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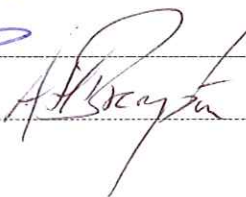
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Summary

Beach Control Structures, Poole

Alternative coastal defence options, Sandbanks to Branksome Dene Chine

Report EX 5200
November 2005

This desk study has been commissioned by Poole Borough Council to review alternative options for maintaining appropriate beach levels, following recharge, for the Poole Bay frontage, specifically from Shore Road, Sandbanks to the Borough boundary near Branksome Dene Chine. This report forms the first stage of a study into the future and long-term management of coastal defences along this frontage. In their recent Poole Bay Strategy Study, Halcrow (2003) concluded that long rock groynes with a spacing: length ratio of 2:1 combined with periodic beach recharge was the best option, although the modelling carried out in that study concentrated on the main Bournemouth frontage further east.

There are concerns that the application of this defence strategy may be inappropriate for the Sandbanks to Branksome frontage as the beach profile shape here is generally flatter, and there is less longshore drift. A recent groyne construction/ beach recharge scheme at the western end of Sandbanks (i.e. to the west of Shore Road) has been successful, but this site is influenced by strong nearshore tidal currents, and therefore this may not be the optimum scheme for the study frontage further east where such currents are unimportant. There has also been considerable interest in the possibility of alternatives to “traditional” groynes along the remainder of the PBC frontage, given the low tidal range. One possibility, for example, is to use submerged reefs that would have a smaller effect on the aesthetics and amenity value of the beaches. This study therefore considers a range of “novel” beach control structures along with similar groynes to those found already in Poole Bay.

Previous work by HR Wallingford, in 1994 and 2000, and more recent survey data, is used to assess coastal processes and beach morphology within the study area. In places, present beach levels are too low to protect the seawall from even a storm with a 1-year return period, especially in the Branksome Chine and Flag Head Chine areas, even though recharge has already been applied. As a result of this, plans are already underway for a further beach recharge, using sand from the intended deepening of the Poole Harbour navigation channel, to improve beach level. In order to retain this extra sand and maintain adequate beach widths, there is a need to decide on improvements to or replacement of the existing short timber groynes that have not worked particularly well as beach control structures.

In this study, past experience of using a range of coastal defence structures has been reviewed, covering groynes (permeable and impermeable), breakwaters and reefs both in the UK and worldwide. This includes work carried out by HR Wallingford by two other consultants working under sub-contract. One sub-contract reviewed beach control structures in Italy, on sandy beaches with similar tidal ranges to Sandbanks. The other was carried out by a consultant based in New Zealand and this includes a review of submerged multi-purpose reefs, and makes recommendations for a scheme using these to improve beaches along the study frontage. Conclusions from these reviews are used to provide guidance on the optimum use of different structures for this specific frontage, and their likely effects on beach evolution.

Summary continued

Based on the above reviews, and the knowledge of the study frontage, a shortlist of ten outline beach control structure schemes has been drawn up. The types of structure, their lengths and spacings have been derived using the conclusions of the reviews, together with previous modelling of wave climates and sediment transport.

The structures proposed to control the evolution of the recharged beach include impermeable groynes (rock or timber), permeable groynes, near-shore reefs and multi-purpose reefs. These short-listed options are subjected to a multi-criteria analysis, assessing them on a variety of weighted criteria, with greatest priority given to maintaining adequate beach widths, slowing the longshore drift and the schemes' construction costs. Consideration is also given to maintenance costs, aesthetics, public safety and past experience. The comparison scheme used is transparent and can be repeated with different weights given to the various assessment criteria.

From this analysis, a scheme using 75m long impermeable groynes, at a spacing: length ratio of 4:1 scores highest, with other impermeable groyne options and the proposed multi-purpose reef scheme also scoring highly. The best three of the schemes have been recommended for more detailed study and modelling during the next phase of the study.

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Appendix 3	Review of Beach Control Structures in Italy
Appendix 4	Beach Profile Analysis

1. Introduction

This report describes an initial study carried out to assess suitable options for beach control structures for the Poole Bay frontage of the Borough of Poole in Dorset. This project was commissioned by Poole Borough Council (PBC) in late October 2004, and is the first stage of a study into the future long-term management of the coastal defences along that shoreline.

1.1 BACKGROUND

A recent Coastal Strategy Study (Halcrow, 2004b) considered a range of options for the future management of the coastal defences in Poole Bay, covering the whole coastline from Durlston Head to Hengistbury Head. The main conclusion reached was that the most attractive option for the PBC portion of that frontage, from the viewpoints of technical feasibility, economics and environmental acceptability was to maintain adequate beach levels by periodic sand recharge schemes. This strategy study also concluded that in order to increase the effective life span of each such recharge, it would be necessary to install beach control structures such as groynes, thus reducing the losses of sand from this frontage.

As part of that study, some modelling was undertaken of alternative types of groynes, i.e. rock and timber structures of various lengths and spacing (Halcrow, 2004b). It was concluded that rock groynes of about 150m length and 300m spacing were likely to be a good choice. However, this modelling concentrated on the central part of Poole Bay (in the vicinity of Bournemouth Pier) and their preferred groyne layout may not be optimal for the coastline further west, i.e. between Sandbanks and the Poole/ Bournemouth boundary, where beach slopes are flatter and longshore drift rates are smaller.

Prior to this strategy study, a number of coast protection schemes were carried out at the western end of the frontage at Sandbanks, in order to remedy an urgent erosion problem along the Sandbanks frontage between the Haven Hotel and Shore Road. These schemes comprised beach recharge operations, using sand dredged from the outer part of the entrance channel to Poole Harbour, and the installation of shorter rock groynes than recommended in the Poole Bay strategy study, and at correspondingly closer spacing. These measures have proved to be successful in creating a much wider beach and appear to be popular with beach users. However, some concerns about the safety of beach users and bathers have been raised by the RNLI, because the groynes project into the East Looe Channel, where there are strong tidal currents.

Along the remainder of the Borough's (Poole Bay) coastline, i.e. from Shore Road to Branksome Dene Chine, the levels of the sand beaches in front of the seawall need to be maintained or improved to provide an acceptable standard of defence. The main concern is that low beach levels may lead to undermining and collapse of the seawalls. The present short timber groynes along this frontage are not very effective in controlling longshore drift, and in places the beaches in front of the seawall can be very narrow, especially in the winter months.

Following discussions with Poole Borough Council, Defra (Regional Engineer, Taunton) and Bournemouth Borough Council, it was agreed that a brief desk study should be undertaken to review alternative methods of maintaining satisfactory beaches along this coastline. The frontage studied stretches from Sandbanks (Shore Road) to the

Poole/ Bournemouth Borough boundary and assumes that intermittent beach recharge operations will be carried out in the future.

As well as considering groynes, there is interest in the possibility of employing other types of structures, for example “reefs” i.e. low-crest breakwaters that may be submerged at some or all states of the tide, to control the evolution of the beach plan shape. This study therefore considers reefs and detached breakwaters as well as more traditional groyne schemes.

A further issue relevant to the optimisation of beach control structures relates to their positioning relative to the main beach access points, i.e. Shore Road, Flag Head Chine and Branksome Chine. It may be preferable from the viewpoint of aesthetics and public safety to minimise the number of beach control structures, and position them carefully in relation to these access points.

Finally, any scheme for improving beaches along the Poole Borough frontage will need to be designed bearing in mind the present (and possible future) groyne system along the adjacent Bournemouth frontage. The long timber groynes just east of the Borough Boundary may not need replacing for 10-15 years. However, the works to improve beach levels along the Poole Borough frontage are planned for the next few years.

The present study has therefore considered a range of beach control structures, of different lengths and spacing, taking into consideration their advantages and disadvantages in terms of aesthetics, public safety and their effectiveness, as proven elsewhere in similar situations.

1.2 AIMS

The overall objective of this initial study is to recommend between three to five viable alternatives for beach control structures. These would be subject to more detailed consideration, including modelling of their likely effects on beach plan-shape evolution, in later stages of the design process. The objectives of this initial study were:

- To review previous experience, in the UK and overseas, of the use of groynes, reefs and breakwaters for controlling the evolution of sand beaches;
- To review both constraints and opportunities for installing beach control structures along this frontage to reflect aesthetic, amenity and public safety objectives;
- To assess the comparative costs of alternative beach control structures, of various dimensions, that would be appropriate for use along the coastline between Shore Road, Sandbanks and the Poole/ Bournemouth boundary;
- To produce outline designs for 3-5 alternative coastal defence schemes, of approximately equal cost, to control the evolution of a recharged sand beach; and
- To make recommendations for the more detailed study and refinement of up to five beach management schemes, to be agreed by the Council. It is anticipated that this more detailed study will include numerical modelling of the long-term evolution of the coastline.

1.3 METHODOLOGY AND REPORT LAYOUT

This study was carried out in a number of steps, each described in a separate chapter of this report, as summarised below:

Chapter 2

A wide-ranging literature review, to provide guidance on successful control schemes for similar beaches in the UK and overseas, together with two sub-contracted reviews carried out in New Zealand and Italy with the same objectives.

Chapter 3

This section of the report provides a review of information relevant to the design of defences along the frontage between Shore Road, Sandbanks and the boundary between Poole/Bournemouth Boroughs. This included reviews of:

- The recent Coastal Strategy Study (Halcrow, 2004b) with particular regard to the choice and evaluation procedures used for the beach control options, including the estimates made on the cost and efficiency of those structures;
- The 1995 “Poole Borough Coastal Strategy Study” report produced by HR Wallingford, which includes information on wave conditions, drift rates etc; and
- Data collected by the Channel Coastal Observatory on beach and nearshore seabed profiles.

Chapter 4

This section assesses a range of possible beach control schemes, based on the specific objectives for beach control structures, using information and analyses summarised in Chapters 2 and 3.

An initial short-list of 10 options has been drawn up, any of which seem capable of being successfully used along the study frontage. However, each of these options would require lengthy refinement at the next stage of the design process, to assess their costs and to optimise their placement and dimensions to produce the best beach plan shape. This is a lengthy process, and it was therefore decided to produce a ranking of the short-listed options by using a multi-criteria analysis, assessing them on a variety of weighted criteria, with greatest priority given to maintaining adequate beach widths, slowing the longshore drift and minimising the schemes’ construction costs. Consideration is also given to maintenance costs, aesthetics, public safety and past experience. The comparison scheme used is transparent and can be repeated with different weights given to the various assessment criteria.

From this exercise, the three most promising options have been selected for possible further consideration.

2. *Review of knowledge: beach control structures*

2.1 CLASSIFICATION OF BEACH CONTROL STRUCTURES

The present study, as mentioned above, seeks to identify and review a wide range of viable schemes for controlling the long-term evolution of the beaches between Sandbanks (Shore Road) and Branksome Chine. In line with Defra guidance, the project does not make any initial assumptions regarding the type of control structures that might be suitable. It does assume, however, that the frontage will be intermittently recharged with suitable sand to replace that inevitably lost from the beaches, mainly under the influence of a predominantly eastward longshore drift. Inevitably, given this wide remit, some of the options considered will be novel, while others will be already familiar to coastal communities around the UK.

Although there are many different types of structures designed and employed to modify the long-term evolution of beaches, these can be divided into four main categories, namely:

Simple groynes i.e. narrow structures built roughly perpendicular to the shoreline, and designed to alter longshore currents, and hence the transport of beach sediments, leading to changes in the plan-shape of the beaches. These structures generally have little direct effect on wave heights or directions at the shoreline;

Fishtail or T-head groynes i.e. groynes which have a Y- or T-shaped head at their seaward end, designed to affect both currents and waves, and providing a degree of shelter from wave action to the shoreline on both sides of their landward end. Such structures have been built using large rocks, and can be regarded as a combination of a groyne and a (short) detached breakwater;

Detached or Offshore Breakwaters i.e. artificial “islands” that break the water surface at some or all states of the tide. These structures are generally elongated parallel to the shoreline and are designed to substantially reduce wave energy along the coastline in their lee. They also alter wave directions in their lee and along the adjacent “unprotected” parts of the coastline; and

Reefs i.e. artificial “banks” or “mounds” built on the seabed, and designed to affect the waves passing over them, changing their direction and usually causing some wave breaking. These structures alter wave directions (and wave heights) along the coastline in their lee.

Some breakwaters built on around the coastlines of the UK, and elsewhere, fall between two of these classes. An example would be a detached breakwater with its crest only breaking the water surface at the lowest tidal levels.

2.2 DESIGN PARAMETERS FOR BEACH CONTROL STRUCTURES

2.2.1 *Simple groynes*

Of these four classes of structure, “simple groynes” are by far the most common around the UK coastline, and have the longest history. It is easy, for example, to find accounts of such groynes that were built 250 years ago, and the earliest examples on our coastlines probably date back more than 500 years. This long period of usage and development has resulted in a wide variety of groynes, although those designed as

impermeable “walls” of planking using tropical hardwood remain the most common type encountered. Numerous other materials have been used including mass concrete, sheet-steel piling or steel tubing, bitumen or asphalt, gabions, i.e. wire-cages filled with stones and, more recently along the south coast of the UK, randomly placed rock. There have also been groynes designed to be permeable, i.e. to reduce longshore currents but not block them entirely, and the two concrete “Makepeace Wood” groynes slightly east of Bournemouth Pier are an example of this type of construction.

The main design characteristics of a groyne scheme can be summarised as their:

- Length (i.e. distance from the crest of the beach to their seaward end);
- Spacing (i.e. distance apart along the shoreline); and
- Profile (i.e. the height of the crest of the groyne above the beach surface).

Note here that the length, and especially the profile of any groyne, varies according to the changing profile and width of the beach. Generally in designing a groyne system, the length of groynes is measured relative to the intended beach crest position, e.g. after an initial recharge.

Other design characteristics include their:

- Permeability (i.e. the extent to which currents and sediment can travel through them);
- Orientation (i.e. some groynes are built at an angle to the beach perpendicular); and
- Roughness/ wave absorption (i.e. the extent to which they dissipate the energy of waves and currents they encounter).

2.2.2 *Fishtail, T-head or other groyne types*

The increased use of imported rock for use in coastal defences has allowed the construction of much larger groynes than had been feasible using timber. Some rock groynes can be described as “simple groynes” in that they are long and narrow, but even these have some capacity to absorb wave energy rather than reflecting it as in the case of timber or mass concrete groynes of the same length. A greater degree of wave energy absorption can be achieved, however, by widening the seaward end of a rock groyne (see Figure 8 of Appendix 3).

A number of more complex groynes have been built around the UK in the last 25 years. The most common type is the so-called “fishtail” groyne, which has a plan-shape reminiscent of the letter “Y”, i.e. with a bifurcated seaward end. Other variations have also been considered, for example groynes in the shape of the letter “T”, i.e. with a shore-parallel segment across its seaward end.

Other variations have also been built, for example groynes with a curved seaward end designed to protect the shoreline immediately down-drift of their own landward ends, or with a roughly circular end (banjo type).

These groynes can be considered as a combination of a simple groyne and a detached breakwater, and their performance depends not only on the parameters listed in

Section 2.2.1 for simple groynes but also many of those presented in the following sections for detached breakwaters.

2.2.3 *Detached breakwaters*

The use of detached breakwaters as components of a coastal defence scheme is a relatively recent development in the UK, with the first example apparently being the one built at Rhos-on-Sea in North Wales in 1983. A number of similar structures have been built since then, but all of these have been of random rock construction, often built using large rock imported from overseas on barges. Such structures are usually built in groups, with the gap between each breakwater similar to their length. Most of those schemes built so far in the UK have their crest well above the highest tidal level, i.e. producing a high degree of shelter against wave action in their lee, although one scheme off the Norfolk coast has been designed to allow partial overtopping by waves.

It is worth noting that in other countries, particularly in Japan and Italy, the use of offshore breakwaters for coastal protection, including the maintenance of adequate beach widths, has been a much more common practice. Indeed in these countries, such breakwater schemes are probably more common than groynes.

The main design characteristics of an offshore breakwater scheme can be summarised as their:

- Length (i.e. distance along their crest parallel to the shoreline);
- Spacing (i.e. distance apart along the shoreline); and
- Distance offshore (i.e. distance from the crest of the beach to their landward side).

Other aspects of the design of offshore breakwaters have an effect on beach evolution, including:

- The crest level of the breakwater relative to high tide level;
- The porosity (or permeability) of the breakwater; and
- The plan-shape and orientation of the breakwater crest.

With the exception of one recent scheme in Norfolk, all the UK offshore breakwater schemes have been designed to prevent wave overtopping except under the most severe combination of high tide levels and large wave heights. Elsewhere, for example in the Mediterranean, offshore breakwaters have been built that have their crests much closer to highest tidal levels.

All UK offshore breakwaters have also been built with straight crests aligned parallel, or very nearly parallel, to the shoreline. There is obviously some scope for altering the alignment of breakwaters, to have different effects on waves approaching from different directions, and perhaps to alter their plan-shape as well, e.g. larger “round-heads” at each end of the breakwater.

2.2.4 *Reefs*

The design of reefs to control the evolution of beaches is an even more recent development in the UK. While schemes have been recently proposed for two sites

along the coastline of East Anglia, none have so far been built in the UK as far as we have been able to determine.

Since one of the objectives of reefs is to change wave directions, there are more factors influencing the effect of a reef on beaches than for a detached breakwater, namely:

- Length (i.e. distance along their crest parallel to the shoreline);
- Spacing (i.e. distance apart along the shoreline);
- Distance offshore (i.e. distance from the crest of the beach to their landward side); and
- 3D- geometry (i.e. their plan shape and variation of depth of immersion).

2.3 INTRODUCTION TO THE LITERATURE REVIEW

A literature review was undertaken at HR Wallingford, and others, as a part of this project to seek out and collate information on the use of these four classes of beach control structure in the UK and overseas. The review undertaken by staff at HR Wallingford is presented in Appendix 1, together with a bibliography listing the papers and books reviewed.

In addition, similar reviews were commissioned as a part of this project from two other consultants, namely ASR Ltd. of New Zealand, and Professor Gianfranco Liberatore of the University of Udine in Italy.

ASR Ltd specialise in the design and execution of artificial nearshore reefs, which can fulfil a number of objectives including modifying the plan-shape and evolution of beaches, improving wave breaking conditions for surfers/ wind surfers and providing a “niche” habitat for marine plants and animals. In view of the fact that no such structures have yet been built in the UK, and indeed there are few reefs designed with the main objective of controlling beach evolution around the world, this subcontract provides valuable and up-to-date information on this type of structure. The report produced by ASR Ltd. is reproduced in its entirety in Appendix 2 to this report.

In Italy, there has been extensive use of nearshore breakwaters to combat beach erosion, and both surface-piercing and low-crest structures have been built in large numbers. As one of the most experienced coastal researchers/ engineers in Italy, Professor Liberatore is particularly well-qualified to provide a review of these and other beach control methods used in Italy, and this is presented as Appendix 3.

The purposes of this three-part review were as follows:

1. To gather together and compare guidance on the main design parameters for the construction of beach control structures on a sandy coastline with a low tidal range (as in Poole Bay);
2. To provide case-histories of schemes on similar coastlines, with an assessment of their performance, i.e. successes and failures; and
3. To provide information on other aspects of such schemes relevant to the choice of beach control structures between Sandbanks and Branksome Chine, including:
 - Design procedures;
 - Construction methods (e.g. materials, sequence, adjustments);

- Longevity and maintenance issues;
- Safety and aesthetic aspects.

In all cases, the emphasis has been placed on the use of control structures for sandy beaches. Hence the review of “simple groynes” has been limited to their use on sandy coastlines, rather than including the considerable body of experience of groyne design on shingle beaches. This is because there are widely acknowledged differences in the design of groynes on the two types of beach, which results from the following two factors:

- Shingle moves almost entirely as “bed load”, i.e. creeping over the surface of the beach/ seabed, while sand also travels as “suspended load”, i.e. with grains being supported within the water column by the effects of turbulence. This alters the way that groynes affect and intercept the movement of sediments along the shoreline; and
- Shingle beaches stand at much steeper gradients, both perpendicular and parallel to the shoreline, than sand. The steep seaward gradient means that groynes on shingle beaches can be shorter. The steep slopes parallel to the shoreline on a shingle beach necessitate both closer spacing and higher crest levels than on sand beaches, to be efficient and to withstand the lateral forces due to the different beach levels either side of the groyne.

For other types of beach control structures, of which there are many fewer in the UK, the literature has included case studies on both sand and shingle beaches.

2.4 SUMMARY OF THE LITERATURE REVIEW

It is clear from the many sources considered, and even from the discussions between various members of the study team, that an overall consensus on the “best” type of beach control structure for sandy beaches is impossible to achieve. Partly this is because different individuals attach different weightings to factors such as aesthetics, safety, ease of access along a beach and the sustainability of different types of construction material.

Even if a more restricted question is asked, however, along the lines of “Which type of control structure is most likely to maintain satisfactory beaches at least cost?” there is still no obvious “best choice”. Much will depend on the particular beach in question, for example the availability of materials, including that of sand for intermittent recharge operations, and of the costs of construction and maintenance of any structures built.

In view of this, the literature review has been used to provide some guidance on the design of control structures for sandy beaches, based on case histories and, where reasonably consistent, on more general advice and guidelines presented in the literature. This seems a sensible basis on which to start the process of suggesting possible schemes for the Sandbanks to Branksome Chine frontage. A further step in this process is the review of the frontage itself as presented in Chapter 3 below, which can be used to convert the following generalised “experience” into more detailed “outline” designs.

2.4.1 *Guidance on design of impermeable groynes for sand beaches*

The present guidance for the design of a groyne scheme on a sandy beach is little different to that available over 100 years ago. Perhaps this is partly due to the fact that

few schemes have been built that radically differ in the most basic parameters of length, height and spacing from those recommended at the end of the 19th century. However, it is probably more likely that schemes built with major changes to these parameters have not been successful and that the basic and long-established guidelines are indeed reliable.

The increased capacity to carry out beach recharge schemes at reasonable cost, however, does provide an option not available to coastal engineers in the Victorian era and hence an improvement in the methodology for installing a successful groyne scheme.

The main guidelines for simple groynes of sand beaches appear to be:

1. The crest of the groyne should not be greater than about 1m above that of the beach profile;
2. The groyne should not extend beyond the landward side of the beach profile bar, if it exists, to prevent sand being diverted offshore into deep water and lost from the beach;
3. The ratio of the spacing between groynes to their length should be between 2:1 and 4:1. Where waves approach the beach at a very oblique angle, a smaller spacing to length ratio is likely to be required, but even in this case the spacing/ length ratio should not be less than 1:1;
4. To avoid problems of beach erosion downdrift of the last groyne, the groyne compartments should be recharged immediately after their construction, and groyne lengths should be “tapered off” in the downdrift direction; and
5. Groynes are not effective on beaches with very low or no longshore drift.

Experience of the existing groynes along the Poole Bay frontage (from Sandbanks to Hengistbury Head) indicates that both impermeable timber and rock groynes have performed well in retaining or increasing beach widths. A beach control system using such groynes should be perfectly feasible from a technical viewpoint, building on well-established practice.

Research based on extensive monitoring of groyne performance along Bournemouth Borough Council’s shoreline (Harlow, pers. comm, 2005), indicates that groynes perform less well where the spacing to length ratio is greater than 2.0 on the Bournemouth frontage.

2.4.2 Guidance on design of permeable groynes for sand beaches

There has been a recent revival in interest in permeable groynes, in the USA, Germany and Italy. Such groynes have been built in the UK in the past, and there are two on the Bournemouth seafront just east of Bournemouth Pier. These are built using a lattice of concrete beams and named after their designer, Makepeace-Wood. In addition, there are permeable timber groynes further east along the Bournemouth frontage. The former have performed well, generally producing a wider beach locally, but are substantial structures that would be expensive to build. In contrast, the rather simpler permeable timber groynes further east have not performed well, and are suffering from damage.

Permeable groynes reduce the unequal beach widths either side of each structure, i.e. they decrease the “saw tooth” pattern in the plan shape of beaches with conventional,

i.e. impermeable groynes. In addition, they reduce seaward flowing rip currents along their flanks, except perhaps for the first, i.e. updrift groyne in any such groyne field. Recent articles (Poff et al., 2004 and Dette, Raudkivi and Oumeraci, 2004) illustrate such groynes in the USA and Germany (respectively) that are built as lines of wooden piles driven into the beach at differing spacing, being virtually impermeable at their landward ends and having greater porosity at their seaward ends. More “formal” groynes with a “walkway” at their crest, similar to the Makepeace-Wood design, have been installed recently in Italy (Lido di Jesolo) and are described in Appendix 3 to this report.

While some authors (Van Rijn, 2004) do not support their use on a recharged beach, those at Lido di Jesolo have been built to help retain a recent recharge and have performed well in reducing losses by longshore drift.

However, unlike impermeable groynes, there is apparently little general advice about lengths and spacing ratios for permeable groynes, with Dette, Raudkivi and Oumeraci (2004) concluding “Not enough data are available to enable the establishment of quantitative design rules for permeable pile groynes”. Similarly Poff et al (2004) state that “Groin spacing is site specific, so that guidance is to set the spacing equal to the length of influence”, implying perhaps a “trial and error” approach to the design of permeable groynes.

While permeable groynes seem to be capable of reducing drift rates, and maintaining nearly equal beach widths on either flank, their design is somewhat hampered by lack of experience and guidance on their appropriate lengths and spacing.

2.4.3 Guidance on design of fishtail groynes for sand beaches

Appendix 1 briefly reviews a number of fishtail groyne schemes built around the coastline of the UK over the last 25 years or so. In general these have performed in line with expectations, maintaining a wide beach in the lee of each “fishtail” on either side of the groyne stem. This locally increased beach width also provides an increased amenity, which can be economically beneficial, as well as improving the standards of defence locally.

Beach widths in the bays between each groyne are narrower, however, and this could result in an unacceptably low standard of defence in such locations if the groynes were too far apart, especially if insufficient beach recharge was carried out immediately after their construction.

No general guidelines on the appropriate spacings for any given fishtail groyne geometry (i.e. length, height, and dimensions of the “arms”) have been published, or derived from analysis of past schemes as far as we can determine. Instead it appears that each scheme has been specifically designed for its particular location, taking into account the beach materials, tidal currents and wave conditions. Such schemes have a greater capacity for retaining sediment on the upper beach either side of the groyne stem, than for simple groynes of the same length.

A critical factor in the consideration of fishtail groynes for any frontage is whether an uneven beach width is likely to be advantageous or not. The extra expense in building the larger groynes may not be worthwhile if the locally enhanced beach width does not provide a worthwhile increase in the standard of coastal defence at those specific locations.

2.4.4 *Guidance on detached breakwaters*

Detached breakwaters have been widely used to protect coastlines in the USA, Japan and around the Mediterranean Sea. As with fishtail groynes, detached breakwaters built to control beach evolution will produce a wider beach in their lee, sometimes to the extent of forming a tombolo, i.e. a “neck” of sand connecting the shoreline to the breakwater at some or all states of the tide.

Where such breakwaters are widely spaced, i.e. the gaps between each adjacent pair are greater than the longshore length of the breakwater, it is likely that the beach between the breakwaters will erode, unless sufficient recharge is undertaken at the time of their construction. This happened at Sea Palling (see Appendix 1), resulting in the removal of beach sand, erosion and removal of the clay shore-platform and scour at the toe of the seawall in the centre of the gaps between each breakwater. Such short term effects can be prevented by ensuring that the beach is recharged at the same time as, or before, the breakwaters are built.

Where the beach behind a breakwater does widen to form a large salient or tombolo, than there is likely to be a significant blocking effect on longshore drift rates. This effect may potentially last for many decades and possibly longer. This may represent a very efficient way of retaining beach recharge material on a coastline with no net drift. However, on coastlines where there is a net drift, there is a danger of serious erosion on the downdrift side of such a breakwater scheme.

Such structures remain popular in many countries, and have the added benefit of greatly reducing wave heights along the shoreline in their lee. Indeed this was the main purpose of the first such “emerged” breakwater built in the UK, at Rhos-on-Sea, in Colwyn Bay, where localised wave overtopping of the promenade was causing severe problems to people and property. However, the subsequent accumulation of beach material to landwards has led to a maintenance dredging commitment.

Reported problems associated with emerged breakwaters include:

- Poor water quality (particularly where tidal ranges and flows are small) and the deposition of fine-grained sediment, seaweed and litter in the area to landwards;
- Scour of the seabed to the seaward of the breakwaters (see Figure 13 of Appendix 3);
- Damage to the rock armour and settlement of the structures requiring maintenance, although this criticism can also be made of other rock beach control structures.

2.4.5 *Guidance of use of reefs as beach control structures*

The use of reefs designed principally as beach control structures appears to be a new concept in the UK, with few case studies available to provide guidance on either their design or success. Undoubtedly, they lend themselves to providing extra opportunities, e.g. for water sports and habitat creation, as well as not obstructing access along a beach or significantly affecting its aesthetics.

As with fishtail groynes, however, it appears that each reef scheme has to be designed specifically for each site, taking into account the particular wave and tidal conditions there. There are apparently no general guidelines on the spacing, offshore distance and reef dimensions (i.e. length alongshore and depth of immersion) that can be readily

applied to provide preliminary predictions of their effects on beach plan shape. A possible exception is for the case where the “reef” is a detached breakwater with its crest just below the water level.

Generally, it can be expected that reefs will produce a beach salient in their lee, thus increasing the beach width locally. If this widening of the beach is not matched by an initial beach recharge, then the longshore drift rates behind the reef and for some distance either side of it will alter temporarily, i.e. until this salient forms.

It appears that some reefs can reduce the net longshore drift in the long-term while others can be designed to restore the existing drift regime after an initial adjustment in the beach plan shape (see Appendix 2).

As with fishtail groynes, therefore, the desirability of creating a locally wider beach needs to be considered in the context of a locally increased standard of protection. An increased beach width will also provide an increased amenity, which can be economically beneficial.

Unlike detached (emergent) breakwaters or fishtail groynes, however, it appears that reefs do not cause as pronounced a narrowing of beaches on either side of the “salient” that the former produce.

3. *Review of coastal processes*

3.1 INTRODUCTION

A review of information relevant to the design of defences along the frontage between Shore Road, Sandbanks and the Poole/Bournemouth Borough boundary has been carried out as part of this Beach Control Structures Study. This included reviews and use of:

- Historic information on coastal geomorphology and processes, including the FutureCoast report produced for Defra by Halcrow (2004a);
- The 1995 “Poole Borough Coastal Strategy Study” report produced by HR Wallingford, which includes information on wave conditions, drift rates etc;
- The recent Coastal Strategy Study (Halcrow, 2004b) with particular regard to the choice and evaluation procedures used for the beach control options, including the estimates made on the cost and efficiency of those structures;
- Recent survey data collected by the Channel Coastal Observatory on beach and nearshore seabed profiles; and
- Calculations of drift rates including potential net longshore drift and the cross-shore distribution of longshore drift.

This chapter draws upon existing information on the coastal and morphological processes within Poole Bay and along the Poole Borough coastline. The full set of documents used in this review is listed in Table 3.1.

The review covers the wave climate, tidal range and currents, analysis of beach and seabed changes and a review of longshore drift and its cross-shore distribution. This provides a sound basis on which to make a judgement on appropriate beach control structures, to be carried forward to the next part of this study, i.e. the selection of the most promising beach control structure schemes for the frontage from Sandbanks to Branksome Dene Chine.

3.2 WAVE CLIMATE

Along the great majority of the coastline of Poole Bay, sediment transport and hence changes in the plan-shapes and profiles of its beaches, is dominated by wave action. Consequently, a good knowledge of wave conditions likely to be experienced along the coastline is a prerequisite to the design of beach control structures.

In previous studies (HR Wallingford, 1995 & 2000) the HR Wallingford HINDWAVE model was calibrated to provide an offshore wave climate using wind data from the anemometer at Portland Coastguard Station. This wind record covered 18 years from January 1974 to February 1992. HINDWAVE uses the JONSWAP wave generation formulation which hindcasts wave conditions at a given location from wind conditions and fetch lengths measured at constant angular increments from the wave prediction point. HINDWAVE produces a set of site-specific offshore wave forecasting tables, giving wave height, period and direction for a wide range of wind speeds, directions and durations. HINDWAVE then uses these tables with the wind data to produce a synthetic wave climate at the offshore wave prediction point.

Table 3.1 List of documents and reports reviewed

Documents/Information	Sub Name	Company/Source	Date
Poole Bay	Hindcasting wave modelling	HR Wallingford EX2406	1991
	Recalibration of hindcasting wave modelling	HR Wallingford EX2508	1992
Sandbanks Coast Protection Scheme	Feasibility study and outline design	HR Wallingford EX3083	1994
Sandbanks Coast Protection Scheme	Tidal flow modelling	HR Wallingford EX3083a	1995
Poole Borough Coastal Strategy Study	Main Report	HR Wallingford EX2881	1995
Sediment inputs into the coastal zone	Fluvial Flows	SCOPAC	1991
Seabed sediment mobility study	West of the Isle of Wight	CIRIA	1998
Regional seabed sediment studies and assessment of marine aggregate dredging		CIRIA	1998
Sandbanks Coast Protection Scheme	Review of Phase 2 scheme	HR Wallingford EX4242	2000
FutureCoast		Halcrow/DEFRA	2004
Poole Bay and Harbour Strategy Study	Computational Model Studies	HR Wallingford EX4555	2003
Poole Harbour Approach Channel Deepening	Hydrodynamic and sedimentation studies	HR Wallingford EX4945	2003
Poole Bay and Harbour Coastal Strategy Study	Main Report plus 10 Annexes	Halcrow	2004
Bournemouth's Coastal Management	Background as to why it is important to challenge the HALCROW report on coastal protection and the principle of constant beach recharge.	Bournemouth Borough Council - Roger West	2004
	<i>Wave Rotation</i>	Black & Mead, ASR	
	<i>Correspondence with Dr. Kerry Black</i>		
	<i>Correspondence with DEFRA</i>		
Beach Profiles	Environment Agency Beach Profiles	Poole Borough Council	2002-2004
Beach and Nearshore Profiles	Channel Coastal Observatory Profiles	Channel Coastal Observatory	
Aerial Photographs		Channel Coastal Observatory	2001 & 03
Coast Protection Maintenance	Existing coast protection of PBC frontage AutoCAD drawing	Poole Borough Council	2004

In a subsequent study (HR Wallingford 2003a) these original predictions of wave conditions were supplemented by wave climate data derived using HINDWAVE with UK Met. Office European Model wind data for a location offshore of Poole Harbour. The offshore point from which the fetches were measured was located at 1°52'0"W 50°36'0"N which lies approximately 6km east of Durlston Head, as shown in Figure 3.1. This data set covers the period October 1986 to March 2001, and information from these two sources was used in this study. A comparison of the two wave climates indicates that there is a small difference in mean wave direction of about 2°. This is thought to be a result of differences in wind conditions resulting from different sources of wind data and in the different time periods considered. However as this difference is small, the longer (1974 to 1992) time-series data will be used for the beach plan-shape modelling. The effects of the small differences in wave directions and heights in the two sets of offshore wave conditions will be taken into account through sensitivity testing.

The offshore wave conditions that have been predicted and measured from these studies indicate that the prevailing offshore wave condition, and the direction from which the largest waves come, is south-west to west-south west (210°N- 250°N). A secondary concentration of waves is also evident from the east-south-east direction (from about 120°N). These two main wave “populations” reflect the long fetch lengths in these two directions.

Inshore wave conditions were predicted at several locations along the shoreline of Poole Bay, typically on the -2m CD contour in the 1995 Strategy Study. The largest predicted waves at these locations are around 5 or 6 metres and the directions of these waves are clustered in a fairly narrow band around the beach normal at each point. However the mean direction of the waves at the shoreline changes along the coastline, from approximately south-east in Studland Bay to near to south-west near Hengistbury Head.

For this study, small changes in wave conditions along the Poole Borough section of the coastline are important in terms of sediment transport along the beaches. Wave roses for inshore points C (Shore Road) and D (Branksome Dene Chine) have been reproduced and are presented in Figure 3.1. It is worth noting that the range of directions from which waves approach the shoreline is a little wider at the eastern end of the Borough’s shoreline, reflecting the reduction in shelter provided by the Isle of Purbeck.

3.3 TIDAL CONDITIONS

There are marked variations in tidal range and asymmetry, with accompanying complex patterns of tidal flow along the central southern coastline of England. Off Portland, for example, the tidal range on mean spring tides is in excess of 3m; there the tidal curve is relatively symmetrical on the ebb and flood, but with a prolonged “stand” at low water. At Poole, on the other hand the range is as small as 1.7m and the tidal curve is strongly distorted, having a double high water and only a short “stand” at low water. Normal tidal levels at the entrance to Poole Harbour, i.e. ignoring any effects due to short-term atmospheric variations, are summarised in Table 3.2 below.

Table 3.2 Tidal levels at Poole Harbour Entrance (mCD)

HAT	MHWS	MHWN	MWL	MLWN	MLWS	LAT
2.6	2.2	1.7	1.6	1.2	0.6	0.0

Flow modelling indicates that the strongest currents in Poole Bay occur in the entrance to the Harbour and around the head of the Sandbanks peninsula. Towards the neck of the Sandbanks peninsula, there is a marked reduction in tidal velocities, which continue to reduce further east along the coastline. From Shore Road Car park eastward, tidal flows are weak and have no significant influence on longshore drift.

The hydrodynamic conditions in the area were also simulated in a more recent study by HR Wallingford (2003) using the 2D, depth averaged flow model TELEMAC-2D. The model results simulated the tidal currents in the Poole Bay area as generally weak with a north-easterly flood and south-westerly ebb direction. Figures 3.2 and 3.3, showing the peak ebb and flood current vectors, highlight the strong currents through Poole Harbour entrance and the reduction of current speeds further eastward along the coast.

3.4 REVIEW OF SEDIMENT TRANSPORT PROCESSES

3.4.1 *Poole Bay*

Sediment transport processes over most of the offshore seabed in Poole Bay, i.e. from Durlston Head in the west to Hengistbury Head in the east, are driven by tidal flows assisted by waves that initially disturb and mobilise the sediment. An overview of sediment transport pathways within the Bay is presented in a report produced by CIRIA (1998) following a study into seabed sediment mobility to the south and west of the Isle of Wight. The main long-term sediment transport pathways identified in that study are shown in Figure 3.4. It should be noted from this figure that there is a net export of sediment out of Poole Bay, both to the east over Christchurch Ledge and into Christchurch Bay and to the south and west past the Isle of Purbeck and into the deeper water of the English Channel.

The only significant sources of sand and gravel within Poole Bay are the eroding cliffs around its margins, particularly between Poole Head and Hengistbury Head, and to a lesser extent from the erosion of the shore-platform seaward of the beaches. The protection of the cliffs by seawalls over the last 100 years has dramatically reduced the major source of fresh beach material for Poole Bay. The net input of sediment from the rivers discharging into the Bay is modest and almost entirely composed of fine sediment, which will rapidly be dispersed into deep water (SCOPAC, 1996).

In shallow water areas just offshore, and along the beaches, wave action plays an important and usually dominant role in sediment transport. As explained in section 3.2, both southerly and south-easterly winds can generate severe waves within the English Channel, which can affect the plan-shape of the beaches. However, it is the waves generated by the more common westerly and south-westerly winds along the long fetches towards the Atlantic, which dominate the hydraulic environment. In the eastern part of Poole Bay the coast is exposed to waves approaching from the south-west travelling up the English Channel from the Atlantic. This produces a net eastwards drift of sediment along the beaches along most of the coastline of Poole Bay.

However, the further one progresses westward from Bournemouth towards Studland, the greater is the shelter provided by the Isle of Purbeck and Handfast Point. Waves from the south-west are refracted and diffracted round these features to approach from the south and south-east, having been much diminished in height and energy.

The wave climate in the sheltered western part of Poole Bay thus tends to be less severe than further east. Longshore currents induced by this wave height gradient are one of the forces driving the morphodynamic processes in Poole Bay. This tends to lead to a

general accumulation of sediment in the western part of Poole Bay, e.g. Hook Sand and South Haven peninsula. In the long-term, there has been a tendency for sediment to be carried out of the centre of the bay to the west and east, resulting in a long-term trend of erosion in the centre of the Bay.

3.4.2 Study frontage – Sandbanks to Branksome Chine

Turning now to the beaches between Sandbanks and Branksome Chine, the predominant transport process is “longshore drift”, i.e. beach sediment transport along a coastline. This is a nearly continuous process occurring throughout the year in both commonly occurring and storm wave conditions. The rate of drift at any moment is sensitive to wave height, wave period and wave direction. There is usually considerable seasonal and inter-annual variability in drift rates, e.g. with larger rates in winter, but the overall total drift varying considerably from year to year depending on the weather conditions experienced. It is also important to introduce the concepts of “gross” and “net” drift. The average gross drift over a year, i.e. the sum of the leftward and rightward drift rates (as seen by an observer facing seawards), is often very much higher than the average net drift rate, i.e. the difference between the leftward and the rightward drift rates.

There have been differences in even the direction of the longshore transport rate along the Sandbanks frontage over time, as revealed in previous studies and analysis of historic maps (HR Wallingford 2000). For example, the shape of the Sandbanks peninsula is close to that of a classical “sand-spit” created by the *westward* movement of sand into the mouth of Poole Harbour. At present, however, the drift is eastward along this section of coastline.

Calculating the quantities of the longshore drift at any point along the Sandbanks frontage is fraught with difficulties. It would require detailed knowledge of the levels on Hook Sand, and accurate computations of the waves diffracting around Durlston Head and subsequently refracting over the complicated contours of the western end of Poole Bay, including the Swash Channel and Hook Sand itself.

The movements of sand in this area are further complicated by the effects of tidal currents. Although the tidal range is small, the large inter-tidal area of Poole Harbour means that a very large volume of water enters and leaves the harbour through its narrow entrance every 12 hours. These tidal flows affect wave transformation patterns, further complicating any computational modelling of waves arriving on the coastline. Of particular concern for the evolution and management of the beaches are the tidal flows and morphological changes associated with the East Looe channel. This channel lies close to and is sub-parallel to the beaches along the Sandbanks frontage.

There is, at present, clear evidence of a net eastward transport of sand from the Haven Hotel along the coast to Shore Road. Since there is no beach along the coastline on the western side of the Hotel, the recent accumulation in the groyne bays in this area indicates an onshore supply of sand from the seabed.

Sand is probably propelled over the crest of Hook Sand especially during severe weather when waves break on its crest. Under typical day-to-day conditions the sediment flux at the seaward end of Hook Sand is thought to be seaward but under storm conditions the sediment flux is reversed (Halcrow, 2004b). The sediment flux from the southern tip of Hook Sand under storm conditions is equivalent to 100-

1,000m³/year and increases considerably further north over the sandbank (Halcrow, 2004b).

The East Looe channel can also carry sand along its length, and some may disperse laterally onto the beaches, especially during the flood tide. Finally, the large spatial variations in the strengths of the tidal currents offshore of the Sandbanks coastline may result in dispersion of suspended sediment from areas of strong currents to calmer areas, e.g. within the bays between the large rock groynes at Sandbanks.

Further east along the coastline, the influence of tidal flows in the East Looe channel diminishes as the channel lies further offshore, is shallower and wider, with less intense flows. Along the frontage near Shore Road, tidal flows are modest and further east again are negligible in terms of sediment transport. There is a suggestion that the flood tidal stream “splits” just offshore, with flows going west along the East Looe channel and east towards Bournemouth roughly parallel to the shoreline (see Figure 2.15 of HR Wallingford, 1994b).

From Shore Road eastwards, it is reasonable to assume that longshore drift rates are not affected by tidal streams, and the drift due to waves arriving obliquely to the shoreline can be calculated on this basis. A previous study by HR Wallingford (1995) did estimate the longshore drift rates, and this report was subsequently used in the recent Shoreline Management Plan produced by Halcrow (1999).

However, the quantities and rates of transport do not tie in well with the results from the site visits, analysis of the beach surveys or the historical map analysis reported by HR Wallingford (2000). It is probably fair to conclude that the net wave-induced longshore drift rate is presently eastwards, and has been in this direction for some decades, with a rate of between zero and perhaps 100,000 cubic metres year (upper limit). It should also be noted that the potential drift rate depends to some extent on which beach contour is chosen to represent the beach orientation, which is one of the factors which introduces variability into the results. Drift calculations carried out as part of this study indicate that a change in beach orientation or wave direction of 1° can vary the drift rate by up to ±10,000 m³/year near Shore Road and by up to ±65,000 m³/year at Branksome Chine and can hence lead to a reversal in the predicted direction of that drift.

Longshore drift rates have also been calculated as part of the strategy study undertaken by Halcrow. The potential drift rates are estimated as between 5,000-30,000m³/year out of the Sandbanks frontage and into the central part of Poole Bay. Along the Central Poole Bay frontage the drift rates are higher, typically between 10,000-100,000 m³/year towards the east (Halcrow, 2004b). The actual longshore transport rates were also calculated along this frontage through the analysis of beach profiles, and are generally lower (typically between 20,000-60,000m³/yr) than the potential drift rates. Further calculations of drift rates are presented later in this report, in the section dealing with the assessments of alternative beach control structures and are also covered in Technical Note CBR3713/TN01: Beach plan shape modelling.

Finally, it is worth mentioning the effect of winds on the movement of beach sand. Since much of the immediate hinterland to the beaches is high cliffs, onshore winds are much stronger than those blowing offshore. This results in favourable conditions for sand being blown inland from the beaches to form dunes where there is sufficient room at or behind the beach crest for these to form. Good examples of this can be found just east of Hengistbury Head and at Sandbanks; in both locations, the installation of groynes created a wider beach that allowed dunes to form.

There is also a predominance of westerly and south-westerly winds in Poole Bay, which in turn leads to an extra mechanism for the transport of sand from west to east along the coastline from Sandbanks to Branksome Chine. Quantifying such “aeolian” transport rates, however, is exceedingly complicated, since it depends upon not only the width of beach from which sand can be picked up, but also on the sand grain size and whether or not it is dry. To date, no study has successfully predicted net volume of transport from the beach to dune system over longer time periods although on nourished beaches in The Netherlands, actual aeolian sediment transport was 2-10% of the predicted potential transport.

In the context of the proposed new beach control structures for the Shore Road to Branksome Chine frontage, the main conclusions regarding the movements of sand along are as follows:

- The longshore drift along the beaches where the scheme is proposed is to the east.
- There is some evidence that there is likely to be a continuing onshore supply of sand from the seabed to the Sandbanks frontage, at least for the next few decades. The long term supply of “new” sand for this frontage is more problematical, and hence beach recharge may well be required in the future.

3.4.3 *Cross-shore distribution of longshore drift*

The cross-shore distribution of the longshore drift along the beaches of Poole Bay (Shore Road to Branksome Chine) is an important factor in the determination of appropriate beach control structures. Longshore sediment transport is related to the energy released by breaking waves, and so the rate of sediment transport at any point down a beach profile depends on the intensity of wave breaking at that point. Thus, in water depths where there is little or no wave breaking, there will be little or no longshore transport.

This drift distribution is not affected by tidal currents, which are very weak along this frontage (section 1.3), but is affected by:

- The incident wave conditions, i.e. heights, periods and directions;
- The variations in tidal level; and
- The variation of seabed level with distance offshore.

In respect of the last of these three factors, the seabed levels do not increase steadily with distance offshore, because there is a “bar” or at least a flat portion of seabed about 100m offshore, as shown in Figures 3.5.

The calculation of the longshore drift rate, and its variation down a prescribed beach profile, requires repeated use of a numerical model for each combination of wave height, wave period, wave direction and tidal level likely to be encountered. The results from each such numerical computation are then “weighted” according to the probability of occurrence of each such combination and summed to provide an overall prediction of the net longshore drift rate and its strength as a function of the distance offshore. For this purpose we have used a computer model developed at HR Wallingford, known as COSMOS; this is described in detail in Appendix 4 to this report.

The cross-shore distribution of longshore drift has been calculated using COSMOS along profiles AE and BE, averaged over all tides, and the results are presented in Figure 3.6. Two graphs are presented for each profile, and each graph shows, in its lower half the variation in seabed level (right hand vertical axis) with distance offshore (horizontal axis).

The top graph of each pair shows the amount of drift (left hand vertical axis) passing to the landward of the offshore distance value (horizontal axis), and the total net drift rate is calculated as about 15,000 cubic metres/ year across Profile AE and about 24,000 cubic metres/ yr across Profile BE.

The bottom graph of each pair shows the net amount of drift passing within a metre long strip measured down the profile (left hand vertical axis) at different distances down that profile. For both profiles there is a peak in this drift rate where the seabed is at a level of about -1m ODN. To the seaward of this, there is a reduction in the drift rate over a nearly-horizontal section of the seabed. This is explained by the fact that waves will either break at the seaward edge of this “plateau” or to the landward of it, but there will be little wave breaking over the plateau itself, and hence little sediment transport along the coast. To the seaward of this “plateau” there is an increase in the drift rate, particularly where there is an offshore bar, in the area where waves approaching the plateau break.

From the top two graphs, it can be seen that about 40-50% of the total longshore drift occurs seawards of the bar (or outer edge of the “plateau”) and this is relevant when considering the possible beach control structures. The choice of an appropriate groyne length, for example, will then be influenced by the position of the low values in the top graphs, since a small increase in the length of the structure will produce very little increase in the amount of drift it “intercepts”. This leads to the possible conclusion that the correct length of a groyne would be such that it stops just short of the offshore bar or the outer edge of the plateau in the beach profile (~ 100 m in this case). There is further support to this view, because of the danger that extending a groyne onto or across an offshore bar is likely to cause that bar to move offshore. This is the advice provided by Van Rijn (2004) based on experience elsewhere, and is supported by observations made by Bournemouth Borough Council (D Harlow, *pers. comm.* 2005) that offshore bars along their frontage are further offshore seaward of the end of groynes than in the groyne bays. Much of the theoretical advantage of intercepting more of the drift by extending groynes or other beach control structures beyond the bar may therefore be lost as the bar moves seaward in response to the installation of that structure.

A structure extending far enough offshore to intercept all the drift would, in the light of these results, need to be at least 200m long at Profile AE and 250m long at Profile BE. However, caution needs to be exercised in the interpretation of the figures showing the longshore drift distribution. While it is very likely that the seabed is covered by sand as far seaward as the “plateau” or offshore bar shown in Figure 3.6, beyond this there may be little or no mobile sediment. The numerical model assumed that the whole profile was covered with sand, and the “potential” drift rates it predicts over the seaward portion of each profile may be greater than the actual drift rate, because there is little or no sand to be transported on that part of the seabed.

Reefs, however, have little or no long-term effect on longshore drift rates, instead serving to reshape the beach locally. Unless they are implemented simultaneously with a beach recharge, the longshore drift in their lee will decrease temporarily until salients are formed there, and then revert to its normal rate.

