptor model.

Will overtopping or breaching events be damaging to land or property?
Will they affect other human needs/ desires such as access, safety, amenity, aesthetics, water quality, for example?
How will the local habitats to landward be altered by an increasing frequency of overtopping, overwashing and roll-back?

7.2.6 Monitoring

The current drive towards improved monitoring of assets, including source, pathway and receptor components, is recognized as a step towards achieving improved knowledge and understanding. Research into best-practice for monitoring methods is not addressed in detail in this scoping study as this topic is dealt with elsewhere in relation to the coastal environment in general. Nevertheless, some issues are raised here which are specific to barrier beaches.

What should be monitored?
How often should monitoring be carried out?
What are the most appropriate monitoring techniques?

7.3 Recommendations for future research

Through an examination of those issues reported in the preceding section, and consideration of the review of the dynamic processes delineated in Section 1.2 (a-g), it is apparent that there are indeed shortfalls in the understanding of processes relating to barrier beaches (and in some instances, all beaches). The Scoping Study has highlighted the gulf in understanding between those processes occurring on sandy coastlines and those occurring on coarse- and mixed-sediment coastlines. Since the majority of barrier beaches around England and Wales consist of these coarser sediments, recommendations are focused solely around these barrier-types.

Evidence from the case histories and issues raised through review and consultation indicate that the basic understanding of pathway component processes is not in line with “end-user” requirements. There is a clear requirement to improve our knowledge of the responses of barrier beaches particularly in storm events, and of the consequences, and a need to review and improve predictive models of their performance. “Best Practice” management guidelines are also needed.

During the course of this study, the project team have discussed the best way forward and it is suggested, that a future research programme be established which concentrates on examining the processes of barrier beaches through experiment and monitoring, with subsequent development of reliable and robust predictive models. The development of Best Practice guidelines would evolve in stages, starting with the review of monitoring and mitigation practices presented in Chapter 6.

It would be necessary to phase the programme such that experiment and monitoring research could usefully inform the development of predictive models. As has been made apparent from the scoping study, typical techniques which are used to investigate barrier beaches include monitoring, physical models and numerical models (where this category includes empirical and parametric models). The recommended research programme expects to make further use of such techniques, and no need for investigating the development or use of alternatives is envisaged at this stage.

The following sections identify topics suitable for investigation in the short- to medium- term, and are ordered in terms of perceived priority rather than relating directly to the discussion of perceived weaknesses presented in the previous section.

7.3.1 Phase 1: Improving pathway component process understanding

Initial review of study methods

The first phase of research should review state-of-the-art methods for studying barrier beaches, using the review presented in this Scoping Study as a starting point. These methods will include, but not necessarily be limited to; physical modelling, numerical modelling and the gathering, collection and analysis of field data.

It is likely that there will still be a need, as there is now, to produce appropriate data relevant to barrier beaches, and that that data can be obtained through physical modelling and monitoring. The following sub-sections describe appropriate research topics.

Physical model experiments

Physical model investigations of barrier beach processes are required to develop reliable flood forecasting tools that are able to estimate flooding arising from overwashing, through-flow, and also the processes influencing the evolution of the barrier crest such that the onset of breaching can be better understood. These investigations should ideally be undertaken at large scale in order to examine the response of shingle and mixed sediment barrier beaches, which are the most frequently occurring. Experiments conducted in scale models could reasonably be expected to enhance our understanding of reliability through improved fragility curves.

Other advantages of increasing the available dataset of concurrent morphodynamic response and associated hydrodynamic forcing conditions include the value that these add to the development and range of applicability of parametric and deterministic predictive tools.

Gathering and analysis of field data

There is some considerable merit in establishing a national database, perhaps allied to the National Flood and Coastal Defence Database (NFCDD), containing details of barrier beaches including beach profiles, aerial surveys and LIDAR. Simultaneous measurements of forcing conditions (waves and tides), through-flow, permeability and over-topping would also be required. Subsequent analysis of data could be based upon on procedures adopted by the Channel Coastal Observatory for beach management in southern England and in context with the framework provided in Section 6.2.