3.5 ANALYSIS OF BEACH AND SEABED CHANGES

3.5.1 *Analysis of bathymetric surveys*

An analysis of bathymetric surveys by Poole Harbour Commissioners has been carried out in previous HR Wallingford studies (1994 & 2000). In the 1994 study it was observed that the East Looe Channel had deepened along most of its length and had moved onshore at its eastern end between 1990 and 1994. Analysis presented in the 2000 study revealed that the floor of the East Looe Channel at the western end had deepened by 1m, but the position of the channel (e.g. its centre line) had remained relatively constant. Further east, between 1995 and 1998 the channel cross-section had reduced as a result of the landwards migration of Hook Sand. However, the position of the centre line of the channel had still remained unchanged, as had the maximum depth. Offshore from the Pavilion, the bed level had fallen by 0.5m between 1995 and 2000, and the former shallow channel some 100m offshore had largely been filled. The study also revealed that to the east of the Pavilion, the chart datum contour had moved landwards in recent years.

Seabed changes from Shore Road to Branksome Chine are examined in more detail as part of this study. A survey carried out in May 2004 has been analysed by extracting levels along profiles AE and BE at Flag Head Chine and Branksome Chine respectively. This analysis revealed that a “bar” or at least a flat portion of seabed exists about 100m offshore, as shown in Figure 3.5. The existence and position of this “bar” is expected to have an affect on the cross-shore distribution of the net littoral drift and therefore have implications on the choice and dimensions of the potential beach control structures.

3.5.2 *Analysis of beach and shoreline surveys*

An analysis of historic shoreline changes along the whole Poole Borough Frontage was undertaken by HR Wallingford as part of the strategy study in 1995. During the period between 1901 and 1925, the high water contour undertook a re-alignment, rotating anti-clockwise about Management Unit 1/9 (Shore Road) by about 2°. Positions to the north-east eroded while accretion occurred to the south-west. From 1925 to 1955 no clear pattern of change was identified and changes that did occur were generally very small except in the groyne bays at Management Units 1/12 to 1/14 (Sandbanks) where the accretion noted during the earlier period continued. Again no very clear pattern of change occurred for the final period of analysis from 1955 to 1993, although there was a tendency for accretion over the north-eastern part of the Poole Borough frontage and erosion closer to Sandbanks.

A more thorough analysis of beach profiles along the Sandbanks frontage was undertaken as part of the HR Wallingford’s studies of the Sandbanks coast protection scheme (1994 & 2000). Prior to the installation of the rock groynes along the Sandbanks frontage, analysis of the beach profiles collected by Poole Borough Council between 1992 and 1994 indicated that the beach above the mean low water mark has tended to erode. The trends identified through analysis of retreat rates were also reflected in the volumetric changes. Following the implementation of the Sandbanks coast protection scheme the beach at the western end has been stabilised. Further along the Sandbanks frontage, the beach has accreted and the sand has blown shoreward to form dunes.

Beyond Shore Road, the frontage is groyned and despite being narrow, shows no indication or record of beach erosion (HR Wallingford, 2000). Photographs from site inspections show much the same beach level at the toe of the wall in both 1991 and 2000. Further along the coast, up to Branksome Chine, recent site visits have always

indicated an eastward drift of sand, as shown by differential levels either side of the short groynes or outfall structures. As part of this study a more detailed analysis of beach profiles (2002-2005) has been undertaken and is discussed in Appendix 4.

3.6 REVIEW OF BEACHES AND EXISTING DEFENCES

The choice of appropriate beach control structures for the coastline between Sandbanks and Branksome Chine will depend, in part, on the specific characteristics of that frontage, taking into account factors such as access points, the details of the plan shape of the seawall and other structures including outfalls, the groynes at the western end of Bournemouth Borough's frontage and those recently built at Sandbanks. For this reason it is appropriate to briefly summarise the existing beaches and defences along the study frontage.

The shoreline of Poole Bay comprises a sandy beach, with some shingle towards the eastern end. With the exception of a short stretch of dunes on the Sandbanks frontage, the whole of the Poole Borough coastline is protected by seawalls, which have been built from about 1890 onwards, fronted by a sloping sandy beach.

From the eastern PBC boundary to Poole Head a variety of stepped or near-vertical concrete walls are in fair to good condition (HR Wallingford, 1995). A recent site visit (30/04/05) undertaken by HR Wallingford revealed the crest of the sand beach at the toe of the seawalls is reasonably high apart from a few isolated stretches along the coastline. The narrowest beach is where the seawall orientation changes between Branksome Chine and Branksome Dene Chine, where there is no beach at mean High Water (Figure 3.7 and Appendix 4).

To the east of the rock groynes at Sandbanks, i.e. either side of Shore Road, the beach is presently much wider than between Poole Head and Branksome Chine, as a result of a recent beach recharge. This section of the frontage presently therefore enjoys a greater standard of protection than that east of Poole Head, and also offers a greater amenity benefit.

The walls along the Shore Road to Branksome Dene Chine frontage are backed by a continuous promenade, which in places shows signs of minor subsidence. Along much of the frontage there are beach huts in front of the cliffs. A toe wall has been constructed along much of the landward edge of the promenade, to reduce erosion at the foot of the cliff face and prevent sand from falling onto the promenade from the cliff.

East of Poole Head the beach is groyned with timber structures 30 to 35m long, spaced at about 120m (i.e. a 1 to 4 ratio). Though generally in fair condition some of these structures are in poor condition with timbers being decayed, and even entirely missing, and metal fixings corroded. At the time of past site visits, there was no significant change in beach levels or widths on either side of these groynes (Appendix 4). This situation is in marked contrast to the effect of the longer and higher groynes just over the borough boundary, which shows the expected pattern of beach accretion against their western faces, and lower beach levels immediately adjacent to the eastern faces. It can be concluded that the present short timber groynes along this section of Poole Borough's frontage are ineffective.

The aim of Poole Borough Council is to achieve and maintain sufficiently high beach levels to ensure an appropriate standard of defence along the whole frontage from

Sandbanks to Branksome Chine. As pointed out in the recent coastal strategy study, this overall aim requires two objectives to be met, namely:

1. To ensure a prolonged life of the seawalls, an adequate beach width and depth of sand at the toe of the walls needs to be maintained. Beach modelling undertaken by Halcrow (2004b) indicated that the minimum beach width to prevent failure of the seawall is 7m (to mean sea level) for a 1:1 year storm, 12m for a 1:10 year storm and 17m for a 1:100 year storm.
2. The beach crest level also has a direct influence on the wave conditions on the seawall and hence its susceptibility to overtopping and scour. Where the beach crest level at the seawall is at 2.4m ODN (beach width of 72m to mean sea level) the overtopping is negligible, whilst where it is 1.1m ODN (beach width of 33m) overtopping during a 1:10 year storm is calculated to be 16 to 21l/s/m (depending on the seawall slope) (HR Wallingford, 1995). This is an unacceptably large overtopping discharge rate that would be exceedingly dangerous to pedestrians and vehicles using the promenade, and would be likely to cause damage to the promenade's surface.

In order to maintain adequate beach widths and levels along the PBC frontage, the Poole Bay Strategy Study therefore concluded that periodic beach recharge and the management of subsequent evolution of the beach through the installation of control structures will be required.

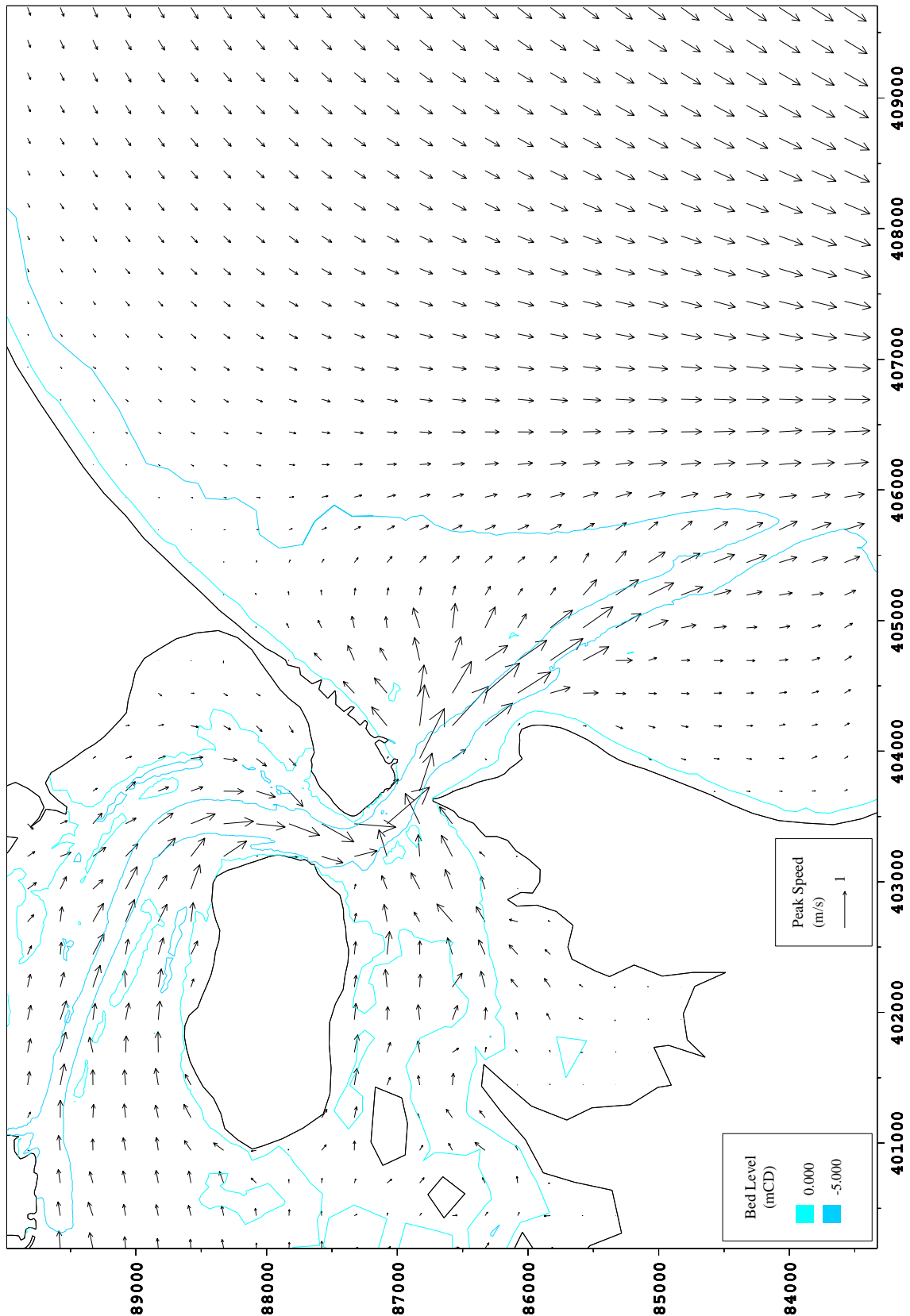


Figure 3.2 Spring tide ebb current vectors. Baseline conditions

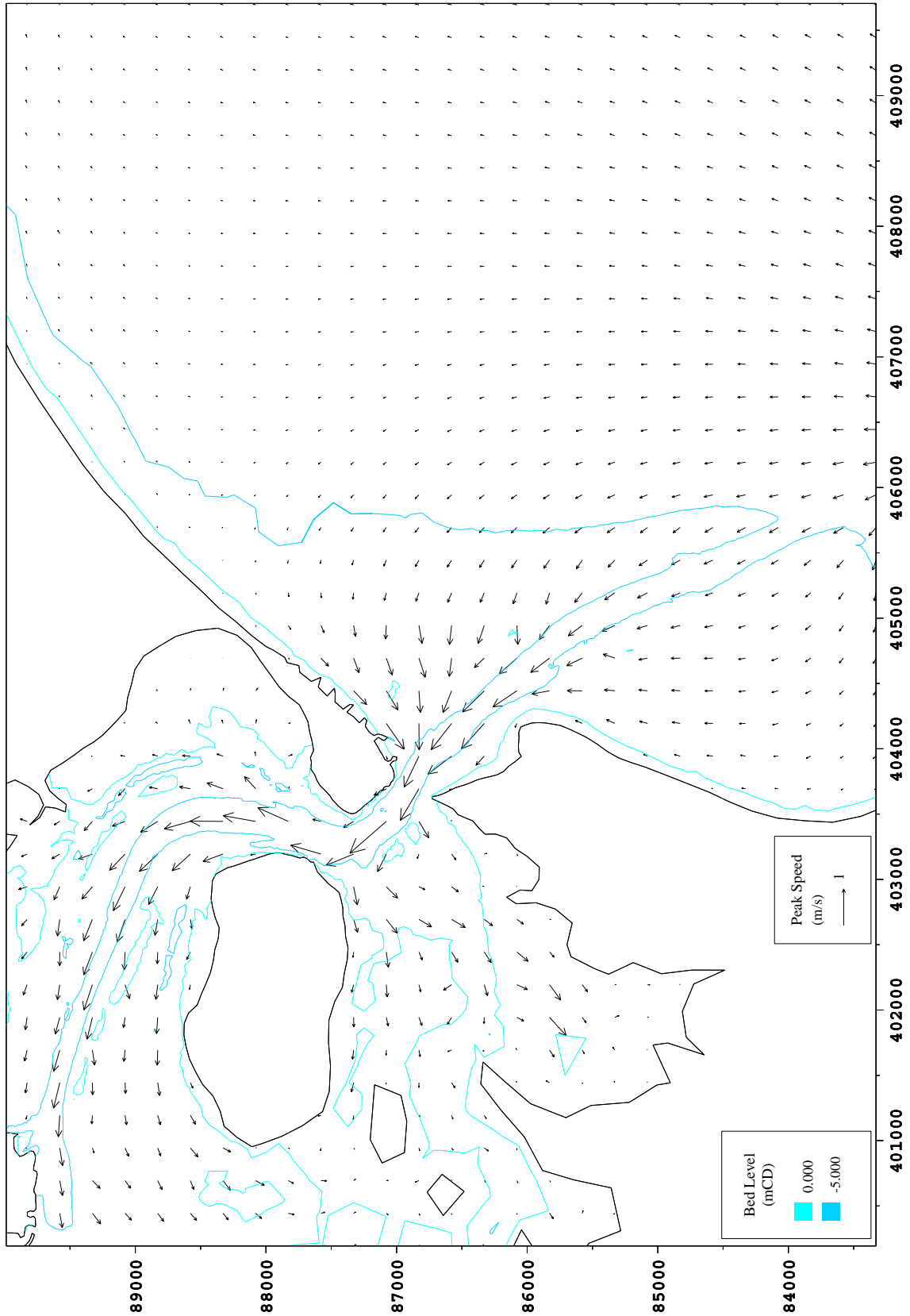


Figure 3.3 Spring tide flood current vectors. Baseline conditions

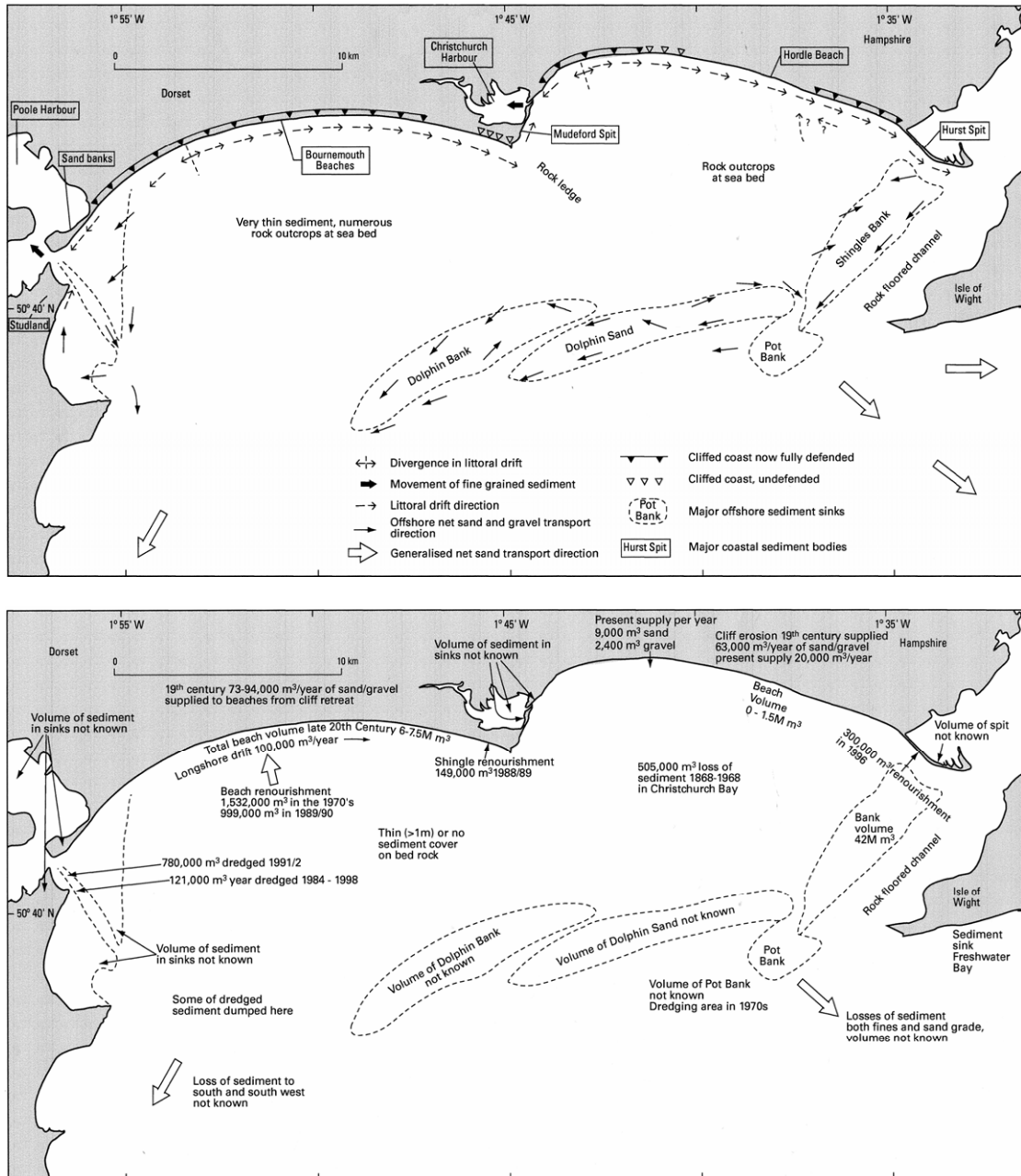


Figure 3.4 Long-term sediment transport pathways

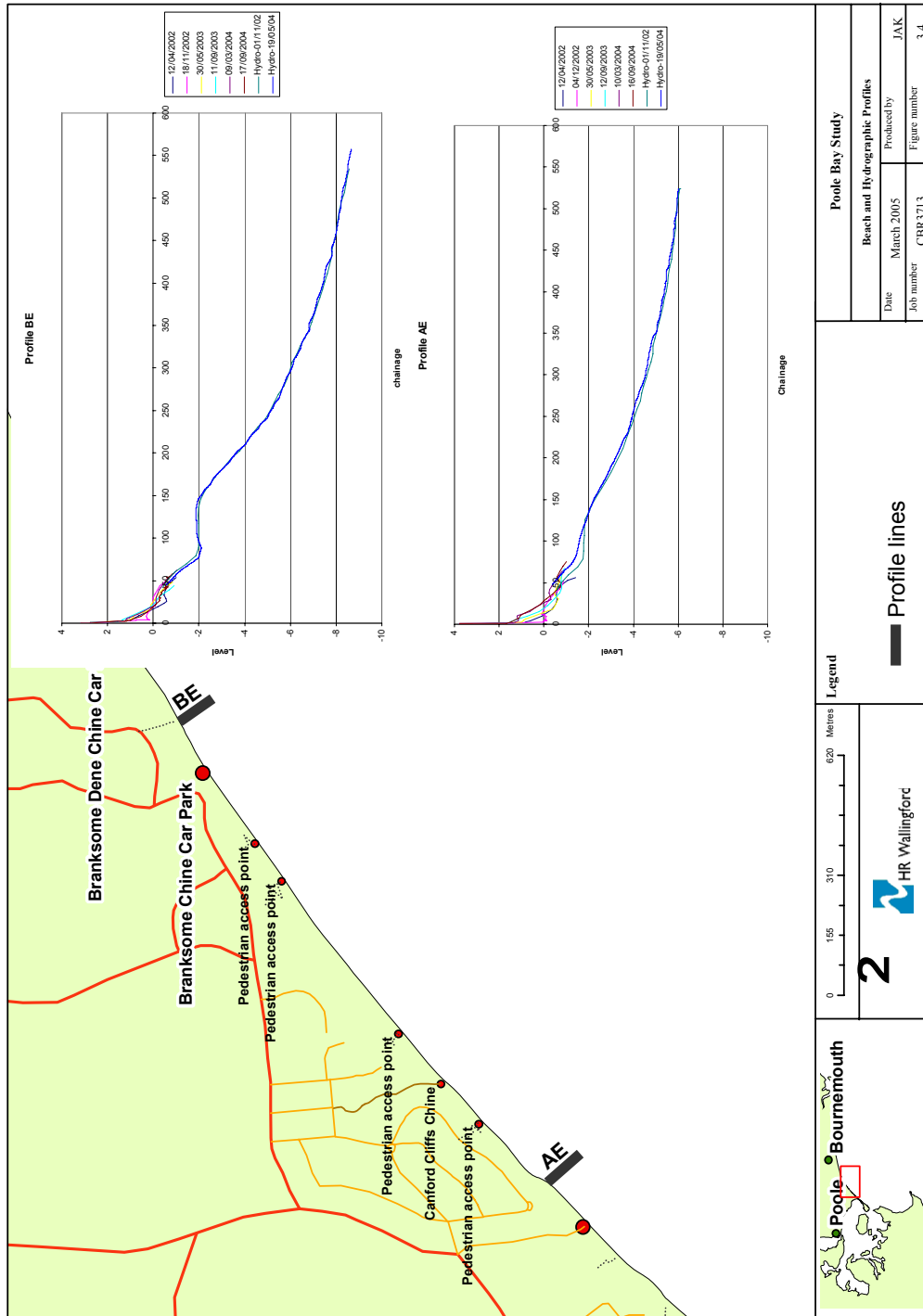


Figure 3.5 Beach and Hydrographic Survey Profiles



Figure 3.7 Location of narrowest beach where the seawall changes orientation

4. *Options for beach control structures*

4.1 BACKGROUND AND OBJECTIVES

The main purpose of the planned improvements to Poole's beaches between Shore Road and the Borough boundary, by recharge of the beaches with sand and the installation of structures designed to control the beach plan shape, is to improve the standard of defence along that frontage. Whilst wave overtopping of the promenade already leads to localised flooding episodes, the main concern is that the seawall along the edge of the promenade may fail. If beach levels at the toe of this wall were to become lower, there would be greater risks of scour and hence of undermining, the most common cause of seawall collapse. Without a competent seawall to protect them, the cliffs behind the promenade would start to suffer from landslides or slips, with significant risks to the houses and infrastructure on the cliff tops.

The present standard of defences along the study frontage is summarised in section 3.1.2 of the Halcrow Strategy Study report (2004b), as follows:

- Between Sandbanks and Canford Cliffs the frontage, recharged in February/March 2003, has a beach that is sufficiently wide to provide a standard of protection greater than 100 years; but
- To the east of Canford Cliffs, to the Borough boundary and beyond, the present standard of defence varies from between 1:1 year and 1:100 years (e.g. at Branksome Chine), depending on the precise location.

Along this latter frontage, beach widths can become particularly narrow at several locations where the seawall has a convex (seawards) plan-shape (see Figure 3.7 and Appendix 4), especially at:

Branksome Dene Chine (406802, 89824)

Southwest of Branksome Dene Chine Car Park the beach becomes particularly narrow as a result of the seawards deflection of the seawall. At this location there was no beach in front of the seawall at MHW in 2005. An analysis of beach levels beside the groyne along the frontage undertaken by Dr David Harlow (Bournemouth Borough Council), see Appendix 4 for more details, reveals that the beach at this location is submerged for 37% of the time (1994-2003). Beach profiles from 2002-2004 have been analysed (Appendix 4) and indicate that the beach width to MSL in September 2004 was 12m, although the beach has been less than 7m wide in the past (i.e. November 2002). The beach width in September 2004, according to the Strategy Study report (2004b), assures protection to the seawall from a storm event with a return period of 1:10 years. Recent site visits and examination of past photos, as part of this study, reveal that the groyne is nearing the end of its functional life and is inefficient, and the toe of the seawall has been damaged (Plate 4.1).

Branksome Chine (405461, 88762)

Further southwest, beyond Branksome Chine car park the beach again becomes narrow. The MHW line measured in 2005 indicates that the beach was 15m wide at high water, which is equivalent to 33m to MSL and therefore provides protection from a storm event with a return period of 1:100 years. Beach analysis by David Harlow, however, reveals that the beach is submerged for 14% of the time and the beach profile analysis again illustrates that the beach in September 2004 was healthy but can reduce to less

than 7m (i.e. November 2002). This means that the present standard of defence is, in places and at certain times, less than the minimum required to protect the sea wall from even a 1:1 year storm, as based on the modelling in the Halcrow Strategy Study (Halcrow, 2004).

Photographs taken in November 2004 indicate substantial beach lowering to well below the toe of the concrete stepped seawall with the beach level judged to be in excess of 2m below the tread of the lowest step (see Plate 4.2). While the method used for monitoring beach scour at high water shown in this Plate is unusual, and not always practicable, it indicates the clear possibility of the groyne and seawall being undermined even in calm conditions. This particular photograph was taken some distance to the west of the location where the lowest beach levels along this part of the frontage are expected to occur.

Flag Head Chine (406384, 89579)

Finally, where the seawall deflects seawards, north east of Flag Head Chine car park, the beach on occasions becomes narrow (Appendix 4). The 2005 survey of MHW indicates a healthy beach at this location with beach widths of 17m and 35m to MHW and MSL respectively. This beach width is greater than the required values to provide protection to the seawall. However, examination of the beach profiles shows that the beach has dropped below the minimum beach width of 7m, again meaning the wall is not providing protection at even a 1:1 year standard. Further examination of the profiles reveals a scour hole in front of the seawall in December 2002 and therefore it would be more vulnerable to overtopping and damage. Evidence of the beach width being narrow in the past is also shown in Plate 4.3; while this may only be a seasonal effect, narrow beaches are likely to occur in winter, i.e. at the time that a severe storm is most likely to occur.

The proposed future beach recharge operations, together with any beach control structures, will therefore need to improve the standard of defence particularly at these “pinch points”, but will also need to improve, or maintain, sufficiently wide beaches at all other locations along this frontage at the same time.

Whilst providing an appropriately improved standard of defence is the primary role of the proposed beach management works, it is fully recognised that other benefits would follow. The sandy beaches of Poole Bay are important environmental, amenity and recreational assets for the region. As a popular destination for holidaymakers and tourists, the beaches indirectly contribute to the local economy. In addition, wider beaches are regarded as beneficial to both the aesthetic and to the natural heritage and conservation attributes of the coastline.

There is therefore not only a need for an improvement in the standard of coast protection along this frontage, but also to ensure, as far as practicable, that any works undertaken take into account the other attributes of the beaches mentioned above.

The above considerations lead to a number of objectives that any beach control structures will need to achieve, namely:

1. To maintain appropriately wide beaches between Shore Road and the Borough Boundary;
2. To maximise the lifetime and benefits to defence standards provided by proposed future beach recharge operations;

3. To be technically feasible, long-lasting and sustainable;
4. To avoid adversely affecting beach levels along the adjacent coastlines, especially to the east i.e. along Bournemouth Borough Council's frontage;
5. To achieve the foregoing objectives at acceptable "whole life" cost, and ensuring that the scheme maximises the economic, environmental and social benefits that result; and
6. To be acceptable from the viewpoints of public safety, environmental impacts and aesthetic appearance.

Some further discussion of these "headline" objectives is warranted, and is presented below.

4.1.1 *Standard of defence required*

The main requirement is to maintain an adequate beach width along the whole frontage between Shore Road and the Borough boundary i.e. that is sufficient to prevent undermining of the seawall. Given that tidal levels, beach sediments, wave action and seawall profiles are reasonably consistent along this frontage, this equates to achieving a consistent minimum beach width.

In this study, rather than re-evaluating this minimum beach width, we have used the values suggested in the recent Coastal Strategy Study (Halcrow, 2004b). This report provides different values for minimum beach widths at Poole, measured between the seawall face and the MSL contour on the beach face, that are necessary to provide different standards of defence (based on the return period of severe wave and tidal level conditions) without causing seawall failure, namely:

Return Period (years)	Min. beach width (m)
1	7
10	12
100	17

These beach widths can therefore be regarded as appropriate "target" values to be achieved all along the study frontage, and at all times of year. If at any location the beach width reduces to below 17m, then the standard of defence at that point will be less than a 100 year standard.

The present beach widths at various points along the frontage, for example near Branksome Chine and Flag Head Chine are already substantially less than these "target" beach widths. Indeed, during winter months at these locations, the standard of defence is currently less than required to withstand a 1:1 year storm, leading to concerns about the stability of the seawall at these locations. The regular and sometimes substantial amounts of wave overtopping flooding the promenade that has been observed in recent years testifies to this assessment of the present poor state of the defences.

It is also worth making the point that once greater beach widths have been achieved, the present problems caused by wave overtopping will also be significantly reduced.

Some variation in beach widths is inevitable given the plan-shape of the seawalls, the effects of existing structures crossing the beach such as outfalls, and the variable but generally eastward drift of beach sand. If beaches are created that are substantially

wider than the required minimum width, these could provide extra amenity, recreational and possibly other environmental benefits. However, doing this could be regarded as a “disbenefit” when viewed from a coastal defence perspective, since a disproportionate amount of the recharge volume would be providing an increased standard of defence locally.

4.1.2 Maximising the lifetime and benefits of the recharge

A satisfactory increase in the standards of defence along the study frontage could be achieved, at least hypothetically, by recharge operations alone. However, the likelihood is that this possible solution would be expensive because of the frequency and volume of “top up” operations that would be needed; in addition, there would be a problem in maintaining minimum beach widths at certain locations along the frontage where the seawall is furthest seawards.

The role of beach control structures is therefore twofold;

- To produce a more even beach width than would be achieved using recharge alone; and
- To reduce the frequency and volume of repeated beach recharge operations by reducing the losses of sand along the coastline under the influence of longshore drift.

The initial and any subsequent maintenance costs of beach control structures thus have to be justified by the reduction in the costs of repeated beach recharge operations during the lifetime of the coastal protection scheme.

4.1.3 Requirements for technical feasibility and sustainability

It is a fundamental requirement that any beach control structures proposed can be successfully built, as designed, at the required locations and that they will last for the proposed lifetime of the overall scheme. It is likely that this latter requirement will be achieved most economically by allowing for intermittent maintenance of the structures, for example to repair damage caused by exceptionally severe storms, rather than by building much more robust structures initially. It therefore follows that these maintenance operations should also be easy to carry out.

This study specifically includes the consideration of novel as well as conventional options for beach control structures. Therefore, the requirement for “technical feasibility” is not a reason for only considering tried and tested structures, such as simple groynes. The assessment of this requirement will only be possible, for novel schemes, when the designs for the structures are nearing completion.

Similarly, the desire to use “sustainable” methods of coastal defence may have a considerable influence on the detailed design of beach control structures. This would be reflected, for example, in the choice and source of materials used for the structures, and indeed for the sand used in beach recharge operations. This level of detail is beyond the scope of this initial study.

However, it is worth making the observation that, in considering “sustainability” in the present study, the concepts of “re-cycling” the materials used and of “flexibility” in the chosen scheme are relevant. In regard to the first of these, any potential for the eventual re-use or disposal of the materials used to build structures may be worth considering.

The need for flexibility in coastal defence management is also emphasised in guidance from Defra, and this needs to be reflected in a consideration of how any particular scheme might be adjusted and refined after its construction, either to reflect changing circumstances, e.g. shifts in mean wave directions, or in light of the performance of the scheme not matching that predicted at the design stage.

4.1.4 *Effects on the adjacent coastline*

From a national or regional viewpoint, the benefit of improving coastal defence standards along one stretch of coastline would be reduced if, as a consequence, there were to be a reduction in the standards of defence along adjacent frontages. As usual, the concern about installing any beach control structures would be that beaches downdrift, in this case to the east along Bournemouth Borough Council's frontage, would be adversely affected. This can be avoided if the scheme adopted by Poole Borough Council does not reduce beach levels or drift rates at the very eastern end of its frontage, i.e. adjacent to the westernmost groyne installed by Bournemouth Borough Council.

In terms of the design of beach control structures along the Shore Road to Borough Boundary frontage, this requirement can most easily be met by matching the efficiency of the beach control structures at least at the eastern end of that frontage to that of the groynes installed by Bournemouth Borough Council. This type of arrangement has been agreed (informally) by other coastal authorities along the South Coast of England and has apparently worked well.

4.1.5 *Maximising the benefits of the scheme*

The very high value of the developments and infrastructure along the cliff tops between Shore Road and the Poole Borough boundary will mean that most beach enhancement/coastal defence schemes will achieve a high "Benefit-Cost" ratio, and this will be important in making a case for grant aid for a proposed coast protection scheme.

By considering a range of beach control structure (plus beach recharge) schemes, the most economically advantageous options can be identified using this criterion. However, there will always be scope to refine a proposed scheme so that this ratio is increased. This may be achieved by reducing the capital or recurring costs of the scheme elements, or by increasing the benefits that the scheme provides, by improving the defence standard or, finally, in less easily evaluated ways, e.g. improving the amenity or natural heritage benefits produced.

4.2 ASSESSMENT OF BEACH CONTROL SCHEME OPTIONS

4.2.1 *Introduction*

The objectives for the future beach management along the frontage between Shore Road and the Bournemouth Borough boundary are very challenging. It has been assumed in this study that beach recharge will be a fundamental element of this management, in combination with a number of "beach control structures". These structures might be groynes or reefs, or possibly some combination of the two, and set out in a fashion that reflects the specific character of the frontage, for example the variations in shoreline orientation, the plan-shape of the seawall with its various salients, and the embayments and the main access points to the beach.

At a later stage, any scheme option suggested will require detailed design and refinement, including computational modelling to predict the long-term effects on the beach morphology, together with consultations within Poole Borough Council and with a range of external organisations with interests in this frontage. It would, however, be prohibitively time-consuming and expensive to carry out such detailed design and refinement for each and every option for beach control structures.

In this preliminary study, we are therefore primarily concerned with identifying a range of possible schemes that can satisfy the “coastal defence” requirements at reasonable cost. As a first step, based on the review of options already presented in Chapter 2 (and Appendices 1-3) and the information on the characteristics of the study frontage summarised in Chapter 3, we have proposed a “short-list” of such schemes in section 4.2.2 below. Even this short-list, however, contains more options than could reasonably be carried through to a detailed design and refinement process.

The next step, therefore, was to compare the various options on the “short-list” in a qualitative manner, taking into account the wide range of objectives for an acceptable scheme as discussed in section 4.1. This comparison was tackled using a “multi-criteria” analysis method that is explained in section 4.2.3; the comparison technique is intended to be transparent, so that others can review the results obtained and carry out a similar comparison themselves, varying the importance given to the different objectives.

It should be noted that in the above summary, only the direct benefits of the options have been examined, i.e. those that relate to coast protection. There are other considerations, and “indirect” benefits associated with each scheme option, for example changes to the aesthetic and amenity characteristics of the frontage; these have been examined separately, in section 4.3.

This reflects that the choice of an appropriate coast protection scheme should initially be made by considering the direct benefits it provides, and then separately considering the options and their potential indirect benefits, considering amenity, public safety and aesthetics, all issues that are particularly important along this coastline because of the importance of tourism and holiday-makers to the local economy.

Once this comparison of schemes is concluded, it is intended that one or more “proposed” options would be put forward for wider consultation, both within the Borough of Poole and with external organisations/ individuals with a particular interest in the stretch of coast. This process should result in one (or more) “preferred” options being taken forward to the detailed design and refinement stage, to better achieve the objectives of maximising benefits, ensuring public safety and improving the “sustainability” of the works that are actually carried out.

4.2.2 Short-listing of beach control scheme options

The review of beach control structures, summarised in Chapter 2 of this report, has shown that there remains a wide divergence of opinion on the advantages and disadvantages of various types of groynes, of detached (emergent) breakwaters and reefs/ low-crested breakwaters. This divergence of opinion in part reflects the availability of construction materials locally, and this is a major factor in the costs of installing such structures in different locations. However, opinions are also influenced by perceptions of the performance, longevity, safety and aesthetics of different types of structures. There is little research or evidence to provide a methodology for comparing these factors for different proposed beach control structures. Coastal managers have

thus often opted for the *status quo* when considering replacing existing structures, albeit with small improvements to the design to reflect past experience on their performance or to reduce construction/ maintenance costs.

While this often avoids the difficulties that always occur when trying to introduce changes to a largely conservative and sceptical public, it inhibits the introduction of new and potentially better schemes. One of this study’s aims was to explicitly consider “novel” beach control structures, as well as “traditional” options. However, given the wide range of possible control structures that are available, it is not practicable to pursue each option, or combination of options, through the necessary numerical model testing, cost estimating and detailed design process before deciding on a preferred scheme for the study frontage. It was therefore decided, from the outset of this study, to produce a “Short List” of beach control schemes suitable for this particular frontage.

Based on the review of experience elsewhere, and based on views expressed during project meetings with council officials and Defra, it was decided not to include the option of “emergent” breakwaters or “fish-tailed” groynes in the short-list, since these have not proved, in our view, to have any clear advantages over other techniques. In addition to the lack of “hydraulic advantages”, there are considerable potential disadvantages in these types of structure, particularly in regard to the likely perception on their impacts on the aesthetics and safety of this stretch of coastline. The remaining options considered can be separated into two classes, namely groynes, of various types and dimensions, and submerged breakwaters/ reefs.

As pointed out above, once a short-list of options has been chosen, and ranked using the weighting of the various criteria proposed, it will then be necessary to carry out further refinement of at least the most promising of these. Because of this, it is not worth trying to define a beach control option to be included in the short-list in great detail; any such scheme will be probably be adjusted at a later stage. Thus, in the present chapter we have not chosen to differentiate between “timber” and “rock” groynes of the same basic functional dimensions, i.e. their length and spacing along the beach. Such “design details”, and their implications on costs, aesthetics and public safety, will be considered at a later stage in the design process, when fewer scheme options are being pursued. However, in Section 4.3, where amenity and aesthetics are examined, such aspects are considered since the material type etc. does have a significant impact, particularly as regards its acceptance by the public.

Table 4.1 lists our recommendations for options that should be included in the “short-list” and is followed by a commentary describing each of these options in greater detail.

Table 4.1 Suggested short-list for beach control options

1. Rock/ timber groynes	Length: 70m-80m	Spacing: 140m - 160m
2. Rock/ timber groynes	Length: 70m-80m	Spacing: 210m – 240m
3. Rock/ timber groynes	Length: 70m-80m	Spacing: 280m – 320m
4. Rock/ timber groynes	Length: 100m-120m	Spacing: 200m – 240m
5. Rock/ timber groynes	Length: 100m-120m	Spacing: 300m – 360m
6. Rock/ timber groynes	Length: 100m-120m	Spacing: 400m – 480m
7. Permeable groynes	Length: 70m-80m	Spacing: 140m - 160m
8. Permeable groynes	Length: 100m-120m	Spacing: 200m – 240m
9. Multi-purpose reefs	(4 No) (See Appendix 2 - dimensions to be refined)	
10. Nearshore reefs	Length: 100m (4 No)	

The following paragraphs briefly describe these ten short-listed options.

Options 1-3 Impermeable groynes 70m – 80m long

Dealing first with “conventional” impermeable straight groynes, we have paid particular regard to the existing groynes in Poole Bay and the recommendations of the recent Strategy Plan for this frontage (Halcrow, 2004b). Our opinion is that groynes as short as those presently found along the eastern end of Poole Borough’s frontage (about 45m long) are too short to be effective, particularly if used in conjunction with major beach nourishment. In contrast, the timber groynes along Bournemouth Borough’s, typically about 70m long, have been shown to be effective by the results of a prolonged data collection campaign, and their design and maintenance has been improved in recent years to further improve their performance.

Therefore, the first groyne scheme suggested in the above short-list of options is essentially that used in the adjacent section of Poole Bay, i.e. about 75m long groynes at approximately 150m spacing. Note that this suggestion does not rule out rock groynes with the same overall length and spacing, with or without a “walkway” along their crests; an example of just such a groyne has been built at Sandbanks recently (Groyne 26), see Plate 4.4. We have allowed for some latitude in the dimensions of these groynes, partly to allow for the widening of the beach after recharge and partly for convenient placing of the groynes relative to particular locations along the frontage.

Whilst the groynes installed along Bournemouth’s seafront have proved effective, the range of wave directions that arrive along Poole Borough’s frontage is less than that experienced at, for example, Boscombe. This reduced range of wave directions suggests that the possibility of widening the gap between 70m – 80m long groynes should be considered. The review of groynes on sandy beaches presented in Chapter 2 of this report indicated that a spacing to length ratio of 3:1 was not uncommon. Indeed, some groyne schemes in the UK have been successful with a ratio as large as 4:1. Any increase in spacing between groynes will obviously reduce the costs of an overall scheme, and its impacts on the aesthetics of the beach. We have therefore proposed two further groyne scheme options on this basis.

Options 4-6 Impermeable groynes 100m -120m long

Some of the groynes built recently at Sandbanks were 100m -125m long (numbers 27-29). These have proved very effective in improving beach levels, although they were actually designed specifically to “repel” the East Looe tidal channel, which is situated just off their ends. The recent Strategy Study (Halcrow, 2004b) also considered the use of groyne lengths of 100m (and indeed 150m) and showed that these were practicable in the central part of Poole Bay. Because the beach gradient is shallower along the Poole Borough frontage than along the beaches further east in Bournemouth, the construction of 100m – 120m long groynes is entirely practicable (i.e. without ending with the groyne ends in water that is too deep). We have therefore included such groynes in our short-list, with spacing to length ratios of 2:1, 3:1 and 4:1, as previously. However, we do not feel that groynes that extend onto the “bar” on the beach profile at Poole are a good idea and we have thus discounted groynes as long as 150m as an option along this frontage. Such long groynes may prove a hazard to boats as well as running the risk of forcing the sand bar further offshore. It is for this latter reason that building out over a bar is not recommended, for example in The Netherlands.

Options 7-8 Permeable groynes

The majority of groynes along the shoreline of Poole Bay are intended to be impermeable, i.e. designed to prevent water (and hence sediment) passing through them. Indeed, this is the type of groyne most widely used around the whole of the UK coastline. There are, however, exceptions to this general “rule”. There are two types of

“permeable” groyne along Bournemouth Borough’s coastline to the east of the Poole frontage. One pair of groynes, situated just to the east of Bournemouth Pier, was built in the 1960’s to a design introduced by an engineer called Makepeace-Wood. These are built using a lattice of concrete beams, both along their length and laterally. Further east still, there are two permeable timber groynes.

The former two groynes have worked well, producing a locally wider beach on either side of each groyne and creating a much less marked “saw-tooth” plan-shape than the timber impermeable groynes nearby. Like the recent rock groynes built at Sandbanks, these concrete permeable groynes have a horizontal walkway along their crest (see Plate 4.5). The main disadvantage of these groynes, however, is the cost of their construction, which greatly exceeds that of conventional timber groynes of the same length.

In contrast, the two timber permeable groynes further east along the Bournemouth frontage have suffered considerable damage and proved much less effective at promoting a locally wider beach.

While UK groynes tend to be impermeable, in other countries there is a growing realisation of the advantages of permeable groynes, as mentioned in the review of literature presented in Chapter 2 and Appendices 1 and 2 of this report. The simplest type of permeable groyne is a row of timber piles, placed at increasing distances as one travels seaward. However, such groynes are particularly susceptible to damage from marine borers and the abrasive effects of sand passing between the piles. More complex structures, such as the Makepeace-Wood groynes, as well as the timber groynes of broadly similar type built at Lido di Jesolo in Italy (see Appendix 3) are more expensive but offer some advantages from a recreational / amenity viewpoint. There may be concerns with such structures that beach users, especially children, might become trapped under the deck, but this has not been reported as a problem as far as we know.

Due to such groynes being novel and hence not widely used, both the prediction of their impacts, and guidance on their dimensions and spacing, are open to more debate than for impermeable groynes such as those already employed along Sandbanks and elsewhere. At Lido di Jesolo, for example, 50 permeable groynes were constructed at a spacing to length ratio of approximately 2.2:1 (see Appendix 3), whilst the Bournemouth permeable groynes have a 2:1 ratio. Dette et al (2004) suggest that optimum spacing is dependent on the length of groyne influence, with a resultant maximum spacing: length ratio of 5:1 therefore being possible. In view of this, and given the novelty of this type of structure, we have suggested in our short-list that such groynes would need to be built at a spacing: length ratio of 2:1, as at Bournemouth.

Options 9 - 10 Nearshore reefs

The use of reefs as beach control structures is a recent development, and one that, so far, has not been used in the UK. By carefully locating such structures to the seaward of parts of the frontage where the beach is particularly narrow, e.g. where the seawall has a seaward-projecting salient, reefs are able to increase beach widths locally, thus increasing the standard of defence at such vulnerable locations.

There is, however, little in the way of general guidance on the required length, distance offshore or crest height of individual reefs, or on the spacing between them, that can be used to define an overall scheme for the short-list. In Appendix 2, an outline, or “conceptual” scheme using “multi-purpose” reefs has been proposed by ASR Ltd., and this is included in the short-list as Option 9. These reefs principally achieve changes in

the beach plan shape by forcing waves to change direction, although they do also cause wave breaking some distance offshore. One disadvantage of such a scheme, however, is that it would not reduce the long-term net drift of sand along the coastline, as groynes would be designed to do.

An alternative to the “multi-purpose” reef would be to build nearshore reefs, which absorb incident wave energy rather than redirecting it. Such reefs have been developed in Italy to replace nearshore detached breakwaters, in order to avoid the adverse effects on water quality and aesthetics that those “emergent” structures have been shown to cause. Previous laboratory testing at HR Wallingford of a scheme involving nearshore reefs, and practical experience of schemes elsewhere in Europe, has indicated that such structures can both increase beach widths locally (i.e. in their lee) and reduce drift rates. However, most schemes of this type have suffered from scour problems along their seaward toe, sometimes leading to settlement and a need for maintenance/reconstruction of the reefs. Furthermore, as has been found for multi-purpose reefs, there is little guidance on the design of the alongshore spacing between reefs. In practice, many schemes have the gaps between reefs at about the same dimensions as the length of each individual reef.

Therefore, for both options 9 and 10, we have assumed that a wide spacing is practicable, so that only four reefs might be needed along the whole frontage between Shore Road and the Borough boundary. It seems likely that more study and refinement of these reef options may be needed than for some of the other short-listed options (impermeable groynes, for example). The optimisation of the multi-purpose reefs, in particular, will require specialised numerical modelling to refine their dimensions, orientation and location.

4.2.3 *Comparison of beach control scheme options*

In order to compare the various options examined in this study, it is necessary to decide on the basis by which one option might be judged to be preferable to another. As mentioned above, there is no “universal” method for this, since in practice this involves using value judgements, as well as applying more tangible criteria such as likely construction costs.

In the following, therefore, we have suggested a number of criteria that can be used to compare options and, for each of these criteria, an associated “weighting” reflecting its importance in the comparison process. These criteria are listed below, with an initial suggestion for their weighting. The subsequent application of this simple “Multi-Criteria” assessment can be repeated very simply, if required, with alternative values for the weighting factors, thus illustrating the effects of any criterion on the ranking of the options considered.

For schemes involving beach recharge, the primary purpose of the beach control structures, as discussed in section 4.1, is to maintain an adequate beach width along all points of the frontage from Sandbanks to the Borough Boundary, so that it is the combination of the seawall and the beach, which provides an appropriate standard of defence against coastal erosion and flooding. This is the most important requirement of any scheme of this type and is therefore given an appropriately high ranking in the table below.

Next we have considered the costs of the beach control structures (initially for construction and subsequently for maintenance), and their efficiency in retaining the

sand used to recharge the beach. Until more detailed plans for the various options are prepared, the comparison both of the costs and of the efficiency of the structures at reducing losses of sand alongshore has been carried out on a qualitative, comparative basis.

Also important is the effect that any scheme on the Poole Borough frontage is likely to have on the adjacent beaches of Bournemouth. Along the Poole frontage, a series of groynes that are too efficient in trapping longshore drift (for example 150m long ones) could alter the hydrodynamics and might cause some unanticipated changes in tidal behaviour on both the Poole and the Bournemouth frontages. We thus regard such a scheme as being “environmentally unacceptable” and deem this criterion as of high importance. However, because sand is readily transported in suspension, none of the schemes that are considered here are likely to have a significant detrimental impact on Bournemouth’s beaches. Indeed, one would argue that any scheme involving beach nourishment along Poole Borough’s frontage in Poole Bay would be beneficial to both the Poole and Bournemouth frontages. For this reason, whilst it is recognised that the criterion of “downdrift impacts” is an important one, the actual weighting attached to it is a neutral one (see the table below).

It is desirable that, in any scheme that is ultimately adopted, it should be possible for any beach control structures to be maintained relatively simply, so as to compensate for normal “wear and tear”. It would also be useful for them to be easily adjusted, to improve their performance (e.g. by means of altering their cross-sectional profile to suit changes in the adjacent beach levels). In practice, however, such adjustments are rarely carried out; alternatives such as mechanical recycling or bypassing of sand may be more appropriate along the study frontage than changing the dimensions of the structures themselves. In view of this we have assessed this aspect of any potential scheme to be of moderate importance.

We have also deliberately suggested that, at this strategic review stage, a low importance should be attached to the consideration of whether a particular scheme is well tried and tested, or indeed whether it is a novel one. It is hoped that by identifying this factor explicitly, and attaching a low weighting to it, novel structures such as reefs or permeable groynes will be considered on a more even footing than perhaps has been the case in the UK in the past.

Table 4.2 Assessment criteria and suggested weighting

Assessment criterion	Importance	Weighting
Maintaining adequate minimum beach width	H	8
Slowing longshore drift /sand losses	M	6
Construction costs (relative)	M	6
Ease/ costs of maintenance/ adjustment	M	4
Impact on downdrift beaches (Bournemouth)	M	5
Tried and tested scheme	L	2

Turning now to possible criteria dealing with the indirect benefits and aspects of a coastal protection scheme, such as aesthetics, amenity and public safety, there are difficulties in considering these in the same manner as the more tangible engineering criteria already discussed above. For example, the public perception of the aesthetic merits of alternative structures is exceedingly difficult to establish, beyond perhaps the reaction that new types of structure are often regarded with suspicion, even hostility initially, but often accepted without comment a few years later. Similarly, there is no

doubt that whatever structures are built, their design and construction will need to take into account their impacts on amenity and public safety, and measures taken to ensure that they are satisfactory in these respects. This may all lead to prolonged discussions about structures that, from a “coastal engineering” viewpoint, have little to separate them from the viewpoints of costs or performance.

We have therefore considered such criteria separately from the more tangible “engineering and efficiency” criteria and discuss the aesthetic/amenity/safety considerations in a separate section of this report (Section 4.3).

4.2.4 *Maintaining adequate beach widths*

The main purpose of the proposed coastal protection works along the Sandbanks to Branksome Chine frontage is to improve the standard of protection to the cliff, provided by the seawall and beach. We have assumed during this study that the seawall is uniform along this frontage, and hence the aim will be to increase beach widths to provide an appropriately increased standard of defence. The proposed recharge will add to the present beach width, but care will be needed to ensure that this advantage is not compromised by allowing the beach to become narrow at particular locations; in this sense the standard of defence is only as good as that at the narrowest part of the beach.

The impermeable groyne options will result in a “saw-tooth” beach plan shape, with narrower widths on the downdrift, normally the eastern side of each groyne. For the impermeable groyne options, the difference in beach width either side of each groyne will depend on the spacing: length ratio of those groynes. Those at wider spacing will produce a greater difference in beach widths than those at closer spacing. Furthermore, for longer groynes this effect will be greater. The scoring for these options therefore reflects the distance between each groyne. For 110m groynes at 4:1 spacing (option 6), i.e. a spacing of 440m, the groyne bay width is almost three times that for the 75m groynes at 2:1 spacing, i.e. 150m. In contrast, permeable groynes are designed to minimise the “saw-tooth” pattern, and they are therefore scored higher under this criterion than any of the impermeable groyne options.

The reef schemes (options 9 and 10) would be specifically designed to increase the beach width at those points along the frontage where the seawall has a salient to seawards, and therefore will have a specific advantage under this criterion. However, as longshore drift removes sand to the east following every recharge, these salients may only be maintained at the expense of narrower beaches along the intervening frontages.

Evaluating the minimum beach width necessary to achieve a particular standard of defence at all points on the frontage, as well as assessing longshore drift rates throughout, can only be made by detailed numerical modelling of each scheme. For the present initial study, however, the comparison of the effectiveness of the various schemes on the short-list in maintaining an even beach width has been carried out using engineering judgement and general experience obtained from inspecting many such schemes. On this basis the scores for each option, for maintaining a suitable beach width, are presented in the table below.

Option scores for this criterion

Option	1	2	3	4	5	6	7	8	9	10
Score (out of 10)	7	6	5	6	5	4	8	8	9	9

4.2.5 Efficiency at retaining beach recharge

The second criterion used to judge the short-listed beach control options reflects the desire to retain the sand imported during beach recharge schemes for as long as possible. While none of the schemes proposed will prevent material from being transported offshore during storms, this is not a serious issue, as beach monitoring has shown that material will tend to return in calmer weather. The greatest net loss of sand will be caused by longshore drift, especially if the beach width in front of the seawall is increased to prevent the present problems of scour. By this reasoning, those options that restrict drift more are to be preferred under this criterion to those that have less effect on the longshore movement of sand.

The importance of this criterion is dependent upon the availability of sediment for recharge. If it was certain that Poole Harbour would be able to supply the annual requirements for beach nourishment indefinitely, then the importance of the control structures' ability to retain material by reducing longshore drift decreases, as pointed out by ASR Ltd (see Appendix 2). In this situation, schemes such as the reefs, which shape the beach rather than trap the sand on anything more than a short-term basis, would score more highly. However, given current concerns for the future long-term sustainability of sand recharge from this source, it becomes more important that the defence schemes are able to retain the recharged beach sand for as long as possible.

The relative performance of impermeable groyne options is assessed bearing in mind their lengths and their spacing. The 110m long groynes are more efficient at blocking sediment transport than the 75m groynes, and their relative performance has been assessed using the information on the cross-shore distribution of the longshore drift presented in section 3.4.3 of this report. This shows that 110m groynes are about 10% more efficient than 75m groynes in retaining drift. Similarly, wider spaced groynes will allow a greater variation in beach positions, as described in the immediately preceding text, and this in turn will lead to a reduction in the efficiency of the groyne at the downdrift end of each groyne bay. A change in shoreline orientation of 10 degrees in each groyne bay will reduce the efficiency of the wider spaced groynes, by about 10% for a spacing: length ratio of 4:1 compared to a scheme using a ratio of 2:1.

Permeable groynes are designed to have less influence on beach plan-shape changes in their vicinity, but at the expense of allowing a great proportion of the longshore drift to pass through them. We have assumed for this exercise that these groynes have a 60% efficiency compared to impermeable groynes of the same length.

Multi-purpose reefs have a strong initial effect on the longshore drift in their lee, but this diminishes very greatly thereafter (see section 2.6 of Appendix 2). We have therefore assumed that, in the long-term, these structures only reduce longshore drift by 10% compared to a 75m long impermeable groyne. Finally, nearshore reefs will also have a stronger initial effect on drift rates as the beach plan-shape changes in their lee. Subsequently, however, their efficiency declines and we have assumed a value of 20% of the efficiency of a 75m long impermeable groyne. This produces the following table for scores of the short-listed options under this criterion.

Option scores for this criterion

Option	1	2	3	4	5	6	7	8	9	10
Score (out of 10)	9	8.5	8	10	9.5	9	5.5	6	2	2

4.2.6 Capital costs

The costs of installing the proposed beach control structures can be evaluated relatively easily for the impermeable groyne options (1-6) on the short-list, based on experience at Sandbanks and along the Bournemouth seafront.

Broadly speaking, the costs for these options will mainly depend on the spacing: length ratio, with schemes at 4:1 spacing being 50% of the cost of those at 2:1 spacing, simply because fewer structures will be required in the former case. The length of each groyne will also have an effect on the overall costs of the scheme, since the cost per metre length of construction of a 110m groyne will be greater than for a 75m long groyne. This is particularly the case for rock groynes with a level crest, since the cross-sectional area of these will increase with the square of the distance offshore. Vertical timber groynes 110m long will also require greater strength at their seaward ends than those that are 75m in length. We have assumed that a 110m long groyne will be 60% more expensive than one of 75m.

There is little in the way of past experience to assess how expensive permeable groynes (Options 7-8) may be to build initially, or to maintain. The simplest type of such a groyne, i.e. a row of piles at variable spacing, may actually be cheaper to build than an impermeable groyne of the same length, but experience suggests that such a structure may be damaged more easily and hence have a substantially reduced life-span. Conversely, more substantial groynes such as the Makepeace-Wood concrete groynes at Bournemouth may have comparable maintenance and life-times to impermeable groynes but will be more expensive to design and build. For the purposes of this initial study, we have assumed, therefore, that the whole life cost of permeable groynes would be twice as much as impermeable groynes of the same length.

Options 9-10 of the short-list envisage a small number of “reefs” and these will require marine construction. Initial costs for the “multi-purpose” reefs suggested by ASR Ltd. (see Appendix 2) have been provided assuming that they are built using sand-filled geotextile containers (in effect large sand bags). The cost of each the four reefs proposed was about the same as that of a conventional 110m long impermeable groyne. There is little information, for such a novel coastal protection scheme, on the likely life-span of a reef of this sort, but according to ASR they may last as long as a conventional timber groyne. We have therefore based the comparative costing of Option 9 on the basis of each reef having the same whole-life cost as a 75m long impermeable groyne.

For Option 10, the nearshore reef scheme, each proposed structure will be larger in volumetric terms than 110m long groynes, and would probably be built by “side-casting” suitable armour rock from a barge moored parallel to the coast at high water. There would be no need, as was necessary for the rock groynes at Sandbanks, to build a walkway on the crest of nearshore reefs. However, any adjustment of the rock to provide the correct cross-sectional shape would be more difficult than an equivalent operation on land. We have estimated the cost of each nearshore reef, therefore, to be 2 times that of a 110m long groyne, or 3.6 times that of a 75m long groyne.

For each of the ten options considered, therefore, we have calculated the number of structures needed and multiplied this figure by the appropriate cost of each (as a multiplier of the cost of a 75m long impermeable groyne). Using this methodology, the least expensive is Option 3 (75m long impermeable groynes with 300m spacing) and the most expensive is the 75m long permeable groyne scheme (Option 7).

Option scores for this criterion

Option	No of Structures	Cost Multiplier	Cost	Score
1	11	1	11	5.5
2	8	1	8	7.5
3	6	1	6	10
4	7	1.6	11.2	5.5
5	6	1.6	9.6	6
6	5	1.6	8	7.5
7	11	2	22	3
8	6	3.1	18.6	3.2
9	4	1.7	6.8	9
10	4	1.6	6.4	9

4.2.7 Ease of maintenance/ adjustment

The design of beach control structures has improved considerably in recent decades, as numerical models of their effects on long-term beach evolution have been developed, allowing extensive testing and refinement of schemes. Nevertheless, experience has shown that there are often opportunities to improve the performance of any scheme after it has been installed, to compensate for factors such as changes in the wave climate, concerns about public safety or aesthetics, deterioration in performance through normal “wear and tear” or simply the inaccuracies in the design process.

Potential changes to beach control structures can involve:

- Changing their crest level/ profile, e.g. by adding planks on a timber groyne;
- Changing their length, e.g. by adding to or removing part of the structure;
- Adding intermediate structures of a similar type; and
- Changing other features, e.g. altering the permeability of a groyne.

It is likely that such post-construction adjustments will be required less for “conservative” designs for structures and more for “novel” designs (see “Tried and Tested” heading below).

For assessing the relative ease with which such maintenance/ adjustment for each short-listed scheme can be executed, we have assumed that working in water will be more difficult than working on the beach, and further that where a structure has been built without using specialised plant or techniques it will be easier to adjust later.

The costs of maintenance and possible adjustments of each scheme will depend on the number of structures proposed. The ratio of the maintenance/ adjustment costs for 75m and 110m long groynes is assessed to be the same as that for the capital construction costs (see above).

It seems likely that maintaining or adjusting reef structures will need specialised plant and perhaps diving teams to guide such operations. We have therefore assumed that the cost ratio for maintaining or adjusting these structures will be substantially greater than for their capital costs, as set out below:

Option scores for this criterion

Option	No of Structures	Cost Multiplier	Cost	Score
1	11	1	11	5.5
2	8	1	8	6
3	6	1	6	10
4	7	1.6	11.2	4
5	6	1.6	9.6	5.4
6	5	1.6	8	7
7	11	2	22	2.25
8	6	3.6	21.6	2.25
9	4	4	16	3
10	4	7.2	29	1.75

4.2.8 Impact on downdrift beaches

In section 4.2.5 above, the effectiveness of the short-listed options at retaining beach sediments was evaluated. There is, however, a disadvantage to schemes that best match this criterion, namely that they will also prevent sand moving onto the adjacent “downdrift” coastline, in this case to the east and into the frontage of Bournemouth Borough.

At present, the short groynes along the eastern end of Poole BC’s frontage are much less effective at preventing sand travelling across the borough boundary than the most westerly groyne erected by Bournemouth Borough Council, which is about 75m long. Some of the short-listed options identified in this report would alter this situation, and could result in “transferring” problems of retaining adequate beach widths from Poole to Bournemouth.

In the light of this, it is worthwhile considering the implications of the various schemes proposed on the beaches further east. At present, it could be reasonably argued that installing groynes that match those built by Bournemouth would not be detrimental to either authority in respect of maintaining beach levels. However, it seems likely that rock groynes with a level crest and 75m long would be more efficient at retaining sand than Bournemouth’s sloping timber groynes of the same length, and might therefore result in some “sediment starvation” further to the east.

This type of effect will need to be examined in more detail for the preferred scheme option(s) selected by Poole Borough Council, for example by modelling the beach plan shape evolution on both sides of the borough boundary. In order to consider the possibility of causing downdrift erosion when assessing the short-listed options, however, a simplified assessment of this concern has been made as follows.

For a hypothetical scenario of no groynes in place, a value of 0% groyne efficiency and no downdrift impacts a score of 10 has been assigned here. At the other extreme, a value of 100% effectiveness, and hence with maximum possible detrimental impact (and hence a score of 0 out of 10) is assigned to a hypothetical groyne 150m long, which is assumed to trap virtually all the longshore drift (in reality it will trap most, but not all of the drift).

Using this technique the 75m long impermeable groynes (options 1 to 3) are assigned a 50% effectiveness in terms of downdrift impacts and hence a score of 5 out of 10. By the same reasoning the 110m long groynes (options 4 to 6) are assigned a 75%

efficiency in terms of downdrift impacts and hence a score of 2.5 out of 10 for avoiding them.

From our experience permeable groynes of the Makepeace-Wood type have about a 60% sediment trapping efficiency, compared to impermeable groynes of the same length. Therefore, the 75m long permeable groynes (option 7) are assigned a 30% effectiveness in terms of downdrift impacts and hence a score of about 7 out of 10. The 110m long permeable groynes (option 8) are then judged to have 45% effectiveness and a score of 5.5.

The multi-purpose reef structures (option 9) have a much lower efficiency with regard to trapping sediment, although they can be expected to produce a salient in their immediate lee. Similarly to groyne structures, it is the impact of the easternmost structure, rather than a combination of all the structures intended, that is important in evaluating downdrift impacts. Although we have only second hand information about their likely effectiveness, we consider that they will be less likely to create downdrift impacts than any other of the options. Therefore a score of 9 is assigned to them. Nearshore reefs (option 10) act in a similar manner to multi-purpose reefs, but, depending on the degree of submergence, are thought likely to be slightly more effective in dissipating wave energy, and hence more effective in reducing sediment transport. Therefore, a score of 8, higher than for the 75m long permeable groynes but lower than for the multi-purpose reefs, is assigned to nearshore reefs.

While this scoring is appropriate for a situation where there are no groynes along the downdrift coast, for this study there are rather long timber groynes immediately east of the borough boundary. Applying the above scoring, those existing groynes would score about 5. In the following table, the scores for all the options considered is given, but it should be noted that options that score less than 5 represent a potential adverse effect along Bournemouth's frontage, while those greater than 5 do not pose any significant risk of "downdrift erosion" east of the Borough boundary.

Option scores for this criterion

Option	Length	Score
1	75m	5
2	75m	5
3	75m	5
4	110m	2.5
5	110m	2.5
6	110m	2.5
7	75m	7
8	110m	5.5
9	N/A	9
10	N/A	8

4.2.9 Tried and Tested Scheme

As part of the remit for this study, it was agreed that both well tried and tested methods and novel approaches to controlling beach development would be considered. Necessarily, there will be less information on which to judge the likely performance, or cost of construction, of a novel beach control scheme than for simply continuing to build structures that have already been designed and installed in Poole Bay. However, the continuation of established practice may not provide the optimum performance or

cost-effectiveness, and new types of structure may prove to have advantages in this regard. It is also worth noting that all schemes proposed would have to be extensively tested, refined and detailed to ensure that it would meet the basic objectives of beach control structures.

In addressing this criterion, we have simply classified the options using the following broad headings:

“Conservative”

Schemes that have been designed well within the limits of well-established guideline, have been built and maintained without great difficulty and have proven to be effective at numerous similar coastal sites in the UK (including nearby frontages).

“Unusual”

Schemes that involve structures whose dimensions, construction type and /or their geometry are approaching or are at the limits of guidelines for beach control schemes for the site under consideration, and which have only been occasionally built elsewhere.

“Novel”

Schemes that involve types of structure that have very rarely or not previously been used in the UK, or which are designed outside established guidelines for their dimensions, construction type and /or their geometry.

Whichever of these categories a “short-listed” scheme is assigned to, that scheme would be subject to detailed modelling and evaluation at a later stage during the detailed design process. However, it is more likely that a “Novel” scheme may prove to be unsuitable (e.g. less effective or more expensive) than a “Conventional” scheme at this later stage. This possibility means that “Conventional” schemes are given a higher “score” under this criterion.

In assessing the scores under this criterion for each option, we have regarded impermeable groynes at a spacing: length ratio of 2:1 as “conservative” and given these a score of 9 (/10). Similar groynes at a spacing to length ratio of 1:3 and 1:4 are regarded as “Unusual” and “Novel” respectively (scores 6 and 3). Permeable groynes and reefs are rather unusual in the UK and are therefore regarded as “novel” and where the spacing between these is proposed to be large, this is interpreted as increasing their “novelty” and hence scoring somewhat lower.

Option scores for this criterion

Option	No of Structures	Score (out of 10)
1	11	9
2	8	6
3	6	3
4	7	9
5	6	6
6	5	3
7	11	4
8	6	3
9	4 (submerged)	2
10	4 (submerged)	2

4.2.10 Summary of direct benefits – Multi-criteria results

The following table presents the results of the multi-criteria analysis method applied to all ten of the short-listed options, using the weightings presented in section 4.2.3 and the scores for each of the criteria used as presented in sections 4.2.4 to 4.2.9.

This table has been produced using a spreadsheet, which can be rapidly adjusted to reflect any desired changes in the relative weightings used for each criterion.

The table shows that option 3 (short impermeable groynes at spacing: length ratio of 4:1) scores the highest with the weightings used (Section 4.2.3), with short impermeable groynes with spacings 2:1 and 3:1 coming second and third respectively, and multi-purpose reefs ranking fourth. Figures 4.1 to 4.2 illustrate potential scheme layouts for the three highest scoring options. All of the rest of the ten options score within the 160-190 range, with the permeable groynes scoring lowest at 161.7 for Option 8 and 167 for Option 7.

Table 4.3 Multi-criteria analysis results

No.	Option	Length	Spacing	Beach width	Slowing drift	Costs	Maintenance	Impact downdrift	Well-proven	Score
1	Impermeable groynes	75m	150m	7.00	9.00	5.50	5.50	5.00	9.00	208
2	Impermeable groynes	75m	225m	6.00	8.50	7.50	6.00	5.00	6.00	205
3	Impermeable groynes	75m	300m	5.00	8.00	10.00	10.00	5.00	3.00	219
4	Impermeable groynes	110m	220m	6.00	10.00	5.50	4.00	2.50	9.00	187.5
5	Impermeable groynes	110m	330m	5.00	9.50	6.00	5.40	2.50	6.00	179.1
6	Impermeable groynes	110m	440m	4.00	9.00	7.50	7.00	2.50	3.00	177.5
7	Permeable groynes	75m	150m	8.00	5.50	3.00	2.25	7.00	4.00	167
8	Permeable groynes	110m	220m	8.00	6.00	3.20	2.25	5.50	3.00	161.7
9	Multi-reefs			9.00	2.00	9.00	3.00	9.00	2.00	199
10	Nearshore reefs			9.00	2.00	9.00	1.75	8.00	2.00	189

By altering the criterion weightings, the above scorings will clearly change, although generally the highest-scoring three or four options described above still retain the same rankings if changes in weightings of only +1 or -1 are applied; for greater changes in weighting, often the highest-scoring 5 remain the preferable options, even if their order varies slightly. The two permeable groyne options consistently remain the lowest placed alternatives, unless some dramatic amendments to the weightings are made.

If the ability of the structures to slow drift becomes less important and the weighting decreased by one point, the rankings shown in the above table remain the same for the highest-scoring options, and similar for the rest. A more significant decrease in this criterion's importance (i.e. presuming that there would be a constant recharge supply) results in the multi-purpose reefs placing second, coming closely behind the short impermeable groynes with a 4:1 spacing: length ratio, with the nearshore reefs ranking third. This finding tallies well with the conclusion drawn in Appendix 2, and may indicate that an alternative scheme with only two such reefs (to improve beach widths in front of the "salients" in the seawall near Flag Head and Branksome Chines) would be worth considering under this assumption. It should be pointed out, however, that recent recharge schemes using sand dredged from the entrance channel to Poole Harbour may not be so readily carried out in future, and it may be necessary to re-nourish beaches in Poole Bay using sand dredged from the offshore seabed.

There are a multitude of possible weightings of the criteria, all with slightly different results, as has been briefly demonstrated above. However, unless priorities change drastically (i.e. away from the importance of maintaining appropriately wide beaches

and reducing longshore drift), the highest-scoring four options consistently remain in the top four positions, although potentially with slight variations to their rankings. Permeable groynes seem to be the least favourable option, despite their apparently increasing popularity in the USA and Germany. This appears to be mainly due to the initial costs of their construction, and the uncertainty of the success of these at a spacing of more than twice their length.

4.2.11 Conclusions from the multi-criteria analysis of short-listed options

As the highest-scoring four options consistently remain the top four, even if slight variations in weighting the criteria are considered, it is recommended that these remain the options carried forward for further study and modelling.

Based on the present weightings, that reflected the primary requirements for a better standard of defence that would be long-lived and economical, in terms of the “direct benefits” they would deliver, the best four options were concluded to be:

- Impermeable groynes, 75m long and spaced 300m apart;
- Impermeable groynes, 75m long and spaced 150m apart;
- Impermeable groynes, 75m long and spaced 225m apart; and
- Four “multi-purpose” reefs, as recommended by ASR Ltd.

4.3 ASSESSING INDIRECT SCHEME BENEFITS

The primary purpose of the beach control structures along the frontage from Shore Road, Sandbanks to the Borough boundary is to help retain a satisfactory beach, as part of the coastal protection for this stretch of coastline. The costs of their construction will be justified primarily on that basis. The direct benefit of a coast protection scheme arise from avoiding damage, for example the losses of cliff top properties, compared to the situation in which nothing is done to prevent such losses.

Where alternative schemes are proposed that achieve approximately the same “cost-effectiveness”, i.e. provide the same direct benefits relative to their costs, with both calculated over the life-time of the scheme, then it is appropriate to consider the options further, taking into account other benefits that they might, or might not, provide. This further consideration may involve refinement of the design of a scheme to further improve the direct benefit: cost ratio it achieves (often by reducing its costs), and/ or by considering how it might be modified to provide other indirect benefits, whether these can be evaluated financially or are less tangible. At this stage, it may even be that the preferred option for a scheme changes because of the differences in indirect benefits that arise.

At Poole, the beaches not only form an important component of the coastal protection to the soft cliffs just landward, but also a very important amenity for the Borough. While indirect, the financial benefits to the local economy of wide, safe sandy beaches are extremely important. Retaining such beaches for longer will therefore, for example, provide a greater benefit than if the recharged beaches were to dwindle quickly.

The “attractiveness” of the beaches to visitors and residents alike is also an important consideration, encompassing aspects such as aesthetic appearance, safety, easy access and recreational opportunities. Some of these indirect benefits are not easily valued, and indeed may not be agreed by everyone.

As mentioned previously, it is considered that the indirect benefits, and potential “disbenefits”, of the proposed schemes are best treated separately from the direct benefits related to their coast protection aspects examined in section 4.2 above.

4.3.1 *Aesthetics*

Any judgement made about the aesthetics of beach control structures is likely to be affected by personal preferences, and perhaps coloured by a leaning towards the “familiar”, rather than in the direction of the “new”. Further, the aesthetics of any scheme will depend to a considerable extent on the detail of the structures, e.g. the use of timber or rock in their construction and the presence or absence of “walk-ways” or other features. At this initial study stage, information on these design is not available.

The aesthetics of any proposed scheme have therefore been assessed primarily on the number of structures that would need to be installed between Shore Road and the Borough Boundary, assuming that many closely spaced groynes would be less attractive visually than fewer and more widely spaced groynes. Schemes involving submerged structures, that will not be visible at all, are therefore rated highest on this basis.

The schemes that score lowest on this criterion are the groyne structures, since they will be visible at all states of the tide. At this stage no difference in aesthetics is assumed for rock and timber groynes. It is assumed that the permeable groynes considered (options 7 and 8) would be built to similar dimensions as the Makepeace-Wood type groynes in the vicinity of Bournemouth pier (see Plate 4.5). It is therefore concluded that permeable groynes (options 7 and 8) would be fairly large and fairly visible. In terms of aesthetics, therefore they have been scored within the same range as the impermeable (rock) groynes in options 1 to 6.

Such distinction does not apply to the reef type structures in options 9 and 10 as they are both likely to be submerged for all or most of the time. Here the visual disturbance is likely to be minimal, except possibly during very low tidal states in combination with long period wave action. Under these conditions the crests of the structures, depending on the design adopted (i.e. the degree of submergence) might conceivably be exposed in the troughs of the waves.

Option scores for this criterion (aesthetics)

Option	No of Structures		Score (out of 10)
1	11		2.75
2	8		4
3	6		5
4	7		4
5	6		5
6	5		6
7	11		2.75
8	6		5
9	4	(submerged)	9
10	4	(submerged)	9

4.3.2 *Public safety*

All proposed beach control schemes will be individually assessed at the detailed design stage, in the light of the hazards they pose to beach users and to “mariners”, i.e. to boats of various sizes, personal water-craft etc. In addition, the requirements of pedestrians,

fishermen etc. will also be taken into consideration. Thus, changes to the design would aim to eliminate hazards as far as is practicable.

Making an assessment of the public safety risks of each of the options in the short-list is necessarily a subjective exercise, since there are no good statistics for accidents associated with the many different types of beach control structures under consideration. Note also that where accidents on beaches are reported, the location of an incident is often given by referring to a nearby groyne, although that structure may have had absolutely no influence on the accident itself. This latter point has been well-proven at Sandbanks where the reporting on minor injuries sustained by beach users has been changed in 2005, to separate the locations of the accident in relation to the groynes. This change dramatically reduced the apparent number of “groyne related” injuries between 2004 and 2005.

In the following paragraphs the safety issues associated with proposed beach control schemes are considered from the viewpoint of a number of different “user groups”, bearing in mind their different interactions with the structures. Any differences in safety aspects between groyne types (rock and timber), are discussed, where appropriate in the text and given different scores with those for rock groynes given in brackets.

Beach users (sunbathers, “sitters”, picnickers, games players etc.)

Beach users are not considered to be as mobile as some of the other user groups (surfers and swimmers for example) and the groyne spacing is therefore not a serious issue.

Both of the reef type options are considered too far offshore to be a hazard.

Children

All of the proposed groyne schemes pose a potential hazard to children and all the schemes involving groynes (permeable, impermeable, rock or timber construction) are assigned a lower than neutral score. Furthermore groyne size and spacing plays a role in safety; the greater the number of groynes there are on the beach the greater the risk of an incident and the longer the groynes the more they project into deeper and potentially more dangerous water. Surprisingly groyne type also matters in respect of public safety. Timber groynes are generally relatively safe and few incidents have been reported with this type of structure (children occasionally try and walk along their crests or try and jump over them). Rock groynes are potentially less safe for children in particular, as there is the possibility of children slipping on the rocks by trying to climb up the groynes. Sometimes children (as well as other users) will also try and jump from one stone to another. For this reason it is recommended that a walkway should be provided on the crest of such groynes to forestall any such tendency for misuse of the groynes. We would not recommend the use of rock groynes on popular tourist beaches without such appropriate protection measures.

Permeable structures such as the Makepeace-Wood groynes used at Bournemouth are considered to be about as safe as impermeable groynes of similar length/spacing, although the potential hazards are a little different. We consider that the risk of children actually getting trapped underneath them is low.

Both of the reef type options are considered too far offshore to be a hazard to children.

Walkers

For walkers, groynes can lower the safety of a beach as they present a potential trip hazard for anyone attempting to walk over them. The spacing, and therefore number of

groynes, is thus an important issue, as the more structures that are present on the beach the more risk they represent to walkers.

The offshore structures (options 9 and 10) clearly do not affect walkers directly and allow walkers unrestricted access to the whole beach and therefore achieve a high score.

Swimmers

Structures that produce rip currents are especially hazardous to swimmers.

Longer groynes are clearly more hazardous than shorter ones, since they extend into deeper water; the risks of collisions, rip currents and beach users falling into deep water are therefore somewhat greater. Wave action at the seaward ends of the structures is also likely to be significantly greater, potentially sweeping the swimmer against the structure. By their very presence such structures will also encourage swimmers to swim out to their seaward ends.

The reflective faces of timber groynes give no “purchase” for those trying to clamber out of the sea in an emergency and are therefore potentially more hazardous than the (sloping) faces of rock groynes. Similarly it is likely that any seaward flowing (rip) currents will be more strongly concentrated along the smooth straight and vertical face of a timber groyne than that of a rock groyne, and hence be more hazardous. The latter type of structure, however, is more likely to cause abrasions to swimmers that venture too close to them.

Thus, overall groynes constructed of rock are likely to score slightly better than timber groynes on the grounds of public safety.

The risks of rip currents becoming a hazard to swimmers may be decreased by installing permeable groynes; by their porous nature these structures will tend to prevent return flows becoming concentrated along their sides. However, they may pose a risk of swimmers being swept through them, or becoming trapped underneath them, a problem that does not arise with impermeable structures. Note that this has not been reported as a particular problem at Bournemouth, where two such structures have been in place for many years and we consider the likelihood of this occurring to be very low.

The hazards posed by multi-purpose reefs are (frankly) unknown. However, if such structures modify the wave climate sufficiently to improve surfing conditions then one can envisage that conditions for swimming may be affected indirectly. It is predicted that quiescent areas will occur in the lee of the structures after the waves have broken. If the multi-purpose reefs just re-orientate the waves, then no quiescent areas suitable for swimming will be created. Appendix 2 points out that (rip) currents in the lee of such structures may be reduced by careful design and this could be a possible advantage if there are problems associated with such strong currents at present. The risk to swimmers will therefore be reduced and a relatively high score will be appropriate for these structures.

Nearshore reefs, whose purpose is to reduce wave energy by breaking, will similarly create quiescent areas in the lee of the structures thus providing a safer swimming area. Therefore a relatively high score seems most appropriate.

Disabled visitors

For the purposes of keeping this assessment relatively straightforward we consider disabled visitors in terms of “mechanical disability” i.e. the possible lack of mobility

rather than any other factors. In that case all groynes structures achieve, in this assessment, a neutral score. The safety factors are not realised in this instance, because disabled visitors are less likely to be clambering over rocks and being hurt.

Both reef options will also achieve a neutral score in that they will have little impact on this user group.

In addition, any scheme that produces a significant build up of beach material could be utilised in providing better access to the beach for disabled people, avoiding a significant “drop” from the promenade onto the beach and therefore presenting less of a hazard. However this criterion has already been considered in section 4.2.4 and further analysis would lead to a potential overemphasis.

Boat Users

For groynes, the main hazards to boat users will be their seaward projection, particularly from any underwater sections of the structures that are not easily visible.

It is generally the case that the further any beach control structure projects out from the shore, and hence into deeper water, the greater the associated hazards to the public will be. Longer groynes will tend to intercept more of the longshore currents and hence produce stronger currents at their heads, which are in deeper water than for shorter groynes. Shorter groynes, which extend into shallower water, are less of a navigation hazard, because for example larger boats will be more likely to have grounded before reaching their seaward ends.

Wider-spaced groyne schemes will involve fewer structures and hence reduce the risks of collisions.

It is difficult to assess the reef designs in terms of public safety, as no such structures have been built yet in this country. These structures will modify the incident wave conditions and if surfing waves are produced these would be a potential hazard to boat users (although boat users are the greater hazard to the potential reef users, see below). The principal concern is that such submerged breakwaters will not be seen, nor recognised as a potential hazard by inexperienced boat users, or those using personal water-craft such as jet skis. It is unclear at present whether such structures would need to be marked as a navigation hazard.

Shore-based fishermen/naturalists/birdwatchers etc.

Rock and timber groynes rate very differently for this user group. This user group is unlikely to be affected (beneficially or otherwise) by the presence of timber groynes; hence a neutral ranking has been assigned to these types of structure.

Rock groynes with a walkway will be very popular with this user group and therefore a certain amount of risk should be assigned to these structures. Similarly permeable groynes such as the Makepeace-Wood groynes at Bournemouth are considered to be as safe as rock groynes.

This group will not be significantly affected by the presence of submerged structures offshore.

Surfers

Any non-submerged coastal structure poses a potential risk to surfers, irrespective of whether they are kite surfers wind surfers or wave surfers, and most of the short-listed schemes will score poorly in terms of the safety of this user group at Poole.

All the groyne options (1 to 8) score poorly under this criterion. As with the concerns about boat users, the fewer groynes present, i.e. those schemes with wider spacing, the smaller the risk of collisions.

It is presumed that option 9 (multi-purpose reefs) have been built with this user group in mind and the safety aspects are therefore not considered for this user group and this option will have a neutral score.

Submerged nearshore reefs (option 10) pose a real hazard to surfers of all types, as they will be almost invisible but possibly shallow enough to cause “snagging”. Experience with their usage in the Mediterranean, in areas with a similarly small tidal range as in Poole Bay, indicates that under-scour (and hence strong currents) may develop around such structures. In our opinion this would almost rule them out on a tourist beach, as we are unsure whether hazards posed by such structures could be eliminated by careful design.

Option scores for public safety (out of 10)

NB Scores in brackets, where applicable, are for rock groynes

User Groups	Scheme options									
	1	2	3	4	5	6	7	8	9	10
Beach users	5	5	5	5	5	5	5	5	9	9
Children	2.5 (2)	3 (2.5)	3.5 (3)	3 (2.5)	3.5 (3)	4 (3.5)	2.5	4	9	9
Walkers	2.5	3.5	4	3.5	4	4.5	4	4	9	9
Swimmers	2 (2.5)	3 (3.5)	3.5 (4)	2 (2.5)	3 (3.5)	3.5 (4)	2.5	4	7	7
Disabled visitors	5	5	5	5	5	5	5	5	5	5
Boat users	3	3.5	4	2	2.5	3	2.5	3.5	2	2
Fishermen etc.	5 (4.75)	5 (4.75)	5 (4.75)	5 (4.75)	5 (4.75)	5 (4.75)	4.75	4.75	5	5
Surfers	2.5	3.5	4	2.5	3.5	4	2.5	4	5	2
Total	27.5	32	34	28	32	34.5				
	(27.25)	(31.75)	(33.75)	(27.75)	(31.75)	(34.25)	28.75	34.25	51	48
Average	3.4	4.0	4.3	3.5	4.0	4.3	3.6	4.3	6.3	6.0

4.3.3 Amenity

The amenity value of any of the proposed beach control schemes can be considered as a useful or pleasant feature or facility that they provide. In context to the coastal zone this would include, but not be limited to, amount of available beach space, beach access and recreation facilities.

Beach users (sunbathers, “sitters”, picnickers, games players etc.)

Amenity value for beach users can be considered in terms of the amount of beach space available i.e. the bigger the footprint of a structure on the beach, the less space there is available for this user group. All successful groyne designs should be such that individual groynes will be spaced sufficiently far apart that beach usage should not be seriously affected by their presence. The amenity value to this user group is therefore based on the number of structures proposed. In this case both the reef options would score highly.

However groynes of all types also have the advantage of providing a wind break, which is quite valuable in the UK climate. Timber groynes have a smaller footprint than rock groynes so there is more beach area to sit on but people often enjoy sitting on the rock groynes; therefore there are no clear differences between the two groyne types.

Children

There is evidence that children are strongly attracted to rock groynes, and enjoy exploring them, looking for marine life within the rocks or fishing from their ends with simple baited lines. The rock groynes will therefore score higher than the other beach control structures.

Walkers

For walkers, groynes can lower the amenity values of a beach by preventing free passage alongshore. The spacing of groynes, and consequently the number of groynes, is therefore an important issue. The higher the number of groynes present on the frontage the more occasions walkers will need to divert onto the promenade. However, in the case of impermeable groynes constructed of rock, and permeable groynes, they can be designed to provide access along their crests, adding to the variety of the views of the coast. With groynes similar to those already built at Sandbanks, it is often possible to walk over their crests at their root, at least in summer when beach levels are higher, reducing the “dissection” of the beach somewhat. The offshore structures (options 9 and 10) clearly allow walkers free passage along the entire frontage and therefore achieve a positive amenity score.

Swimmers

As far as amenity is concerned, it would be best for swimmers to be discouraged from coming near such structures. Therefore, all the groynes have a neutral score in terms of improving/enhancing amenity (for swimmers). Both reef options will create quiescent areas, ideal for swimming, in the lee of their structures and will score highly.

Disabled visitors

The main amenity value for this user group comes from the accessibility of the beach. Any scheme that provides easier access to the beach will score highly. Access along rock or permeable groynes with a walkway on their crests provides an opportunity for disabled visitors that cannot be provided by conventional timber groynes. The multiple-purpose and nearshore reefs do not provide any amenity value to this user group.

Boat Users

All the structures score neutrally in terms of amenity, since none are intended to be used by boat users. Indeed, warning notices are likely to be provided, warning boat users from approaching too closely to such structures. It is possible that one or more of the rock groynes could be converted into a mooring facility, with dry access to the beach, or the reefs could provide unique fishing grounds for commercial fishermen but we have not considered this possibility within the present study.

Shore-based fishermen/naturalists/birdwatchers etc.

Rock groynes with a walkway such as the ones at Sandbanks are known to be very popular with this user group.

Like timber groynes the permeable groynes are unlikely to impact seriously on this user group with the exception of the few naturalists who might be interested in marine life around the structures.

Again, this group will not be significantly affected by the presence of submerged structures offshore, although shore fisherman may derive benefits from the niche habitats provided by both rock groynes and offshore reefs.

Surfers

Groynes (options 1-8) and submerged nearshore reefs (option 10) are unlikely to provide any amenity value to the coastal zone. By contrast multiple-purpose reefs achieve a high score from the viewpoint of improving amenity for surfers.

Option scores for this criterion (out of 10)

NB Scores in brackets, where applicable, are for rock groynes

User Groups	Scheme options									
	1	2	3	4	5	6	7	8	9	10
Beach users	5	6.5	7	6	7	8	5	7	8	8
Children	7.5 (8)	7 (7.5)	6.5 (7)	7 (7.5)	6.5 (7)	6 (6.5)	8	7	2	2
Walkers	3 (5)	3.5 (5)	4 (5)	3.5 (5)	4 (5)	4.5 (5)	5	5	9	9
Swimmers	5	5	5	5	5	5	5	5	9	9
Disabled visitors	5 (8)	5 (7.5)	5 (7)	5 (7.5)	5 (7)	5 (6.5)	8	7	5	5
Boat users	5	5	5	5	5	5	5	5	5	5
Fishermen etc.	5 (7.5)	5 (7)	5 (6.5)	5 (7.5)	5 (7)	5 (6.5)	7.5	6.5	5	5
Surfers	2.5	3.5	4	2.5	3.5	4	2.5	4	9	2
Total	38 (46)	40.5 (47)	41.5 (46.5)	39 (46)	41 (46.5)	42.5 (46.5)	46	46.5	52	45
Average	4.8 (5.8)	5.1 (5.9)	5.2 (5.8)	4.9 (5.8)	5.1 (5.8)	5.3 (5.8)	5.8	5.8	6.5	5.6

4.3.4 Summary of indirect scheme benefits – Multi-criteria results

The various potential impacts are now summarised and presented in tabular form. The method of analysis employed is less complex than the one used to assess “engineering criteria” in Chapter 4 above. This is because the criteria such as public safety and amenity are difficult to judge in anything but a comparative sense. For this reason we have not complicated matters by including weighting for the importance of one criterion against another or indeed one user group against another, although this could be included at a later stage if required. We have also deliberately used a neutral score (of 5) for situations where there is little measurable difference between one scheme and another.

In the following table a score of 1 to 10, taken from the averaged scores of the previous tables, has been used for each criterion. Using the average, instead of the total, allows a fair comparison between each of the criteria before any potential weighting is applied. However, we could have equally used a rating of good to poor as the scores given below are indicative rather than absolute values.

Table 4.6 Multi-criteria analysis results – indirect benefits

No.	Option	Length	Spacing	Aesthetics	Safety	Amenity		Score	
						(Timber)	(Rocks)	(Timber)	(Rocks)
1	Impermeable groynes	75m	150m	2.75	3.40	4.75	5.75	10.9	11.9
2	Impermeable groynes	75m	225m	4.00	4.00	5.10	5.90	13.1	13.9
3	Impermeable groynes	75m	300m	5.00	4.30	5.20	5.80	14.5	15.1
4	Impermeable groynes	110m	220m	4.00	3.50	4.90	5.75	12.4	13.3
5	Impermeable groynes	110m	330m	5.00	4.00	5.10	5.80	14.1	14.8
6	Impermeable groynes	110m	440m	6.00	4.30	5.30	5.80	15.6	16.1
7	Permeable groynes	75m	150m	2.75	3.60	5.75	5.75	12.1	12.1
8	Permeable groynes	110m	220m	5.00	4.30	5.80	5.80	15.1	15.1
9	Multi-reefs			9.00	6.30	6.50	6.50	21.8	21.8
10	Nearshore reefs			9.00	6.00	5.60	5.60	20.6	20.6

This table has been produced using a spreadsheet, which can be rapidly adjusted to reflect any desired changes in each criterion or in the potential weightings. It shows that option 9 (multi-purpose reefs) scores the highest with no weightings used, with nearshore reefs and long impermeable groynes with a ratio 4:1 coming second and third respectively. Overall the rock groynes score higher than the timber ones but the choice in material makes little difference to the ranking of the options.

4.4 COMPARISON OF SCHEME OPTIONS – MULTI-CRITERIA RESULTS

In Section 4.2, a multi-criteria analysis of so-called ‘direct’ scheme benefits (i.e. those relating to coastal protection) was undertaken with the top four ranking schemes being Option 3 (short impermeable groynes with a 4:1 spacing) ranked first with the weightings used, followed by Options 1 (short impermeable groynes with a 2:1 spacing), 2 (short impermeable groynes with a 3:1 spacing) and 9 (multi-purpose reefs) respectively. These highest scoring options are recommended for further design and refinement before a final option is implemented. A simpler analysis of the ‘indirect’ scheme benefits, namely aesthetics, public safety and amenity value, is described in Section 4.3, and the results for the recommended options are considered below.

- Option 3 (Impermeable groynes 75m long at 300m spacing):**
Neutral score for aesthetics as act as a viewpoint for the surrounding coastline but there is a large number of visually intrusive structures; low safety scoring due to number of structures involved; positive score for amenity (especially if rock groynes are used) as provide function for all beach users whether ‘sitters’, walkers, children, fishermen or disabled visitors.
- Option 1 (Impermeable groynes 75m long at 150m spacing):**
Poor values for safety and amenity due to number of structures necessary; and a medium score for amenity (again higher for rock groynes).
- Option 2 (Impermeable groynes 75m long at 225m spacing):**
Negative value for aesthetics, again as a result of the number of structures; low score for safety due to number of structures involved (more than for Option 3); medium score for amenity (although higher for rock groynes) as provide some function for beach users, not so much use for water-based user groups.
- Option 9 (multi-purpose reefs):**
Score very highly for aesthetics as structures are submerged; positive score for safety, as they are too far offshore to present a hazard to most of the user groups, with boat users most at risk; positive score for amenity as they do not

restrict beach users' space or free passage and improve amenity for swimmers and surfers.

As described above, although Option 3 is the highest rank in terms of fulfilling the coastal protection objectives, it scores much lower in the indirect benefits analysis (as do most of the groyne options), with Option 9 (the submerged and unobtrusive multi-purpose reefs) faring better in these categories.

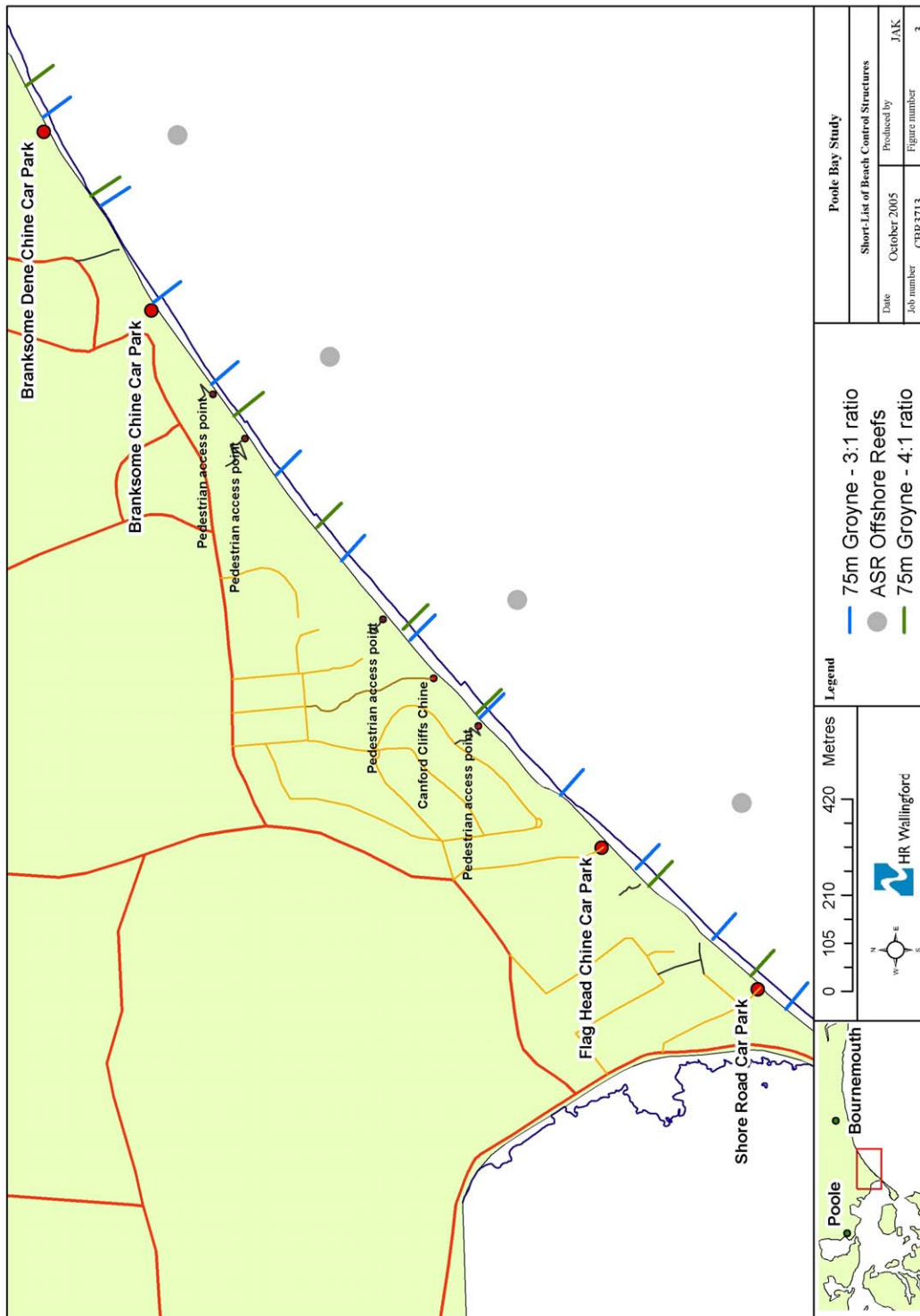


Figure 4.1 Structures shortlist scheme layout



Plate 4.1 Low beach levels and damaged seawall near Branksome Dene Chine



Plate 4.2 Potential beach scouring in front of seawall



Plate 4.3 Narrow beach width near Flag Head Chine



Plate 4.4 Rock groyne at Sandbanks with concrete walkway



Plate 4.5 Concrete Makepeace-Wood groyne at Bournemouth

5. *Conclusions and Recommendations*

5.1 STUDY CONCLUSIONS

5.1.1 *High-level review of beach control structures*

1. As part of this study, a wide-ranging review has been carried out seeking national and international guidance on structures that can be used to control plan-shape changes and wave-driven longshore drift on sandy beaches in areas of low tidal range. This review was undertaken to allow consideration of all practicable options for improving the performance and longevity of the future beach recharge operations along the study frontage, which are foreseen as underpinning Poole Borough Council's long-term strategy for coastal protection.
2. Despite reviewing practice in many countries where such beach management has been carried out, the existing technical literature, i.e. textbooks, manuals and scientific papers, on general best-practice and on case histories of beach control structures provides little agreement on the best types of structures, or on how to choose between them.
3. Even with their widespread use over centuries, for example, there appear to be no generally accepted "rules" or even firm guidelines for the "optimal" dimensions, spacing or crest levels of impermeable groynes.
4. There is continuing development of novel types of beach control structures and/ or of their construction method, e.g. sand-filled geotextile bags, which appear promising but for which there is limited practical experience.
5. Structures built to control the plan-shape of different beaches, which may have very similar characteristics (i.e. hydrodynamics, sediments and morphology) and problems, are often very different. Such differences not only occur between countries but often within the same country, or region or locality.
6. The choices made are often influenced by the availability and cost of construction materials, but also by less tangible factors such as perceptions of the aesthetics of new types of construction and concerns about possible adverse effects on public safety and the amenity usage of the beaches.
7. While different authorities and organisations often strongly support one particular type of beach control structure, there is little available guidance providing disinterested evaluations of the comparative advantages or disadvantages of different types of structure for in given situation. It has not been possible, for example, to validate or disprove various assertions made regarding the advantages of one type of structure over another in terms of public safety, their effects on ecology or their aesthetic acceptability.

5.1.2 *Review of beaches and defences – Sandbanks to Branksome Dene Chine*

8. The rise and fall of the tide is almost identical along the whole frontage from Poole Harbour entrance to the PBC boundary with Bournemouth. However, while tidal currents are influential west of Shore Road, they are unimportant in

terms of beach processes east of Shore Road, i.e. along the frontage considered in this study. This change affects both the nearshore seabed levels and the way that sand is moved across and along the beaches. The design of control structures for the beaches either side of Shore Road may therefore reflect the different tidal flows along those frontages.

9. Wave conditions also vary significantly along the shoreline between Poole Harbour entrance and the Bournemouth Borough boundary. West of Shore Road, the beaches are partly sheltered by Hook Sand, and by Durlston Head from waves approaching from the south-west sector. East of Shore Road, there is no shelter from Hook Sand and the protection afforded by Durlston Head gradually diminishes as one travels further east. This variation in wave conditions has not only brought about the overall curved plan-shape of the western part of the Poole Bay coastline of PBC over a period of several thousand years, but dictates the continuing evolution of the beaches east of Shore Road.
10. The net longshore transport of beach sediment is presently eastwards along the whole frontage from Sandbanks to the boundary with Bournemouth, and beyond. The rate of transport of the sand, however, increases as one travels eastward, because of the changes in wave conditions described above. This increase in sand transport leads to a gradual erosion of the beaches along the whole study frontage. Historically this would have led to erosion of the sandy cliffs between Poole Head and Branksome Dene Chine, which in turn would have provided extra sand to supply the beaches. Since the cliffs are now protected by the seawall along this frontage, the eastward increase in longshore drift causes a lowering of beach levels in front of the walls.
11. Beach sediments and gradients are broadly similar along the whole study frontage, although at the western end, i.e. near Shore Road, the -4m contour lies rather further offshore than it does further east at Branksome Dene Chine.
12. Beach levels west of Shore Road have been greatly improved in recent years by both the construction of long rock groynes and the pumping ashore of sand dredged from the approach channel to Poole Harbour (Swash Channel). This beach recharge has also improved beach levels just to the east of Shore Road, because some of the placed sand has moved east under the influence of the longshore drift.
13. Over most of the study frontage, however, beach levels have fallen in recent years. Beach widths are particularly narrow where the seawall has a seaward salient or “bulge”, particularly near Flag Head and Branksome Dene Chines. At these locations, the present standard of defence was much lower in early 2006 than was thought at the time of compiling the Poole Bay Strategy Study (Halcrow, 2003), and beaches fell to dangerously low levels in the winter months. It is likely that the defence standard at this time was no better than a 1:1 year standard, i.e. that serious overtopping and the attendant risks of undermining or structural failure of the seawall could occur under storm wave and tidal conditions that could be expected to occur once every year, on average.
14. The existing short timber groynes east of Shore Road have proved largely inefficient at controlling the longshore drift of sand, and most are at or near the

end of their useful lives. There is a substantial contrast in performance between these groynes and those installed by Bournemouth Borough Council immediately to the east of the study frontage.