There is generally a shortfall of data relating to specific storm events. Hydrodynamic data describing barrier performance under extreme conditions, such as measurement of volumes of water overtopping or flowing through barriers should be obtained. Information regarding waves and tides corresponding to overtopping and through-flow events should be gathered simultaneously. A broad study including many sites is needed in order to provide detail at sites where the geometry and grain size are wide ranging and where conditions are variable. The case study sites (Chapter 5) illustrate this variability.

The nature of barrier beach evolution is such that storm events are episodic and planning of a short-term field-based programme would not guarantee results within a defined timescale. The rarity of such data, however, means that obtaining records during just a single event could be regarded as a success. The existing field monitoring programmes (funded by DEFRA) could be refined to provide appropriate levels of data. On-going data collection as part of the southeast and south west regional coastal monitoring programmes could provide appropriate site-specific data to assist in this particular research objective. The long-term deployment of waverider buoys and tide gauges can provide hydrodynamic input to this programme.

There is a considerable quantity of raw data becoming available that could enable a description of the performance of barriers to be developed at both decadal scales and, to a lesser extent, storm event scale. Much of this data has not been analysed previously in context with barrier performance or management. A considerable proportion of this data is already held by the Channel Coastal Observatory (for southern England).

Data obtained by through-storm measurement of nearshore waves and gravel beach morphology, using shallow angle LIDAR for example, would prove pioneering and provide valuable insight into the behaviour of barrier beach faces. A research programme such as this would benefit from support by the Environment Agency and contribute to further understanding some of the processes active on barrier beaches.

Such investigations could usefully be combined with laboratory tests under controlled conditions to focus on testing and development of more robust and wide ranging predictive techniques.

7.3.2 Phase 2: Improving pathway component prediction tools

Refinement/ development of existing/ new tools for predicting performance

The main method of assessing condition and performance of barrier beaches will be through the collection of further field data and refining existing or developing new predictive methods. The first phase of recommended research (described above) relates to the collection of further data as required to aid the refinement of existing numerical models, or the development of new numerical models, which is the recommended topic for the second phase of recommended research.

Existing numerical models include SHINGLE (Powell, 1990), the dimensionless barrier inertia model (Bradbury, 1998) and ANEMONE (Dodd et al., 2000) and any others which may come to light as a result of the Phase 1 of research.

Numerical modelling

Predictive tools are currently very limited in scope and application. Tools which are actively applied consist of Powell's (1990) SHINGLE model, and Bradbury's (2000) dimensionless barrier inertia model. Other tools which are not in routine application include the OTTP-1D process-based numerical model, developed by HR Wallingford as part of the ANEMONE suite (Dodd et al., 2000) with funding under MAFF commission FD0204.

The data collection and analysis research (Phase 1 of the proposed research above) is intended to lead to improved understanding of the pathway component processes. It is expected that the datasets and the improved understanding gleaned through analysis of the data will contribute to improved predictive tools. As these tools develop, so too will process understanding.

Bradbury's dimensionless barrier inertia model provides a first approximation for the prediction of overwashing threshold conditions; this can be refined further, by the selective testing of conditions close to the overwashing threshold, under more closely-controlled conditions, with minimal spatial variability (of the barrier profile). Near prototype-scale random wave-flume studies would: (a) aid the development of confidence in the modelling methodology; (b) minimise the scale effects; and (c) provide confirmation of the functional relationships over the lower part of the barrier profile (these cannot be measured, practicably, in the field). The influence of shingle grading on barrier-crest evolution should also be examined. Future development should be supported by the large-scale physical model tests and field investigations suggested as part of the Phase 1 research.

Other empirical frameworks developed for sand beaches could be examined further, but these are generally even less sophisticated.

The SHINGLE model, the use and development of which has been reported extensively during this review, while not strictly applicable to barrier beaches, could nevertheless be augmented by the proposed monitoring. The SHINGLE model is simple to apply, and is currently being applied to solve barrier beach problems. Developing this rapid spreadsheet model, therefore, would be a justifiable task with the potential to offer a low-cost solution to some of the problems faced during flood-risk assessment.