5.1.3 Comparison of beach control structures

15. There are many possible methods for successfully maintaining wider beaches along the study frontage, provided that beach recharge operations are carried out from time to time, as recommended in the Poole Bay Strategy Study. Longshore drift rates, and their variations along the study frontage are substantially less than at locations further east, i.e. along the Bournemouth and Southbourne frontages, where a similar strategy for coastal protection has been implemented for the last 30 years.
16. As well as the smaller drift rates, there are further important differences between the study frontage and the beaches further east in Poole Bay that will affect the choice of beach control structures, namely (for the PBC frontage):
 - The more gently shelving beach profiles, especially along the western part of the frontage;
 - The greater ease (and hence lower cost) of delivering rock from the Isle of Purbeck; and
 - The smaller wave heights, and less substantial variations in wave directions.
17. There may be other differences too, for example in regard to the perceptions of the aesthetics of different types of structure and the amenity usage of the beaches, although these are not easy to evaluate or quantify.
18. These differences, together with one of the fundamental aims of this study, namely to investigate “novel” as well as more conventional types of beach control structure, meant that neither copying the groyne system that has been developed and refined along the Bournemouth frontage nor accepting the recommendation in the Poole Bay Strategy Study (Halcrow, 2003) were necessarily the “best” solution for PBC.
 - In view of this, a variety of permeable groyne schemes (different lengths and spacings) were considered in this study.
 - While impermeable groynes (of timber or rock) are familiar structures along the coastline of the UK, permeable groynes are less common. In other countries such as the USA and Germany, however, there is considerable interest and experience in using permeable groynes; one successful recent scheme in Italy is also reviewed in Appendix 3.
 - In many countries with low tidal range, particularly in Japan, the USA and countries with Mediterranean coastlines, detached offshore breakwaters have often been used to control longshore drift rather than groynes. The possibility of using these structures in Poole Bay also formed part of this study.

- There is particular interest in using submerged “multi-purpose” reefs along the Poole Bay coastline, a novel type of structure patented by ASR Ltd. of New Zealand. As part of this study, therefore, ASR were sub-contracted not only to assist in the literature review but also to suggest an appropriate scheme for controlling the beach using such structures (see Appendix 2).
19. Using knowledge of the particular characteristics of the study frontage, and those guidelines that do exist, a short-list of ten different beach control structure schemes was compiled, each of which were thought suitable for the PBC frontage of Poole Bay.
 20. Refining each of these ten short-listed schemes to obtain an optimal design, for example by studying exactly where structures might be placed, and whether slight changes in their dimensions would be beneficial, was beyond the scope of this study, and indeed would be a very substantial and lengthy task during a “design stage” study. In the light of this, it was decided to provide a “ranking” of the options based on a multi-criteria analysis method.
 21. To compare schemes using this method, a total of six criteria were developed, each of which reflected important issues in the consideration of beach control structures. These were:
 - Maintaining an adequate minimum beach width along the whole frontage;
 - Slowing the longshore drift rate and hence reducing sand losses;
 - The relative construction costs of the short-listed options;
 - The relative maintenance costs and ease of adjustment of the short-listed options;
 - The comparative impacts on public safety of the short-listed options; and
 - Whether or not the option was a well tried and tested scheme.
 22. The multi-criteria analysis allows for different “weights” to be attached to each of these criteria, so that different opinions on their importance can be taken into consideration.

Based on the present weightings, that reflected the primary requirements for a better standard of defence that would be long-lived and economical, in terms of the “direct benefits” they would deliver, the best four options were concluded to be:

- Impermeable groynes, 75m long and spaced 300m apart;
 - Impermeable groynes, 75m long and spaced 150m apart;
 - Impermeable groynes, 75m long and spaced 225m apart; and
 - Four “multi-purpose” reefs, as recommended by ASR Ltd.
23. Further refinement of these schemes will be required at later stages of the design process, and this may alter the order of these options, and possibly promote other of the short-listed options to the status of a “preferred scheme”. However, it is suggested that these four options are taken forward and discussed within the Council and with external consultees before any further design study or modelling is undertaken.

24. It is also considered that a combination of schemes may actually provide the most suitable option, in particular at the borough boundary, where there is a requirement not to adversely affect beach levels along Bournemouth Borough Council's frontage. Further discussions will also be necessary on this topic before any further detailed design is undertaken.

5.2 RECOMMENDATIONS

1. This study has been undertaken from a "coastal protection" perspective and has reviewed possible beach control structures and schemes principally from that viewpoint. It is recognised, however, that the beaches of Poole Bay are one of the major natural and economic assets within the Borough, and that the structures will have an influence on the amenity, recreational and tourism attributes of the coastline.
2. The evaluation of such influences is far from straightforward, and will depend to a considerable extent on the detailed design of whatever structures are built. Such details are not likely to affect the coastal protection function of the structures and have not been considered in this study. For example, it has not been necessary to differentiate between timber or concrete groynes (such as used in Bournemouth) and rock groynes (such as at Sandbanks). These issues can usefully be discussed before any further modelling or refinement of the dimensions of beach control structures is carried out.
3. The multi-criteria analysis method developed in this study can be adjusted to allow different weightings of the seven criteria identified, making these either less or more important or even eliminating one or more of them entirely. This will hopefully allow others within or outside Poole Borough Council to examine the consequences of different views on what structures should be built.
4. Once opinions on the conclusions of this study, and ideas resulting from the discussion of the options proposed have been collated, it may be that options other than the preferred four identified here will need to be considered further. In any event, each option carried forward will need to be studied in more detail, to optimise its performance in terms of providing an appropriate and long-lived coast protection standard along the whole frontage, and to refine the costs of construction and maintenance. The former requirement will require modelling of the effects of beach recharge operations and the proposed structures on future beach plan-shape changes, over a prolonged period, and comparing each option on the basis of their benefits and costs.

6. References

CIRIA. 1998. *Seabed Sediment Mobility Study – West of the Isle of Wight*. Project Report 65, London.

DETTE, H.H. et al. 2004. Permeable Pile Groyne Fields. *Journal of Coastal Research*, Special Issue 33, pp145-159.

HALCROW. 1999. *Poole and Christchurch Bays Shoreline Management Plan*. Report to Poole Bay and Harbour Coastal Group. Halcrow Group Limited, Swindon.

HALCROW. 2004a. *Futurecoast: research project to improve the understanding of coastal evolution over the next century for the open coastline of England and Wales*. Report and CD-ROM produced by Halcrow-led consortium for DEFRA. Halcrow Group Limited, Swindon.

HALCROW. 2004b. *Poole Bay and Harbour Coastal Strategy Study: Assessment of Flood and Coast Defence Options*. Report to Poole Bay and Harbour Coastal Group. Halcrow Group Limited, Swindon.

HR WALLINGFORD. 1994. *Sandbanks Coast Protection Scheme Feasibility study and outline design*. EX3083. HR Wallingford, Wallingford.

HR WALLINGFORD. 1995. *Poole Borough Coastal Strategy Study*. EX2881. HR Wallingford, Wallingford.

HR WALLINGFORD. 2003. *Poole Bay & Harbour Strategy Study, Computational Modelling Studies*. EX4555. HR Wallingford, Wallingford.

POFF, M.T. et al. 2004. Permeable Wood Groynes: A Case Study on their Impact on the Coastal System. *Journal of Coastal Research*, Special Issue 33, pp131-144.

RENDEL GEOTECHNICS and UNIVERSITY OF PORTSMOUTH. 1996. *Sediment Inputs into the Coastal Zone: Fluvial Flows*, Report to SCOPAC. Rendel Geotechnics, Birmingham.

VAN RIJN, L.C. 2004. *Principles of Sedimentation and Erosion Engineering in Rivers, Estuaries and Coastal Seas*. Aqua Publications, Amsterdam.

Appendices

Appendix 1

Beach control structures a literature review

Appendix 1 *Beach Control Structures - A Literature Review*

1. *Introduction*

This Appendix summarises a literature review carried out by HR Wallingford into the various types of structure that are used, in the UK and elsewhere, to control longshore sediment transport and hence long-term beach evolution. The purpose of this review was to identify different types of beach control structure that might be suitable for use along the Poole Bay frontage, and summarise past experiences and design guidance for these.

2. *Groynes*

2.1 DEFINITIONS AND OBJECTIVES

A groyne is essentially a narrow “rib-like” structure that is built approximately perpendicular to the shoreline in order to trap a portion of the longshore sediment and accumulate beach material (Komar, 1998). On sandy beaches, they achieve this by interfering with longshore currents created by obliquely incident waves and/or tidal currents, reducing the strength of these within the groyne “bays”.

Groynes typically extend across a part or the entire inter-tidal zone and may be built in groups (known as groyne fields) to protect an entire frontage length (CIRIA, 1996).

Groynes are very common, particularly on UK beaches and, as a general guideline, are most appropriate to frontages where there is a low net and high gross drift (CIRIA, 1996). Groynes have been successfully employed on both shingle and sand beaches but in the latter case, tend to perform best in micro-tidal environments where the spatial distribution of transport due to waves and tidal currents across the foreshore is limited. Sand beach groynes do not generally trap all the drift but should be long enough to control a sufficient part of the profile and hence protect the upper beach from severe erosion. In practice, a large number of groyne fields have not performed satisfactorily (Brampton and Motyka, 1983). Sediment that is transported along the shore is trapped against the updrift side of the structure. Consequently, the supply to downdrift beaches is reduced and erosion occurs. The erosion problem is therefore transferred along the coast (Bird, 1996).

Groynes are built to serve three main purposes (Van Rijn 2004):

- To stabilise and widen an eroding beach by trapping sand from littoral drift.
- To stabilise the placement of beach fill material on nourished or man made beaches.
- To prevent the movement of littoral material out of a littoral cell.

In addition to the three main groyne functions as described above, a well-designed system can also (Fleming 1990):

- Deflect strong tidal currents away from the shore,

- Control seasonal shifts of material alongshore and hence the distribution of sediments within and embayment and,
- Increase the depth of beach material cover to an otherwise erodible seabed.

Groynes cannot usually stop coastal erosion completely but effective systems can retard shoreline recession by a factor of 2 or 3. It stands to reason that groynes will perform best at sites with a significant longshore drift component and an oblique angle of wave approach. Additionally, it is important to note that conventional groynes will have little or no effect on offshore movement of sediment.

2.2 EFFECTS OF GROYNES ON BEACHES

In providing a physical barrier to longshore transport, groynes are intended to alter the orientation of the beach to be more closely aligned with the incoming wave crests (CIRIA 1996). On coasts with a dominant wave approach, sediment is deposited on the updrift side of the groyne whilst erosion occurs in the lee of the structure. Consequently, there is often a step-type difference in elevation across each groyne. On coasts with a highly variable wave climate, a curved, bay-type beach will form (Van Rijn 2004). Examples of the shoreline position within groyne compartments are shown in Figure X.

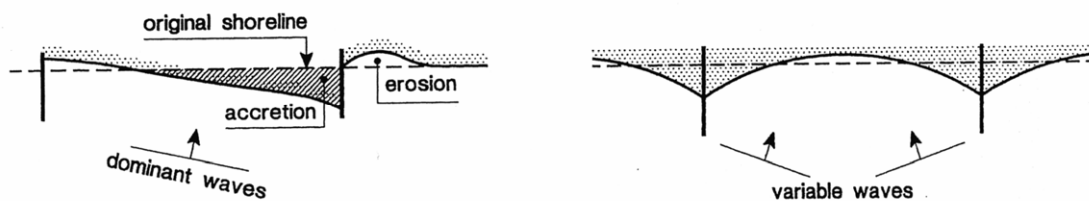


Figure 1 Shoreline position inside groyne compartments (left = single dominant wave direction, right = variable wave direction) (from Van Rijn, 2004).

Over time, the width of the beach within the groyne compartments increases but in doing so may cause erosion downdrift of the groyne field. Groyne systems can only trap a finite quantity of sand and once filled, the groynes allow any longshore transport to resume around the toe of the structure. To reduce downdrift erosion, artificial nourishment of the groyne bays may be carried out so that the overall littoral sediment budget is not adversely affected (Komar, 1998).

The effectiveness of a groyne is dependent on the type of beach material, the wave climate and tidal regime, the dimensions and the hydraulic characteristics of the structure (CIRIA 1996). The design of a groyne system will always be a compromise between providing for long-term beach stability and minimising damage due to large storm waves.

2.3 TYPES OF GROYNES

Groynes can be constructed from a wide range of materials and the choice is essentially based upon the specific requirements of the scheme and the physical characteristics of the site (although price and material availability are often defining criteria). (CIRIA, 1996). Two main types of groyne may be distinguished:

Impermeable, high crested structures. These are generally concrete or sheet piling structures but also include grouted rock and rubble mound groynes (Van Rijn, 2004).

Groynes of composite construction with steel sheet piling or concrete at lower levels and timber at upper levels are common (Fleming, 1990), whilst more diverse structures comprising grout or geotextile filled bags have also been constructed. The crest of these structures is always above mean high water (MHW) level and the resultant shoreline is orientated perpendicular to the dominant wave direction within each compartment. Impermeable groynes are intended to inhibit longshore transport until the beach volume has increased sufficiently for sediment to spill over the crest or around the toe of the structures (Bird, 1996). The major drawbacks of impermeable groynes are their tendency to reflect wave energy and to create offshore-directed currents particularly along their downdrift face. These hydrodynamic effects are quite localised but can have an adverse effect on the beach, for example forming scour channels alongside the downdrift face of the groyne.

Permeable groynes can be used on beaches that have sufficient quantities of sediment to allow a proportion to pass through or over the structure without limiting its efficiency (Fleming, 1990). The crest level of permeable groynes is usually fixed between the high and low water marks in order to reduce eddy formation at high tide (Van Rijn, 2004). These types of groynes are generally used on beaches with lower sediment depletion in order to create a more regular beach plan-shape. Timber, rock and pre-cast concrete have all been used in the construction of permeable groynes.

2.4 FUNCTIONAL DESIGN

2.4.1 *Crest level*

As sand is transported in suspension throughout the water column, then the impact upon longshore transport will be a function of the height of the groyne (CIRIA, 1996). The required beach level should be used to determine the crest level for an artificial beach or where a recharge scheme is planned. In order to produce steady accretion of a natural foreshore, it is recommended that around 0.5m of the structure protrudes above the beach profile and that at the extreme landward end of the groyne, the crest level should actually be slightly lower than the beach. At the seaward end of the groyne, the crest level should be slightly higher than MLW and around 1m above the seabed so that all bedload transport is blocked. Submerged groynes are most effective in low wave energy conditions. Gomez-Pina (2004), and Aminti et al. (2004) state that their crest level should be set at 0.5 – 1m above the subaerial portion of the profile in order to hold the submerged portion of the beach profile.

2.4.2 *Groyne Length*

As with crest level, a long high structure that extends across the entire width of the surf zone will have a greater impact than a short, low structure. In practice, the maximum groyne length is determined by the mean low water spring line in tidal environments although the tip should always be within the surf zone to allow sand to pass round it (Van Rijn, 2004).

For estimating a suitable groyne length, it can be assumed that the majority of the longshore drift occurs on the upper part of the profile although sand is still mobile under both wave and tidal currents at relatively large water depths (Fleming, 1990). This being the case, groynes on sand beaches need to be longer than those constructed on shingle. For example, Van Rijn (2004) states that in the UK, groynes on shingle beaches do not generally exceed 60m whereas on sand beaches the length is usually in excess of 100m. In the Netherlands, groynes on sand beaches may be up to 200m long. The optimum length of groynes also depends partly on the dominant angle of wave approach. Wave

crests arriving between 40°-50° to the shoreline are most effective in driving longshore transport (Bird, 1996) and groynes in these locations must be relatively long in order to be effective. For locations where the angle of approach is greater or less than this, the groynes can be shorter.

2.4.3 Spacing

The optimum spacing of the groynes depends primarily on the nature of the beach material (Bird, 1996) and is related to the structure length. The spacing of groynes on shingle beaches is less than that on sand beaches. This is because sand beaches do not re-orientate themselves as quickly as shingle beaches during storm conditions. As a general rule, the spacing should be 2-4 times the groyne length in order to prevent the generation of rip currents and excessive erosion between the groynes (Van Rijn, 2004). The spacing should decrease with increasing wave angle to ensure a uniform distribution of sediment within the compartment (Bird, 1996). In the UK, the spacing/length (S/L) ratio is between 0.8 and 3 whilst in Holland, S/L is typically between 2 and 4 (Van Rijn, 2004).

2.5 CASE STUDIES

2.5.1 Physical Model Studies on Groyne Design (Russell, 1960)

Experiments carried out in the Coastal Model at the Hydraulics Research Station, Wallingford during the 1950s provide some optimum design criteria for groynes on sand and shingle beaches. According to the results of the experiments, groynes operate purely by reducing littoral drift, causing an accumulation first at the updrift end of the system, which travels slowly, down-drift and fill all the compartments in turn. They would not have any beneficial effect where there was no net littoral drift. They did not cause sand to come in from offshore.

According to the results of these experiments, it is possible by building groynes 1m high, c75m long and spaced 55m apart, to arrest practically the whole drift that would be moving along inshore of the low water mark. Normally, this should not be done because it results in sand being washed from the beach out to sea. If the groynes are made lower or spaced more widely, the littoral drift is not completely arrested, but then no sand is washed out to sea. The criterion for no loss of sand from the beach is that when oblique waves cause the contours to pivot and make the saw tooth pattern, the sand on the updrift side of the groyne should reach the crest of the groyne and be able to pass freely over it. It is possible to justify this conclusion on hydraulic grounds by considering the subsequent behaviour of water that has run up the beach and momentarily collected on the updrift side of the groynes. If the groynes are high above the sand, the water can only run down the updrift side of the groynes, taking sand with it. If, on the other hand, the groynes are low and level with the surface of the sand, water passes over them and flows back down the beach on the downdrift side of the groynes. On this side of the groynes, the beach is lower, the mean depth of water is greater and the same quantity of water flowing down the beach accordingly carries less sand with it.

A very short system of groynes containing say only 5 groynes can be built with the groynes high and closely spaced because the loss of sand from the 5 groynes may be less than the extra drift this is collected by building the groynes high or closely spaced. On the other hand if there were 50 groynes, the loss offshore being 10 times as great, might exceed the quantity arrested. It was found that groynes 1m high and 55m apart lost from each compartment 1/8 of the drift that was arrested: accordingly, if a large

number of these groynes were built, those downdrift of the eighth groynes could not be expected to build up the beach.

Small variations in the orientation of the groynes were not found to have any effect on the slowing down of the drift. However, the contours on the down drift side of the groynes were kept farthest away from the sea wall or cliff if the groynes were angled away from the drift.

2.5.2 *Rock Groynes at Montrose*

The design of groynes on the sandy beach at the entrance to the estuary of the South Esk River, in Angus, is described in a paper by Banyard and Mannion, 2001. They write that coastal protection schemes, particularly with respect to groynes, are typically dominated by wave processes. However, in some locations, such as near the entrance to estuary mouths strong tidal currents can influence the performance of coastal structures and cause long-term changes in sediment transport processes. The impact of tidal currents on sediment transport and the resultant coastal protection requirements formed part of a shoreline management study for Montrose Bay in Scotland. This scheme is highly relevant to the present study, as there are similarities between this site and the Poole Bay frontage at Sandbanks. Indeed the original (conceptual) design for groynes at both locations was developed at HR Wallingford, in 1994 for Sandbanks and in 1995 for Montrose.

Montrose is located on the eastern coast of Scotland on the banks of the River South Esk. The tidal currents at the entrance to Montrose Basin are in the region of 1m/s. The Montrose Foreshore Protection Scheme extends along the north side of the estuary mouth over a 600m frontage. The scheme comprises 3 rock groynes of between 60 and 140m long that are equally spaced at 150m intervals. The central groyne is at least twice the length of the structures to its north and south. The southern most groyne is curved slightly whilst the other two are straight. The groynes were constructed with a higher than normal permeability as their primary function is to drive tidal currents offshore rather than to reduce sediment transport from wave attack. The groyne bays were nourished with 60,000m³ of sand dredged from the navigational channel of the River Esk.

Calculations of beach volume showed that between summer 1999 and summer 2000, the upper beach had accreted by 18,000m³ whilst the lower beach had eroded by 14,000m³. Initial monitoring of the Montrose scheme indicates that rock groynes can be effective in deflecting tidal flows in order to encourage beach stability but careful consideration must be given to the rock grading used and the design profile of the groyne. The decision to include beach recharge will depend on the availability of suitable natural material from offshore sources.

2.5.3 *Groynes on the East Frisian Islands (Kunz, 1997)*

Some 124 groynes have been constructed on the East Frisian Islands (North Sea) comprising both 'beach groynes' to prevent coastal erosion and 'stream groynes' to divert tidal currents. This differentiation is similar to that made in the case of the groynes at Montrose (see section 2.5.2), where in the former the currents are produced by obliquely breaking waves and in the latter by nearshore tidal currents.

The groynes and other coastal protection units (seawalls etc.) at this site have been constructed since the middle of the 19th century in response to beach erosion and dune

depletion. These initial structures were not successful in preventing coastal erosion and therefore research was carried out to determine how the morphological and hydrodynamic regime could be adequately controlled by defence structures. As a result, the groynes have now been incorporated into an engineering coastal defence system on Norderney Island have succeeded in stopping the migration of tidal channels and prevented further coastal recession. The groynes were not able to halt the coastal erosion *per se* but were used to stabilise the post-recharge beach fill. The Norderney coastal defence system is summarised below.

- Stream groynes to stop the migration of the tidal currents;
- Beach groynes to stabilise the beaches in front of the seawalls;
- Seawalls and revetments to protect the dunes from erosion; and
- Beach nourishment to groyne bays to maintain beach levels during storm conditions.

2.5.4 Pile Groynes

Simple pile groynes are regarded a cost-effective solution for sheltered beaches in microtidal conditions and have been used extensively in the USA, Germany and Poland (Dette et al, 2004, Poff et al, 2004 and Trampenau et al, 2004). For example, around 2000 wooden pile groynes have been installed along the Baltic Sea coasts where they have been found to reduce the littoral drift and eliminate local rip currents (Trampenau et al, 1996).

Permeable pile groynes are essentially a series of (usually wooden) piles that are driven into the beach with predetermined spacing that may vary along the groyne. As a result, the shoreline remains continuous and does not display the characteristic 'saw tooth' shoreline plan view of normal permeable groyne fields.

As a specific example, 17 pile groynes were constructed at Warnemunde on the German Baltic coast during 1991-1992. The structures are 120m long and are spaced at 80m with the crest level at +0.5m MSL. The permeability of the groynes varies both along the structures and between groynes in the field. In the centre of the groyne field, the permeability of the structures is 0 (i.e. they are impermeable) for the first 80m of their length. Thereafter, the spacing between the piles increases seawards. These central groynes exhibit the highest permeability whilst groynes at either end of the system were constructed with the lowest permeability. The response of the shoreline to the groyne construction was slow but after a period of 3-4 years, the beach and underwater terrace showed significant accretion. Additionally, profile of the inter-tidal zone was found to have flattened thereby reducing the wave energy loading per unit area.

Design considerations of permeable pile groyne fields are: (Van Rijn, 2004).

- Groyne length should extend to the seaward flank of the innermost breaker bar;
- Spacing should be between 1:1 and 1:2;
- Piles should be driven in to approximately 60% of their length;
- The crest level should be around +0.5m MSL;
- Permeability should not exceed 40%;

- A double width groyne is more effective than two widely spaced single structures; and
- Permeable pile groynes should not be used in conjunction with beach fill.

2.5.5 Groyne Notching

The construction of long groynes has led to severe downdrift erosion problems at many locations (Van Rijn, 2004). It is possible to remedy this by either shortening the groyne or removing the upper layer of the structure to permit sand bypassing. This process is known as groyne notching.

Wang and Kraus, (2004) studied three groyne notching options in a physical model experiment (notching at the breaker line, mid-surf zone and in the swash zone). The construction of a notch in the swash zone provided the greatest increase in sand bypassing. The other notches reduced the rip currents around the groyne tip but did not result in increased bypassing. Therefore, a gap in the swash zone and a lower profile in the outer surf zone would appear to provide the most suitable solution.

Groyne notching has been carried out on the northern New Jersey coast (Donohue *et al*, 2004). A series of groynes were constructed from timber, timber and steel, timber and rock and rubble mound. The length of the structures varied from 15 to 200m and the groyne bays were artificially nourished every 6-7 years. Based on the results of GENESIS numerical modelling, it was concluded that the groyne field would perform better if all groynes longer than 150m were removed. Rather than take out the entire structure, a 30m wide section was removed from a point about 15m seawards of the beach fill toe with the purpose of allowing a partial passage of littoral drift.

Groyne notching along the New Jersey and Long Island coasts has been studied by Rankin *et al*, (2004) to determine the optimum notch location. The location of the notch controls the directionality of the sediment transport through the notch as well as its magnitude. The study concluded that the optimum location for a groyne notch is in the swash zone. Additionally, it is advisable to remove only the upper layers of the groyne leaving the lower layers intact to prevent localised scour. The notch should be sufficiently wide to prevent the generation of strong local currents and any modifications to the structure should be carried out post beach nourishment once an equilibrium profile has been established.

2.5.6 Groyne scheme at Keta, Ghana

The Keta sea defence system, which was built between 2000 and 2003, is described in a paper by Nairn and Dibajnia (2004). The scheme comprises a revetment and 6 long groynes together with beach nourishment to protect a 5km stretch of rapidly eroding shoreline. The groynes are 190m and are spaced at 750m intervals ($S/L=1$ to 4). The groyne bays were pre-filled (by a beach recharge) to minimise downdrift erosion. The groynes in the centre of the field were constructed first, followed by the updrift group and then the downdrift group. Post project monitoring shows that accretion has occurred around the groynes and that there is no evidence of scour around the groyne tips.

2.5.7 Groyne Experience in USA (Galgano, 2004)

The effectiveness of the groynes on Bethany beach, a sandy beach along the Atlantic coast of Delaware, USA has been assessed by Galgano (2004). The original scheme

proved ineffective, as the structures were too short. Galgano (2004) presents the following guidelines.

- The groyne bays should be nourished either during or immediately after construction;
- The first groyne should be placed at the downdrift end of the scheme and continue in an updrift direction;
- Groynes should be relatively short and low to maintain downdrift beaches; and
- The spacing should never be less than 1:1.

3. *Fishtail and T-Head groynes*

These types of groyne are a recent development, and there have been a limited number of independent reviews of their performance. Those for which information has been located are described below (alphabetical order). In addition, there are known to be similar schemes at Cleethorpes, Felixstowe and Skinningrove.

Dinas Dinlle

A scheme to provide an appropriate standard of defence along the Dinas Dinlle frontage in Caernarfon Bay was carried out in 1994. The scheme comprised the following elements.

- A rock bastion outside the Marine Hotel;
- A fishtail groyne at Dinas Dinlle
- Beach nourishment and
- Flood protection embankments, an associated drainage ditch and raising of a highway behind the defences.

Five years after construction, the performance of the scheme was reviewed by Williams and Davies (1998). There has been general longshore drift diversion around the village frontage with accretion of sand-sized sediments behind the bastion. Significant lower beach accretion along the whole frontage has been observed as indicated by a general seaward movement of the mean low water mark. This has manifested itself specifically in the formation of pocket beaches a) between the outer arms of the breakwater structures and b) on the north side of the fishtail groyne structure.

Jaywick

A scheme comprising four fishtail groynes together with beach nourishment was implemented on the Jaywick frontage, south and west of Clacton. The scheme did not perform as anticipated. The groynes promoted substantial beach accretion in their lee, and along the immediately adjacent sections of coastline. However, this accretion was at the expense of erosion of the recharged beach between the structures, where the required standard of defence was not achieved. Supplementary works were later built, between the original major groynes, to provide a satisfactory beach width along the whole frontage.

Llandudno

Three rubble mound fishtail groynes were installed on West Shore, Llandudno during 1991 as part of a coastal protection scheme. The scheme is described by Bull et al, (1998). Monitoring of one of the groynes found that whilst the structure was effective in protecting the adjacent seawall, it was not effective in increasing beach levels. Additionally, a large volume of fine material had accreted in the lee of the groyne, which reduced the amenity value of the beach. This build up of material was attributed to the quiescent conditions behind the groyne; it appears that the conditions created there are far calmer than anticipated at the design stage.

Llanelli

The scheme comprises four breakwaters, of which one is a fishtail groyne, and a recharge of the sand beaches between the breakwaters. The scheme has provided adequate standards of defence along the whole frontage. Surveys of the beach areas between the structures showed no overall loss of nourishment volume after four years service.

Machynys

The scheme comprises four breakwaters of which two are fishtail groynes, the construction of a new promenade and beach nourishment. The aim of the scheme was primarily to protect the shoreline from erosion and also to remove or bury evidence of industrial waste. The scheme has provided adequate standards of defence along the whole frontage and there have been no significant adverse effects on coastlines adjacent to the scheme.

Morecambe Bay

A coastal protection scheme comprising 13 fishtail groynes was implemented in Morecambe Bay. An analysis of the impacts of this scheme on sediment deposition is presented by French and Livesey (2000). The scheme achieved its primary objective in that sediment accumulation has been significant and considerable foreshore protection has been attained.

Sand Bay

A series of five fishtail groynes were constructed at Sand Bay. The structures have lengths of 60m, 58m, 60m, 40m and 55m and were spaced at distances of 45m, 110m, 75m and 55m apart respectively. The scheme has fulfilled its specified performance brief and no significant adverse effects have been recorded.

Walney Island

An existing seawall had at the end of its life and was suffering scour problems. Breaching occurred in 1990 and emergency works were carried out until the new fishtail breakwater scheme was completed. The scheme comprised a new seawall to provide protection to the hinterland and a fishtail groyne to enhance beach levels along the frontage. The main arm of the groyne is 115m long and the shorter arm is 40m long. The fishtail groyne has performed as expected and has resulted in accretion.

4. *Detached Breakwaters and Reefs*

4.1 DEFINITIONS AND OBJECTIVES

Detached breakwaters are generally built in the nearshore zone to shelter an adjacent stretch of coastline from wave action (Bird, 1996). These structures may also diminish the longshore drift and can reduce offshore sediment transport during storm conditions (Pope and Dean, 1986). Offshore breakwaters are most effective in tide-less or microtidal environments such as the Mediterranean or Adriatic. The structures are usually orientated parallel to the coast, although there are locations where breakwaters have been constructed parallel to the predominant wave angle (Bird, 1996). They may be constructed as single units or as a series (segments) that are designed to protect a long stretch of coastline (Komar, 1998).

Waves break against the structures and are diffracted through the gaps before they reach the beach. The inherent sheltering effect generally causes the beach in the lee of the structure to accrete, leading to the formation of beach cusps or “salients” behind each breakwater. If there is sufficient sand supply, a tombolo will form that links the structure with the shoreline. In such cases, the longshore drift may be blocked almost entirely, or diverted offshore into deep water, in either case resulting in downdrift erosion.

4.2 TYPES OF DETACHED BREAKWATER

Detached breakwaters were first constructed as early as the 1930's (Magoon, 1976) but during the past 20 years, offshore structures have been employed extensively as coastal protection measures (Komar, 1998). There are many variations in the design of detached breakwaters but they have often been divided into two categories – emergent and submerged.

4.2.1 *Emergent breakwaters*

Emergent breakwaters are large structures whose crest width is generally of the order of the local water depth. The crest height is typically ~2m above MHWS and the structure is designed specifically to prevent wave overtopping. Circulation cells form close to the tips of emergent breakwaters, bringing sand into the lee zone and promoting salient and tombolo formation. Emergent breakwaters have been successfully applied in micro-, meso- and macro tidal conditions against erosion of headlands, and recreational (artificial) beaches. In general, beaches behind the structures are relatively wide with well-developed salients or tombolos as shown in Figure 2

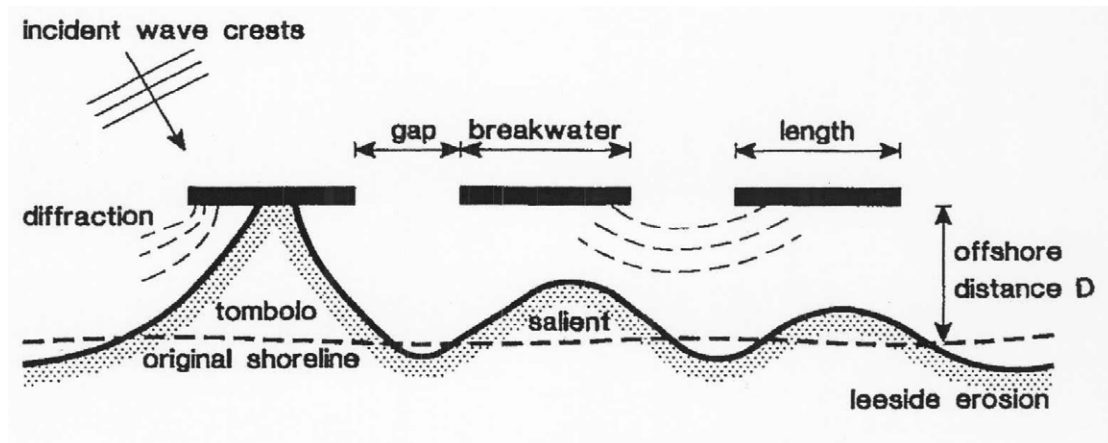


Figure 2 Definition sketch for detached breakwaters as beach control structures

4.2.2 Submerged Breakwaters/ Reefs

Submerged breakwaters are also known as reefs. The crests of submerged breakwaters or reefs are positioned below the still water level. These structures are only effective in trapping sand for salient formation if the crest level is relatively high (0.5 to 1m below MSL) in a microtidal regime and the beaches that form are narrower than to those formed behind emergent breakwaters. Longshore and cross-shore rip currents tend to form in the gaps between multiple structures due to the transport of water over the crest. This may result in erosion in the lee of the structures and the effect increases with decreasing crest level.

Submerged breakwaters are not particularly effective at reducing shoreline wave heights during extreme storms or surges and hence do not provide much added protection along the coastline during such conditions. Submerged breakwaters or reefs can be used to generate breaking waves at popular surfing locations but these can also present a hazard to boats operating in the nearshore zone. Compared to emergent breakwaters, artificial reefs are relatively cheap to construct and maintain. Numerous examples of these reefs have been successfully applied in low energy microtidal environments such as the Mediterranean.

4.2.3 Summary

The crest elevation of a detached breakwater determines the amount of wave energy transmitted over its crest. The highest “emerged” breakwaters prevent overtopping by all but the largest waves, whereas “low crested” breakwaters allow frequent overtopping. The amount and frequency of overtopping can affect the formation of salients or tombolos and/ or remove or modify these features once they have formed.

Submerged breakwaters (reefs) allow almost continual transmission of waves over their crests during both calm and storm conditions, but can be designed to produce some degree of wave breaking, for example to improve conditions for surfing.

4.3 HYDRAULIC AND MORPHODYNAMIC EFFECTS OF DETACHED BREAKWATERS

The basic characteristics of detached breakwaters are shown in Figure 3 and discussed briefly below.

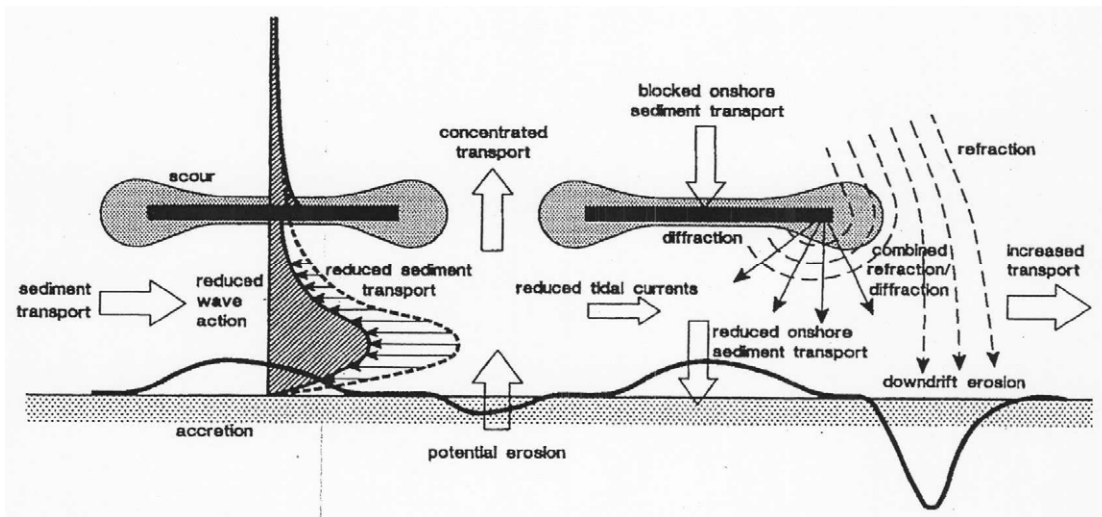


Figure 3 Hydrodynamic and transport processes around detached breakwaters

Wave energy at the shoreline is reduced due to breaking and reflection against the structure. However some of the incident wave energy will propagate into the lee zone due transmission through the structure and diffraction around the ends of the breakwater.

Diffacted and transmitted waves will continue to propagate to the shoreline but the longshore transport capacity in their lee will be substantially reduced. Therefore beach sediment transported into the sheltered zone will be deposited.

In the case of normally incident waves, the diffracted waves will transport sand from the adjacent beaches into the lee of the structure until the waves break parallel to the shoreline. The longshore transport will be effectively reduced to zero in the lee of the structure yielding a symmetrical salient or tombolo formation. However, the shoreline will erode on both sides of the structure.

In the case of obliquely incident waves, the longshore transport along the coastline behind breakwater will be affected at least initially. In some circumstances, the shoreline behind the structure forms a salient (see Figure 2). The smaller waves behind the structure break at a more oblique angle to the shoreline and can transport the same amount of sediment as the larger waves updrift and downdrift of the breakwaters, i.e. the longshore drift returns to its previous value.

In other cases, the breakwater will continue to block the drift for a prolonged period, eventually forming a tombolo (see Figure 2). Subsequently, the updrift side of the tombolo may be filled, allowing sediment to pass around the seaward side of the breakwater and hence eventually restoring drift to the downdrift coast. There is however a danger that sediment could be diverted offshore into deep water and lost from the beach.

4.3.1 *Depositional Patterns (for emerged breakwaters)*

Some initial indication of the effects on a beach of a detached breakwater scheme can be obtained by simple “rules of thumb” developed in the USA (Pope and Dean, 1986). In the following:

L = Length of the breakwater;

G = Gap width between adjacent breakwaters; and

D = distance of the breakwater offshore.

L/D > 3 = Permanent tombolo

The breakwater length should be larger than the gap length to form a tombolo. Increasing this ratio increases the amount of energy transmitted through and over the segments while decreasing the diffraction effects. No erosion opposite the gaps will occur for $G/D < 0.8$.

- A tombolo behind a breakwater with $L=200\text{m}$, $D=200\text{m}$ in a water depth of 3m can be formed in 1 to 3 years.
- Tombolos will be formed if the structure is placed too close to the shore well within the breaker zone or if the longshore transport rate is relatively large.
- Tombolos can eventually completely block longshore sediment transport.
- Tombolos will lead to the formation of rip currents that transport sand offshore as a result of water piling up in the embayment between adjacent tombolos.
- Tombolos will lead to severe lee side erosion in conditions where the dominant wave angle is not parallel to the shoreline.

L/D 2 to 3

- A permanent tombolo or a semi-permanent tombolo will form (i.e. one whose extent depends on wave conditions) or perhaps a well-developed salient.

L/D 1 to 2 well-developed salient to incipient tombolo

- Salients create less erosion of the beach downdrift due to sand bypassing.
- Salient formation can be promoted by allowing sufficient wave energy in the lee area by increasing offshore distance and gap length, by reducing the crest level or by increasing the permeability of the structure.

L/D 0.5 to 1 = Weak to well developed salient

L/D = 0.2 to 0.5 = Incipient to weak salient

L/D > 0.2 = No effect on beach plan-shape

4.3.2 *Erosional Patterns*

G /D<0.8 = no erosion in the gap between breakwaters

G /D 0.8 to 1.3 = minor beach erosion in the gap between breakwaters

G /D>1.3 = major beach erosion in the gap between breakwaters

The implication of this admittedly simple guidance is that if breakwaters are positioned so that adjacent breakwaters have their ends further apart than their length, there is likely to be a problem of erosion between them.

It should be noted however that in areas of large tidal range, the distances between a breakwater and different contours varies. Consequently, for example, this can result in a tombolo forming at low water but only a salient at high water. Further the above guidelines are based on experience on wave-dominated coastlines; at Sea Palling in Norfolk, where tidal currents close inshore are strong, (Thomalla and Vincent, 2004) found that the performance of four offshore breakwaters did not match that expected using these guidelines (see below).

4.4 BREAKWATER CASE STUDIES UK

Because of the large tidal range and severe wave conditions that occur on many parts of the UK coast both emergent and submerged breakwaters do not generally represent a cost effective solution to coastal defence problems. However, shore parallel breakwaters have been constructed at a number of locations around the coast of the UK. The majority of these structures are on shingle or mixed beaches and in many cases are used in conjunction with other coastal defence structures such as groynes or sea walls.

4.4.1 *Sea Palling*

In 1990, the National Rivers Authority (now EA) implemented a 50-year sea defence strategy for the 14km coastline between Happisburgh and Winterton. The strategy was intended to protect several villages and 6000 hectares of low lying land from tidal inundation and included the construction of a series of shore parallel breakwaters. A scheme comprising a total of 16 segments was designed to be built over a 9-year period. Four segments were constructed during the initial phase of the study to the following specifications:

Structure length = 220m
Gap width = 280m
Offshore distance = 200m

During the first 12 months after construction, updrift bed levels accreted significantly, particularly on the nearshore sand bar located ~ 200m offshore. In the immediate lee of each structure, large accumulations of sand built up rapidly after construction. However the beaches opposite the breakwater gaps had cut back to the seawall as a result of wave diffraction around the ends of the structures. The breakwaters had been constructed with a fairly high crest level in order to protect the coast from storm surges and consequently some longshore sediment transport was sacrificed. Beaches downdrift of the structures

began eroding immediately after construction and this in conjunction with low sediment feed resulted in the exposure of the seawall toe piles by early 1996.

A review of the structures highlighted the following points:

- Due to the accretion of beaches updrift of the structures, it was decided to abandon plans to build more breakwaters in this area.
- A plan to lower the crest level of the existing structures was considered in order to reduce the sediment accumulation in their lee. However this was abandoned given the benefits of the updrift accretion.
- In developing future reefs, on the downdrift side of the existing structures, the gap between the segments would be reduced to prevent local erosion.

A further five segments were installed further south in Phase 2 of the scheme. Based on the experience gained during Phase 1, the new structures were designed with the following specifications:

Length = 160m
Gap width = 160m
Offshore distance = 200m

The new structures have a slightly lower crest height than those built during Phase 1. Although these have now been installed, no review of their effect on the beach plan-shape has been located.

4.4.2 *Leasowe Bay*

The use of offshore breakwaters at Leasowe Bay on the Wirral was the first application of these structures in the UK. The reconstruction of the Wallasey embankment in Leasowe Bay began in 1972 but it became evident that due to continuous erosion of the north Wirral foreshore, the new defences would deteriorate rapidly. Traditional groynes had proved unsuccessful at Kings Parade to the North and it was ultimately decided to experiment with an offshore breakwater to control beach levels in the Leasowe Bay. The Wallasey breakwater was completed in 1981. The structure is 240m long and 52m wide and is located at a 45° angle to the shoreline.

The second breakwater at Leasowe Bay was constructed in response to a different set of environmental conditions. The shoreline at the proposed location was subject to a combination of flow from the bay, wave action in the bay and a wave induced current at the junction of the bay and the west end of the Leasowe revetment. This had led to scouring of the beach at the end of the revetment and it became necessary to realign the angle of wave approach to reduce the impacts on the reconstructed wall. A breakwater with a shore-connected link was proposed to protect the new sea wall. The shore parallel section took the form of a submerged porous structure constructed from wave absorption blocks. Ultimately, it was decided not to fully connect the breakwater with the shore parallel section in order to divert wave energy into the lee of the breakwater. The structure was completed in 1982 and has a length of 200m. The crest width is 30m and the offshore distance is approximately 150m. Both structures are emergent at all states of the tide.

Two further detached breakwaters were constructed along the King's Parade frontage in order to intercept wave activity. The proposed structures were designed to be submerged at high water and possess a variable permeability. For this reason, the

breakwaters were eventually constructed from a combination of armour stone and pre-cast concrete reef blocks. (see Barber and Davies, 1985 and Davies, 1985).

4.4.3 Elmer

A series of eight shore-parallel detached breakwaters were constructed at Elmer in West Sussex to protect an eroding mixed sand/shingle beach and adjacent high value residential property. The scheme involved the enlargement of two breakwaters constructed previously by Arun District Council, the placement of six additional shore parallel structures and a major beach nourishment. The breakwater segments vary in length from approximately 70m to 140m. The longer segments are located in the centre of the scheme. The structures are positioned between 115m and 140m offshore. A terminal rock groyne has also been built at the eastern end of the scheme. The breakwaters are emergent at all states of the tide.

The monthly average wave height is around 0.8m in winter and 0.3m in summer. Storm waves may exceed 3.5m offshore. The toe of the structures is exposed on all low tides except extreme neaps. Therefore, at low tide, the breakwaters are above the water line and they begin to operate as the water level rises. Well-developed salients have formed in the lee of the structures and are clearly visible at high tide. At low tide however, it is apparent that sand tombolos have formed such that as the tide recedes, a series of individual bays form inshore of the structures (Axe et al, 1996).

During storm conditions, beach profiles of the downdrift beaches were observed to suffer more than those in the lee of the structures. Longshore transport between the 'bays' is maintained until the ebbing tide exposes the salients. On the whole the scheme fulfils its design specification of shingle retention but severe downdrift erosion has been observed. This is contradictory to the results of a physical modelling study, which predicted that the structures would not have a negative impact on downdrift beaches. The disruption to longshore transport has been caused by the accumulation of sand in the lee of the breakwaters (King et al, 2000).

4.4.4 Monks Bay

Monks Bay is located at Bonchurch to the east of Ventnor on the south coast of the Isle of Wight. The beaches in the area have suffered continual erosion and 19th Century defences comprising a seawall and timber groynes had ceased to function by the end of the 1980's. The beaches along this frontage are predominantly shingle and mixed sand/shingle. The material for the Monk's Bay beaches had been derived largely from the erosion of the Ventnor cliffs. Due to the protection of these frontages, the supply of available beach material had virtually disappeared. Following a series of landslides, a major coastal protection scheme was instigated. Following physical and numerical modelling studies, a scheme comprising rubble mound groynes, a detached offshore breakwater and beach nourishment was proposed. The beach is free to move within the confines of the groyne system and to build up against the eastern groyne during south-westerly wave conditions. The breakwater ensures that the central section of the beach is retained during all wave conditions.

4.4.5 Llanelli

A scheme comprising 4 rock armour breakwaters (including one fishtail groyne) was constructed at Llanelli in 1992. The main aim of the works was to prevent erosion of the inter-tidal zone due to the migration of an estuary low water channel and to build local

beach levels. Additionally, the existing defences were becoming increasingly unstable. Beach nourishment was also carried out within two years of the breakwater construction.

The scheme has provided adequate standards of defence along the whole frontage but has suffered from low beach levels at two locations prejudicing the integrity of the seawall foundations. Surveys of the beach area between the breakwaters showed no loss of nourishment volume during the first 4 years after placement. No further information about the structure specifications is available.

4.4.6 *Dengie*

A novel detached breakwater scheme was constructed along the Dengie Peninsula in Essex during 1985 to prevent further erosion of the saltmarshes. The scheme comprised a series of disused barges (Thames Lighters) that were filled with locally won material and sunk parallel to the coastline some distance offshore. One year after construction, a site visit confirmed that the breakwaters have been successful in preventing further erosion and increasing beach levels in their lee. Eventually, however, erosion of the muddy seabed directly in front of their seaward face caused problems with this scheme.

4.4.7 *Rhos on Sea*

A detached rubble mound breakwater was constructed at Rhos on Sea (North Wales) in 1983. The aim of the scheme was to alleviate flooding of the residential area behind the sea wall caused by wave overtopping. The breakwater is positioned at the low water mark facing east-north-east. The structure is 200m long and is located approximately 150-175m offshore. An armour stone groyne was also constructed to the north of the breakwater in order to prevent north easterly waves from entering the gap at the northern end of the breakwater and running along the seawall. Since completion of the structures, sediment has accumulated in the lee of the breakwater and in the lee of the groyne, requiring intermittent dredging.

4.4.8 *Clacton*

A proposal has been submitted to construct a series of detached breakwaters along the frontage between Clacton on Sea and Holland on Sea. The proposed structures have a length of 200m, a gap width of 200m-300m and are positioned approximately 200m offshore.

4.4.9 *Gorleston*

The 50-year strategy for the Gorleston Coast Protection Scheme proposes the construction of nine shore-parallel reefs positioned approximately 175m offshore from the seawall in the north and around 90m at the southern end. The reefs will be positioned inshore of the zone of littoral drift so as not to cause any disruption to the transfer of material downdrift. The aim is to utilise the cross-shore transport of sediment by initiating wave breaking and the subsequent deposition of material in their lee. Note: Following consultation and modelling studies, the layout and specification of the structures may have changed.

4.5 BREAKWATER CASE STUDIES USA

A large number of detached breakwater schemes have been built as coastal defences on the oceanic and Great Lakes shorelines of the USA. Basic details are presented below.