Development would include giving the model the ability to be run repeatedly for a variety of different loadings, with increased capacity to represent the overwashing process derived from physical modelling tests and monitoring – essentially deriving a probabilistic risk-based method of assessing the performance of barrier beaches. Output from the model could be expected to consist of an improved expression of the fragility of the barrier, which could then be used to better inform the RASP-type flood inundation analysis.

The focus of the short-term future numerical model development is therefore based on using Bradbury's barrier inertia framework or Powell's SHINGLE model as a starting point. However, there is no reason at this time to disregard the possibility of the development of a process-based numerical model over the medium-term. The basis for such a model could be the MAFF-funded (now Defra) OTTP-1D model developed by HR Wallingford, for example.

The OTTP-1D model was built to simulate surf-zone hydrodynamics over porous beaches, and provided predictions of overtopping rates assuming an immobile beach, and accounts for permeable structures. To further develop this model (or even the 2D version, OTTP-2D) towards full morphodynamic capability would enable detailed process-based deterministic assessment of barrier beach design in relation to standards of flood defence. Whilst this implies considerable time and resource investment, it is nevertheless considered a worthwhile task, and would result in a generic industry standard tool for assessment of likelihood of breaching of barrier beaches.

The feasibility of the 2D option predicting the development and spatial variation of the plan-shape of a shingle barrier beach due to the combined influence of longshore transport and overtopping should be investigated. The sensitivity of the barrier profile response to spatial variation of the barrier geometry should be examined in the systematic assessment of 3-dimensional response; this would require an extensive test programme to provide statistically-valid data.

The longer-term aim should be focussed on the development of a methodology to represent barrier beaches within a broad-scale systems model (such as that being developed under the FLOODsite and FRMRC research programmes) such that longshore connectivity and cross-shore processes are considered in tandem. Useful tools and concepts developed under the RASP methodology, such as fragility and resilience, could serve to enable such a model development whilst maintaining practical computational effort and user operability. Methods for prediction to be developed in research proposed under Super Work Packages 2 and 6 of FRMRC2 (to commence in 2008) into breach may be relevant, and should be reviewed as necessary.

7.3.3 Providing effective and efficient management guidelines for the pathway component

Although it would be convenient to label the provision of management guidelines as a “Phase 3” research programme, i.e. to wait for a significant advancement in understanding before issuing guidance, the reality is that there is an urgent need for advice and methods in the short-term. It is suggested that first research need is a “Best Practice” document which focuses on the use of existing methods and understanding, including monitoring practice. Tools for the development of site-specific schemes of management are adequate for shingle beaches (physical models) but there is no generic guidance available that assists with the design of suitable beach geometry to enable the beach evolution to be controlled adequately.

The fundamental difficulty is to assess the volume of water passing over/ through a barrier beach under a given scenario (i.e. defined wave/ tide conditions and perhaps an assumed future beach profile). Over a longer-term it may be necessary to judge when a barrier beach will retreat over an important hinterland asset such as a coastal road. The former will need the “predictive tools” mentioned above which will involve the acquisition of information on past events that caused problems, together with the database resulting from the proposed monitoring. The second issue can be addressed through analysis of beach profiles, maps, and any data on the episodic nature of crest retreat with some degree of success.

In many cases, the first thing coastal managers will want to do is assess the need to manage a barrier beach. This will typically need a flood-risk assessment (more rarely an erosion risk assessment), which will provide an indication of the requirement to “manage”, or otherwise.

Given a good reason to manage the beach, i.e. showing that it needs maintaining or improving to reduce flood-risk (or more precisely at this stage to improve defence standards), then one can turn to deciding what to do. It is unlikely that much can be done about the source, although improved wave/ tidal prediction can be made with site-specific data collection. Similarly, the receptor could be improved so that it could accommodate greater flood/ erosion risks (e.g. move the asset out of harms way), but the source and receptor behaviour is beyond the remit of this scoping study.