4.5.1 *Lorain, Ohio on Lake Erie*

Date of Construction: 1977

Purpose: To protect placed beach fill and maintain a recreational park

Construction Aspects: (from Herbich, 1999)

The Lakeview Park project consists of three breakwater segments, two terminal groynes and beach fill (Pope and Dean, 1986). Each breakwater segment is 76.2m in length and the units are positioned at an average offshore distance of 146m and the gap width between the structures is 49m (Pope and Rowan, 1983). The breakwaters were constructed in a water depth of around 2.4m and the crest height of the structures was set at 2.4m.

Wave conditions:

The average nearshore breaking wave height is 0.8m and the average period is 3.5 seconds. The predominant wave direction is from the east and wave heights of up to 2.5m occur annually.

Beach fill:

84,000m³ beach fill was placed during the original nourishment programme and an annual recharge of ~4000m³ was proposed. However, the recharge was only carried out twice during the first 5 years of the project.

Shoreline Response:

Once equilibrium beach profile had been reached, only minor shoreline variations were observed (Pope and Dean, 1986). After five years, the beach had been aligned in a more westerly direction than originally intended and the western breakwater maintained only a narrow beach. The two other segments maintained the beach fill and produced some shoreline accretion. Cuspate salients formed in the lee of these two structures. The shoreline now maintains a stable configuration with accretion rates of around 2300m³ per year.

4.5.2 *Winthrop Beach, Massachusetts*

Date of Construction: 1931-1933 (completed 1935)

Purpose: To stabilise an eroding coastline and protect existing seawall.

Construction Elements:

The Winthrop beach project Consists of five breakwater segments (Pope and Dean, 1986). The segments are 100m in length, the gap width is 30m and the total structure length is around 625m. The structures are located 300m offshore in 3m water (MLW) and the crest height is 4.1m. The normal tidal range is 2.7m (Herbich, 1999).

Wave conditions:

Winthrop Beach is exposed to north-easterly and south-easterly waves. The maximum significant wave height is around 7m (0.1% annual exceedence).

Beach Fill:

Beach fill has been placed north and south of the breakwater along with a groyne field during the first 20 years after construction (Herbich, 1999).

Shoreline Response:

During high tide, the structure behaves as a combined entity resulting in single cusped salient (Herbich, 1999). During low tide, individual tombolos are formed behind several of the segments resulting in low tide headlands (Pope and Dean, 1986). The breakwater has succeeded in protecting the seawall and forming a recreational beach.

4.5.3 *Haleiwa, Oahu, Hawaii*

Date of Construction: 1965

Purpose: To develop and protect a recreational beach

Construction Aspects

A detached single breakwater was constructed 91m offshore. The unit is 49m in length and the water depth and crest height are 2.1m and 1.2m respectively.

Wave conditions:

This location is exposed to large high-energy storm waves during the winter months.

Beach Fill: 69,000m³

Shoreline Response:

A cusped salient formed in the lee of the structure. The breakwater was damaged during a severe storm and the shoreline began eroding as a result. Additional beach nourishment was subsequently carried out. The breakwater has also provided a sheltered swimming area.

4.5.4 *Santa Monica, California (from Herbich, 1999 and Handin and Ludwick, 1950)*

Date of Construction: 1933

Purpose: To provide a harbour for small boats

Construction Aspects:

The Santa Monica breakwater consists of a rubble mound structure built 610m offshore. The structure is 640m in length and the crest height was fixed at 3m above MLLW. The water depth at the structure is around 8m.

Wave conditions:

The mean significant wave height and period is 0.37m and 10.5s respectively

Beach Fill: None

Shoreline Response

The shoreline in the lee of the structure accreted by approximately 240m and blocked the littoral drift causing severe downdrift erosion. Higher wave transmission due to structural degradation re-established the littoral drift after a number of years. A broad salient remains in the lee of the structure and accretion of the beaches to the north has also occurred.

4.5.5 *Channel Islands Harbour, California (from Herbich, 1999)*

Date of Construction: 1960

Purpose: Provide shelter for a small craft harbour and entrap sediment for bypassing

Construction Aspects:

A single rubble mound structure was constructed 550m offshore in approximately 9m water. The structure is 700m in length.

Wave conditions:

The predominant wave direction is from the north and northwest. Breaker heights of between 1m and 3m are common.

Beach Fill: None

Shoreline Response:

The shoreline has experienced rapid accretion. During the first seven years, 10,712,00m³ sediment was artificial bypassed to maintain downdrift beaches.

4.5.6 *Ala Moana Park, Honolulu, Hawaii*

Date of Construction: 1964

Purpose: The structure initially formed part of a larger recreational development that was never completed.

Construction Aspects:

The design incorporates two shore-attached breakwaters at either end of four detached rubble mound structures placed in an overlapping fashion. The structures vary in length from 15-20m.

Wave conditions:

The breakwaters are situated in a sheltered location although storm waves in the area can be severe.

Beach Fill: An artificial beach was created with beach fill.

Shoreline Response

A high percentage of the beach fill has been retained. A small salient has developed in the lee of the innermost breakwater. Some erosion has occurred along the eastern shore.

4.5.7 *Colonial Beach, Virginia*

(from Pope and Dean, 1986 and Herbich, 1999).

Date of Construction: 1982

Purpose: To protect a cohesive bluff and shore road and to create a recreational beach.

Construction Aspects:

Two segmented breakwater schemes were constructed. The Central Beach project comprises four detached rubble mounds placed 64m from the original shoreline. Each segment is 64m long with 46m gaps. Castlewood Park Beach Project consists of a three detached rubble mound breakwaters of which two are 61m long whilst the third is 91m long. The gaps between the structures are 27m and 46m. The structure was placed 46m offshore. The breakwaters were constructed perpendicular to the predominant wave direction at a crest elevation of 0.91m.

Wave conditions:

Average wave heights vary from 0-0.3m whilst locally generated storm waves vary from 0.5-1.4m.

Beach Fill: 40,400m³ beach fill was placed at Central Beach and 11,500m³ were placed on Castlewood Beach.

Shoreline Response:

Tombolos formed behind all seven breakwaters. The Castlwood Park Beach has been relatively inactive. Water movement through the gaps has been restricted and a marshy, tidal flat has developed in the embayments. Central Beach is more dynamic and has developed a stable tombolo in the lee of the structure, which now serves as an attractive recreational amenity.

4.5.8 *Venice, California*

(from Herbich, 1999)

Date of Construction: 1905

Purpose: Protection of an amusement pier that has since been removed.

Construction Aspects:

A single rubble mound structure of 183m length was constructed 370m from the shoreline (MHW). During the 1960s, a timber groyne was constructed in the lee of the structure creating a T head groyne.

Wave conditions:

The average nearshore wave height is 0.4m. The largest storm waves range from 3.7-4.6m

Beach Fill: 11 million m³ of dredged sand was deposited on the shoreline in 1948.

Shoreline Response

The shoreline showed significant accretion and a tombolo formed in the lee of the breakwater. This eventually eroded but a salient remained. Construction of the T groyne resulted in the formation of a permanent tombolo. The structure now maintains a wide recreational beach.

4.5.9 *Redington Shores, Florida*

(from Herbich, 1999)

Date of Construction: 1985

Purpose: The breakwater was constructed to protect a sea wall and to maintain a recreation beach.

Construction Aspects:

A low crested (0.5m MSL) breakwater was constructed at 450 to the shoreline. The structure is 46m in length and has a permeability in excess of 50%. A second, 79m long segment was placed parallel to the shoreline.

Wave conditions:

The yearly mean wave height ranges from 0.06m to 0.3m although hurricanes occasionally pass through the area.

Beach Fill: 23,000m³ beach fill was placed in front of the sea wall.

Shoreline Response:

A well-developed salient formed in the lee of the offshore breakwater but 200m downdrift of the structures, the beach has retreated back to the wall. Some armour units were removed to combat this problem.

4.5.10 Presque Isle, Pennsylvania

(from Pope and Dean, 1986 and Herbich, 1999)

Date of Construction: 1978

Purpose: To ensure the stability of the Presque Isle Peninsula and to provide recreational beaches.

Construction Aspects

A total of 58 breakwaters have been constructed to protect a 10km sandy spit. Three initial segments were constructed as a test case. The segments are 40m in length and are spaced at 60m – 90m intervals. The units have a crest height of 1.2m.

Wave conditions:

The largest waves come from a westerly direction. The majority of waves are less than 0.7m H_s .

Beach Fill: The entire beach was artificially nourished.

Shoreline Response

The shoreline response varies seasonally. During low water levels and low wave energy, a tombolo forms behind the updrift segment with two smaller salients behind the other two structures. The low crest elevation permits sufficient overtopping during higher wave energy conditions to disconnect the tombolo and re-establish longshore transport. The shoreline is relatively stable and the beach fill is maintained.

4.5.11 Holly Beach, Louisiana

(from Herbich 1999)

Date of Construction: 1985

Purpose: To protect a highway and revetment.

Construction Aspects:

A six segment breakwater scheme was constructed. One segment is of rubble mound construction, whilst the remaining five involves various configurations of rock, riprap, and timber piles and used tyres. Each segment is around 45m in length and the gap widths vary from 89 to 93m. The tyre and pile segments are more permeable than the rubble mounds.

Wave conditions:

The wave climate at Holly Beach is moderate with an average breaker height of 0.5m. The predominant wave angle is from the south and southeast. The spring tidal range is around 0.75m.

Beach Fill: None

Shoreline Response

Salient formation occurred rapidly at the extremities of the scheme as sediment was driven into the protected area from both directions. The rubble mound structure created

a low tide tombolo whilst the tyre and pile structures created variable salients as function of their permeability. Damage to the tyres and timber poles occurred shortly after placement and hence this type of structure may be seen as a temporary solution rather than a permanent method of shoreline protection.

4.5.12 Gloucester Point, Virginia, on the York River

(from Herbich, 1999)

Date of Construction: 1983

Purpose: To mitigate erosion caused by groyne placement

Construction Aspects:

A three-segment gabion and riprap breakwater was constructed within an embayment between two groynes. The southern and central sections are 12m and 10m offshore respectively and are 11m in length. The gap length between the two segments is ~15m. The northern segment is 7.3m in length and is placed 15m offshore. This unit is 18m from the central unit. The crest level is approximately at MHW.

Wave conditions:

Waves are predominately from the north and northwest.

Beach Fill None.

Shoreline Response

Tombolos have formed in the lee of the southern and central units at low tide. During higher water levels, the tombolos become submerged leaving cusped salients. The shorter, northern segment has only developed a salient at low water and has experienced some shoreline erosion. On the whole, the project has stabilised 75% of the shoreline within the embayment.

4.5.13 Lake Forest, Illinois (western shore of Lake Michigan)

(from Herbich, (1999)

Date of Construction: 1987

Purpose: To protect eroding bluffs and to maintain an artificial recreational beach.

Construction Aspects

Two shore-connected breakwaters were constructed at either end of the site and three T groynes were placed at intervals in between creating a series of sediment cells. The crest height of the structures is 2.4m allowing for overtopping during storms. Two of the offshore structures are 76m in length and have shore connected sections of 46m long. The third segment is 61m long and is connected by a 52m groyne. The gap widths are 61m and 76m.

Wave conditions:

A maximum significant wave height of 3m was predicted and the predominant wave angle is from the NE.

Beach Fill: Sufficient beach fill was placed on the shoreline to allow for the predicted response to the structures.

Shoreline Response

Well-developed sinuous salients have formed in the lee of the structures. Very little beach material has been lost and there has been no downdrift erosion due to the structures. A wide recreational beach has been maintained.

4.5.14 *Palm Beach, Florida*

(from Browder et al, 1996 and Dean et al, 1997)

Date of Construction, 1992

Purpose: To reduce shoreline recession and provide protection from storm waves.

Construction Aspects

A total of 330 pre-case concrete reef units were placed approximately 73m offshore to form a continuous submerged reef measuring a total of 1260m in length. Each reef unit measured 1.8m high, 4.6m wide and 3.7m long. The units were placed in 3m water depth. All units were placed directly on the sandy seabed with no foundation material.

Wave conditions: No available statistics

Beach Fill: None

Shoreline Response

The reef was found to cause a 5-15% reduction in incident wave height and also modified nearshore currents. A three-year monitoring programme beginning immediately post construction, found that erosion had occurred throughout the project area (particularly in the lee of the reef. The erosion rate was measured to be 2.3 times higher than the pre-project rate. Consequently, the structure was removed in 1995.

4.5.15 *Vero Beach Florida*

(from Smith *et al*, 1998).

Date of Construction: 1996

Purpose: Shoreline protection and maintenance of recreational beaches.

Construction Aspects:

A 915m long prefabricated concrete submerged breakwater was placed offshore of the City of Vero Beach. The structure comprises 215 interlocking concrete units each measuring 4.6m x 3.7m x 1.8m. Based on the experience gained with the failed Palm Beach breakwater, this structure was placed in eleven segments ranging in length from 50m to 160m and alternating in an onshore, offshore configuration. The crest level was set at 0.3 – 0.9 (for onshore and offshore units) below MSL.

Wave conditions: No available information

Beach Fill: None

Shoreline Response

Wave measurements indicate that the reef reduced incident wave heights by 12% after the initial installation and then by 9% after settling had occurred. Following breakwater installation, both shoreline and volumetric data suggest a net accretion in sediment over the entire project length.

4.5.16 *Cameron Parish, Louisiana*

(from Nakashima et al. 1987 and Underwood et al. 1999).

Date of Construction: 1991-1994

Purpose: To reduce long-term shoreline recession.

Construction Aspects:

A total of 85 shore-parallel breakwaters were constructed along 12km of shoreline. The segments have a length of 45 to 55m with gaps of 91m. The structures are positioned between 75 and 185m from the shoreline in water depths of 1.2 to 1.8m. The crest level is 1.2m above MSL. The L/D ratios are 0.3 to 0.6 and the G /D ratios are 0.5 to 1.2.

Wave conditions:

Average breaker heights are in the region of 0.5m with a 5 second period.

Beach Fill: None

Shoreline Response

Overall beach response has been net accretion since 1995 but this has been confined to the eastern end of the scheme (with 50 breakwaters). The remaining 35 breakwaters are receiving little or no sediment from the longshore drift system. At some locations, the gap widths appear to be too large resulting in shoreline retreat opposite the gaps. Those breakwaters that were situated the furthest from the shore also tended to cause shoreline retreat, as the trapping rate was too low. Modifications of the existing structures are recommended to address erosion problems at specific locations. This would include locating the eastern most segments further offshore to provide more available sediment for downdrift beaches.

4.5.17 Raccoon Island - Louisiana

Stone et al. (1999) studied the morphological coastal response after construction of detached segmented breakwaters along Raccoon Island (one of the Isles Dernieres barrier island chain), Gulf of Mexico, Louisiana. In the summer of 1997, eight segmented emergent breakwaters were constructed approximately 90m off the eastern seaboard of the island. The construction objective was to mitigate beach erosion by reducing wave energy at the coast. The structures are 90m long with a crest width of 3m and crest height in excess of 2m. The gap width was also 90m.

The sequence of morphological changes after completion of the breakwaters were as follows:

2 months after construction 10-15m salients were observed in the lee of the breakwaters whilst shoreline recession of around 5 to 10m had occurred opposite the gaps between the structures.

7 months after construction, substantial sand accumulation had occurred directly behind the breakwater forming a 'reverse salient'.

12 months after construction, sand accumulation in the lee of the breakwater continued and had extended to the gaps between segments resulting in a continuous stretch of sand connecting several breakwaters. Sand had also accumulated up to 20m seawards of the structures.

14 months after construction the accumulated sand to seaward of the breakwaters had eroded but sand accumulation shoreward of the structures continued as before.

Wave transmission through the structures was found to be minimal but monitoring of wave periods shoreward of the breakwaters showed that high frequency waves were more effectively reduced than low frequency waves.

4.6 DETACHED BREAKWATERS IN JAPAN

Until the 1960s, structures built to combat beach and coastal erosion in Japan were dominated by groynes and seawalls (Hsu et al, 1999). Following the successful application at Ishizaki in 1966, detached breakwaters have been constructed almost indiscriminately as coastal protection measures. In 1962, there were 205 examples of detached breakwaters in Japan (Toyoshima, 1984). By 1996, there were a total of 7371 such structures (Kawata and Shibayama, 1998). Some examples of detached breakwater construction in Japan are given below.

4.6.1 *Ishizaki*

The first detached breakwater system in Japan was constructed in 1966 on the Ishizaki coast in Hokkaido. The structure was built to promote the regeneration of a beach that had eroded due to the construction of a seawall during the early 1960s. The breakwater is a shore-parallel tetrapod construction, 60m long and located 35m from the seawall. A tombolo formed in the lee of the first structure and consequently a total of eleven segments were placed along the shore and the beach erosion problems have been overcome.

4.6.2 *Kaike*

Toyoshima, (1976) described shoreline changes behind a series of emergent breakwaters that were constructed along the Kaike coast in 1971. A total of 29 structures were placed 110m offshore in 5m water depth. The structures were 150m long and had 50m gaps between them. The crest level was 2m and the structures were built in around 5m water. Kaji et al, (1989) studied the effects of the Kaike breakwaters on erosion and accretion. A large amount of sand accumulated in the lee of the breakwaters and accretion rates varied between 250m² – 600m² per year during the first 10 years. The annual fluctuations of the accretion areas were much larger than the long-term accretion values; fluctuations as high as 1500m² per year (erosion and accretion) were observed and can be attributed to seasonal effects. Strong rip currents of around 0.6m/s were generated through the breakwater gaps during storm conditions transporting sand offshore. During summer, sediment in transported onshore through the gaps and deposited in the lee of the structures.

4.6.3 *Niigata*

Constructed 1973

A series of submerged detached breakwaters were constructed to the east and west of the Shinano River mouth in an attempt to combat severe coastal erosion. The scheme comprised groynes and seawalls in addition to the structures. The breakwaters suffered from severe toe scour and subsidence. In order to improve efficiency, the crest width was increased to 50m whilst the permeability and wave reflection coefficients were decreased. Sawaragi (1988) describes the construction and morphological effects of the structures. The breakwaters were of rubble mound construction and the crest level was set at 2m below MSL. The structure length was 200m and the gap width was 40m. The structures were placed between two existing emergent breakwaters. The shoreline facing the submerged structures did not show accretion but did not erode either. The shoreline facing the emergent structures advanced seawards as salients formed in the lee of the breakwaters.

4.6.4 *Oarai*

The port of Oarai is location north of the Kashimanada Coast, which is a bow shaped sandy beach about 70km long between the Naka River to the north and the Tone River in the south. The predominant wave direction at Oarai is southwards. A harbour built at Isohama was built but was prone to siltation before it was completed. An L shaped breakwater was constructed between 1977 and 1981 to deflect the littoral drift seaward. In 1981 a 45° arm was added, which caused further downcoast beach erosion. An offshore detached breakwater was also added in 1981 and this compounded the erosion problems.

Kuriyama et al, (1988) studied shoreline evolution behind five detached (emergent and submerged) breakwaters along the Pacific coast of Japan. The structure lengths are between 65 and 260m long and the gap lengths vary between 0.3L and 0.7L. The breakwaters were constructed between 40m and 170m offshore. The local beaches were artificially nourished after installation of the breakwaters. Well-developed salients were observed in the lee of the structures and the beaches remained stable under both calm and storm conditions.

4.6.5 *Satsuma Peninsula*

The Nagasaki coast is located on the Pacific coast of Japan and is subjected to intense typhoons. Mutugami et al, (2001) describe the effects of submerged reefs that were installed to protect eroding bluffs. The first reef was constructed in 1996 with the crest fixed at approximately mean low water level. The structure length is around 200m. Post construction surveys showed that the beach in the lee of the reef had accreted by up to 20m whilst the beaches outside the lee area had retreated.

4.7 DETACHED BREAKWATERS IN SPAIN

4.7.1 *De Estepona Beach – Malaga*

Estepona beach is composed of medium to coarse sand and runs along the Estepona frontage to the port at its southern end. Five rock groynes (of which one was semi-submerged) were built along the northern section of the beach in 1973 to protect the promenade. However these structures did not offer protection from storm waves and also restricted longshore transport resulting in beach erosion at the southern end of the frontage. The present regeneration scheme comprised the following elements:

- Demolition of the four emergent rock groynes and elongation of the semi-submerged structure to a depth of –5m.
- Construction of a 180m long shore parallel breakwater.
- Supply of 196,000m³ sand for beach nourishment.

4.7.2 *El Palo Beach – Malaga*

The gradual reclamation of seafront land for building works in the Malaga area has increased the coastal defence requirements in the area. The El Palo beaches extend approximately 1.8km from Jaboneras Brook to the port at El Candado. The average beach width of this frontage is around 10m, which is insufficient to meet the recreational amenity demands of the area. A large-scale regeneration scheme was carried out comprising the following elements:

- The supply of 135,000m³ sand for beach nourishment;
- Three 90m emergent breakwaters;
- Three fishtail groynes; and
- Four submerged breakwaters.

4.7.3 *Pedregalejo Beach – Malaga*

The Pedregalejo district to the east of the city of Malaga has a long history of beach erosion problems. Ultimately, all beach material was lost from the frontage resulting in damage to seafront building during winter storms. A major regeneration scheme has been undertaken in order to provide protection to seafront property and to provide recreational beaches to boost tourism in the area. The scheme comprised the following:

- The construction of a promenade
- Supply of sediment for beach nourishment
- The construction of two shore parallel detached breakwaters
- Two fishtail groynes and three smaller ‘headland’ groynes
- An L shaped terminal groyne at the western end of the frontage.

4.7.4 *Castell De Ferro Beach – Granada*

The beach at Castell de Ferro became seriously depleted after the illegal construction of a seafront promenade, which restricted the natural supply of sediment. A regeneration scheme was carried out comprising the following:

- The demolition of the promenade and other seafront buildings;
- The construction of a shorter promenade after a suitable planning operation;
- The construction of three 80m shore parallel breakwaters. These structures are linked by two 150m submerged sills; and
- The supply of 130,000m³ sand for beach nourishment.

4.7.5 *Aguadulce Beach – Almeria*

The objective of this scheme was to regenerate the tourist beaches between Hortichuelas gully and the Canarete cliffs. The beach consisted of a narrow strip of sand and gravel that could not cope with the tourism demands during the summer months. The following works were undertaken:

- The construction of a 145m shore parallel detached breakwater at the eastern end of the frontage; and
- The supply of 240,000m³ sand for beach nourishment.

Monitoring carried out since the completion of the scheme shows that the structure is performing as expected and that beach volumes have been maintained.

4.7.6 Rihuete Beach – Murcia

Rihuete beach was a stable sandy frontage between the port to the west and the Los Lorentes delta to the east. After construction of the Mazarron port breakwater, sediment accumulated on the updrift side of the structure whilst the beaches to the east disappeared. A regeneration scheme was carried out including:

- The construction of three shore-parallel detached breakwaters of 90, 85 and 80m. The structures are low crested to allow overtopping;
- A small rock groyne was constructed adjacent to the port breakwater; and
- The supply of 180,000m³ of sand for beach nourishment.

4.7.7 Almazora Beach – Castellon

The Almazora coastline is located immediately to the south of the port of Castellon. Large volumes of sand have accumulated against the updrift arm of the port breakwater whilst downdrift beaches have become increasingly depleted. An attempt to remedy the situation was made by the construction of two shore parallel breakwaters but this did not solve the problem as there was no available sediment supply. During the second phase of regeneration, 117,000m³ of sand and gravel were to nourish the beach. Sediment volumes along the frontage are maintained by the offshore breakwaters and tombolos have formed in the lee of the structures.

4.7.8 Ribes Rotges Beach – Barcelona

The beach at Ribes Rotges became increasingly unstable after the construction of the port at Villanova. Large volumes of sand accumulated against the harbour breakwater whilst the other end of the frontage suffered severe erosion. The regeneration scheme included the construction of a low crested, shore parallel breakwater approximately halfway along the frontage. A terminal groyne was built at the western end of the frontage. Since construction a tombolo has formed in the lee of the breakwater and beach levels along the entire frontage have increased.

4.7.9 San Antonio de Calonge Beach – Gerona

Beach erosion at Calonge began eroding following the construction of the port at Palamos. The problem was partially solved by the construction of three detached breakwaters but the beaches to the east of the structures continued to erode until no beach remained. Two semi submerged rock groynes have since been placed at the eastern end of the frontage and a 120,000m³ recharge scheme has been implemented to rebuild the beach.

4.7.10 Review of Structures along Spanish Mediterranean Coast

Berenguer and Enriquez (1988) studied the morphological response of beaches behind 24 multiple emergent breakwaters along the Spanish Mediterranean Coast. The breakwater lengths (L) are between 100m and 300m, the gap lengths (G) are between 60m and 200m and the offshore distances (D) are between 100m and 270m. The structures are built in water depths of 1.5 to 5.7m. Tombolos or salients developed behind the segmented breakwaters and the equilibrium accretion area behind the structures was found to be about $0.4D(L+G)$.

5. *Reefs/ Submerged breakwaters*

Submerged breakwaters are also known as reefs. The crest of submerged breakwaters is positioned below the still water level. These structures are only effective in trapping sand for salient formation if the crest level is relatively high (0.5 to 1m below MSL) in a microtidal regime and the beaches that form are small compared to those behind emergent breakwaters.

Longshore and cross-shore rip currents tend to form in the gaps between multiple structures due to the transport of water over the crest. This may result in erosion in the lee of the structures and the effect increases with decreasing crest level. Submerged breakwaters are not particularly effective during storm conditions or surges and hence do not provide adequate protection to the coastline during extreme conditions. Submerged breakwaters can be used to generate breaking waves at popular surfing locations but these can also present a significant hazard to boats operating in the nearshore zone. Compared to emergent breakwaters, artificial reefs are relatively cheap to construct and maintain. Numerous examples of these reefs have been successfully applied in low energy microtidal environments such as the Mediterranean.

The crest elevation of a submerged breakwater determines the amount of wave energy transmitted over the top of the breakwater. High crest elevations preclude overtopping by all but the largest waves, whereas low crested structures allow frequent overtopping. Overtopping can prevent tombolo formation or remove it once it has formed. Submerged breakwaters (reefs) allow almost continual overtopping of waves during both calm and storm conditions.

6. *Bibliography*

1. Abdul-Khader, M. H., and Rai, S. P., 1980. A study of submerged breakwaters. *Journal of Hydraulic Research* 18 (2). 113-121.
2. Allsop, N. W. H., 1994. Design of rock armoured beach control structures. MAFF Conference of River and Coastal Engineers, 1994. 2.1.1-2.1.12
3. Aminti, P., et al., 2004. Evaluation of beach response to submerged groyne construction at Marina di Ronchi, Italy using field data and a numerical simulation model. *Journal of Coastal Research*, SI 33. 99-120.
4. Andrews, J., and Powell, K, A, 1993. Monks Bay scheme, Isle of Wight. MAFF Conference of River and Coastal Engineers, 1994. 2.1.1 to 2.1.15.
5. Axe, P., Ilic, S. and Chadwick, A., 1996. Evaluation of beach modelling techniques behind detached breakwaters. *Proc. International Conference on Coastal Engineering*. 2036-2049.
6. Banyard, L. S. and Mannion, M.B., 2001. Design of rock groynes and sand beach replenishment under strong tidal flows. *ICE Breakwaters, Coastal Structures and Coastlines Conference 2001*.
7. Barber, P. C. and Davies, C. D., 1985. Offshore breakwaters – Leasowe Bay. *Proc. Institution of Civil Engineers, Part 1* p 85-109.
8. Basco, D. R., and Pope, J., Groyne functional design guidance from the coastal engineering manual. *Journal of Coastal Research*, SI 33. 121-130.
9. Berenguer, J., and Enriquez, J., 1988. Design of pocket beaches, the Spanish case. *Proc. International Conference on Coastal Engineering*. 1411-1425.
10. Bird, E. C. F., 1996. *Beach Management*. J.W. Wiley & Sons: 281pp
11. Bird, P. A. D., Davidson, M. A., Ilic, S. Bullock, G. N., Chadwick, A. J., Axe, P. and Huntley, D. A., 1996. Wave reflection, transformation and attenuation characteristics of rock island breakwaters. *Advances in Coastal Structures and Breakwaters*. Thomas Telford. 93-106.
12. Brampton, A. H., Motyka, J. M., 1983. The effectiveness of groynes. *Shoreline Protection*. Thomas Telford. 151-156.
13. Brampton, A. H., and Smallman, J. V., 1985. Shore protection by offshore breakwaters. *HR Wallingford Report No SR 8*. 33pp.
14. Browder, A. E., Dean, R. G., and Chen, R., 2000. Performance of submerged breakwater for shore protection. *Proc. International Conference on coastal Engineering*. 2312-2323.
15. Bull, C. F. J., Davis, A. M., and Jones, R., 1998. The influence of fishtail groynes on the characteristics of the adjacent beach at Llandudno, North Wales. *Journal of coastal research*. 14 (1). 93-105.

16. Carter, C. H., Monroe, C. B. Guy, D. E., 1986. Lake Erie shore erosion: the beach width and shore protection structures. *Journal of Coastal Research* 2 (1). 17-23.
17. Carver, R. D., and Bottin, R. R., 1997. Reef breakwater design for Burns Waterway Harbour, Indiana. *Journal of Coastal Research* 13 (4). 1267-1281.
18. CIRIA, 1990. Groynes in coastal engineering: data on performance of existing groyne systems. CIRIA Technical Note 135. 124pp.
19. CIRIA, 1996. Beach Management Manual. CIRIA Report 153. 448pp.
20. Dattari, J., Raman, H., and Jothi-Shankar, N., 1979. Performance characteristics of submerged breakwaters. *Proc. International Conference on Coastal Engineering*, 1978. 2151-2171.
21. Davies, C. D., 1985. Offshore Breakwaters at Wirral. *Municipal Engineer*, 2. 198-207.
22. Dean, R. G., Chen, R. and Browder, A. E., 1997. Full scale monitoring study of a submerged breakwater, Palm Beach Florida, USA. *Coastal Engineering*, 29 (3/4): p 291-315.
23. Debailon, P., Sergent, P. and Zhang, B., 2001. Morphological evolution behind a detached shore parallel breakwater. *Proc. International Conference on Coastal Dynamics*. 46-54.
24. Dette, H.H., et al, 2004. Permeable pile groyne fields. *Journal of Coastal Research*, SI 33. 145-159.
25. Donohue, K.A. et al., 2004. Experience with groyne notching along northern New Jersey coast. *Journal of Coastal Research*, SI 33. 198-214.
26. Fleming, C.A., and Hamer, B., 2000. Successful implementation of an offshore reef system. *Proc. International Conference on Coastal Engineering 2000*. 1813-1820.
27. Fleming, C. A., 1990. Guide on the use of groynes in coastal engineering. CIRIA Report, 119. 114pp.
28. French, P. W., and Livesey, J. S., 2000. The impacts of fishtail groynes on sediment deposition at Morcambe, North-West England. *Journal of Coastal Research*, 16 (3). 724-734.
29. Galgano, F.A., 2004. Long-term effectiveness of a groin and beach fill system: a case study using shoreline change maps. *Journal of Coastal Research*, SI 33. 3-18.
30. Galofre, J. and Montoya, F. J., 1999. Coastal structures applied to shoreline stabilisation: Tarragona case. *Proc. International Conference on Coastal Structures '99*, Balkema. Vol 2, 993-1000.
31. Gomez-Pina, G., 2004. The importance of aesthetic aspects in the design of coastal groynes. *Journal of Coastal Research*, SI 33. 83-98.

32. Hamer, B. A., Hayman, S. J. Elsdon, P. A., and Fleming, C. A., 1998. Happisburgh to Winterton sea defences: Stage Two. Proc. International Conference on Coastlines, Structures and Breakwaters 1998. 119-134.
33. Handin, J. W., and Ludwick, J. C., 1950. Accretion of beach sand behind a detached breakwater. US Army Corps of Engineers Beach Erosion Board, Technical Memorandum, No 16, 13pp.
34. Herbich, J. B., 1990. Pile and offshore breakwaters. In Herbich, J.B., (ed) Handbook of Coastal and Ocean Engineering, Vol. 1: Wave Phenomenon and coastal structures. Gulf Publishing. 895-920.
35. Herbich, J. B., 1999. Offshore (Detached) Breakwaters. In Herbich, J. B., Ed: Handbook of Coastal Engineering. McGraw-Hill. 5.1-5.97.
36. HR Wallingford Ltd., 1992. Elmer 3D physical model study. HR Wallingford Report EX 2529. 44pp.
37. HR Wallingford, 1999. Afon Dysynni to Aberdyfi coastal defence scheme. HR Wallingford Report EX 3941. 56pp
38. Hsu, J. R. C., and Silvester, R., 1990. Accretion behind single offshore breakwater. Journal of Waterway, Port, Coastal and Ocean Engineering. 116 (3). 362-380.
39. Hsu, J. C. R., Uda, T. and Silvester, R., 1999. Shoreline Protection Methods – Japanese Experience. In Herbich, J. B., Ed: Handbook of Coastal Engineering. McGraw-Hill. 9.1-9.77
40. Jackson, L. A., Tomlinson, R., McGrath, J. and Turner, I., 2002. Monitoring of a multifunctional submerged geotextile reef breakwater. Proc. International Conference on Coastal Engineering. 1923-1935.
41. Kaji, T., et al. 1989. Field investigation of nearshore current and littoral transport around detached breakwaters on the Kaike coast, Japan. Coastal Engineering in Japan, 32 (2). 173-186.
42. Kawata, Y. and Shibayama, T. (eds), 1998. Guideline of creating sediment transport environment. Current Status Review sub-committee, Committee on Coastal Engineering, JSCE. Tokyo: Japan Society of Civil Engineers.
43. King, D.M., Cooper, N.J., Morfett, J.C., and Pope, D.J., 2000. Application of offshore breakwaters to the UK: A case study at Elmer Beach. Journal of Coastal Research, 16(1). 172-187.
44. Komar, P. D., 1998. Beach Processes and Sedimentation. Prentice Hall: 544pp
45. Kraus, N. C. Hanson, H., and Blomgren, S. H. 1994. Modern functional design of groyne systems. Proc. International Conference on Coastal Engineering. 1327-1342.
46. Kunz, H., 1997. Groynes on the East Frisian islands: history and experiences. Proc. International Conference on Coastal Engineering, 1996. 1256-2572.

47. Kuriyama et al. 1988. Follow-up surveys of artificially nourished beaches. *Coastal Engineering in Japan*, 31 (1). 105-120.
48. Laustrup, C., 1988. Erosion control with breakwaters and beach nourishment. *Journal of Coastal Research* 4 (4). 677-685.
49. Loveless, J., and McLeod, B., 1999. The influence of set up currents on sediment movement behind detached breakwaters. *Proc. International Conference on Coastal Sediments*. 2026-2041.
50. Magoon, O.T., 1976. Offshore breakwaters at Winthrop Beach, Massachusetts. *Shore and Beach* 44 (3). 34.
51. Ming, D. And Chiew, Y. M., 2000. Shoreline changes behind detached breakwater. *Journal of Waterways, Port, Coastal and Ocean Engineering* 126 (2). 63-70.
52. Mohr, M. C., and McClung, J., K., 1999. Coastal response to a detached breakwater system; Presque Isle, Erie, USA. *Proc. International Conference on Coastal Sediments*. 2010-2025.
53. Moller, J. T., 1992. Balanced coastal protection on a Danish North Sea coast. *Journal of Coastal Research* 8 (3). 712-718.
54. MOPU – Direccion General de Puertos Y Costas, 1998. Actuaciones en la costa. *International Conference on Coastal Engineering*. 143pp.
55. Mutugami, M., Nishi, R. and Uda, T., 2001. Optimal design of artificial reefs in a pocket beach at Nagasakibana, Japan. *Proc. International Conference on Coastal Dynamics*. 36-45.
56. Nairn, R. B., and Dibajnia, M., 2004. Design and construction of a large headland system, Keta Sea Defence Project, West Africa. *Journal of Coastal Research*, SI 33. 294-314.
57. Nakashim, L.D., Pope, J., Mossa, J., and Dean, J. L., 1987. Initial response of a segmented breakwater system, Holly Beach, Louisiana, USA. *Proc. International Conference on Coastal Sediments*.
58. Nir, Y., 1982. Offshore artificial structures and their influence on the Israel and Sinai Mediterranean beaches. *Proc. International Conference on Coastal Engineering*. 1837-1856.
59. Owens, J. S., and Case, G. O., 1998. *Coast erosion and foreshore protection*. The St Brides Press Ltd. 144pp.
60. Pilarczyk, K. W. and Zeidler, R. B., 1996. *Offshore Breakwaters and Shore Evolution Control*. Balkema. 559pp.
61. Pilkey, O., et al., 1998. Evaluation of alternative or non traditional shoreline stabilisation devices. *Journal of Coastal Research*, SI 26. 269-272.
62. Poff, M.T. et al., 2004. Permeable wood groynes: case study on their impact on the coastal system. *Journal of Coastal Research*, SI 33. 131-144.

63. Pope, J., and Dean, J. L., 1986. Development of design criteria for segmented breakwaters. Proc. International Conference on Coastal Engineering. 2144-2158
64. Pope, J., and Rowen, D. D., 1983. Breakwaters for beach protection at Lorain, OH. Proc. International Conference on Coastal Engineering. 753-768.
65. Rankin, K. L. et al., 2004a. Nearshore currents and sediment transport measured at notched groynes. Journal of Coastal Research, SI 33. 237-254.
66. Rankin, K. L. et al., 2004b. Cross-shore distribution of alongshore currents and sediment fluxes in the vicinity of notched groynes. Journal of Coastal Research, SI 33. 255-268.
67. Rosen, D. S., and Vajda, M., 1982. Sedimentological influences of detached breakwaters. Proc. International Conference on Coastal Engineering. 1930-1949.
68. Russel, R. C. H., 1960. Coast erosion and defence – nine questions answered. Hydraulics Research Paper, No 3. Department of Scientific and Industrial Research. HMSO London. 14pp.
69. Sanchez-Arcilla, A., Gironella, X., Verges, D., Sierra, J. P. Pena, C., and Moreno, L., 2001. Submerged breakwaters and ‘bars’ from hydrodynamics to functional design. Proc. International Conference on Coastal Engineering. 1821-1835.
70. Sawaragi, T., 1988. Current shore protection works in Japan. Journal of Coastal Research 4 (4). 531-541.
71. Shabica, C. et al., 2004. Evolution and performance of groynes on a sediment starved coast; the Illinois shoe of Lake Michigan, north of Chicago. Journal of Coastal Research, SI 33. 39-56.
72. Shimoda, N., Murakami, N., and Iwata, K., 1991. Beach erosion by a submerged floating structure. Proc. International Conference on Coastal Engineering 1990. 2740-2753.
73. Silvester, R., 1990 Design of Seawalls and Groynes. In Herbich, J.B., (ed) Handbook of Coastal and Ocean Engineering, Vol. 1: Wave Phenomenon and coastal structures. Gulf Publishing. 1059-1080.
74. Smallman, J. V., Allsop, N. W. H., and Brampton, A. H., 1986. The hydraulic design of offshore breakwaters in coast protection. HR Wallingford Report No IT 305. 20pp.
75. Smith, J. B., Pope, J., and Tabar, J. 1998. Examination of a prefabricated submerged breakwater, - preliminary results: Vero Beach, Florida, USA. Proc. International Conference on Coastlines, Structures and Breakwaters 1998. 94-105.
76. Stauble, D. K. and Tabar, J. R., 2003. The use of submerged, narrow crested breakwaters for shoreline erosion control. Journal of Coastal Research, 19 (3). 684-722.

77. Stone, P. W., Wang, P. and Armbruster, C.K., 1999 Unanticipated response to detached, segmented breakwaters along Raccoon Island, Louisiana. Proc. International Conference on Coastal Engineering. 2057-2072.
78. Swart, D. H., and Horikawa, K., 1986. Design and evaluation of beach protection schemes. Proc. International Conference on Coastal Engineering 1986. 2268-2287
79. Syamsudin, A. R., Tsuchiya, Y., and Yamashita, T., 1995. Beach erosion in Kuta Beach, Bali and its stabilisation. Proc. International Conference on Coastal Engineering, 1994. 2683-2697.
80. Thomalla, F., and Vincent, C.E., 2004. Designing offshore breakwaters using empirical relationships: A case study from Norfolk, UK. Journal of Coastal Research, 20 (4). 1224-1230.
81. Toyoshima, O., 1976. Changes of sea bed due to detached breakwaters. Proc. International Conference on Coastal Engineering. 1572-1589.
82. Toyoshima, O., 1984. New type block for seawall slope protection. Proc. International Conference on Coastal Engineering, 1984. 2536-2545.
83. Trampenau, T., Goricke, F., and Raudkivi, A. J., 1996. Permeable pile groynes. Proc, International Conference on Coastal Engineering 1996. 2142-2151
84. Trampenau, T et al, 2004. Hydraulic functioning of permeable pile groynes. Journal of Coastal Research, SI 33. 160-187.
85. Underwood, S. G., Chen, R., Stone, G. W., Zhang, X., Byrnes, M. R., and McBride, R. A., 1999. Beach response to a segmented breakwater system southwest Louisiana. Proc International Conference on Coastal Sediments. 2042-2056.
86. US Army Corps of Engineers, 1981. Low cost shore protection, a guide for engineers and contractors. US Army Corps of Engineers Report.
87. US Army Corps of Engineers, 1992. Coastal groynes and nearshore breakwaters. Engineering Manual. Report EM. 1110-2-1617.
88. Van de Graaff, J., Steetzel, H., Blick, B., and de Vroeg, H., 1998. Shore parallel breakwaters and beach nourishments. Proc. International Conference on Coastal Engineering. 1706-1720.
89. Van Rijn, L. C., 2004. Principles of sedimentation and erosion engineering in rivers, estuaries and coastal seas. Aqua Publications.
90. Walker, D. J., Dong, P., and Anastasiou, K., 1991. Sediment transport near groynes in the nearshore zone. Journal of coastal research 7 (4). 1003-1011.
91. Wang, P., and Kraus, N. C., 2004. Movable bed investigation of groyne notching. Journal of Coastal Research, SI 33. 342-367.
92. Williams, A. J., and Davies, M. F., 1998. Dinas Dinlle coastal works. Proc. International Conference on Coastlines, Structures and Breakwaters. 223-236.

93. Zyserman, J. A., Broker, I., Johnson, H. K., Mangor, K., and Jorgensen, K., 1998. On the design of shore parallel breakwaters. Proc. International Conference on Coastal Engineering. 1693-1705.

Appendix 2

Desk study of alternative Coastal Defence options,
Sandbanks, Poole

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DESK STUDY OF ALTERNATIVE COASTAL DEFENCE OPTIONS SANDBANKS, POOLE



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DESK STUDY OF ALTERNATIVE COASTAL DEFENCE OPTIONS SANDBANKS, POOLE

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**Report prepared for HR Wallingford
on behalf of Poole Borough Council**

Executive Summary

An assessment of modern methods of coastal protection is undertaken for considered application at Sandbanks, Poole. Beach erosion at Poole requires constant intervention and the latest round of works is now required. The aim of this assessment is to update the technology applied in line with modern trends and knowledge. An assessment is made of the current trends in coastal works, and the status and success of these works. Given the high social and commercial importance of beaches, particularly in the Sandbanks region, it is essential to consider all of the coastal processes, engineering capabilities/costs and social background to decision making.

After considering a range of options, the following conclusions were reached:

- 1. If up to 50,000 m³ of nourishment was placed on the Poole beaches, only very small works (if any) would be needed.**
- 2. The nourishment programme should be finalised before making any decisions about control works.**
- 3. If works are undertaken, four small offshore reefs with volumes of 3000, 3500, 4000 and 5000 m³ would be sufficient. However, if up to 50,000 m³/yr of nourishment was guaranteed, then a maximum of 2 reefs would be recommended with volumes of 3000 and 4000 m³.**
- 4. As the fixed and mobilization costs are common to all reefs, the total project costs are smaller if the reefs are built together. Also, there will be cost savings if the sand can be obtained from Poole Harbour at no additional cost to the reef construction project. The cost for 4 reefs is £1,215,100 and the cost for 2 reefs is approximately half at £650,000.**
- 5. If groynes were adopted, up to 18 short groynes would be needed. The number of groynes may be less if regular nourishment was planned.**
- 6. The groynes have been very successful near the entrance to Poole Harbour, but in the wave-dominated environment to the north-east, reefs are expected to provide a better solution, especially in the socio-economic categories of visual appeal, access, safety and costs.**
- 7. Offshore breakwaters are not acceptable since they are better suited to situations where alongshore sediment transport is low, cannot incorporate the sophistication of submerged reefs (to address both across and alongshore sediment transport issues), and would significantly impact on the visual aesthetics of the area.**

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1 Background

1.1 Introduction

Following discussions led by HR Wallingford and held with Poole Borough Council, Defra (Regional Engineer, Taunton) and Bournemouth Borough Council, a desk study to review alternative methods of coastal protection was initiated. The goal is to find best ways to maintain satisfactory beaches along the coastline from Sandbanks (Shore Road) and the Poole/Bournemouth Borough boundary. The technology to be applied would be groynes, detached breakwaters, “reefs” or a combination, assuming that periodic beach recharge (nourishment) will also be carried out at Poole.



Figure 1.1. Sandbanks Poole on the north side of the Harbour entrance to Bournemouth Pier. The borough boundary is the black dash line to the east of Branksome Chine.

ASR Ltd has long experience of the beaches of Poole Bay working on behalf of Bournemouth Borough Council to develop a multi-purpose reef at Boscombe (Black *et al.*, 2000 and 2004a) and other beaches around Britain (Newquay and Borth, for example). Consequently, ASR Ltd was commissioned to:

- suggest alternative layouts for beach control structures (whether reefs, detached breakwaters or groynes) that would be suitable for the western end of Poole Bay.

The study considers existing coastal processes and the relative costs of alternative beach control structures, of different lengths and spacing, and their advantages and disadvantages in terms of aesthetics, public safety and their effectiveness, as proven elsewhere in similar situations. It will recommend between three to five options for further more detailed consideration, including modelling of their likely effects on beach plan-shape evolution, in later stages of the design process.

The complete team is broad, with HR Wallingford leading, ASR as second consultant for their expertise on offshore reefs, plus 3 international specialist experts.

1.2 Background

ASR Ltd has undertaken field studies and numerical modelling of Poole Bay (Black et al., 2000; Black et al., 2004a). The potential for a surfing reef at Boscombe in Bournemouth, UK, was recognized by Councillors, Leisure and Tourism Services of the Bournemouth Borough Council (BBC) and members of the public. When Leisure and Tourism began to pursue their goal “To develop a reef for recreational and commercial benefit in Boscombe”, BBC requested Dr Kerry Black of ASR Ltd (then Professor of Coastal Oceanography at the University of Waikato and NIWA) to undertake a feasibility study. The aim was to determine if any sites were suitable for an artificial reef and, if so, to consider the relative advantages and disadvantages of the various sites. An executive summary produced for the project is included as Appendix 1.

Black and Mead (2001) showed that offshore reefs can be used to protect the beach by breaking the waves offshore and thereby reducing the power of the waves to move sediment. In addition, they showed that reefs can be used also to rotate the waves onto preferred alignments. By rotating to make the wave crests more shore parallel, the strength of the longshore current can be reduced, and thereby reduce the losses of sediment. In the first report by ASR Ltd for the Bournemouth Borough Council (Black et al., 2000), a long-term strategy was proposed which would involve the use of a multiplicity of reefs, which would each rotate the waves and thereby negate the losses of sand to the east. While the cost would be high to do this at one time, the vision involved slowly developing the reef system by exchanging groyne replacement (once it became due) with substituted reefs. The net benefits over the long term of around 20-25 years would be substantial as the losses of sand and requirement for nourishment would be substantially reduced. The cost of this scheme was found to be no more or similar to the cost of groyne replacement, especially once full life cycle costing was done and consideration was taken of the sustainability of the current practice of using hardwoods in the groynes.

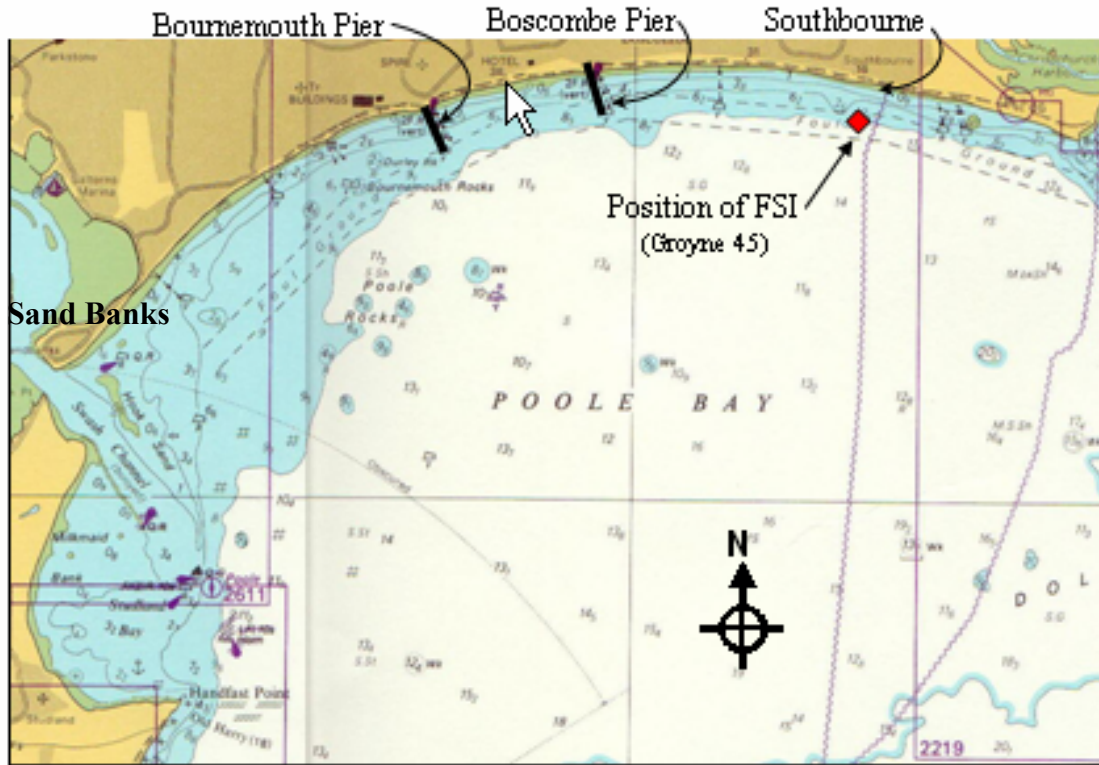


Figure 1.2 Location map, Poole Bay.



Figure 1.3 Boscombe Pier and the beach, Bournemouth.

More recently, the Poole Bay Strategy study was independently produced by Halcrow. It descriptively considered a range of options for the future management of the coastal defences in Poole Bay. The main conclusion reached was that the most attractive option, from the viewpoints of technical feasibility, economics and environmental acceptability, was to maintain adequate beach levels by periodic sand recharge schemes and to manage the subsequent evolution of the beaches by the installation of control structures such as groynes.

As part of that study, Halcrow carried out modelling of alternative types of groynes, i.e. rock and timber structures of various lengths and spacing, and concluded that longer rock groynes of about 150 m length and 300 m spacing were likely to be a good choice.

However, HR Wallingford in their review noted that this modelling concentrated on the central part of Poole Bay (in the vicinity of Bournemouth Pier) and their preferred groyne layout may not be optimal for the coastline further west, i.e. between Sandbanks and the Poole/ Bournemouth boundary, where beach slopes are flatter.

Recent coastal defence schemes along the Sandbanks frontage between the Haven Hotel and Shore Road have used shorter rock groynes than recommended by Halcrow, at correspondingly closer spacing. In HR Wallingford's assessment these have proved to be successful in creating a wider beach and appear to be popular with beach users, although some concerns about the safety of beach users and bathers have been raised by the RNLI and members of the public.

In addition, there is local interest in employing "reefs", i.e. low-crest submerged structures that may be submerged at some or all states of the tide, to control the evolution of the beach plan shape.

A further issue relevant to the optimisation of beach control structures relates to their positioning relative to the main beach access points, i.e. at Shore Road, Flag Head Chine and Branksome Chine. It may be preferable from the viewpoint of aesthetics and public safety to minimise the number of beach control structures, and position them carefully in relation to (i.e. away from) these access points.

Finally, any scheme for improving beaches along the Poole Borough frontage will need to be designed bearing in mind the present (and possible future) groyne system along the adjacent Bournemouth frontage. The long timber groynes just east of the Borough boundary may not need replacing for 20 years, while new defences along the Poole Borough frontage are planned in the next few years.

1.3 Aims

The specific aims of the overall study are:

- To review previous experience in the use of rock groynes, of emerged breakwaters and of reefs in controlling the evolution of sand beaches, in the UK and overseas;
- To assess the comparative costs of alternative beach control structures, of various dimensions, that would be appropriate for use along the coastline between Shore Road, Sandbanks and the Poole/ Bournemouth boundary;

- To review planning constraints on and opportunities for installing beach control structures along this frontage to reflect aesthetic and amenity concerns and objectives;
- To produce outline designs for 3-5 alternative coastal defence schemes, of similar cost, to control the evolution of a recharged sand beach;
- To recommend up to five beach management schemes for more detailed study, including numerical modelling of beach evolution.

In collaboration with other members of the team, ASR will suggest alternative layouts for beach control structures (whether reefs, detached breakwaters or groynes) that would be suitable for the western end of Poole Bay. Consideration will be given to the situation at the boundary between Bournemouth and Poole Boroughs, particularly with regard to the existing groynes on the eastern side of that boundary. If possible, the layouts should be similar to schemes already tried and tested elsewhere, although some degree of novelty is expected. There could be considerable savings in adopting novel forms of construction, particularly for reefs, and ASR will provide outline costs for building reefs with geotextile bags and rock.

Some comparisons of alternative layouts would be undertaken in terms of their efficiency, i.e. percentage of longshore drift retained, and their public safety implications. These are likely to be "value judgments" and so expert views on these aspects would be required. The aim would be to get a balanced view of the advantages and disadvantages of alternative structures based on experience wherever possible.

This initial study will also need to provide guidance on which options should be taken forward to a more detailed examination and how that second study should be carried out. In that stage, a numerical model of beach plan shape evolution (initially for the Poole Borough frontage) will be developed to predict beach plan-shape changes over a few decades. In addition, there would need to be some refinement in both the layout and costing of the options to get a better view on the costs of each option.

At the end of the first desktop study, it will be necessary to choose a sub-set of the numerous options considered for further and more detailed evaluation. Consequently, the initial desktop study needs to provide a broad view on all options, their advantages and disadvantages, based wherever possible on past experience. Decisions on which options to investigate further will be based on estimated costs, value judgements on the efficiency and public of schemes and on the willingness of stakeholders to develop the solutions.

2 An Assessment of Coastal Protection Options to Reduce Erosion

2.1 Introduction

Successful long-term coastal protection solutions seek to directly address and work with the natural physical processes responsible for the erosion problems, not just the effects of erosion. Thus, in order to select the most appropriate device(s) for a particular site both an understanding of the existing physical environment of the site (e.g. Sections 3 and 4 of this report address the coastal processes at Sandbanks and Poole Bay) and an understanding of the function and impact of various coastal protection options is required.

This Section considers the existing knowledge of the function and impacts of various coastal protection options that are presently used world-wide. In the present case, the options under consideration are groynes, submerged reefs and detached breakwaters in association with beach nourishment. Although these coastal protection methods are the focus of the report, seawalls and dune management options have also been described. It is noted that the coastal protection options described here do not include all available options; there are various other strategies such as beach drainage, beach scraping, cliff stabilisation, managed retreat, wave dissipation units, etc., that are not considered applicable to the present case.

Nowadays, two major factors are driving the increase in coastal protection options that are considered ‘environmentally-friendly’ in terms of both the natural and human environments. The first is the changing attitude of people the world over to the common amenity that the coast provides and the need to preserve and/or create these characteristics; wide sandy beaches, natural character, easy access, safety, and so on. These are all valued highly for recreational and aesthetic reasons (e.g. Bush *et al.*, 1996; Pilkey and Dixon, 1996; Komar, 1997). In addition, economic value of the beach amenity has become recognised as a valuable asset by local and regional governments (e.g. Raybould and Mules, 1998; Ove Arup and Partners International, 2001; Houston, 2002). Secondly, in the past few decades very large advances have been made in the field of coastal oceanography and littoral science, due to modern instruments for data collection and computers for data analysis and numerical modelling. Today there is a much clearer understanding of the way the coastal system works and the dynamics of the impacts that any particular coastal structure may cause, which has led to advances in the way coastal protection issues are addressed.

2.2 Seawalls

Seawalls or revetments are shore parallel structures that are put in place to impose a landward limit to coastal erosion and to provide protection to the area behind it. Seawalls are classic hard engineering structures that are put in place to protect the land and associated land-based amenities behind them. While these structures are usually termed “coastal” protection structures, ‘land protection’ is most certainly a better description than coastal protection, since they do not address the causes of erosion and in many cases accelerate erosion on their seaward side. In addition, these structures can be aesthetically unappealing and may hinder access to the beach (US Army Corps of Engineers, 1995; Black, 2000).

Seawalls are commonly built of many materials including timber, steel, concrete, rock, gabions, geotextiles, tyre mattresses, and specially designed armour units, and have a face that is either vertical, curved, stepped or sloping (US Army Corps of Engineers, 1995).

Seawalls need to cover the full length of the littoral cell within which they are located in order to prevent down-coast effects. That is, erosion around the ends of the wall can lead to collapse of the adjacent unprotected coast – this is commonly known as the ‘end effect’ (Fig. 2.1). In addition, isolated sections of seawall may exacerbate erosion on unprotected sections of a beach by denying sediment down coast during storms and by deflecting wave energy (Nielsen, 2001). For example, Dyer (1994) attributes the higher erosion in the central areas of the St Clair seawall (Dunedin, New Zealand) to the presence of the seawall causing reflected wave energy, which increases the rate of sediment loss from the beach.



Figure 2.1. The ‘end effect’ that has resulted in erosion at the eastern end of the St. Clair seawall (Dunedin, New Zealand).

An interesting concept is the use of ‘rip-rap’ (assorted sized rocks), rather than a smooth concrete face, to reduce the amount of reflected wave energy. The permeable rock wall breaks up the wave energy and adds turbulence so that the backwash is much reduced. This methodology is very useful in some cases, especially where the structure continues well below the low tide mark (e.g. a detached breakwater, port or marina walls). However, it is often wrongly assumed that the same effect of backwash/reflection reduction can be achieved at the top of a beach. While waves that break up on the structure itself will be dissipated, on a smaller scale, where the rock wall meets with the sandy beach, the erosion processes are accelerated by the presence of the rocks, and the structures are undermined. Thus, continual maintenance of the structures is required and the beach continues to be lost. Noosa Beach (Australia), Yeppoon Beach (Australia), Waihi Beach (New Zealand) and Orewa Beach (New Zealand) are good examples where rock walls on the beach have led to accelerated erosion and the need for increased nourishment (Raudkivi, 1980; Black et al., 2001; Piorewicz, 2002).

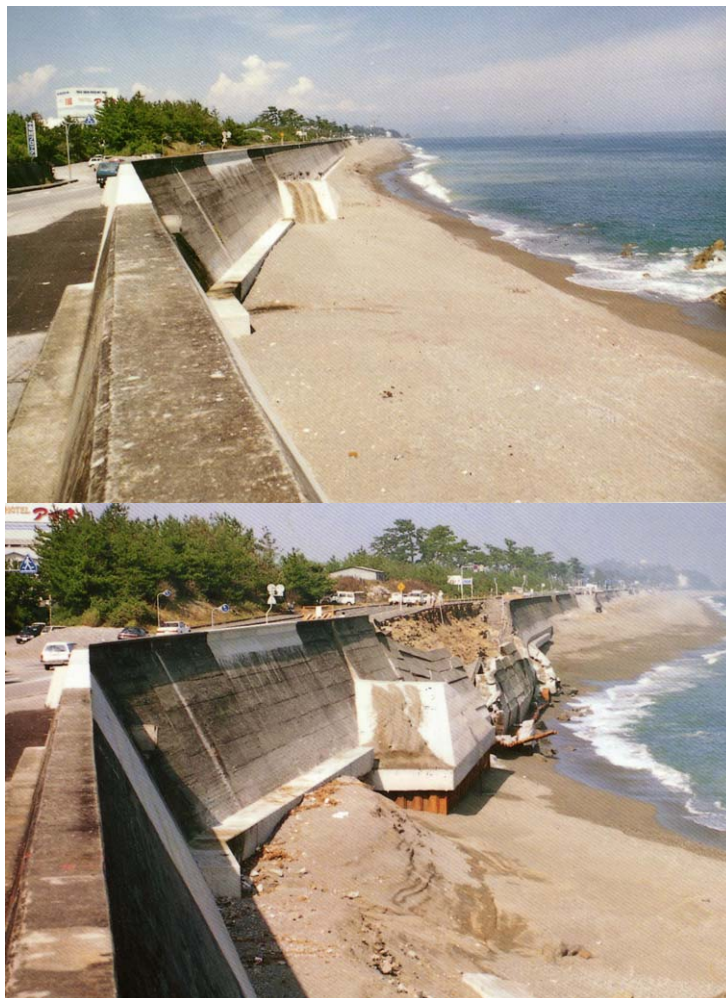


Figure 2.2. Failure of a seawall in Japan. The coastal processes causing erosion continue to operate on the shore and foundation failure results.

Seawalls are not a form of coastal protection, they are designed to protect the land and address the effects of the erosion and not the cause. Because the coast continues to ‘do its own thing’, in cases where erosion is occurring the seawall will inevitably fail – this process is well demonstrated in Figure 2.2. Seawalls have their place as coastal structures, but are more suited to low wave energy areas such as inside developed harbour cities where they can successfully protect valuable infrastructure (e.g. coastal roads) and the relatively low wave exposure does not result in the undermining of foundations.

Seawalls are not an option for beach erosion control on the exposed coast because:

- ***They are ‘land’ protection structures that do not prevent erosion of the beach:***
- ***They do not address the causes of erosion, indeed erosion may be exacerbated, and;***
- ***They have a negative impact on the beach amenity, access and aesthetics.***

2.3 Groynes and Artificial Headlands

Groynes and their massive relatives, artificial headlands, are coastal structures built of similar materials as seawalls, but oriented approximately shore-normal. They form a cross-shore barrier that traps sand that moves alongshore, thereby increasing the width of the beach on the upstream side. Thus, they function best on beaches with a predominant alongshore transport direction (Basco and Pope, 2004). In the present case, groynes are used to slow the predominant alongshore transport of nourishment material in order to lengthen the periods between recharge. However, as in many parts of the world, the timber groynes in Poole Bay have received negative criticism on the grounds of swimming safety, aesthetic impacts and reduced alongshore beach access (e.g. Short, 2003).

Groynes function in the same way that natural headlands block the sediment transport pathway and create beaches (Grigg, 2004). However, natural headlands represent the boundaries between littoral cells; when groynes are placed within a littoral cell the usual outcome is to ‘shift the problem down the coast’. Indeed, groynes form an artificial headland that results in erosion on the downcoast side of the structure, not unlike that shown in Figure 2.3. This effect is well known and has been documented world-wide (e.g. JCR, 2004). In some parts of the world, groyne-fields are used to counter the effect of the previous, up-drift groyne (e.g. as in Poole Bay - Fig. 2.4).

For groynes to be effective there must be a longshore supply of sand that is trapped on the up-drift side of the groyne and accretes. This reduces sand supply down-drift of the groyne and erosion can result. Down-drift erosion can be reduced by filling the groyne embayment under a beach nourishment programme, and dune management measures can

be used both up-drift and down-drift to accommodate changes in the beach and dune systems (Nielsen, 2001). Special Issue 33 of the *Journal of Coastal Research* (2004) presents an up-to-date assessment of the effectiveness of groynes, design aspects and case studies. Chronic erosion and undesirable distorted shorelines are the result of many groyne placements (Hanson and Kraus, 2004), and many methods have been applied (some successfully) to reduce these impacts. Basco and Pope (2004) outline the general principles and functional properties of groynes, which include the necessity to use groynes throughout the entire littoral cell in order to negate down coast impacts, the use of tapering of the vertical sides of groynes to increase sediment exchange between groyne cells, the need to artificially ‘fill’ groyne compartments with sand, and the ‘tapering’ of the length of successive groynes to reduce impacts to the downcoast beach if the entire littoral cell is not to be protected (as has been applied at Sandbanks – discussed below).

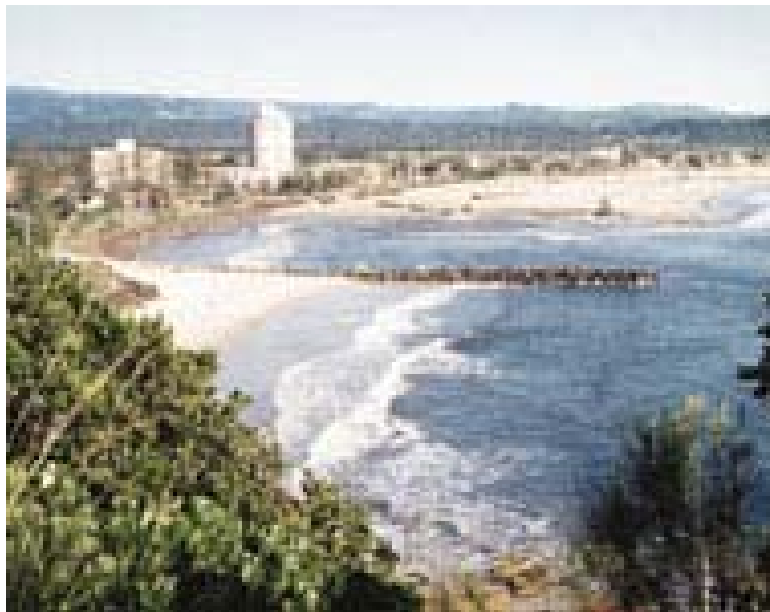


Figure 2.3. Down-drift erosion (upper side of the groyne) caused by a groyne forming a ‘blockade’ against alongshore sediment transport.



Figure 2.4. The groyne field in Poole Bay, Southbourne, England.

Groynes do not reduce the cross-shore movement of sand during storms. Indeed, offshore directed wave-driven currents adjacent to groynes can lead to accelerated cross-shore loss of beach sand (Johnson and Hansen, 2003) (Fig. 2.5). This negative effect is likely to increase with increasing groyne length (Basco and Pope, 2004), as has been proposed by Halcrow for Poole Bay. Therefore, groynes are not effective as a means of managing short-term erosion in the form of cross-shore sediment transport, other than their effect in building up a sand buffer. This is a concern in the present case.

Groynes are most effective when there is a predominant alongshore transport direction and when used in conjunction with beach nourishment (as at Sandbanks and in Poole Bay).

Downcoast impacts can be reduced by continuous groyne-fields within a littoral cell or decreasing consecutive groyne length, 'tapering' (as at Sandbanks and in Poole Bay).

However, groynes

- ***Can impact negatively on swimming safety, aesthetic and alongshore beach access, and;***
- ***Are ineffective at preventing cross-shore sediment transport and can increase the loss of beach sand through the creation of strong offshore-directed rip currents if they are too long for the particular site.***

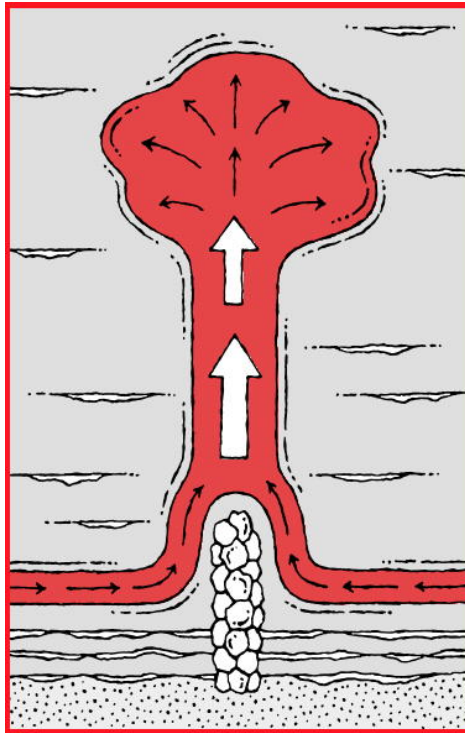


Figure 2.5. A simple schematic showing how groynes can generate offshore directed rip currents, as described by Johnson and Hanson (2003) and Basco and Pope (2004). Source, Sea Grant 2001.

2.4 *Dune Management*

During storms, sand is removed from the beach to form an offshore bar and, during intense storms, a demand for dune sand is created which involves the sand buffer reserves held in the dunes. The maintenance of sufficient sand storage relies on effective dune management, which is based on maintenance of a satisfactory primary, secondary and tertiary vegetative cover (Chapman, 1989). This requires the introduction of appropriate plant species and defined access/walkways over the dunes to maintain their health.

Beach vegetation stabilises the sand that returns within a few weeks of the return of fine weather following storms. Secondary species have an important function of stabilising seaward of the permanent dune crest, and permanent tertiary species on and behind the crest of the foredune that are slow growing, provide shelter to the secondary species and, most importantly, act to modify locally the onshore wind field favourably for aeolian sand deposition onto the dune field (Nielsen, 2001).

Dune restoration and management has been very successful in many parts of the world (e.g. New Zealand, Australia, South Africa) and in many areas the local government or groups of local residents regularly undertake planting of large area (WCC, 1994; Black and Mead, 2002; Burkinshaw and Illenberger, 2001; Savioli et al., 2002). These areas

provide a 'buffer zone' that stores the sand so that during storm events it is available to naturally protect the coast.

It is interesting to note that in some areas of the world (e.g. parts of the west coast of France; the Cape St Francis coast in South Africa (Burkinshaw and Illenberger, 2001)) are experiencing down-coast erosion (much the same as that described for groynes below) due to the stabilisation of very large areas of dune – the sediment that would normally move along the coast and maintain wide sandy beaches is 'tied-up' in the dune system.

Dune management is very successful where a wide dune area is or was previously present and there is an abundant source of sediment. Where only a small width of beach is present, dune management can be applied in conjunction with other coastal protection measures that result in the widening of the beach. However, existing coastal processes, landscape and vista aspects need to be considered before dune management is applied.

2.5 Beach Nourishment

Where there is insufficient sediment on a beach to meet storm erosion or long-term sediment loss, additional sediment can be placed by mechanical means, as is presently done in Poole Bay and in many other parts of the United Kingdom. Indeed, beach nourishment is a common coastal protection method world-wide (e.g. Australia – Jackson & Tomlinson, 1990; New Zealand – Mead et al., 2001; USA - Bush *et al.*, 1996; Bruun, 2000; Houston, 2002). Beach nourishment involves the artificial placement of sand onto a beach to replace sand that has been lost through erosion. The nourishment forms a 'buffer' against storm erosion, with periodic topping up to satisfy the natural erosion processes. To be successful, nourishment is best undertaken over the whole beach profile including the part of the beach below the water out to depths where storm waves are breaking, not only the beach above the low-water mark. This approach of profile nourishment has been used successfully on the Gold Coast for many years (Jackson & Tomlinson, 1990).

As well, the reef built at Narrowneck was constructed with coincident beach nourishment (Fig. 2.10). The nourishment extended for several kilometres along the Surfers Paradise beaches and was planned as part of the project. In several reef projects, ASR Ltd has recommended that nourishment be done in conjunction with the reef construction to eliminate short-term impacts on adjacent locations when the salient is forming. This is not always needed, particularly in regions with adequate cross-shore or longshore transport. It is often ASR's recommendation that nourishment should accompany most coastal construction projects, whether it be reefs, groynes, breakwaters or rock walls.

Beach nourishment is a favoured means of beach protection for resort and high amenity beaches because it promotes amenity and natural character in the form of a wide sandy

beach and, unlike some other structural measures, generally it does not have adverse effects on adjacent areas of the coastline (Savioli et al., 2002). Provided sufficient sand is used, beach nourishment can provide total protection. However, it may be an expensive means of control. To prevent excessive offshore losses of the placed material, the nourishment sand should be similar in size or, preferably, slightly coarser than the natural beach material (Nielsen, 2001). However, in recent years use of coarser beach materials has become a point of controversy, since in many cases this will lead to steeper beach profiles and consequently ‘harder’ breaking waves. Many consider this a safety issue and a degradation of public amenity.

These days, beach nourishment is most often undertaken in conjunction with some form of sand retention device such as groynes (as presently used in Poole Bay), submerged reefs (e.g. Narrowneck on the Gold Coast in Australia), or detached breakwaters (e.g. East Anglia, England). This combination of coastal protection methodologies is undertaken to lower costs of nourishment (since the material remains in place for a greater period of time) and address the restraints of sufficient source materials and sustainability.

Nourishment has provided a successful option for erosion protection in Poole Bay because:

- ***It works within the natural processes and provides a buffer zone in the form of beach sand;***
- ***It has a positive impact on the beach amenity and aesthetics in the form of a wide sandy beach.***
- ***Sand retention devices are used to increase the residence time of the nourishment material***

However, the present practise of using groynes to slow the loss of nourishment material has been questioned with respect to negative impacts on beach amenity and aesthetics, as well as beach safety (e.g. Short, 2003).

2.6 Offshore Submerged Reefs

Offshore, or multi-purpose, submerged reefs are a relatively new coastal zone management concept that is being very well received by the public (Nielsen, 2001; Pilarczyk, 2003). The concept attempts to blend the socio-economics with the coastal protection imperatives, thereby offering the opportunity for coastal engineers to act on the community’s requests and aspirations (Black, 2000; Pilarczyk, 2003). While the multi-purpose aspects are relatively new, offshore reefs (also known as submerged breakwaters and artificial reefs) have been successfully and commonly applied world-wide (Adams and Sonu, 1986; Pilarczyk, 1990; Pilarczyk and Zeidler, 1996; Smith et al.,

2001; Van der Meer and Pilarczyk, 1998; Harris, 2002; Pilarczyk, 2003) and are particularly popular in Japan (Fig. 2.6). In Figure 2.6, the reefs are close together and can be considered as a single reef, which is confirmed by the presence of the one large salient with only small cusps. Larger gap/reef ratios are more cost effective, although the small gaps in the “single” reef of Figure 2.6 have some benefits for reducing volume and cost. Figure 2.6 confirms the benefit of the reefs, by breaking the waves offshore and eliminating the power of the waves at the shoreline.

There are many examples of coastal structures in the form of natural reefs and islands that provide protection to the shoreline in their lee. Andrews (1996) identified several hundred such cases around the New Zealand and New South Wales, Australia coastlines. Figure 2.7 is a classic example of a protective reef that has created a natural widening of the beach in the form of a salient. It is an impressive example of the effects of a submerged reef. Site inspections by Dr Black and knowledge of the geology have shown that the salient is formed by the offshore reef, not by underlying geology. As can be seen from Figure 2.7, the hinterland is composed of sandy sediments on a barrier spit, very similar to the situation at Poole.



Figure 2.6. Submerged reefs for coastal protection in Japan. In this case the spacing between the reefs is very close, and so rather than the usual cusped formations in the lee of each reef (e.g. Black and Andrews, 2001a), they are working as a single structure (i.e. the salient features have ‘welded’ into a single feature).



Figure 2.7. A salient formation in the lee of a natural submerged reef.

There are many examples of submerged reefs for coastal protection worldwide, although the majority are not multi-purpose and do not incorporate rotation functions – these structures are mostly designed for wave attenuation. Even so, these existing structures give a good indication of reef effectiveness and wave transmission for different depths of water above the crest, which is also relevant to tidal range. Anecdotal evidence suggests that submerged reefs are more effective in areas with small tidal ranges. When tidal ranges at sites with existing successful submerged reefs are considered, it is found that many have significantly greater tidal ranges than that at Poole (2.1 m). For example, at Santa Monica in California, a successful submerged breakwater is located in an area with a 3.4 m tidal range (Adams and Sonu, 1986).

Submerged reefs function through wave dissipation and wave rotation, which leads to salient growth in the lee of a reef. Wave energy is dissipated on the reef resulting in less energy at the beach in the lee of the reef and the consequent deposition of sediment. Wave rotation is a novel approach to coastal protection and is well described by Pilarczyk (2003):

“It is also worth noting that Black and Mead (2001) have introduced a new concept of coastal protection by applying wave rotation due to the presence of submerged structures. Wave rotation targets the cause of the erosion, i.e. longshore wave-driven currents. Offshore structures are oriented to rotate waves so that the longshore current (and sediment transport) is reduced inshore. The realigned wave angle at the breaking point (in harmony with the alignment of the

beach) results in reduced longshore flows and sediment accretion in the lee of the rotating reef.”

Thus, where oblique incident waves are the main cause of erosion, as in much of Poole Bay (Black et al., 2000), wave rotation can play a significant role in the functional aspects of submerged reefs. This is achieved with or without waves breaking on the reef, and so is not reliant on wave transmission (e.g. Ahrens, 1984).

Recreational and public amenity can be incorporated through surfing, diving, sheltered swimming, water games, fishing and/or marine habitat. The inclusion of amenity, however, requires the amalgamation of different purposes in the reef design and, consequently, can make the design more difficult than that which may be required for coastal protection only. Even if only coastal protection is required from an offshore reef, it is important to thoroughly understand the local physical processes in order to optimise placement. Indeed, poorly designed and positioned submerged reefs can accelerate erosion if placed too close to the shore by ‘compressing’ the surf zone and increasing alongshore currents.

Multi-purpose, submerged reefs are increasingly being selected as coastal protection structures because of the many advantages associated with them when compared with other coastal protection devices. Submerged reefs can provide shoreline protection with low environmental impact. Because the structure is underwater, the visual amenity is not impaired. Furthermore, submerged offshore reefs can lead to eliminating the need for rock works on beaches. A submerged reef may:

- unify coastal protection and amenity benefits into a single structure placed offshore;
- enhance the coastal amenity value by incorporating multiple use options of surfing, diving, marine habitat, water games and sheltered swimming, and;
- preserve or enhance the beach amenity.

The depth of the reef, its size and its position relative to the shoreline determine the level of coastal protection that may be provided by the reef (e.g. Black and Andrews, 2001a, b; Black, 2003). This ability to vary the protection level as part of the reef design is a feature of the offshore reefs and allows far greater flexibility in their design function compared to other types of coastal protection device.

It is well accepted (e.g. A. Brampton, pers. comm.) that a submerged reef will reduce wave heights in its lee, and cause a re-orientation (or “rotation”) of the beach contours and waves for some distance up and down the coast on either side of the sheltered area. This must, at least initially, lead to deposition of sand in the area affected by the reef. Similarly, the many salients behind natural reefs and islands show that this feature is pseudo-stable and permanent, while acknowledging that the salient will partially cut and fill as the waves go through their storm/swell cycles. As noted above, the sand in the

salient is taken from the system, can be detrimental in some circumstances. Consequently, we normally recommend that initial beach recharge be undertaken after reef construction to provide the salient material. On the Gold Coast Reef, the response of the beach has been the same as that calculated and modelled (Black, 1998; 1999) according to four years of monitoring of the reef (Jackson *et al.*, 2005) and no downcoast impacts have been observed by the monitoring (Jackson *et al.*, 2005; Turner *et al.*, 2001).

Since submerged reefs are placed a sufficient distance offshore to ensure that tombolo formation does not occur and form a ‘groyne-like’ blockage, sand can pass between the reef and the shoreline. As such, once the equilibrium salient is formed conservation of volume (as the volumes remain unchanged on average) dictates that no more material is being captured by the salient in the long term, and so there can be no reduction of material reaching downdrift beaches. Our studies suggest that this equilibrium takes about 6-12 months to occur and the short-term impacts can be eliminated by initial recharge. Another way of explaining this phenomenon is that the drift rate across a line drawn perpendicular to the shoreline through the centre of the reef is reduced temporarily, not permanently. The only exception is when the reef is poorly designed and a tombolo forms. The sand tombolo acts like a groyne which causes a permanent deflection of sand offshore and a change to the longshore drift. Thus, we recommend that reefs be developed to form salients, rather than tombolos so that the beach widening occurs, but the obstruction to longshore drift does not. There are some exceptions when a tombolo is beneficial, such as cases requiring an artificial headland. In some cases, the sand tombolo may be a better method of protection than a groyne.

If we compare a groyne field with a reef field, the following observations are pertinent. It is recognized that a short groyne field will re-orient the beach and, like reefs with salients, the drift rate eventually reduces as the groyne compartments take up a stable plan shape. Indeed, the groyne fields in Poole Bay are an example of groynes providing some measurable reduction in longshore drift. The distinction between groynes and reefs, however, is that the groynes are immovable. The sand salient, on the other hand, can adjust and move in accordance with the wave conditions, and so the solution is much “softer” and more natural. The obstruction by the groyne must lead to deflection of the currents offshore along the side of the groynes, which carries sand into deeper water. This problem has led engineers to shorten the groynes and thereby place the tip of the groyne in shallower water. The shorter length, however, makes it necessary to reduce the groyne spacing in order to realign the beach to its “equilibrium” orientation.

Groynes can also act as a “one-way valve”. That is, when the waves are coming from a different direction, the groyne prevents sand movement upstream, relative to the average net drift. With its soft salient, the reef does not have this weakness, and so the movement of the sand up and down the beach occurs naturally with reefs. The one-way valve effect is well recognized at Bournemouth, where eventually all sand in the compartments is lost to the east due to beach erosion. On the contrary, the reefs could be designed to simply adjust the wave orientations and intensities at the shoreline and breakpoint until the net drift was close to zero.

Thus, offshore reefs may be used when:

- a widening of the beach is required which can adjust naturally around its equilibrium position during the storm/swell cycles;
- hard construction on the beach is not wanted or may not be suitable;
- the natural character of the beach is to be preserved or reinstated, and/or;
- a safer and improved recreational and environmental amenity value is required.

A good example of a multi-purpose reef is the Narrowneck reef on the Gold Coast in Queensland, Australia. The Narrowneck case is applicable to the present location since there is also an on-going beach nourishment scheme in place for the Gold Coast, and although the wave climate is smaller in Poole Bay than at Narrowneck, the tidal range at Poole is very similar (i.e. 2.1 m). The Gold Coast is Australia's primary tourist destination, with the wide sandy beaches being a major attraction. The erosion problem on the Gold Coast was confined to a hotspot at Narrowneck, where only the coastal road separates the Broadwater from the sea. This causeway was breached several times in the previous century and coastal protection was proposed as part of the Gold Coast Beach Protection Strategy to address this problem. The Gold Coast has a predominant south easterly swell direction, which results in large net sediment transport ($\sim 500,000 \text{ m}^3/\text{yr}$ to the north), although reversals of sediment transport direction occur frequently.

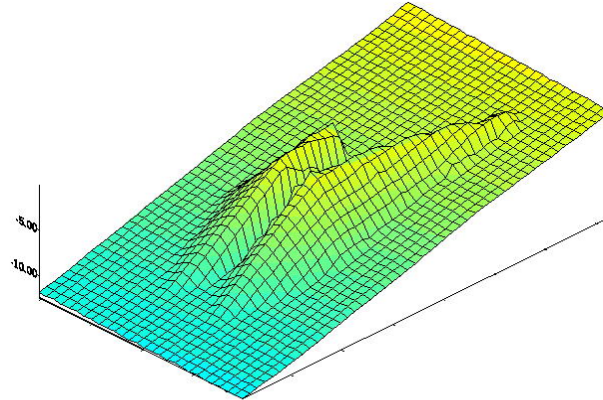
Traditional coastal protection methods were considered (e.g. groynes, tipped rock walls, etc.), however, a socio-economic assessment found that for every dollar spent on enhancing the beach, \$60-80 was returned via tourism using the reef (Raybould and Mules, 1997). Consequently, an offshore submerged reef was proposed and design works were undertaken by ASR consultants (Black *et al.*, 1998; Black, 1998; Hutt *et al.*, 1998). Many other multi-purpose reefs world-wide are also being driven by socio-economic factors (e.g. Black *et al.*, 2000; Black *et al.*, 2002; Black *et al.*, 2004a, b; Mead, 2004; Mead *et al.*, 2004a, b).

The aims of the project were:

- to protect and widen the beach and dunes along Surfers Paradise Esplanade.
- to improve the surfing climate at Narrowneck.

A comprehensive field programme was undertaken, with the results being utilised for reef design and sediment transport modelling (i.e. to assess the functional performance of the reef). The resulting final design was a $120,000 \text{ m}^3$ submerged reef (Fig. 2.8 and 2.9) – reef crest level is $\sim 0.5 \text{ m}$ below LAT. The main purpose of the Narrowneck reef is to retain sand nourishment material that was pumped onto the beach from the Broadwater. Figure 2.10 demonstrates how successful the Narrowneck submerged reef has been at retaining nourishment material on Surfer's Paradise Beach. Argus coastal imaging has

shown that wave energy is dissipated by the reef for up to 90% of the time and that Narrowneck reef is an erosion control point on the coast (Turner *et al.*, 2001).



Narrowneck Reef Design

Figure 2.8. 3-dimensional representation of the Narrowneck multi-purpose reef.



Figure 2.9. The view of Surfer's Paradise with the multi-purpose reef in the foreground.

The Gold Coast reef has been a huge success, not only in terms of coastal protection, but also providing a surfing facility (recent reports describe the reef as the 'best surfing spot on the coast') and a 'natural' reef ecosystem that supports a dive trail (Fig. 1.12). An important outcome of the project was the confirmation (via beach profile monitoring and Argus coastal imaging) of no downdrift impacts on the coast. In 2000, the Narrowneck reef project won the prestigious Queensland State environmental award. Recent re-assessment of the economic impacts of the reef has confirmed a benefit:cost ratio of 70:1 (McGrath, 2002). In addition, recently released statistics from the Gold Coast Surf Lifesaving Council show that there are some 67% less rescues in the vicinity of the reef in comparison to the rest of the coast (Jackson *et al.*, 2005).

One disadvantage of a reef is that it occupies space offshore which the fisherman may initially raise a dispute over the seabed access rights. They may prefer to see the

construction on the land, particularly trawlers. With one exception, our experience has been that this objection is quickly eliminated once the fishermen are able to view movies of the improved marine ecology that occurs on the reefs, as this improves their fish stocks.

There is no hazard to navigation (even if some mariners claim the reef to be an inconvenience), unless the reef is to be placed in a shipping channel, which is not the case at Poole. It is necessary, of course, to warn the boat owners about the presence of the reef, through notices to mariners, changes to the marine charts, advertising and possibly markers lit at night. However, reefs are common along the British coast and the British mariner is fully aware of how to avoid them by steering away.

In relation to rip currents, changes to currents are mostly constrained to the reef, which is placed well beyond the normal surf zone – this is verified by many recent modelling studies (e.g. Mead *et al.*, 2004). As such and assuming the reef is placed well offshore, rip currents at the beach are not strong, due also to the reduced wave heights inshore. In addition, the majority of local current changes result in currents directed inshore (due to waves breaking over and along the submerged reef), rather than offshore as can occur with groynes (Basco and Pope, 2004).

Submerged reefs are a feasible option for long-term erosion protection Poole Bay because:

- ***They can be designed to address the critical causes of erosion by locally neutralising longshore currents (either through dissipation of wave energy, wave rotation or a combination of both) without downcoast impacts;***
- ***Their offshore location reduces cross-shore sediment losses;***
- ***There are no, or low, negative aesthetic, access or safety impacts, and;***
- ***Amenity value can also be enhanced.***

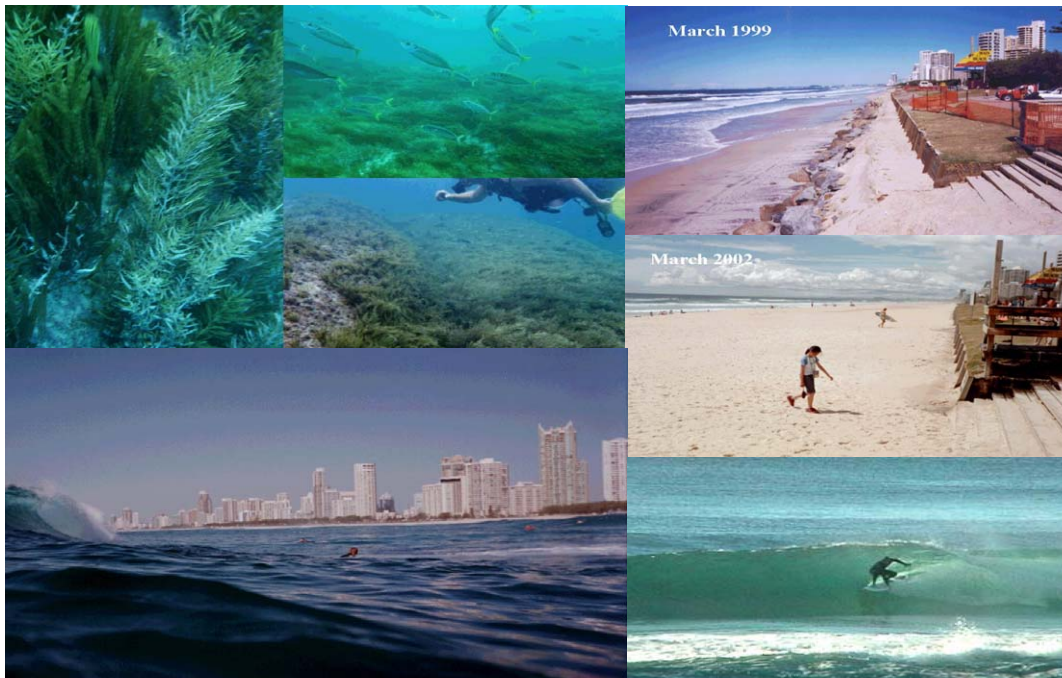


Figure 2.10. The Narrowneck multi-purpose reef. Clockwise from top left, colonization of the reef has resulted in a dive-trail; before and after reef construction (construction commenced in August 1999); surfing on the reef; the view from the surf.

2.7 *Detached Breakwaters*

Detached breakwaters are oriented approximately parallel to the beach but, unlike a seawall, are placed some distance offshore. They protrude above water level and can be continuous or consist of a series of segments and can be built of similar materials as the previously mentioned coastal protection works. While these kinds of structures have not been used for coastal protection in New Zealand, they have been used extensively for a number of years in Japan, the USA, Singapore and in Europe (Fig. 2.11). Overseas, detached breakwaters have been used typically along coastlines with small tidal fluctuations to control the cross-shore sand transport processes (Nielsen, 2001).



Figure 2.11. An example of a detached breakwater project in East Anglia, Great Britain. Note the formation of tombolos in the lee of the breakwaters. The sand was placed as nourishment, not trapped from the natural system.

In general, detached breakwaters are suited to coastlines that require higher levels of protection at sensitive points and do not have sensitive down-drift beaches, as detached breakwaters may cause down-drift erosion on a beach where there is a strong net transport of littoral drift. Indeed, detached breakwaters are generally used in situations where there is no alongshore net sediment transport (NSWG, 1990).

Detached breakwaters provide protection by reducing the energy in their lee and thereby, reducing the wave-driven currents that cause coastal erosion. Material is deposited in the zone of reduced wave energy forming a salient (a bulge in the coast) or tombolo (a connection between the coast and the offshore obstacle). However, like a groyne, a tombolo will block alongshore sediment transport and cause erosion on the down-drift side of the structure. Eventually, sediment transport may resume along the seaward face of the detached breakwater, but often the water is too deep for large amounts of transport to occur. This situation may be desirable if the intention is to create a pocket beach or to reclaim part of the foreshore. However, it requires a large and continuous sediment supply as well as accompanying renourishment on the down-drift side until equilibrium has been reached. In the case shown in Figure 2.11 with large tombolos, negative downdrift impacts beyond the breakwater field were severe and required emergency works. In the background of Figure 2.11, lower breakwaters have salients in their lee. This may relate to the lowered crest, but because of the large tombolos upstream, the salients may be the result of sand starvation from up-coast. It is always our opinion that

structures should only be built once the physical system is well understood, otherwise the structure is just a “full-scale experiment”, which the community must pay for. It should be always acknowledged that the system is permanently altered by any hard structures and the sand formations in their lee. In most cases, a tombolo is an indication of over-design errors by the team responsible for the structure.

A salient allows sediment to pass between the offshore obstacle and the coast, and so the ‘one-way valve’ effect doesn’t occur and sediment can move in both directions without causing long-term depletion on the down-drift side. The beach is protected in the lee without totally blocking sediment movement. It is therefore critical to place the detached breakwater of the correct length at the correct distance offshore to achieve a salient and not a tombolo, which is closely coupled with an understanding of the coastal dynamics of the area.

Well-designed detached breakwaters could provide a long-term coastal protection option at Poole Bay. However, such structures lack the sophistication of similar devices (e.g. submerged reefs) in terms of both coastal processes and amenity enhancement. For example, detached breakwaters do not function to rotate waves to an alignment closer to shore parallel to mitigate alongshore currents and nor do they provide as much opportunity for subtle adjustment of the structure’s geometry and position to achieve design criteria and added amenity. Waves mainly break on the emerged structure. In relation to rotation, the only form of rotation occurs by incidental diffraction or refraction around the breakwater at each end which is only a very small fraction of the wave crest that impacts on the structure. A substantial rotation of the full wave crest can be achieved if the whole structure is underwater, and waves will still break on the crest during storm events in the small tidal range at Poole.



Figure 2.12. A view from the shore of detached breakwaters on the East Anglia, Great Britain (shown in aerial view in the previous Figure).

In addition, the protrusion of detached breakwaters above sea level reduces the natural character of the coast and from the perspective of persons on the beach replaces the feeling of the open coast with that of being enclosed (e.g. Fig. 2.12).

Detached breakwaters are a possible option for Poole Bay erosion control, however,

- *They would not well address the primary cause of erosion (i.e. alongshore sediment transport), since these structures are better suited to areas with little or no alongshore sediment transport;*
- *Similar, but more sophisticated offshore structures (i.e. submerged reefs) can provide a better solution (e.g. incorporate wave rotation, do not negatively impact on aesthetics, etc.);*
- *Detached breakwaters protrude above water level in the surf zone and so are prone to damage and hence maintenance, and are also more difficult to construct than land-based or submerged solutions, both of which makes them expensive to construct;*
- *Because they protrude above the water level, they negatively impact on the aesthetics of the beach, and;*
- *Very little additional amenity is provided by the structure.*

2.9 Summary

Successful long-term coastal protection solutions seek to directly address and work with the natural physical processes responsible for the erosion problems, not just the effects of erosion. At Poole Bay, beach nourishment needs to be undertaken in conjunction with another form of sand retention device in order to reduce sediment loss by alongshore sediment transport. This review of existing knowledge has examined the function and impacts of various coastal protection options that are presently used world-wide, i.e. groyne, detached breakwaters, “reefs” or a combination of these are the most suitable devices to reduce alongshore sediment transport at Poole.

The options are summarised in Table 2.1. Of these devices, detached breakwaters are the least suited since they are most effective in areas with little or no alongshore currents. Reefs have several advantages with respect to offshore options (e.g. can incorporate rotation, far less visually obtrusive, etc.). Groyne and submerged reefs are better suited to the Poole Bay situation, with reefs having the advantages of reducing cross-shore sediment losses due to their offshore location, lower negative aesthetic, access or safety impacts, and the potential to incorporate amenity value.

Consideration of the most appropriate options in the present case must also consider the local coastal processes and construction costs. These are addressed in following sections.

Table 2.1. Summary of coastal protection devices for Sandbanks/Poole. Note that groynes, reefs and detached breakwaters would all be used in conjunction with nourishment. The problem of swimmer safety around groynes was reported by RNLI (Short, 2001) and anecdotally in Poole Bay. While some safety problems should be expected due to currents that are known to flow offshore against groynes in wave dominated areas, no detailed worldwide study of groyne dangers for swimmers has been compiled to our knowledge.

	Groynes	Submerged Reefs	Detached Breakwaters	Nourishment
Effectiveness	Most effective when there is a predominant alongshore transport direction (wave-driven or tidally driven). Can result in cross-shore loss of sand if too long due to the jetting of sediment offshore	Most effective in areas where erosion is driven by waves. Can incorporate both wave dissipation and wave rotation principles to retain sediment on the beach.	Most effective in areas with low alongshore sediment transport, unless well-designed to ensure no tombolo formation. Good dissipation, but cannot incorporate wave rotation aspects for sediment retention.	Most effective in areas of low alongshore sediment transport, unless used in conjunction with control measures (e.g. submerged reefs, groynes or detached breakwaters). Sustainable issues related to source
Aesthetics	Immediate intrusion on beach aesthetics and natural character. Can block alongshore beach access.	Very low aesthetic impacts, since always covered by water.	Exposed crest reduces the natural character of the coast and from the perspective of persons on the beach replaces the feeling of the open coast with that of being enclosed	Positive aesthetic impacts, as long as similar colour and grain size is used.
Public Safety	Creates strong offshore directed wave driven circulation currents adjacent to groynes due to compartmentalization. Prevalent for longer groynes at Bournemouth, and further studies are recommended.	Increased public safety, lower waves and currents at the beach (e.g. 67% less rescues in the vicinity of the Gold Coast reef (Jackson et al, 2005). Any changes to currents are confined to the immediate area of the reef (e.g. Mead et al, 2004) and currents are directed inshore, rather than offshore as can occur with groynes.	Increased public safety at the beach (as long as no tombolo formation). Potential safety problems on the structure, since protrudes above water level	No negative impacts to public safety unless courser grain size is used (can lead to stronger plunging waves)

3 Recommendations for Sandbanks, Poole

3.1 Introduction

Field site inspections were conducted by Dr Black in October, 2004 with HR Wallingford and Poole Borough Council staff. In combination with the previous studies, several conclusions about the circulation and beach dynamics were made by Dr Black, as described below.

In addition, the previous investigations of the Bay by ASR Ltd and others showed that the wave approach directions relative to the shoreline orientation in Poole Bay cause the wave heights to increase from Poole to Southbourne because of the protection afforded by the headlands to the west (Handfast Point, Peveril Point and Durlston Head) (Fig. 3.1). Consequently, the bay has developed a spiral shape (Fig. 3.1) and it is believed that the net longshore transport is to the east and increases towards Southbourne (Harlow, 2000).

In the Bournemouth Borough, cross-shore beach profiles show that beach volumes tend to gradually and systematically diminish at decadal time scales - to the limit of the surveys at 450 m offshore of the sea wall (Harlow, 2000). The dropping beach levels in the context of the net longshore transport means that sand leaked from between the groynes is lost from the beach system, with loss rates that vary from about 80,000 m³yr⁻¹ at the west end of the borough to about 110,000 m³yr⁻¹ at the east (Harlow, 2000). As such, nourishment has been required approximately every 15-20 years. While wave energy is smaller, Poole beaches also require management to maintain a healthy and stable beach.

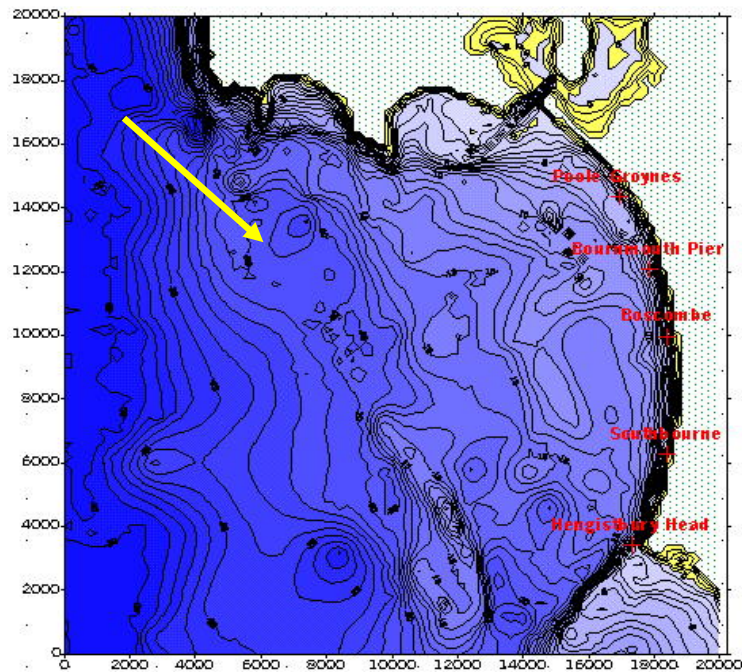


Figure 3.1a. Bathymetry of the Bay and dominant wave approach direction.