To improve our management of the pathway component it is proposed, as a first step, to improve our knowledge of the pathway. This step is aimed at providing more detailed data collection (Phase 1) and modelling (Phase 2), e.g. short-term intensive measurements of beach response to validate predictions of morphological changes under “normal”, rather than extreme, forcing conditions enabling “weak points” along the barrier to be identified.
The next option is then to consider intervention methods, as discussed in Section 6. These include:

Recycling beach material from over-stocked to under-stocked areas
Beach re-profiling – but ideally recovering material from the landward side rather than scraping the front face upwards
Adding temporary or permanent crest level enhancement (gabions etc.)
Adding a seawall or rock revetment partly buried in the crest or to the rear
Adding beach sediment – either small-scale trickle charging using construction or excavation waste or large-scale operations.

These methods would be examined alongside the enhanced predictive tools discussed above, with due consideration of the whole-life costs and environmental impacts. These might include compensating for coastal squeeze if the barrier is prevented from retreating.

Present methods make substantial simplifications of the actual morphological events that might occur as a barrier beach responds to extreme events, and of the processes of overtopping or flow through partial breaches. A review and assessment is therefore needed of the existing methods that have been and are used to calculate the risks of flooding landwards of barrier beaches. The future improvement to such methods will be greatly assisted by validation using information from the monitoring of the changes in barrier beaches during storm events and of the consequences such as the extent and depths of flooding.

There are numerous sites around the coastline of England and Wales where the problems of flooding behind barrier beaches are managed on an “emergency” basis. It is recommended that information on these schemes is also sought, in the first instance to assess the effectiveness and costs of coastal flood warning systems and emergency responses e.g. closing roads, installing demountable secondary defences and evacuation. The “post-event” actions needed at such locations, for example the pumping out of flooded areas and clearing up debris and sediments washed over the beach crest would also be worth reviewing at the same time.

The first benefit of collecting this information would be to collate, assess and disseminate good practice. However, it is also recommended that guidance is drawn up for the monitoring of future similar events, so that better information on the performance of barrier beaches can be accumulated, e.g. times of recorded through-flow, overtopping or breaching to assist in validating warning systems and extents and amounts of flooding to assess the severity of the risks associated with such events.

A revised “Best Practice” guide would evolve in much the same way that has been proposed for the Beach Management Manual II. If time-scales were to permit, a short-term review of current management practice could form part of the Defra/EA revision the Beach Management Manual II. Revisiting the online consultation method initiated as part of the present scoping study would provide assistance.

7.3.4 Cost estimates for recommended research

Clearly it is very difficult to provide accurate estimates of the costs of the recommended research. However, the research can be divided into broadly 3 categories:

- Physical modelling plus a review of existing methods for predicting performance
  - This may cost in the region of £100,000 to £150,000 dependent upon exact scope.

- Review of monitoring data/ procedures
  - This is likely to cost in the region of £75,000 - £100,000
  - Implementation of appropriate monitoring cannot be costed until such as review is made.

- Development of numerical models combining above
  - This may cost in the region of £150,000 to £250,000 depending on the exact scope.

- Producing Best Practice guidelines
  - Such guidelines are an evolving process, but to scope their development, it is thought that £30,000 to £40,000 would be required in the first instance.

Justification for these costs can be made through consideration of the possibility that there may be of the order of 50km of barrier beach where flood risks might be reduced by improved management practice. If an assumption is made that coastal defence schemes might work out to cost £2000 to £5000 per metre every 5 years, then there is a £2M to £5M saving in costs if the guidance saves 5%, and ten times that in benefits.

So the research might save £20m in 5 years, and if the costs are of the order £0.5M – then a Benefit/Cost ratio of 40 can estimated.

7.4 Delivery tools

A dedicated Web-site (mapping and pooling UK experience) has been established as part of this Scoping Study. Findings from this research programme will be made available on the Web. The address of the Web-site is:

http://www.barrierbeaches.org.uk.

During the course of the Scoping Study, managers and academics, together with other interested parties, were invited to contribute to a pooling of experience of barrier beach management around England and Wales. A proforma to ease the provision of information was made available. In the event, there was poor response to the request for provision of information in this manner, with only one form returned completed. A more effective method of obtaining the required information was found to be through meeting those managers with direct experience of barrier beach management.

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