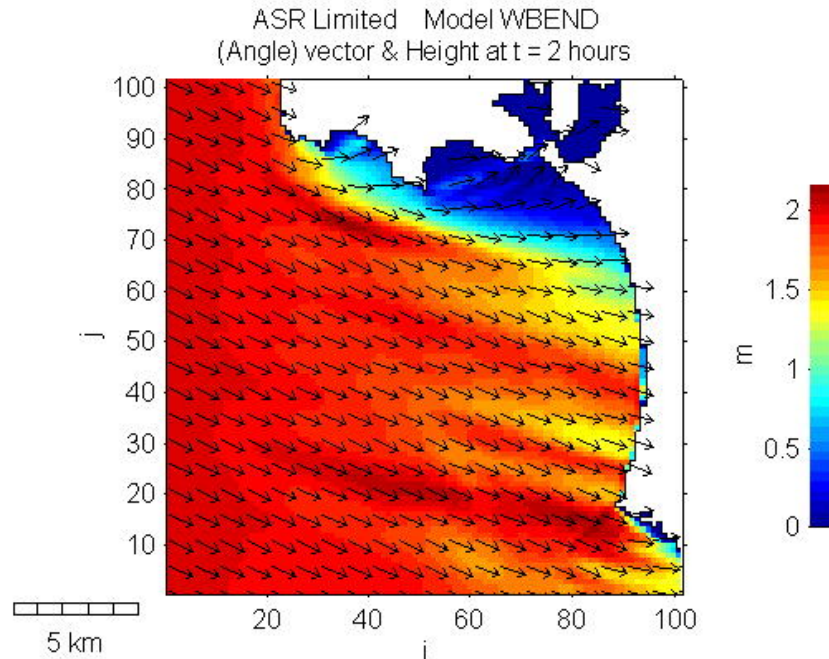


Figure 3.1b. Wave refraction modelling using model WBEND from the 3DD Suite (from Black et al., 2000). Wave heights increase to the east.

3.2 *The Beach System*

The physical dynamics at Sandbanks is composed of three dominant elements:

1. tidal circulation through the entrance and around the ebb-tidal delta;
2. wave-driven longshore transport, and;
3. wind-driven longshore transport.

The ebb-tidal delta (Fig. 3.2) is a large sub-tidal feature that extends to seaward along both sides of the main channel from the entrance. Figure 3.2 shows that the ebb-delta essentially diffuses in the north-east direction around Poole Head, where the isobaths running cross-shore around the delta, join onto the isobaths running longshore up the beach. Black et al. (1989) and Black (1987) studied these types of features in detail, and there are many international scientific studies in the literature, which allow the circulation and dynamics of the region to be inferred.

It can be inferred from the bathymetry that the net currents are offshore (ebb directed) in the deep entrance channel through Poole Harbour (Fig. 3.2). (Net tidal circulation is the long-term averaged movement of water.) A compensating return current (flood directed) flows along the east side of the delta off Sandbanks. The loop is completed by net flood-directed tidal flows through the channel off the beach at Sandbanks. Thus, we can expect

stronger flood than ebb flows through that channel, which would bring sand back towards the entrance, on average. The presence of this flow is supported by the growth of the spit from Poole Head to the south-west. These long-term dynamical patterns are summarised in Figure 3.2.

On the crest of the delta to the north of the main channel, the deposition has occurred in the neutral zone, which must occur where the strong ebb and flood circulation on either side of the bank is neutralised. Overall, the sandy sediments are therefore predicted to be slowly migrating around the bank in an anti-clockwise loop.

Superimposed on the tidal circulation is the wave driven flow. Wave currents at the beach are dependent on wave height and the angle of the waves to the shore. It is known that most of the larger swell waves approach from the English Channel from the south-west (Black et al., 2000). Thus, it can be expected that the net wave-driven currents are towards the north and east i.e. along the beach towards Bournemouth. This is relatively well accepted at the site, and was shown to occur with models and measurements at Boscombe by Black et al. (2000; 2004a). Around Sandbanks, when wave and tidal currents are combined, the net movement of sand at the beach may be quite small as these two forces are opposed. However, the tidal currents are expected to be strongest offshore in the channel, while the wave currents are strongest in the surf zone. This could mean that sand on the beach could be moving towards Bournemouth, while sand in the channels offshore is moving towards the Poole Harbour entrance.

The wind-driven flows are expected to be towards the east on average, due the overriding predominance of westerly quadrant winds at the site. Modelling by Black et al. (2000) showed the presence of some eddy circulation in the bay, which may modify this pattern at times. However, the overall trend is for sand to move east around the beaches due to winds.

In summary, our analysis of the physical system indicates that there are two distinct compartments (Fig. 3.2), That is:

Zone 1: A region between the entrance to Poole Harbour and Poole Head / Flag Head Chine

Zone 2: A region beyond Flag Head Chine up to the Bournemouth Borough boundary

The distinction is that Zone 1 is subject to a balance of wave and tidal currents within the confines of the ebb-tidal delta, while zone 2 is beyond the direct tidal influence of the ebb delta where the sediment dynamics is mostly driven by waves and wind.

The shoreline in Zone 1 is approximately linear, while Zone 2 is arcuate. The linear shoreline is indicative of the balanced wave and tidal processes, and the wave rotation due to refraction over the offshore delta.

The arcuate shape in Zone 2 relates to slow changes in wave exposure. To the east in Poole Bay, the angle of wave attack changes as the beach slowly comes out of the shelter of Durlston Head (Black et al. 2000). The west swell conditions become more dominant to the east, causing the beach to align more into that direction of swell. While at the Poole end of the Bay, the shelter of Durlston Head protects the coast from west waves, while the easterly waves (coming from the English Channel and France) become more dominant. As a consequence, the beach alignment is constantly changing around the Bay.

There are many similar curved beaches worldwide, with some of the seminal work to understand the alignments being conducted by Black and Rosenberg in Port Phillip Bay, Australia (Black and Rosenberg, 1992). They showed that beach orientations in the bay were a response to the dominant wave climate at every location around the Bay, which was partially determined by the presence of headlands, as in Poole Bay. Similar causes of bay alignment have been demonstrated in New Zealand and California, supported by detailed field measurements and modelling (e.g. Mead et al., 2001; Mead et al., 2004a).

Black et al. (2004a) noted that the beach at Boscombe is about 11° out of alignment with the wave climate, leading to the net movement of sand to the east, driven by the combination of waves in the surf zone and the dominant winds from the west quadrant. While no detailed analysis of Poole Bay in Zone 2 has been completed, numerical modelling of the Bay supports the contention that the beach is slightly out of alignment with the waves, leading to a net sediment movement to the east (i.e. towards Bournemouth).

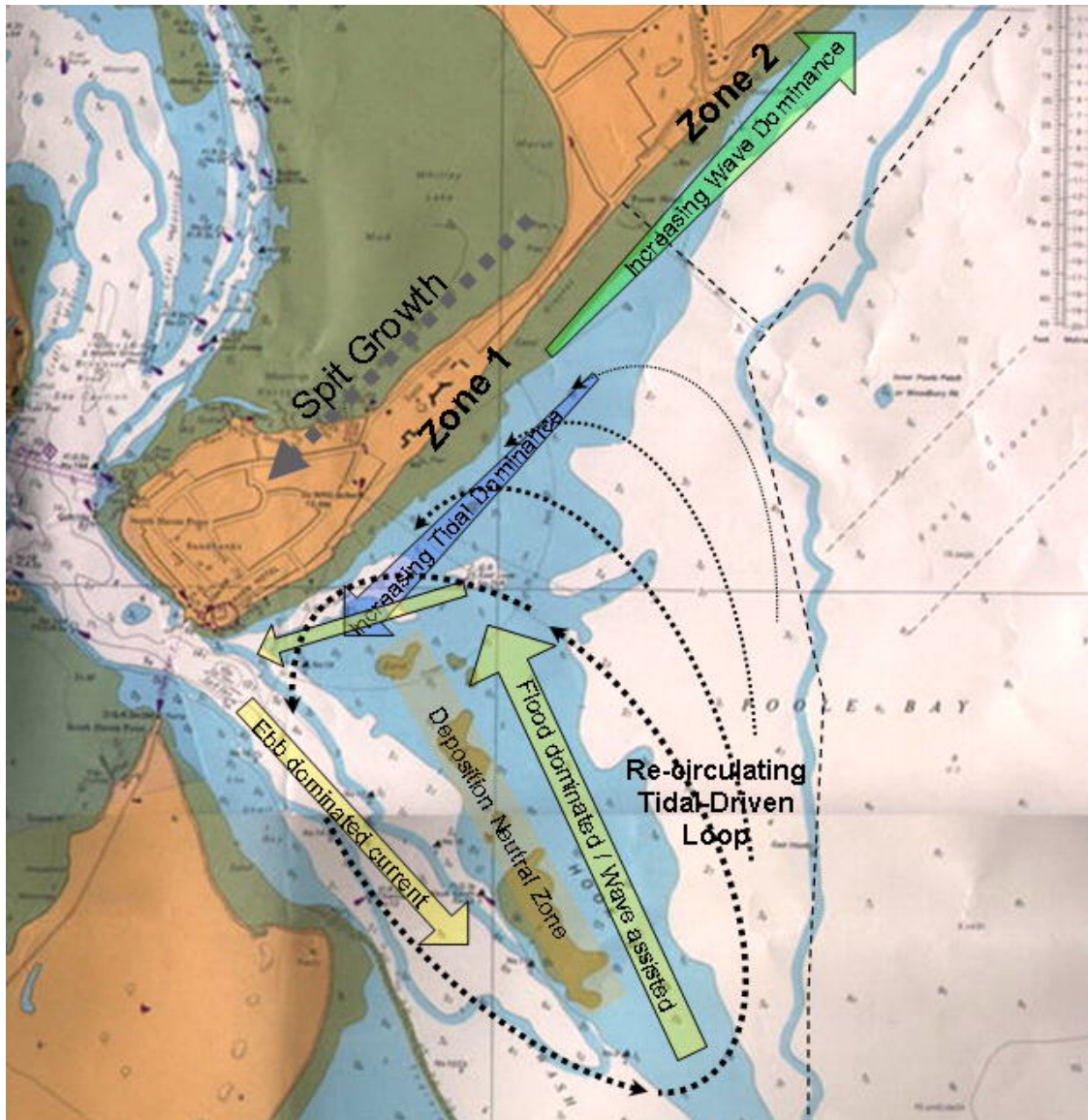


Figure 3.2. Summary of the predominant coastal processes at the entrance to Poole Harbour and Sandbanks.



Figure 3.3 Aerial photograph of the groynes constructed at Sandbanks near the entrance to Poole Harbour.

3.3 Implications of the Sandbanks Dynamics for Coastal Protection

3.3.1 Description of Existing Works at Sandbanks

In Poole Borough, short groynes have been constructed near the entrance to the harbour along the foreshore (Fig. 3.3). These have been filled by nourishment and the beach within the groyne field is currently wide and “healthy”. The project is viewed as a success by many, as it provides a useful beach with adequate width, although the quality of sand used for the nourishment was temporarily criticised by some residents. On the contrary, the sand used for the recharge was largely from Poole Harbour entrance channel and well matched to the indigenous sediments (A. Brampton, pers. comm.)

The groynes at Sandbanks have a low profile and many are buried at the inshore end. On top of each groyne is a concrete footpath that provides good public amenity and viewing of the surf zone. This is popular and a major beneficial outcome. The Sandbanks groynes are a contrast to the wooden groynes at Bournemouth. These are made with the single purpose of blocking longshore drift. Except for the wind shelter they provide, these

narrow wooden groynes have no public amenity (to our knowledge) and have been criticised by lifesavers as being dangerous for inexperienced swimmers.

Notably, the groynes at Sandbanks are constructed to minimise visual impact, but there have been criticisms from some residents that the groynes are both dangerous for swimmers and an imposition on the beach.

It is well known that all estuarine entrances are mobile and migrate at yearly and decadal time scales. Indeed, they are not usually a good place to build a house! They are subject to strong physical processes (particularly strong waves and tidal currents). Thus, the balance between contrary processes leads to constant changes in the spit position in response to changing weather patterns. As the position of the spit results from a delicate balance between two large forces, a slight shift in either force (due to a windy year or decade, larger swell than normal, the North Atlantic Oscillation, etc.) will result in migration of the spit. Once houses are built on this fragile and mobile region, any slight change to the sediment supply, such as a new jetty, breakwater, loss of a sand dune etc., will also lead to adjustments in the position of the spit. Another factor is a change to the offshore delta (due to natural change or perhaps due to dredging), which will lead to altered wave refraction and changes to the wave angle at the shore. Wave angle determines current strength and once again the balance will be affected. Of course, Rule 1 is not to live or build on sand spits!! However, once this has occurred, there are few exceptions worldwide where human intervention to stabilise the location of the spit has not been needed. Normally this will take the form of a hard structure at the shoreline. In the case under consideration, the groynes provide the required stability.

The modern short groynes at Sandbanks (Fig. 3.3) were designed to stop the erosion of the beach, due to the strong alongshore tidal currents. Once the groyne is placed, the natural balance is further disturbed and management of sand supply is needed. At Poole, this is achieved through nourishment.

It is our conclusion that:

- **The existing groynes, in association with beach recharge, solves the problem of the natural (but unwanted) migration and erosion of Sandbanks Spit.**
- **We also conclude that the short groynes are better than longer groynes. This is because the long groynes would project out further towards the nearshore flood-tidal channel. This channel returns sand towards the entrance to potentially feed the beach system, and so this flow should not be inhibited by structures. Consequently in the area of existing works, we cannot support the suggestion of Halcrow that the groynes should be longer.**
- **We support the decision-making that led to the construction of the existing groynes in that area. While there is a visual and social**

impact, there is no way that houses can be placed on a sand spit without some form of human control of the shoreline movement. Surfing reefs in this location may be suitable, but the beach processes result from an interaction of opposed wave and current processes and it would be difficult to justify the use of reefs.

- The use of successively shorter groynes to smooth out the “end effects” is effective, and the benefits can be seen on the aerial photographs which show minimal end effects beyond the groyne field (Fig. 4.1). (End effects are the scour hole that cuts into the beach on the downstream side of groynes).
- There is one compartment (between the groynes closest to the entrance) that is suffering from scour. It is our understanding that the compartment was filled with recharge sand, but this sand has been lost. Our expectation is that the mechanism responsible for the sediment losses is a combination of eddy circulation within the compartment, due to passing tidal currents, and the steeper cross-shore profile at this compartment with the East Looe and Swash channels being very close to the shore. A solution for this problem can undoubtedly be found. Our recommendation is that micro-scale modelling of the tidal circulation and wave interactions should be undertaken to determine the physical cause and, with this knowledge, a solution can be recommended. It is likely that the solution will relate to deflection of tidal flows in combination with recharge.

3.4 Beyond the Existing Works

We have concluded that works in the existing area near the entrance to Poole Harbour are satisfactory, although some additional study is needed to determine the cause of the sediment losses within the groyne compartment closest to the entrance. Beyond this region to the north-east, recommendations are needed for future works. At Sandbanks, Zones 1 and 2 are different dynamical zones, suggesting that different solutions may be needed.

It is our opinion that control structures are needed:

- 1. At the interface between zones 1 and 2, near Flag Head Chine**
- 2. Along the shoreline in Zone 2**

The main goals would be to:

- Treat the 2 compartments individually

- Reduce the losses of sediment to the east

3.5 Recommendations for Works

As described in Chapter 2, the most suitable coastal protection methods at the site are groynes or reefs, combined with nourishment. We reject the breakwater option outright. This is due to many factors, including the higher cost with the raised crest and the unsightly visual impact. Breakwaters are not a suitable option for a beach backed by possibly the most expensive beach real estate in Britain. It is out of the question to place big breakwaters offshore, as they totally dominate the seascape and, in our opinion, the public would not commit to them. In addition, they are a “heavy-handed” option that cannot incorporate wave rotation in the small wave climate, when a more sophisticated multi-purpose reef will provide a similar outcome.

3.5.1 Groynes

Taking the groyne option first, two distinct possibilities are,

- a groyne field similar to existing practice (i.e. a succession of short groynes separated by 150-300 m, Fig. 3.4a), and;
- fewer but longer groynes that separate the beach into larger compartments. If the Halcrow recommendation was adopted, the groynes would be about twice as long as the existing groynes and double the spacing (Fig. 3.4b).

Long groynes

Our recommendation is that if groynes were to be adopted, then the long groynes should not be used. This is because they would extend over the longshore bar causing significant changes to the sediment dynamics. Such an option would be experimental as it has not been proven, and there is a strong likelihood that it would deflect sand offshore and into deeper water at times (Basco and Pope, 2004). There is also the possibility that the longer groynes would cause stronger currents and greater safety problems, particularly as the stronger currents may discharge into deeper water at the groyne tip. Finally, the long groynes would not overcome the unwanted construction on the beach, or the problem of visual impact. They would, in fact, worsen the visual impact.

As noted in the brief, care should be taken to ensure that there are no serious impacts on Bournemouth at the boundary with Poole Borough. A large groyne there would strongly impact on the adjacent borough. Similarly at the interface between Zones 1 and 2, a large impact downstream may be expected.

Short groynes

Short groynes have not been totally successful along the Bournemouth shoreline, as the sand needs to be replenished every 10-15 years, even with the groynes in place. Thus, they are not solving the problem of beach sediment losses. In addition, there are concerns about amenity and safety (Short, 2001). For instance, many residents would now like to have an uninterrupted stroll along the beach, which is not possible with the groynes that have been adopted at Bournemouth.

If low profile, rock groynes were adopted, then the visual impact is reduced, assuming that the beach can be nourished in order to keep the groynes mostly buried above the high tide line. There is a possibility of eliminating the shore connection of the groynes, above the high tide line. **This should be possible, if the nourishment supply was guaranteed.**

In summary, short groynes have potential to be a viable option, but their weaknesses are as follows:

- social attitudes regarding construction on the beach and visual impacts;
- physical end effects;
- the existing groynes are not preventing sand losses at Bournemouth, where nourishment is still needed every 10-15 years;
- If the groynes are spaced at 150 m, then over 2.5 km in zones 1 and 2, some 16-18 groynes would be needed at a very substantial cost, which would exceed the cost of a reef solution;
- Concerns about safety of swimmers, and;
- Lack of uptake of potential significant amenity benefits that can be accrued.

The small groynes have been used in the past, but several convincing concerns are noted above. A more detailed modelling study would be needed to determine the best length and spacing.

3.6 Reef options

As summarised in previous chapters, multi-purpose reefs have the potential to provide coast protection, enhanced ecology, and improvements for all forms of surfing (board, kite and wind surfing). In Poole, the board surfing potential is limited due to the small wave climate, although some days are expected to be suitable. However, the kite and

wind surfers would benefit greatly. For example, the reef allows wind surfers to do wave jumping, which is not possible in a coastal surf zone as they do not have the open water to gather speed against the coast. On the contrary, they can do this over the open water between the natural surf zone and the reef.

While the amenity is a bonus, the goal at Poole is to protect the coast, with a solution that would justify grant aid from DEFRA. One of the biggest reef benefits is versatility. A reef is fully adjustable and modular. Factors that can be adjusted are reef size and shape, reef orientation, distance offshore, wave rotation, dissipation character and depth of crest. Indeed, a reef is a sculpted shape to optimise coastal protection and amenity. With the low tidal ranges at Poole relative to other parts of the British coast, previous studies by ASR have shown that reefs in Poole Bay would be effective for protecting the coast and widening the beach.

Thus our recommendations in relation to reefs are:

- **A reef on the boundary between Zones 1 and 2, to widen the beach in a critical region at the end of the ebb tidal delta (Fig. 3.4c). With the small wave climate and shallow depths, it is anticipated that a small reef of no more than 3,500 m³ would be needed.**
- **That 3 reefs be built in Zone 2 over the 2.3 km length (Fig. 3.4c). This puts the spacing of the reefs at about 780 m. The reefs should rotate the waves and be successively larger to the east. This would have the effect of providing more protection, as the wave climate gets bigger to the east, while also rotating the beach by widening the beach more to the east. With the small wave climate and shallow depths offshore, it is estimated that the size of the 3 reefs should be 3,000, 4,000 and 5,000 m³ respectively and a crest height just below low tide level. However, further studies would be needed to do a detailed design. In all cases, the reefs would be placed about 50-100 m offshore of the bar (Fig. 3.4c) and studies would be needed to determine the size and amplitude of the salient that would form.**

3.6.1 Discussion

The layout and spacing is chosen in accordance with our experience of natural reefs and our many years of research. Black and Andrews (2001a,b) assessed hundreds of natural cases to obtain relationships of reef size / placement to the response of the salient (size and geometry). Subsequent numerical modelling, physical modelling and commercial studies (some confidential) of ASR's consultants over nearly a decade have found that the salient size depends on the reef length and distance offshore and that the length of the salient is considerably more than the length of the reef (some 4-8 times) (Black, 2003; Mead et al., 2004). The calculations are very applicable to Poole due to the similar tidal

levels. As noted above, these results have been confirmed by four years of monitoring on the Gold Coast Reef.

Having said that, the brief of HR Wallingford states that there, “*are likely to be "value judgments" and so expert views on these aspects would be required.*”. Consequently, our expert views and value judgements were requested. The next stage of confirmation would require a more detailed study which would include numerical modelling of the reefs and beach system, which ASR normally does to undertake detailed design in each reef project.



Figure 3.4a Long groyne option: This option is not recommended



Figure 3.4b Short groyne option.



Figure 3.4c 4 reefs option. If nourishment was available, 2 reefs only would be required.

3.7 *Nourishment*

The chosen options are highly dependent on the proposed nourishment programme. Assuming that the losses per year from the beach are up to about 80,000 m³ and the nourishment could be as much as 50,000 m³, then only very small works (if any) may be needed. Thus, before making the final decision about control works, it is essential that the nourishment programme is finalised. This means that the timetable and quantities of the nourishment programme should be agreed before any structures are designed (reefs or groynes).

Our assessment indicates that structures are needed only to bridge the gap between the volume that can be placed as nourishment and the volume that is lost each year due to natural attrition. Our “value judgment” assessment is that the number of reefs could be halved if 50,000 m³ of nourishment is available per year. We would similarly expect that groyne numbers could be reduced also, because full re-orientation of the beach onto the neutral alignment in the groyne compartments would not be needed, as they could leak up

to 50,000 m³ per year. Less sand leakage (with reefs or groynes) could be designed into the structures if further widening of the beach was required.

3.8 Summary

- **If up to 50,000 m³ of nourishment was placed on the Poole beaches, only very small works (if any) would be needed.**
- **The nourishment programme should be finalised before making any decisions about control works.**
- **Four reefs with volumes of 3000, 3500, 4000 and 5000 m³ will be costed in the next chapter. If up to 50,000 m³/yr of nourishment was guaranteed, then a maximum of 2 reefs would be recommended with volumes of 3000 and 4000 m³.**
- **If groynes were adopted, up to 18 short groynes would be needed. This could be significantly reduced if regular nourishment was planned.**

Chapter 4 Costing of reef options

4.1 Introduction

The following costing is based on work that has been recently completed for the proposed Artificial Surfing Reef at Boscombe in Poole Bay and during the tendering process at Mt Maunganui in New Zealand. It assumes that there will be similarities between these reefs and those suitable for coastal protection purposes at Sandbanks. The reef can be made of rock or geotextile sand-filled containers (SFC's). If surfing is wanted, then rocks are not usually suitable, due to the danger and irregularity of the surface. A further advantage of a geotextile solution is that they can be readily removed if any unforeseen negative effects are observed.

4.2 Geotextile Reefs

Geotextiles are a family of synthetic materials including polyester and polypropylene which are formed into flexible and durable sheet fabrics that are resistant to tension and tear. Geotextile materials are now commonplace in the construction industry and are used in some form in the majority of modern Civil Engineering projects for applications such as embankment reinforcement or foundation preparation. The use of geotextiles is also becoming increasingly popular for coastal applications such as groynes (Restall et al. 2002), dune reinforcements (das Neves et al. 2004) and artificial reefs.

Geotextile sand containers are purpose built containers for encapsulating sand, gravel and other materials to provide building elements for the construction of structures such as sea walls, groynes, flood control berms, river protection, and containment bunds. Sand filled geotextile containers exist in many forms, including small (1 tonne) geobags, large (100-300 tonnes) geocontainers and geomattresses. A comprehensive investigation of the available geotextile container products has been carried out during studies for numerous international reef projects. Based on these studies, two preferred methods of geotextile reef construction have been identified:

- 1 Large geotextile mega-containers placed using a split-hull barge.
- 2 Custom designed, compartmentalised geotextile reef units, filled in-situ.

These techniques are briefly described below and a breakdown of costs is assigned to each method for reef construction in Poole Bay.

4.3 *Sand-Filled Geotextile Mega-Containers*

An artificial surfing reef for coastal protection and amenity enhancement was constructed at Narrowneck, Australia using large 160-300 tonne sand-filled containers (Figure 4.1). The containers used at Narrowneck were 20 m in length and 3-5 m in diameter and were placed on the seabed using a split-hull barge. A similar technique would be considered suitable for construction of reefs in Poole Bay.



Figure 4.1. The Narrowneck Artificial Surfing Reef at Narrowneck, Australia.

Placement of the individual containers is achieved using a split-hull barge which would operate from Poole Harbour. The empty mega-containers are loaded onto the vessel, laid out in the hull and then hydraulically filled with sand using the outlet from a dredge or slurry pump. Once the container has been filled, the barge is moved to the correct installation location using DGPS equipment which allows highly accurate positioning of the vessel. When the barge is at the correct installation position, the geotextile container is placed by splitting the hull of the barge along its axis which allows the container to fall through the hull and onto the seabed (Figure 4.2). This process is then repeated to bring the reef up to its final designed profile using layers of geotextile containers.



Figure 4.2. Geotextile container placement using a Split-Hull Barge.

The cost information provided in Table 4.1 has been compiled with assistance from local or international contractors with experience of working in the Poole Bay area along with international geotextile manufacturers. As no detailed design for the reefs is available, for the purpose of this costing it is assumed that the reef will be constructed using medium size mega-containers with a volume of 120 m³. It is hoped that if the project were to go ahead, it may be possible to make beneficial use of the sand dredged from Poole Harbour to fill the geotextile containers which would allow significant cost savings. However the costing provided in this report assumes that the sand required to fill the containers is obtained from a commercial source.

Reef Volume	3000m3	3500m3	4000m3	5000m3
No. of Geotextile Containers	21	25	28	36
Total Geotextile Cost	£52,500	£62,500	£70,000	£90,000
Fixed Plant Costs	£112,000	£112,000	£112,000	£112,000
Placement Costs inc. plant and labour	£158,000	£188,250	£210,850	£271,000
Sand Cost	£43,350	£50,575	£57,800	£72,250
TOTAL COST	£365,850	£413,325	£450,650	£545,250

4.4 *Compartmentalised Geotextile Reef Units*

A second potential method for reef construction in Poole Bay makes use of compartmentalised geotextile reef units made specifically for surfing reef construction. The containers are multi-celled, compartmentalised units similar to that shown in figure 4.3 and are designed to be hydraulically filled with sand in-situ at the reef site. Similar units have been developed in the US specifically for the construction of a multi-purpose reef at Oil Piers, California (Mead et al. 2004a). The prefabricated units allow the reef to take the exact designed cross-sectional shape once filled, ensuring that the quality of the surf on the reef is optimised. The reef is built up using a number of these modular reef units which are filled to the full reef height and placed one beside the other to form the entire reef volume.



Figure 4.3. Example of a compartmentalised geotextile structure. Installation of the reef units is achieved by winching the containers from the work vessel down to a pre-prepared grid of seabed restraint anchors. Once securely in position on the

seabed, the individual compartments of the reef units are hydraulically filled with a sand/water slurry from a hopper barge using a slurry pump. The filling process is controlled by a team of commercial divers.

For the purpose of this costing it is assumed that each container has plan dimensions of 35 x10 m, an average crest height of 3 m above the seabed and an approximate total volume of 500 m³.

Table 4.2 Cost information for construction using compartmentalised reef units.

Reef Volume	3000m³	3500m³	4000m³	5000m³
No. of Reef Units	6	7	8	10
Total Geotextile Cost	£192,000	£224,000	£256,000	£320,000
Fixed Plant Costs	£10,000	£10,000	£10,000	£10,000
Placement Costs inc. plant and labour	£96,000	£112,000	£128,000	£160,000
Sand Cost	£51,000	£59,500	£68,000	£85,000
TOTAL COST	£349,000	£405,500	£462,000	£575,000

4.5 Total Project Costs

As the fixed and mobilization costs are common to all reefs, the total project costs are smaller if the 4 reefs are built together. Also, there will be cost savings if the sand can be obtained from Poole Harbour at no additional cost to the reef construction project. Table 4.3 shows the total project costs for 4 reefs with these conditions.

Table 4.3. Total project costs for 4 reefs. Note that costs would be approximately halved if 2 reefs were required.

Construction method	Compartmentalised containers (£)	Mega-containers (£)
Total geotextile cost	992,000	275,000
Fixed plant costs	10,000	112,000
Placements costs, incl. plant and labour	496,000	828,100
Total cost	£1,498,000	£1,215,100

References

- Adams, C. B., and C. J. Sonu, 1986. Wave Transmission across Submerged Near-Surface Breakwaters. *Proceedings 20th Coastal Engineering Conference*, ASCE, 1729-1738.
- Ahrens, J. P., 1984. Reef Type Breakwaters. *Coastal Engineering - 1984*, B. L. Edge, ed., 1985. pp. 2648-2662
- Basco, D. R., and J. Pope, 2004. Groin Functional Design Guidance from the Coastal Engineering Manual. Special Issue 33, *Journal of Coastal Research*, 121-130.
- Black, K.P. (1987) *A numerical sediment transport model for application to natural estuaries, harbours and rivers*. In: 'Numerical modelling applications to marine systems'. ed: J. Noye. North Holland/Elsevier. Mathematics Studies 145:77-105.
- Black, K.P.; Healy, T.R. and Hunter, M. (1989) Sediment dynamics in the lower section of a mixed sand and shell-lagged tidal estuary. *Journal of Coastal Research*. 5(3): 503-521.
- Black, K.P. and Rosenberg, M.A., 1992. Natural stability of beaches around a large bay. *Journal of Coastal Research*. 8(2): 385-397.
- Black, K.P. 1998. *Narrowneck Reef - Report 3: Sediment transport*. Joint Centre of Excellence in Coastal Oceanography and University of Waikato report prepared for Gold Coast City Council, 1998.
- Black, K. 1999. Designing the shape of the Gold Coast Reef: sediment dynamics. *Proceedings of the Coasts & Ports '99 Conference*, 14-16 April 1999, Perth, Australia. Vol 1, pp.58-63.
- Black, K.P., Mead, S., McComb, P., Jackson, A. and Armstrong, K., 1999. *New Plymouth City Foreshore Redevelopment: Reef and Beach Feasibility Study*. Prepared by the Centre of Excellence in Coastal Oceanography and Marine Geology, for the New Plymouth District Council, March 1999.
- Black, K. P., S. Mead and A. Jackson, 2000. *Beach Amenity Options and Coastal Protection at Bournemouth*. Technical Report Prepared for Leisure and Tourism Services, Bournemouth Borough Council, May, 2000.
- Black, K. P., 2000. *Artificial surfing reefs for erosion control and amenity: theory and application*. International Coastal Symposium (ICS2000) Invited paper. Rotorua, April, 2000.

- Black, K. P., S. T. Mead and J. Mathew, 2001. *Design and Approvals for an Artificial Reef for Protection of Noosa Main Beach: Detailed Investigations and Modelling*. Final report for Noosa Council and ICM Ltd, June 2001.
- Black, K.P. and S.T. Mead, 2001. *Wave Rotation for Coastal Protection*. Proceedings Coasts and Ports 2001. 25-28 September, Goldcoast, Queensland, Australia.
- Black, K. P., and C. Andrews, 2001a. Sandy Shoreline Response to Offshore obstacles Part 1: Salient and Tombolo Geometry and Shape. *Journal of Coastal Research*, Special Issue 29: 82-93.
- Black, K. P., and C. Andrews, 2001b. Sandy Shoreline Response to Offshore obstacles Part 2: Discussion of Formative Mechanisms. *Journal of Coastal Research*, Special Issue 29: 94-101.
- Black, K.P. and S. T. Mead, 2002. *Review of Dune Contouring Criteria For Christchurch Beaches*. Report to the Christchurch City Council, December 2001.
- Black, K. P., S. T. Mead, P. McComb, J. Mathew and C. Blenkinsopp, 2002. *Developing a Surfing Reef at Newquay Bay: Feasibility Study*. Report to Newquay Artificial Reef Association, May 2002
- Black, K. P., 2003. *Numerical Predictions of Salient Formation in the Lee of Offshore Reefs*. Proceedings of the 3rd International Surfing Reef Conference, June 23-25 2003. ISBN 0-473-09801-6
- Black, K. P., C. Blenkinsopp, B. Beamsley, D. Johnson, S. T. Mead and J. Mathew, 2004a. *Boscombe Surfing Reef Detailed Design: Field Data and Initial Design Report*. Report prepared for Bournemouth Borough Council, August 2004
- Black, K. P., S. T. Mead, P. McComb B. Scarfe, and B. Beamsley, 2004b. *Studies for Resource Consent: Opunake Surfing Reef. Detailed Design and Physical and Biological Impact Studies needed to obtain Resource Consents for the Artificial Surf Reef at Opunake, Taranaki, New Zealand*. Report prepared for the South Taranaki District Council and the Opunake Artificial Reef Committee
- Burkinshaw, J., and W. Illenberger, 2001. *The Cape St Francis headland bypass dune system from a geomorphological perspective*. Illenberger & Associates, December 2001.
- Brunn, P., 2000. Coastal Protection Structures. *Proceedings Coastal Structures '99*, Losada (ed.), Balkema, Rotterdam. Pp. 737-746.

- Bush, D. M., O. H. Pilkey Jr., and W. J. Neal, 1996. *Living by the Rules of the Sea*. Duke University Press, Durham and London, 1996. Pp. 179.
- Chapman, D.M. (1989), Coastal Dunes of New South Wales – Status and Management. A report of the University of Sydney and Soil Conservation Service of NSW, University of Sydney Technical Report 89/03, December, 1989.
- das Neves, L., Gomes, F.V. & de Lurdes Lopes, M., 2004. *Coastal Erosion Control using Sand Filled Geotextile Containers: A Case Study from the NW Coast of Portugal*. 29th International Conference on Coastal Engineering.
- Dyer, M. J., 1994. Beach Profile Change at St. Clair Beach, Dunedin. Unpublished Masters of Science Thesis, University of Canterbury.
- Gough, V.J., 1999. *Assessing the Economic Effects of Recreation Facility Development: Proposed Artificial Surfing Reef, Mount Maunganui, New Zealand*. Unpublished 0516.590 Directed Research Project For Honours Degree in Social Sciences, Department of Geography, University of Waikato, New Zealand. Pp. 58 + appendices.
- Grigg, G. B., 2004. Headlands and Groins: Replicating Natural Systems. Special Issue 33, *Journal of Coastal Research*, 280-293.
- Hanson, H., and N. C. Kraus, 2004. Advancements in One-Line Modelling of T-Head Groins: (Genesis T). Special Issue 33, *Journal of Coastal Research*, 315-323.
- Harlow, D. A., 2000. *Bournemouth Beach Monitoring, 1974 to 2000*. Bournemouth Borough Council, Development Services Directorate, Technical Services Division, Coast Protection Section, 2 Volumes.
- Harris, L., 2002. Submerged reef structures for habitat enhancement and shoreline erosion abatement. U.S. Army Corps of Engineers Coastal & Hydraulic Engineering Technical Note (CHETN), Vicksburg, MS.
- Houston, J. R., 2002. *The Economic Value of Our Beaches*. US Army Engineer Research and Development Centre.
- Hume, T. M., K. P. Black, J. W. Oldman and R. Vennell, 1997. *Signatures of Wave and Current Forcing on the Seabed About a Large Coastal Headland*. Proc. Combined Australasian Coastal Engineering and Ports Conference, ChCh, New Zealand, 1997.
- Jackson, L.A. & R.B. Tomlinson (1990). *Nearshore nourishment implementation, monitoring and model studies of 1.5M³ at Kirra Beach*. Proc. 22nd I.C.C.E., ASCE, Delft, 1990.

- Jackson, L. A., R. Tomlinson, I. Turner, R. Corbett, M Daggata and J. McGrath, 2005. Proceedings of the 4th International Surfing Reef Conference, Manhattan Beach, Californai, 12-14 January 2005.
- JCR, 2004. Functioning and Design of Coastal Groins: The Interaction of Groins and the Beach – Process and Planning. Special Issue 33, Journal of Coastal Research, p.367.
- Johnson, H. K., and H. Kofoed-Hansen, 2003. *Modelling morphological evolution in the vicinity of groins using MIKE 21 CAMS*. DHI Water and Environment, Agern Allé 11, DK-2970, Hørsholm, Denmark.
- Komar, P. D., 1997. Beach Processes and the Erosion of Coastal Properties. *Proceedings of Pacific Coasts and Ports '97*, 7-11 September, 1997, Christ Church, New Zealand. Vol. 1, 1-6.
- M^cComb, P.; Black K.; Healy, T. & Atkinson, P. (1999). *Coastal and sediment dynamics at Port Taranaki, New Zealand: a large, multi-faceted, field experiment*. Proceedings of Coastal Structures '99 Conference, Santander, Spain. Vol. 2: 823-832.
- McGrath. J., 2002. *Northern Gold Coast Beach Protection Strategy – July 2002 Update*. GCCC report from Coastal Management Engineer.
- Mead, S. T., K. P. Black and P. McComb, 2001. *Westshore Coastal Process Investigation*. Technical report for Napier City Council, September 2001.
- Mead, S. T., 2004. *Nanuku Surfing Reef: Feasibility Study for a Surfing Reef at Nanuku Island, Fiji*. Prepared for R. Hatherly, June 2004.
- Mead, S. T., K. P. Black, B. Scarfe, L. Harris, J. Sample and C. Blenkinsopp, 2004a. *Oil Piers Reef: Phase II – Detailed Design and Environmental Impact Assessment*. Report prepared for the US Army Corp of Engineers, January, 2004.
- Mead, S. T., K. P. Black, B. Scarfe, C. Blenkinsopp, B. Beamsley and J. Frazerhurst, 2004b. *Orewa Beach Reef – Feasibility Study for a Multi-Purpose Reef at Orewa Beach, Hibiscus Coast, Auckland, New Zealand*. Report prepared for Rodney District Council and the Orewa Beach Reef Charitable Trust, April 2004.
- NSWG, 1990. NSW Coastline Management Manual. New South Wales Government, September 1990, ISBN 0730575063

- Nielsen, L., 2001. *Newcastle Coastal Zone Management Plan: Coastal Engineering Advice*. Technical report for Umwelt (Australia) Pty Ltd, March 2001.
- Ove Arup and Partners International, 2001. *Assessment of Potential Contribution of Marinas and Watersports to Increasing Prosperity in Cornwall*. Report prepared for Cornwall Enterprise, Sept 2001.
- Pilarczyk, K. W. 1990. *Coastal Protection*. Published by A. A. Balkema, Rotterdam.
- Pilarczyk, K. W. & R. B. Zeidler, 1996. *Offshore Breakwaters and Shore Evolution Control*. Published by A. A. Balkema, Rotterdam. 560p.
- Pilarczyk, K. W., 2003. *Design of Low-Crested (Submerged) Structures – An Overview*. Proceedings of the 6th International conference on Coastal and Port Engineering in Developing Countries, Colombo, Sri Lanka, 2003.
- Pilkey, O. H., and K. L. Dixon, 1996. *The Corps and the Shore*. Island Press, Washington, D. C. / Covelo, California. Pp. 272.
- Piorewicz, J., 2002. *Proceedings of the Public Workshop – Yeppoon 2002*. Ed. J. Piorewicz, Central Queensland University Press.
- Raudkivi, A., 1980. *Orewa Foreshore*. Report to Rodney County Council Meeting, November 1980
- Raybould, M. and T. Mules, 1998. *Northern Gold Coast Beach Protection Strategy: A Benefit-Cost Analysis*. Report prepared for the Gold Coast City Council, February 1998.
- Restall, S.J., Jackson, L.A., Heerten, G.I. & Hornsey, W.P. 2002. *Case Studies Showing the Growth and Development of Geotextile Sand Containers: An Australian Perspective*. Geotextiles and Geomembranes, Vol. 20, No. 4.
- Savioli, J., K. Mangor and S. Szykarski, 2002. *Shoreline Management and Evaluation of Restoration Measures*. Proceedings of Coast to Coast 2002.
- Sea Grant, 2001. *Rip Currents*. Produced by NC Sea Grant, NC State University.
- Short, C., 2003. *Poole and Bournemouth Water Related Incidents Report*. RNLI Beach Rescue, September 2003.
- Smith, J.T., Harris, L.E., and Tabar, J., 1998. Preliminary evaluation of the Vero Beach, FL prefabricated submerged breakwater. Beach Preservation Technology '98, FSBPA, Tallahassee, FL.

- Turner, I. L., T. D. T. Dronkers, C. Roman, S. G. J. Aarninkhof and J. McGrath, 2001. *The Application of Video Imaging at the Gold Coast to Quantify Beach Response and Sand Nourishment to Construction of an Artificial Reef*. Proceedings of the Australasian Coasts and Ports Conference, Gold Coast, Australia.
- US Army Corps of Engineers, 1995. *Design of Coastal Revetments, Seawalls, and Bulkheads*. Department of the Army, US Army Corps of Engineers, Washington DC 20314-1000.
- Van der Meer W. and K. W. Pilarczyk, 1998. *Stability of Low-Crest and Reef Breakwaters*.
- WCC, 1994. *A Draft Landscape Development Plan for the Lyall Bay to Palmer Head Coastline*. A report prepared for the Wellington City Council. Pp. 32.



Appendix 1

Boscombe Surfing Reef Information Sheet

Appendix 3

Review of Beach Control Structures in Italy

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Review of Beach Control Structures in Italy

1. Introduction

This report presents a review of the various types of beach protective structures used in Italy and in particular groynes, detached breakwaters and composite structures (i.e. structures composed of detached breakwaters and groynes).

In the following, although reference to other regions will also be made, structures built in three Adriatic regions in particular will be considered namely (*Figure 1*):

- 1) Veneto
- 2) Emilia Romagna
- 3) Marche



Figure 1. Map of Italy

The reason for this particular choice, apart from a better personal knowledge, is because in each of these regions a consistent approach has been adopted for coastal defence, particularly in the Emilia Romagna and Marche.

Along the Veneto coast, the traditional solution for coastal protection is groynes, which were built intensively during the period 1950-1970, with some groynes being built even before 1940.

This tendency has been continued in major recent projects, like those at Jesolo, Cavallino and Isola Verde (see below). In all these cases, groynes have been used to support artificial beach recharge schemes.

Also in the case of Pellestrina (see below) groynes have been used in combination with submerged breakwaters to protect the fill material.

Some detached breakwaters (emergent type) were also built along the Veneto coast, but at present, the only system of emergent breakwaters is found along the beach to the west of the mouth of the River Tagliamento.

Along the coasts of Emilia Romagna and Marche, the traditional coast protection structures are detached breakwaters, which still protect long stretches of coast.

Emergent breakwaters were the preferred solution until about 1980, when their shortcomings became clear namely; erosion of downdrift beaches, which often led to the necessity to "propagate" defence schemes downdrift; unfavourable aesthetic effects and adverse environmental effects on water quality and sediments in the sheltered area. Even their effects on the beaches protected were not always satisfactory, because a lack of sediment transport and continuing subsidence.

In the 1980's, there was a change of approach, and an important turning-point was the decision of the regional authorities to carry out thorough investigations on the condition of the entire coast and to propose solutions for the eroding beaches which would carefully consider the consequences on the adjoining beaches and on the coastal environment, thus avoiding "case by case" solutions as in the past.

Artificial beach nourishment was recognised as the best solution for shore protection, and most new works carried out since then have followed this guideline.

However, artificial nourishment has not been carried out on its own and complementary structures were always built to reduce losses of the fill material.

This approach was favoured because of the scarcity and cost of the sediment used for the recharge, which was (at that time) obtained from land pits.

The typical basic choice for auxiliary structures, designed to reduce sand losses without adversely affecting the environmental qualities of the beaches, consisted of a composite structure comprising a submerged breakwater parallel to the beach, with partially submerged groynes built perpendicularly, connecting the beach to the breakwater.

All the submerged structures were composed of sand-filled geotextile bags.

At first, low barriers with crests well below sea level were used. Subsequently structures with crests less deeply submerged and larger sections were used. (*Figure 2*)

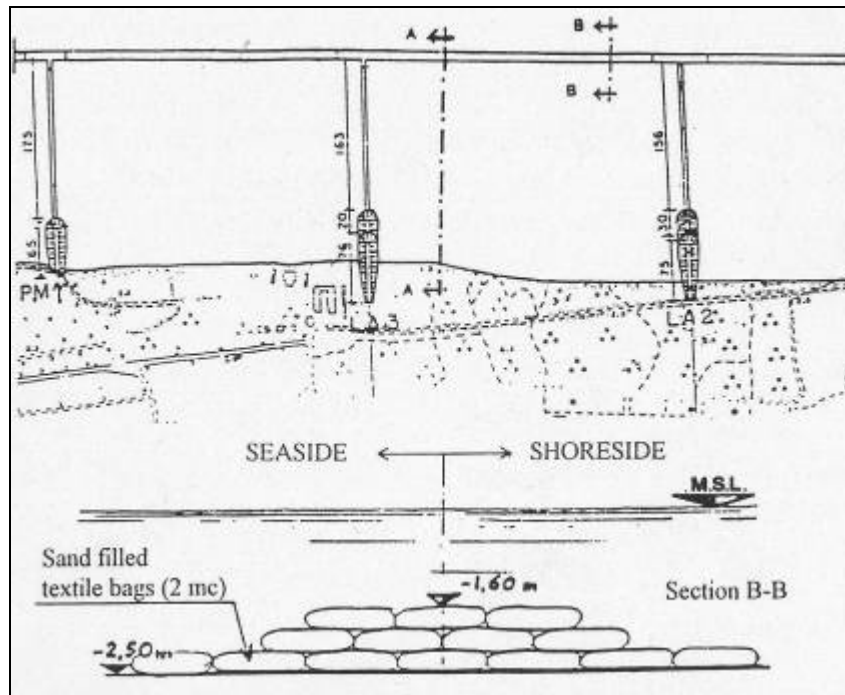


Figure 2. Protective structures used in Emilia-Romagna (Lamberti and Mancinelli, 1996).

The functional effects of this kind of solution were rather controversial, with good results reported in some cases. However, the structural weakness of the bags appeared to be a significant problem for the long-term survival of the structure, so that no new structures of this kind were built on the coast after the 1990's. Artificial beach nourishment, using sediments extracted from offshore deposits became the only recommended solution, whereas the use of rigid structures, whether of emergent or submerged types, was no longer recommended.

No examples of newly built submerged structures and only few examples of new low-crested structures (rubble mound structures with their crest at sea water level) are found in Emilia Romagna (see below).

In the case of Emilia Romagna, more than 50 km of beach (of a total of about 130 km) are now protected by detached breakwaters (emergent type) and about 14 km by submerged breakwaters.

Along the Marche coast, emergent detached breakwaters were and still are the most common protective structure built to protect against beach erosion.

After 1980, new types of coast protection were also used, such as artificial nourishment schemes protected by submerged breakwaters of various kinds.

In this case, small volumes of fills, of the order of a few thousand cubic metres, were used to protect short stretches of beach; rubble mound structures were also often used as submerged breakwaters.

Furthermore, in this region the construction of emergent breakwaters has not been almost completely abandoned as in Emilia Romagna, but continued, although at a slower rate than in the past.

In the case of Marche, about 40 km of beach (of a total of about 170 km) is now protected by detached breakwaters (emergent type) and about 15 km by submerged breakwaters.

2. Examples

2.1 Cavallino Beach

Cavallino beach is a 12 km long coastal barrier separating the North-East side of the Venice lagoon from the Adriatic Sea, and stretching from the inlet of River Sile to the northern jetty of Port of Lido (*Figure 3*).



Figure 3. The barrier beaches separating the lagoon of Venice from the Adriatic Sea (*Consorzio Venezia Nuova, 2005b*).

To protect this beach from erosion, in 1996-97 about 2 millions cubic metres of sand fill were placed on it and a system of 32 rubble mound groynes built to reduce sand losses (*Figures 4-5*). The fill was dredged from offshore deposits in a water depth of 20m, about 15 km from the coast.

The approximate length of groynes is 80m (65m measured from the shoreline), with their head placed in water depths of 3-3.5m and their spacing variable from 240 to 450 m (average spacing of more than 300 m). Their structure is rather robust, with a concrete cap at a level of more than 2 m above MSL.

After about 8 years after the completion of the works, a loss of sand fill of about 10% of the initial nourishment volume has been estimated from beach surveys carried out by the 'Consorzio Venezia Nuova'.

This seems a rather good result, although the quantity of lost material has not been compared with losses that would have occurred on the renourished beach without the protective system of groynes.

Unfortunately no data on losses of beach material before the new interventions are available at present.



Figure 4. View along Cavallino Beach (looking North East, against the direction of the net longshore drift) (*Consorzio Venezia Nuova, 2005b*).



Figure 5. View of Cavallino Beach (looking North East) (Consorzio Venezia Nuova, 2005b).

2.2 Isola Verde Beach

To protect the beach of Isola Verde (a 2.7 km long beach between the inlets of the rivers Brenta and Adige, to the south of the Venice lagoon, *Figure 6*) from erosion, about 400,000 cubic metres of sand fill (taken from the mouth of Adige river) were placed on the beach and a system of 8 rubble mound groynes built to reduce sand losses (*Figures 7-9*). At this site also, the groynes are rather large and reach a level of more than 2m above MSL.



Figure 6. View of Isola Verde Beach (Consorzio Venezia Nuova, 2005b).

The approximate length of groynes is 85m (measured from the shoreline), with their head placed in a water depth of 2.5m and their spacing is about 300 m.

About 3 years after the completion of the works, only a small loss of sand fill (of the order of a few thousands of cubic meters) has been detected by the 'Consorzio Venezia Nuova'.

(Source: *Consorzio Venezia Nuova, Private Comm*; www.salve.it).



Figure 7. View of Isola Verde Beach (looking North, in the direction of net longshore drift) (Consorzio Venezia Nuova, 2005b).



Figure 8. View of a groyne at Isola Verde (*Consorzio Venezia Nuova, 2005b*).



Figure 9. Aerial view of groynes at Isola Verde (*Consorzio Venezia Nuova, 2005b*).

2.3 Jesolo Beach

On the beach of Jesolo, a 12 km long beach north of the lagoon of Venice, between the inlets of the rivers Piave and Sile (*Figure 10*), a recharge scheme was carried over a stretch of about 9 kilometres to rebuild the beach that had suffered erosion problems.

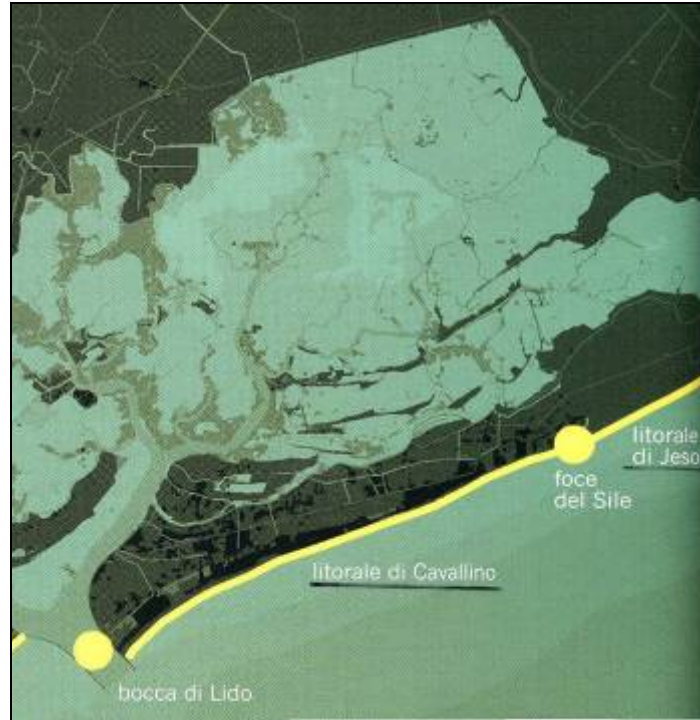


Figure 10. Map of Jesolo Beach (*Consorzio Venezia Nuova, 2005b*).

The fill material was taken, as for Cavallino beach, from offshore deposits in water depths of 20 m, about 10 km from the coast. About 500,000 cubic metres were placed on the beach. To prevent sand losses, a system of 50 groynes of permeable type (concrete piles supporting a wooden deck) was built (*Figures 11-12*).

The approximate length of groynes is 82 m, with their head in a water depth of about 2.5m and their spacing of about 180 m.

The particularity of the system is the use of permeable groynes, causing a more regular shoreline than would result from building impermeable ones.

About 3 years after the completion of the works, only a small loss of sand fill (of the order of a few thousands of cubic meters) has been detected by the 'Consorzio Venezia Nuova'.



Figure 11. Groyne system at Jesolo (Consorzio Venezia Nuova).



Figure 12. Building a piled groyne at Jesolo (Consorzio Venezia Nuova).

2.2. Detached breakwaters (emergent type)

2.2.1 Detached (emergent) breakwaters built in Emilia Romagna

Detached breakwaters of the emergent type have been built along the Emilia-Romagna coast since about 1930 and were the preferred solution for coastal protection until about 1980. In subsequent years, only a few emergent breakwaters have been built along the coast (in the period 1981-1993 a system of 4 segments has been built north of Lido Adriano and 1 km of beach has been protected by emergent breakwaters south of Cesenatico).

The structures consist of rubble-mound breakwaters, generally located in water depths of 2.5-3m; their distance from the shoreline varies from 25m to 300m. The length of the breakwater segments is about 100 m, with gaps of 30 to 40 m. The breakwaters are either parallel to the shoreline or inclined south-east (normal to the direction of dominant seas).

The typical breakwater cross-section (*Figure 13*) has a seaward slope of 1:2 and landward slope of 2:3; the crown height is about 1.0 to 1.5m above MSL, and its width is about 4 to 5m (corresponding to the width of 3 armour stones). The weight of the armour stones is 3 to 7 (metric) tons.

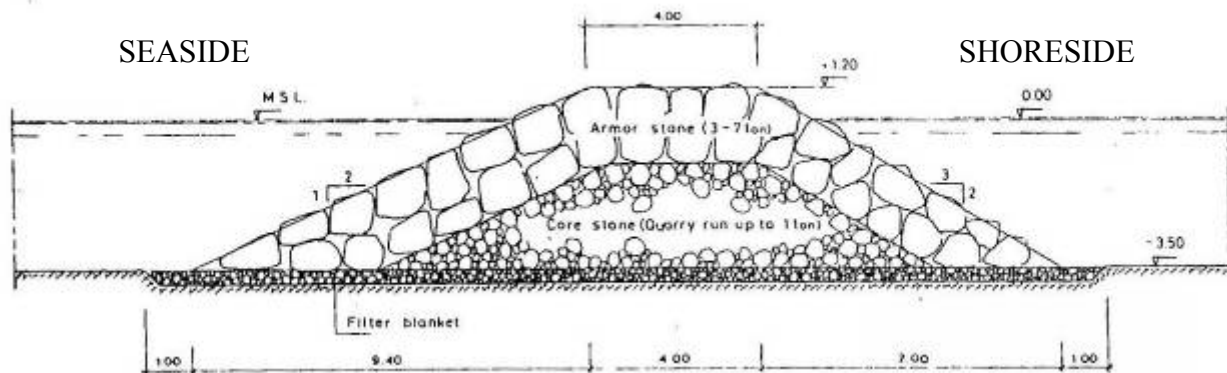


Figure 13. Typical cross-section of breakwaters used in Emilia Romagna.

The system built to protect the coast between Rimini and Cesenatico is considered as an example (*Figure 14*). This stretch of coast, with a total length of about 20 km, is protected by a long belt of detached emergent breakwaters, which represent the longest continuous protective system of detached breakwaters present in this region, probably in Italy and is composed of about 180 segments.

The necessity to protect the beach downdrift of the port of Rimini became evident some years after the last extensions of the jetties, which were completed in 1925. These extensions caused an offset in the shoreline of about 500m, with accretion of the beach of Rimini and erosion of the downdrift beaches for several kilometres. The first breakwaters were built at Viserba in 1950, and the protective system was completed in 1980.

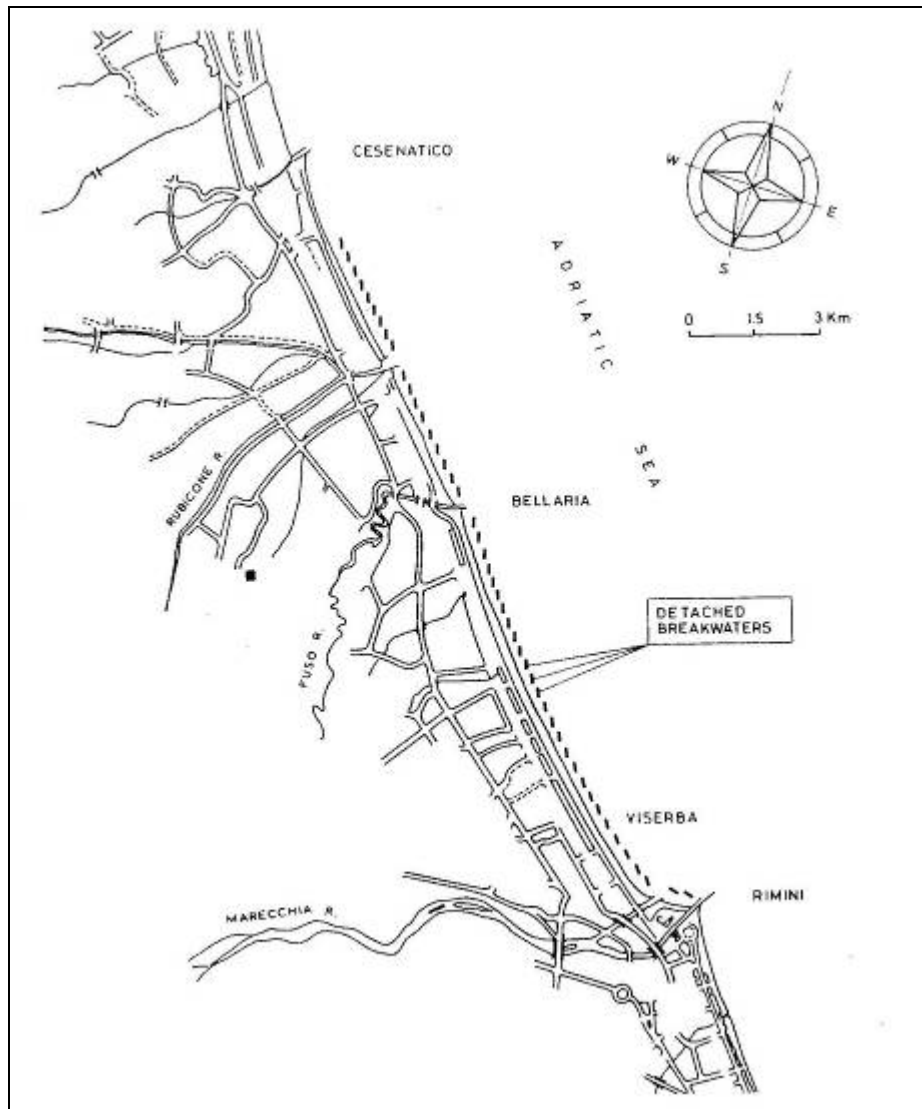


Figure 14. System of emergent breakwaters protecting the coast between Rimini and Cesenatico. (Liberatore, 1992)

In general, it is considered that the systems of emergent breakwaters built to protect the coast helped in countering erosion, at least in the short term. However, significant beach accretion was only observed near the river inlets, especially on the northern (downdrift) side of the river mouths. Little benefit was obtained on the most downdrift beaches, and in some cases the protection afforded by the breakwaters was unsatisfactory, and nourishment was deemed necessary to improve the conditions of the beach.

In the case of the coast of Emilia Romagna it should also be pointed out that a significant role has been played by subsidence that has affected the coast. In the last 50 years a subsidence of more than 100 cm has been detected along the northern stretch of the coast, whereas 60 to 70cm of subsidence has been detected at Rimini and Bellaria (middle stretch). Only along the southern part of the coast are subsidence effects negligible (as at Cattolica).

This certainly can at the least partly explain the inadequacy of any rigid coast protecting scheme to counter erosion problems along this coast; this seems to be confirmed by the protective system (of 18 emergent

breakwaters) built in 1961-71 at Cattolica, in the south of the region, which is still effective from the point of view of erosion control.

Another interesting aspect of the effects of detached emergent breakwaters on the submerged beach is the strong erosion of the offshore beach that has been observed in front of emergent breakwaters some years after their construction (*Idroser, 1996*). This results in increase of the level of the protected beach, and a decrease of the level of the offshore beach. In this way, the barrier results in a separation line between the two parts of the beach (*Figure 15*). This effect was also observed at other sites in Italy e.g., on the Tuscany coast (*Aminti et al, 2003*).

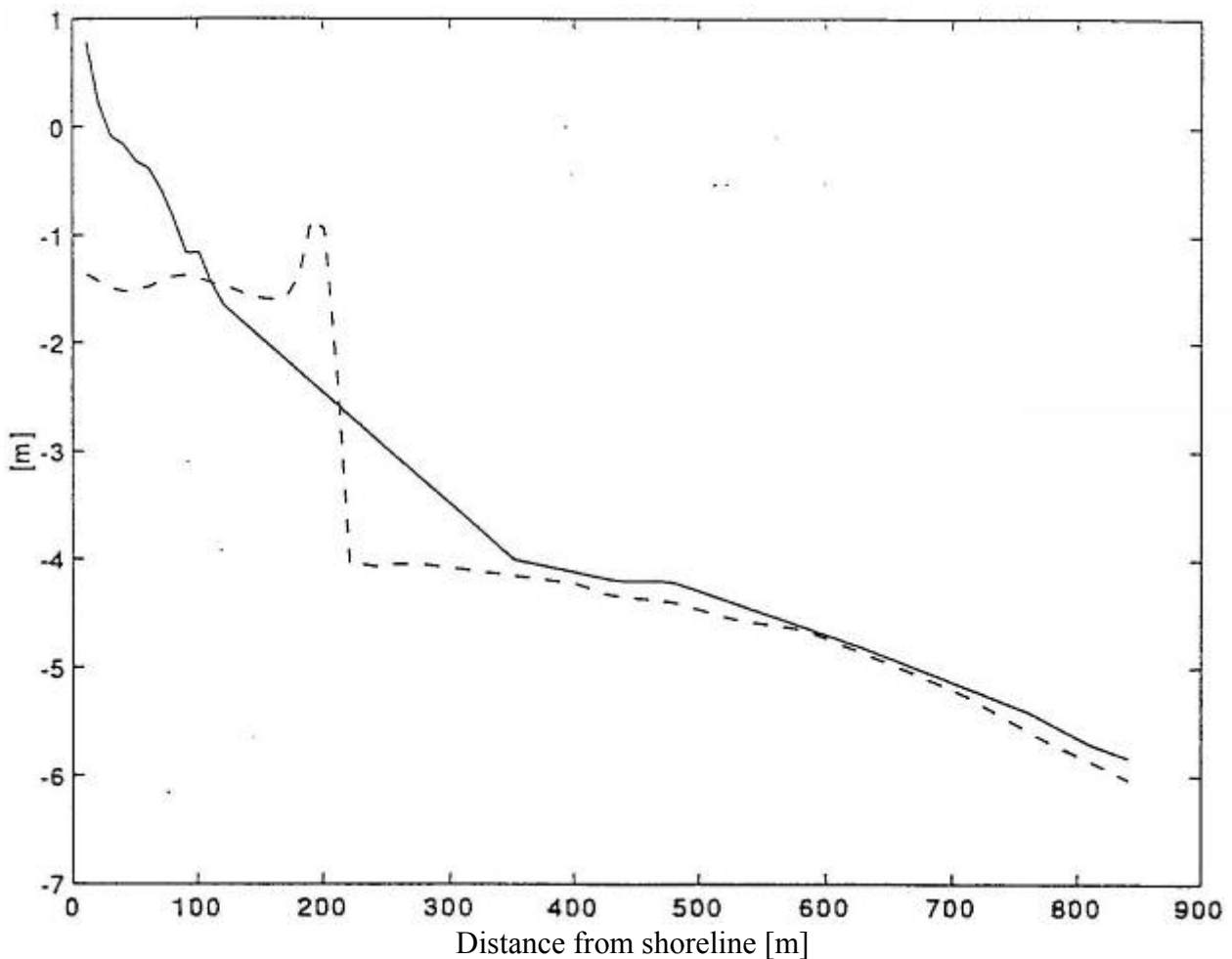


Figure 15. Beach profiles at Cesenatico in 1984 (—) and in 1993 (---). (*Idroser, 1996*)

The experiences in Emilia Romagna also evidence the difficulties and costs of maintaining a system of emergent breakwaters over time. Continuous maintenance works were necessary to repair and re-shape the breakwaters that were subject to damage by storms and the effects of subsidence; continuous modifications were also necessary in an effort to combat local erosion phenomena. In many cases this has resulted in shortening the gaps or in adding segments of breakwaters, or even in closing the gaps with submerged breakwaters.

2.2.2 Detached (emergent) breakwaters built in Marche

Detached breakwaters of emergent type used in Marche are similar to those used in Emilia Romagna. They are generally built in water depths of about 3 m; height of crown is 1 to 1.5 m above MSL with a width of about 3m; offshore slopes are 1:2 to 2.5 and inshore slopes 1:1 to 1.5. Length of segments is about the same as in Emilia Romagna.

This scheme had no significant modifications in the last years. The only difference regards orientation of barriers, which now is always parallel to the beach (formerly they were also, in some cases, inclined normally to the direction of prevailing waves).

Also in Marche the maintenance of breakwaters has been a problem, leading in several cases to the elimination of gaps or to their partial closure with submerged breakwaters, in an effort to increase the protection to the beach and to improve stability of the breakwaters ends.

2.3. Detached breakwaters of submerged or low crest type

2.3.1 Detached (low crest) breakwaters built in Emilia Romagna

After the nourishment schemes carried out until about 1990 that were supplemented using submerged structures composed of sand-filled bags, only a few low-crest structures have been built along this coast.

These are, in particular, the breakwaters built at Cesenatico and Punta Marina, whereas the third site at Lido di Dante, see below, is a composite structure. In all cases, the crest of the breakwaters is about at MSL.

2.3.1.1 Cesenatico

A long continuous barrier (with a length of about 800 m) was built in 2002 immediately north of the armoured inlet of Cesenatico to protect recharge material placed on the beach. The crest of the breakwater is at MSL.

Although this intervention seems not to have been monitored adequately, no negative effects on the protected beach or on downdrift beaches have been reported.

2.3.1.2 Punta Marina

About three kilometres of beach have been protected by a system of low crest barriers (crest level at MSL). The length of the segments is about 100 m, with a distance from the shoreline of about 250 m.

Also in this case, poor monitoring of the beach after the intervention seems to have been carried out. Results of this intervention seem, however, not to be particularly favourable.

2.3.2 Detached (low crest) breakwaters built in Marche

The ‘first generation’ of submerged breakwaters built in Marche in the 1980 had a crest level about 1m below sea level, with a crest width of 3 to 4 m, a seaward slope of 1:2 to 2.5 and a landward slope of 1:1 to 1.5. Structures were usually built as segmented breakwaters (with typical lengths of about 100 to 120m and gaps of 20-30m), although in some cases also continuous structures (without gaps) were built. They were usually placed in water depths of about 3m.

In the 1990’s, structures with crests nearer to sea level (submerged by less than 0.3-0.4 m) and of larger cross-sections were built, with crest widths of 10-12m and less steep slopes (of 1:2 to 1:3 on both sides). Both measures were taken to improve the stability of the structures and to increase their efficiency in decreasing the height of transmitted waves.

Toe protection was also provided to prevent scouring of the structures and *emergent* piled navigation markers were installed.

The use of submerged structures has not always proved successful, and in cases when they replaced older emergent barriers, a reduction in beach width was generally observed. The disappearance of tombolos at some locations, notwithstanding the associated environmental advantages, was regarded as detrimental by beach managers, and this concern resulted in a return to emergent barriers in some cases.

Increased return currents were observed at the gaps, which then in many cases were partially closed with submerged sills to prevent sand losses and improve the stability of the barriers.

2.3.2.1 Marina di Montemarignano

Four submerged breakwater segments (with a crest width of about 10 m, *Figure 16*) were built at Marina di Montemarignano for protection of a gravel beach North of the inlet of the Esino river.

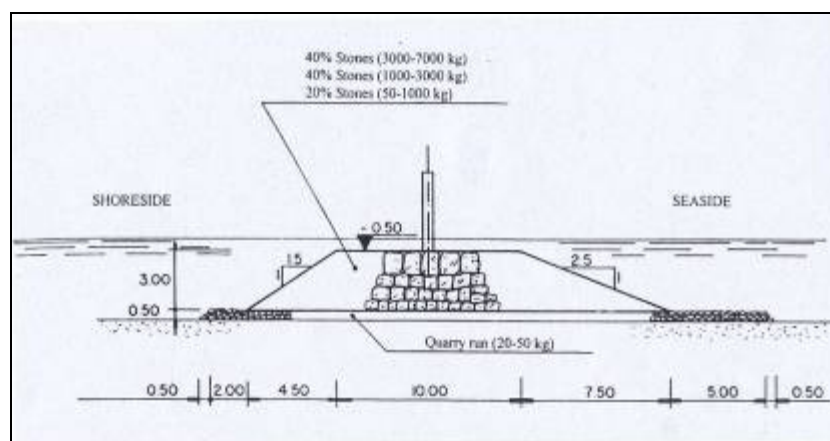


Figure 16. Section of the breakwater built at Montemarignano (*Lamberti and Mancinelli, 1996*)

These improved the beach behind the breakwaters, although to a lesser degree than behind the emergent breakwaters (*Figure 17*), whereas severe erosion occurred on the downdrift coast (*Figure 18*).

As a remedial measure for the downdrift coast, artificial renourishment with submerged groynes to protect the fill has been proposed.



Figure 17. View of Montemarignano beach (looking North, in the direction of net littoral drift: a system of submerged breakwaters built downdrift of a system of emergent breakwaters) (*Idroser, 1996*)



Figure 18. Erosion phenomena at Montemarciano, downdrift of submerged breakwaters. (*Idroser, 1996*)

2.3.2.2 Senigallia

On the beach downdrift of the jetties of the channel at the Port of Senigallia that has been subject to erosion, a system of emergent breakwaters was built, which proved successful in combating erosion. As a consequence, cusps and tombolos appeared on the beach (*Figure 19*).

Some years later, it was decided to install submerged barriers (with a crest width of about 10 m) to replace the existing emergent barriers. A more regular shoreline was achieved and the previous tombolos disappeared (*Figure 20*), although the width of the beach was preserved.



Figure 19. View of Senigallia Beach, protected by emergent breakwaters. (Idroser, 1996)



Figure 20. View of Senigallia Beach, protected by submerged breakwaters. (Idroser, 1996)

2.3.3 Detached (low crest or submerged) barriers built in other regions of Italy

2.3.3.1. Lido di Ostia (Lazio, Tyrrhenian coast)

At Lido di Ostia, an important tourist resort south of the River Tevere inlet (*Figure 21*), nourishment of the beach was carried out in 1990 using sediment extracted from land quarries.

To protect the fill material (about 1,300,000 m³), a 2500m long, continuous submerged breakwater with a crest width of 15 m and a crest level about 1.50m below sea level was built in a water depth of about 4 m and at a distance of about 110 m from the shore.

The effectiveness of this coast protection system was not particularly good, and important losses of fill material occurred in the following years due to longshore currents generated by obliquely incident waves. This was also enhanced by settlement of the breakwater and damage suffered by the rubble mound structure which was not able to withstand the wave forces, causing a decrease of the crest level to 2 to 2.5 m below MSL.

The general view about this intervention is that it was not particularly successful in retaining beach sand fill and is considered to have been less effective than the composite system built at Pellestrina (with groynes as well as a submerged breakwater similar to that built at Ostia), which is considered to have worked satisfactorily.

In the following years (1998-2004), the structural stability of the submerged breakwater at Ostia was improved using heavier armour stones. The level of the crest was increased to 0.5 to 1m below MSL and its width was increased to 20m (*Figure 22*). Furthermore, a system of 5 partially submerged groynes has been built recently to decrease longshore currents along the protected beach. More fill material was also placed on the beach in the years from 1998 to 2003. After these new interventions, the situation seems to be improving.



Figure 21. Map of coast at Lazio (Franco et al, 2004)

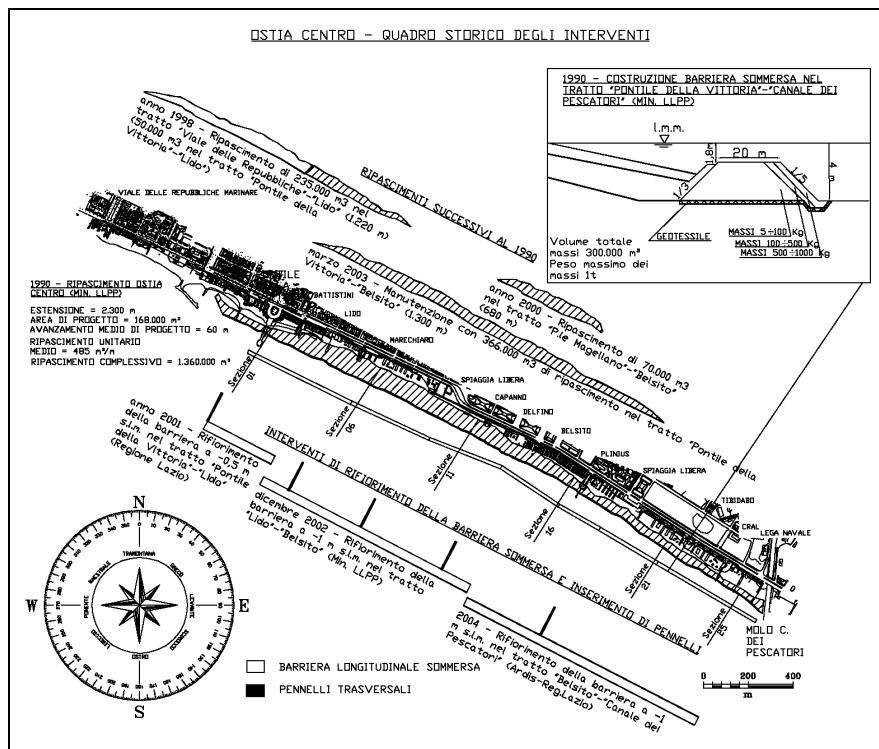


Figure 22. Protective barrier built at Ostia (Franco et al, 2004)

2.3.3.2 Beach of Casalbordino (ABRUZZO, Adriatic Sea)

In the eighties, two very long submerged barriers, separated by a gap of 120 m, were built in a depth of 4 to 4.5 m to protect the beach (*Figure 23*).

The first ('A') is about 1700 m long, with a crest level at 0.5 m below MSL; the second ('B') is about 1200m long and has a crest level of about 1.5 m below MSL. Both barriers have a crown width of 4 to 5 m.

A third submerged barrier ('C') was built in the 1990's in a water depth of 3m. Its length is 800m, with a crest 8m wide at a level of -1.5 m. The breakwater is "bracketed" by two partly submerged groynes connecting its extremities to the beach. This breakwater was built to increase protection of a sand recharge (50,000 m³) placed on the beach.

Very severe scour (reaching 12 m below MSL) occurred at the gap between the two barriers A and B, due to return (i.e. seaward flowing rip) currents.

No particular erosion effects were observed, however, for barrier C; also protection of the artificial fill is considered satisfactory.

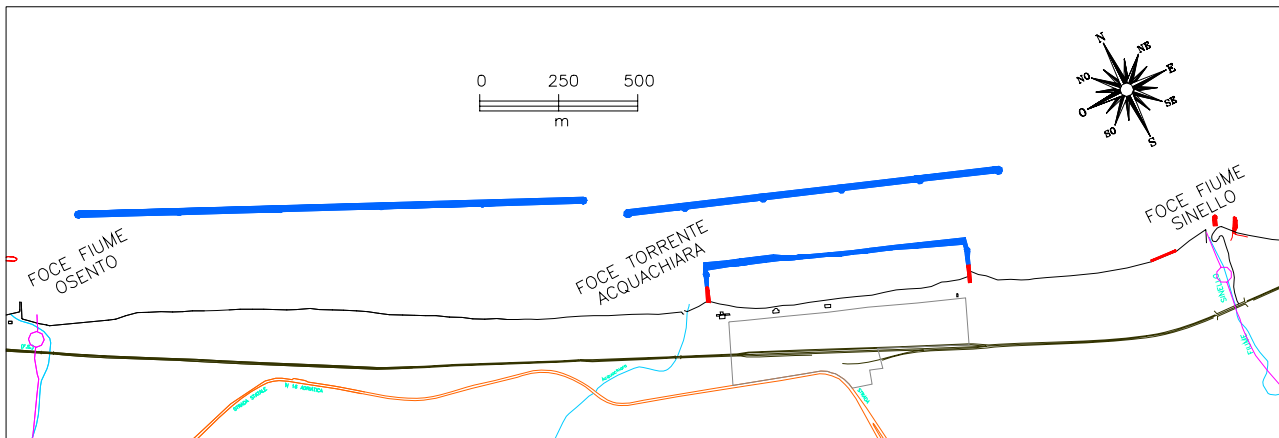


Figure 23. Protective system built at Casalbordino (De Girolamo, 2005)

4. Composite structures

4.1 Pellestrina (Veneto)

At Pellestrina, one of the two islands of the coastal belt which separates the lagoon of Venice from the Adriatic Sea (*Figure 24*), a protective beach was built in 1998 using about 4,000,000 cubic metres of sand dredged from offshore deposits.

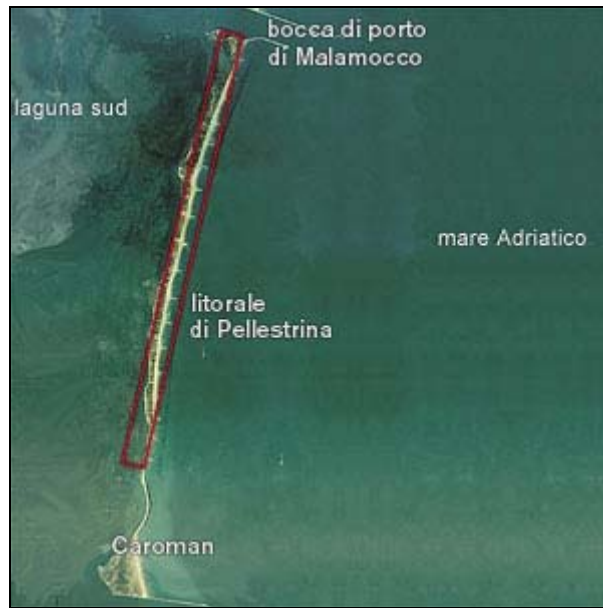


Figure 24. View of Pellestrina Island. (Consorzio Venezia Nuova).

The fill was protected by a composite structure, i.e. a system of 18 rubble mound groynes (partly emergent, partly submerged) and a submerged breakwater (Figure 25-28).

The groynes are connected to a submerged breakwater built at a distance of about 300m from the shoreline at a depth of 4 to 4.5 m; the length of the emergent part of the groynes is 150 to 200 m and their spacing variable from 400 to 500 m.

The submerged barrier, parallel to the beach, has a crest level of 1.50m below MSL.

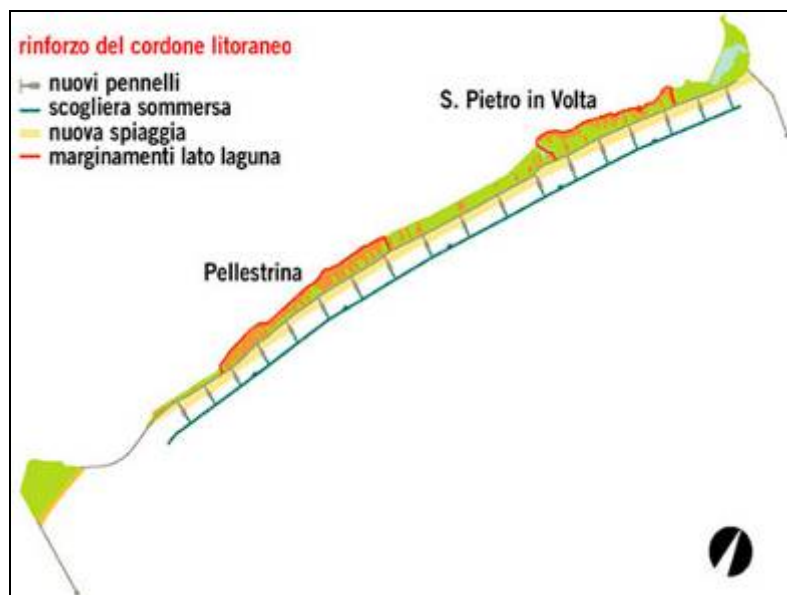


Figure 25. Protective system at Pellestrina. (Consorzio Venezia Nuova).

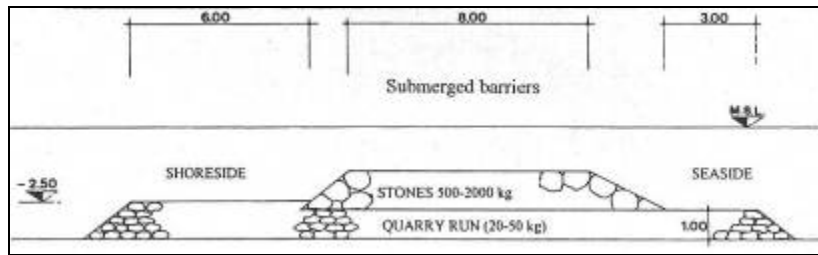


Figure 26. Cross-section of submerged barrier.



Figure 27. View of protective system at Pellestrina. (Consorzio Venezia Nuova).



Figure 28. View of protective system at Pellestrina. (*Consorzio Venezia Nuova*).

The general view of this intervention is that it has succeeded in limiting the losses of sand fill. According to *Consorzio Venezia Nuova (2005a)*, only small losses of sand were detected after about six years from the completion of the intervention (the average annual rate of loss has been estimated to be less than 3%).

No effects on downdrift beaches can occur in this case, since the system in practice protects the whole coast of Pellestrina island, which is confined by long jetties at both ends.

4.2 Lido di Dante (Emilia Romagna)

At Lido di Dante, a small resort on the coast of Emilia Romagna, a composite structure was built in 1995-96 to protect a sand fill placed on the beach.

The protective structure consists of a rubble-mound, low-crested breakwater about 800 m long (with a crest height at 0.5 m above MSL and with a gap at its centre) and three groynes (with their landward segments

above beach level and their seaward segments submerged) delimiting two square cells with a side length of about 400 m (Figure 29).

This system was thoroughly investigated by means of mathematical and physical models and prototype measurements. These revealed the occurrence of strong currents at the gap of the barrier and in the zones between the breakwater and the groynes, with resulting stability problems for the groynes.

The general view is that the efficiency (see *e.Arpa, 2002*) of this system is rather poor during high water conditions, and that erosion of downdrift beaches is also occurring.

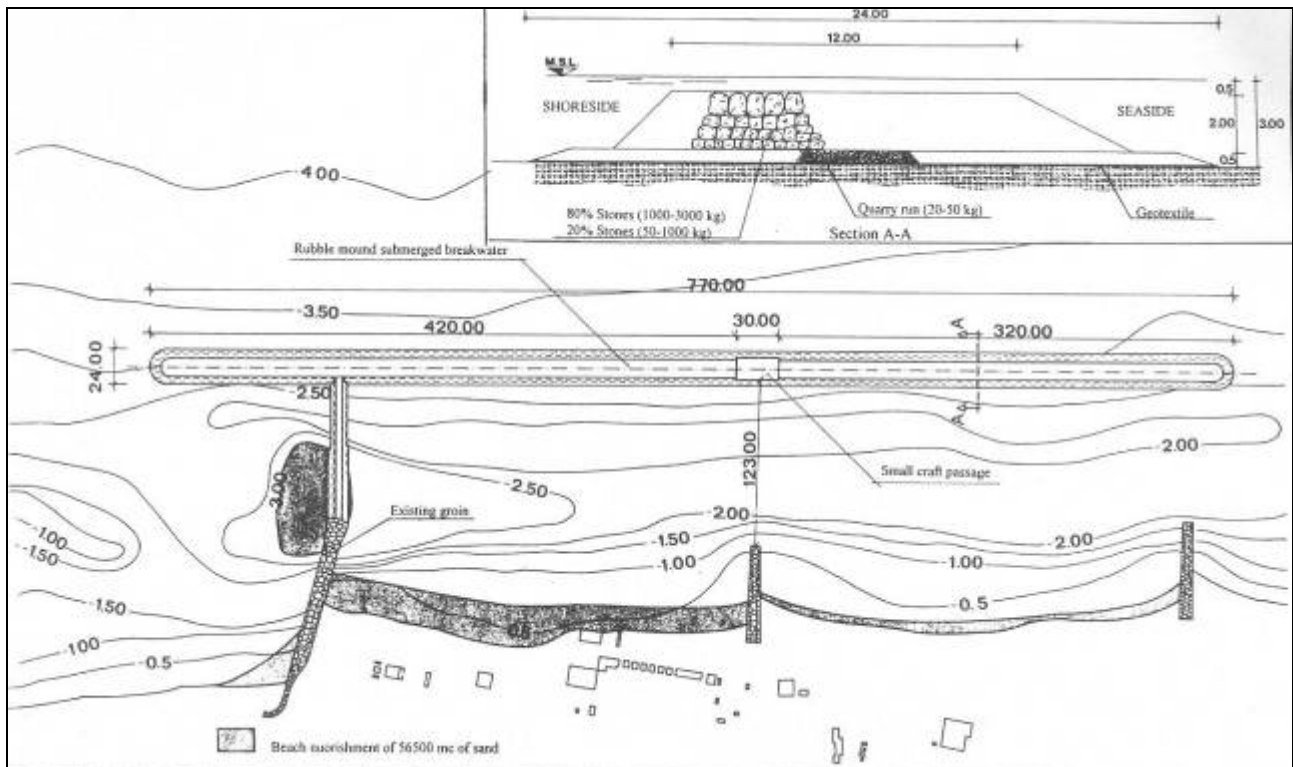


Figure 29. Protective system built at Lido Adriano (plan and cross-section of the barrier) (Lamberti and Mancinelli, 1996)

4.3 Beach of Follonica (Tuscany, Tyrrhenian Sea)

A number of emergent barriers and some groynes had been built to face severe erosion phenomena which the beach of Follonica had suffered from several years.

In an effort to rationalize the protective system, and improve it from an environmental point of view, a new protective system was designed in 2000, consisting in a long submerged barrier replacing the emergent breakwaters and three partially submerged groynes. The barrier is composed of three long segments separated by gaps. The crest of the barriers is at 0.5 m below MSL. the length of each segment is 285 m and the gap width is 40 m. The gaps are protected by submerged structures with their crown at 1.50 m below MSL. (Not yet built)

5. Concluding remarks

From above, it is evident that in Italy wide use has been made of emergent breakwaters until about the 1980's. They were usually built as segmented breakwaters (with typical lengths of about 100 m and gaps of 25 to 40 m).

The effects of these structures are rather controversial: in several cases (but not always) they have had a good degree of success in achieving an increased width for the protected beaches, but at the expense of the downdrift beaches, which were subject to erosion. Furthermore, negative effects on the quality of the water and sediments in the protected areas were generally observed.

As a consequence, in the following years several submerged breakwaters were tried, aiming at avoiding the above negative effects. These structures were frequently used to protect artificial fills placed on the beach.

At first, breakwaters with crest level well below MSL were used, particularly in Emilia Romagna, aiming at providing some degree of protection to the protected beach (by creating something like an artificial bar) without affecting downdrift beaches and the quality of water and sediments in the protected zone too negatively.

Then, structures with less submergence (about 1.5-2m below MSL) were built, but more frequently structures with their crest at MSL or slightly below.

Also, an increased width of the structure is observed in more recent designs, aiming at increasing stability of structures and greater attenuation of waves.

Submerged structures were built either as segmented breakwaters (with lengths of segments and width of gaps often similar to those used for emergent breakwaters) or as long continuous structures. In the first case, the protection of gaps with submerged structures was frequently necessary to counteract strong scouring caused by return currents.

The effects of these structures were variable; in some cases they were successful in achieving an accretion of protected beaches, although at a reduced rate compared to emergent barriers.

In general, submerged or low-crested breakwaters are still considered experimental structures and their design is considered to be rather difficult. The main problems regard their distance from the coastline, which influences greatly their effectiveness, and the occurrence of strong return currents.

After some experiences on this kind of solution, showing poor defence of protected beaches during high-water conditions, erosion problems along downdrift beaches and the creation of strong return currents and loss of sediments, the effectiveness of this kind of structure is considered doubtful.

To control return currents due to overtopping waves, transverse structures (generally, partly submerged) are associated to submerged or low-crested barriers in most recent designs: the protective system so obtained may be considered a composite structure.

This solution, aiming at limiting water and sediment exchanges with adjacent beaches, seems to be preferred for protection of sand fills with detached breakwaters.

An alternative solution is that of systems of groynes as those built in Veneto.

The latter solution is generally considered preferable from an environmental viewpoint and for use by tourists when they are accessible.

Submerged protective systems are considered preferable, instead, from an aesthetic point of view.

6. References

Aminti, P. and L. Cappiotti (2003), *Sea bottom scour near gaps in coastal protection structures*, Proc. Sixth Conf. On Mediterranean Coastal Environment, Vol.3, 1719-1730

Archetti, A., Tirindelli, M., Gamberoni, G. and A.Lamberti (2003), *Analysis of Currents around a Low Crest Barrier*, Proc. Sixth Conf. On Mediterranean Coastal Environment, Vol.3, 1731-1740

Barbanti,C.(2004) “*Modellazione e valutazione dei benefici della difesa del litorale di Pellestrina*”, Boll. AIOM, n.30, 2004

Caielli, A e Ardone, V. (2004), ‘*Programma di gestione degli interventi di difesa dei litorali veneziani*, Boll. AIOM, n.30, 2004

Consorzio Venezia Nuova (2005a), *Personal Communications*

Consorzio Venezia Nuova (2005b), www.salve.it

Consorzio Venezia Nuova (2002), Quaderni Trimestrali 2.02

Consorzio Venezia Nuova (2000), Quaderni Trimestrali, 1/2.00

De Girolamo, P. (2005), *Personal communication*

Delos, (2004), www.delos.unibo.it

Franco, L., Di Risio, M., Riccardi, C., Scalonì, P. e M.Conti (2004), *Monitoraggio del ripascimento protetto con barriera sommersa nella spiaggia di Ostia Centro*, Studi Costieri n.8

Idroser, Regione Emilia Romagna (1996), *Progetto di piano per la difesa dal mare e la riqualificazione ambientale le litorale della Regione Emilia Romagna*, Bologna,

Lamberti, A., and Mancinelli, A.(1996), *Italian experience on submerged barriers as beach defence structures*, Proc. 25th International Coastal Engineering Conference, 2352-65

Liberatore, G. (1992), *Detached breakwaters and their use in Italy*. Short Course, in "Design and Reliability of Coastal Structures", Proc. of the Short Course, 23rd International Coastal Engineering Conference, Venice, 1992, 373-395

Preti, M.(2003), *Messa in sicurezza dei tratti critici del litorale emiliano-romagnolo mediante ripascimento artificiale*, in: Il mare e la fascia costiera del'Emilia-Romagna, le reti di monitoraggio per il controllo dell'ecosistema marino costiero, Atti se. 27.9.2001, Verso la gestione integrata delle zone costiere, I quaderni di Arpa, 2003

Regione Marche, Università degli Studi di Ancona (2005), *Studi, indagini e modelli matematici finalizzati alla redazione del Piano di Difesa della Costa*, www.autoritabacino.marche.it

Progetto Delos (2004), *Final Meeting –Ostia 13.2.2004*, Boll. AIOM, n.30

Appendix 4

Beach Profile Analysis

Appendix 4 *Beach profile analysis*

1. *Introduction*

This Appendix summarises an analysis of the beach profiles surveyed by Poole Borough Council from 2002 to 2005, reviews the Bournemouth beach monitoring work undertaken by David Harlow and analyses past MHW positions against the minimum beach widths defined in the recent Coastal Strategy Study. This detailed analysis has been undertaken to identify any existing narrow sections of the beach, identify areas of erosion and accretion and extract trends to help predict future areas of erosion/accretion.

2. *Beach Profile Analysis*

A beach may be defined as ‘a deposit of non-cohesive material (e.g. sand, gravel) situated on the interface between dry land and the sea ... and actively “worked” by present-day hydrodynamic processes (i.e. waves, tides and currents) and sometimes by winds’ (CIRIA, 1996). The upper and lower limits of the beach can be taken as the beach crest (at the normal limit of wave induced run-up) and the seaward limit of sediment motion respectively. The beach volume thus includes all the potentially mobile material between the beach crest and the lower limit of wave action. Beach morphology is influenced not only by wave energy, but also by:

- Material added to the beach from slumping and mud flows from cliffs
- Aeolian processes
- The reworking of beach sediments by anthropogenic factors, such as vehicular disruption/digging

Beach profile changes occur over a variety of timescales, which vary from a single tide or storm through to seasonal variations and long term trends lasting thousands of years. Most beaches exhibit a seasonal variation in profile variability and volume in response to changing wave energies. During the summer months most beaches build up to produce a high beach with a berm above the high tide mark, and in the winter higher waves comb down the beach moving sand down to, and below, the low water mark.

The volume of the true beach material is very difficult to obtain and therefore, a measure of beach volume is found by calculating the volume of a geometrically developed beach prism, including all material (whether true beach sediment or not). The volume is calculated as volume per unit width (cross sectional area) of a shore-normal beach profile. This profile is constrained by horizontal planes at the lower limit of wave action, a vertical plane at the landward limit of the beach system (such as the beach crest, cliff toe, or seawall), and the beach surface.

Beach profiles have been collected along the Poole Borough Frontage from Sandbanks to the borough boundary bi-annually since 2002. As part of this study, survey data collected along 19 of these profiles, with locations as shown in Figure A.1, have been incorporated into the HR Wallingford Beach Data Analysis System (BDAS) and a statistical analysis carried out.

2.1 BEACH DATA ANALYSIS SYSTEM (BDAS)

The Beach Data Analysis System (BDAS) has been developed at HR Wallingford to store, recall, present and analyse large volumes of cross-section beach survey data. The main functions of the system are as follows:

- To store beach profile data, from different sites and dates, in a standard format, in a computer database.
- To add extra profile information to the database as it becomes available, with in-built data quality checking procedures.
- To recall profile data and present it "on-screen" or graphically.
- To carry out statistical analyses of beach levels, gradients, cross-sectional areas and other parameters usually as a function of time.

Cross-sections are normally repeated at different dates along the same "line"; to avoid confusion with nomenclature, we define each "line" as a "station", and generally give it a number and name (e.g. Station 1, PBCBL). Surveys at different dates are then stored together for each station, for later analysis. Apart from the surveys, a station number and title, BDAS has the capacity to store further information for each station. This information includes the National Grid co-ordinates of the zero-chainage point, the bearing (Grid North) looking seaward down the profile line, and an optional "base" profile which can show the promenade, sea wall and, if known, the level of the solid rock stratum below the beach.

For each profile, data is stored as a set of chainage-level pairs together with the survey date. The levels of the beaches along the Poole Borough Frontage have been reduced to Ordnance Datum, and chainages measured to a fixed point near the beach crest.

2.2 DATA QUALITY CHECKING

Before any calculations are started, quality control checks on the cross-sectional profiles have to be carried out. For each of the stations, BDAS itself is used to produce plots of all the surveyed profiles. Apparent errors, such as the occasional "rogue" beach level, consistent shifts in chainage values, or simple data input errors, are then identified visually, and necessary corrections made, within the computer database, i.e. without having to re-enter the data.

Further quality checks were carried out as the analysis proceeded, and the same approach to amending the data was adopted. If further information is available to correct, or confirm the data questioned in this part of the process, the database can be altered, and any analysis can be repeated, at a future time.

2.3 DESCRIPTION OF BEACH PROFILE ANALYSIS RESULTS

The primary use of BDAS is to gather together profiles from the same station, surveyed on different dates, and then carry out comparisons and statistical analyses of them. Changes over time can be separated into long-term trends, seasonal changes and short-term fluctuations. This type of analysis provides predictions of future beach changes, and a more detailed understanding of past events.

The analyses have been carried out for the following "measures" of the beach to determine the overall trends (i.e. over time) in the:

- Changes in beach levels at various fixed locations (chainages) along each profile (Figure A.1);
- Changes in the position (chainage) of the MHW line (+0.6m OND), (Figure A.2); and
- Changes in beach cross-sectional area above the 0m ODN contour, giving a measure of the changes in sediment volume on the upper part of each profile (Figure A.3).

2.3.1 *Changes in beach levels*

The most straightforward way to present such a statistical analysis of the beach data is by a "mean profile plot" for each station. In this type of plot, information is given on maximum, minimum and mean beach levels, and on the long-term rate of change in beach level, during the period considered. The long-term trend is calculated using a least-squares analysis method, and shown in metres/year upward (accretion) or downward (erosion). However for this study the long-term trend of the beach levels has been plotted in GIS, see Figure A.1, to allow an easy comparison of all the profiles along the beach and aids in the identification of areas of erosion.

Based on the analysis of the beach profiles between 2002 and 2005, there is little overall change in the beach levels along the study frontage. Small differences can be observed between the beach profiles at the western end of the frontage, which exhibit more of an increase in beach levels to those at the eastern end.

2.3.2 *Changes in MHW shoreline position*

The BDAS software can also perform an analysis on the distance to a particular beach contour and then super-imposes a "trend" line which identifies an underlying linear long-term (secular) trend using multi-linear regression methods. As part of this study the MHW line has been analysed at each profile and the calculated trend has been extrapolated between profiles and plotted into GIS, see Figure A.2. This allows a complete analysis of the changes in the MHW line along the study frontage.

Based on the results of this analysis, as presented in Figure A.2, the MHW line is advancing by up to 8 m/yr at the western end of the frontage i.e. between profiles PBCZ and PBCAP. Further east along the frontage, the MHW line continues to advance but at a lesser rate, up to 4 m/yr. Only at Branksome Chine, profile PBCBD, does the MHW line shows signs of retreat. This coincides with the location of the seawards bulge of the seawall and is a known area of narrow beach widths from site visits.

2.3.3 *Changes in beach cross-sectional area*

Another useful output produced by BDAS is the calculation of the cross-sectional area underneath each profile as it provides an indication of the beach volume. For this study the area above the 0m contour line has been calculated for each profile and output into a spreadsheet. The long-term trend was then calculated using a least-squares analysis method and the results have been extrapolated between profiles to calculate the beach volume. The results of the analysis of trend in beach volumes have been plotted into GIS (Figure A.3) for ease of interpretation.

Examination of the change in beach volumes indicates continuing accretion along the whole study frontage. The largest amount of accretion occurs at the western end, near Flag Head Chine, with less accretion occurring at the eastern end, near Branksome

Chine. Based on the analysis of the survey data along the beaches along the Poole Borough Frontage appear to be rather healthy and gaining more sand.

3. *Review of MHW shoreline positions*

The main objective that any beach control structure will need to achieve is to maintain appropriately wide beaches between Shore Road and the Borough Boundary (EX5200) i.e. that is sufficient to prevent undermining of the seawall. A minimum beach width (7m for a 1:1 year return period) has been ascribed to this frontage as part of the recent Coastal Strategy Study (Halcrow, 2004) and is discussed in more detail in report EX5200. These minimum beach widths have been plotted in GIS, along with several Mean High Water (MHW) lines provided by Poole Borough Council, and are presented in Figure A.4. The analysis of past MHW lines against the minimum beach widths allows the locations of where the beach has been narrow in the past to be identified. Based on the provided surveys the beach width at both Flag Head Chine and Branksome Dene Chine has been less than 7m in the past.

4. *Review of Bournemouth beach monitoring work*

A study into the direction of longshore drift has been undertaken by David Harlow of Bournemouth Borough Council. The work involves a simple yet effective measure of the direction of longshore drift based on a groyne's ability to intercept drift and reduce coastal erosion. Observations of sand accumulation on the updrift side of groyne has been made at the groynes along the Poole Borough Council's frontage since 1994. E is recorded when there is an accumulation of sand on the west side of the groyne is observed and indicates there is littoral drift to the East. Similarly, W is recorded when there is an accumulation of sand on the east side of the groyne, thus indicating Westerly drift. If no difference in beach levels is observed on either side of the groyne an X is recorded. This indicates there is either no drift occurring (i.e. the waves are normal to the groyne) or the groyne is inefficient at intercepting drift. Finally S is recorded if the beach is submerged and beach levels can not be observed, indicating possible low beach levels. A "good" groyne will score high amounts of E & W observations because it is successful in intercepting the drift. It will therefore show low amounts of X & S observations.

The results for the groyne observations are stored in a database and are regularly updated. For this study the observations from 1994 to 2003 have been incorporated into a GIS, shown in Figure A.5, for ease of interpolation of the spatial distribution of drift direction and most importantly identification of areas of lower beach levels. Along the Poole Frontage there were very few occasions when the drift direction was observed to be in a west direction and there are only two locations where the observed easterly drift is above 60%, at Canford Cliffs Chine and at Branksome Chine. For many of the groynes no difference in beach levels on either side of the groynes was observed. These observations indicate that the groynes along this frontage are in need of repair as they are not intercepting any drift or that in these locations there is no drift occurring. An important result from this work is the percentage of time the beach was submerged as this provides information on where the beach levels are low. From the observations made between 1994 and 2003 the beach at Branksome Dene Chine, where the seawall takes on a convex shape seawards, has been submerged for 36% of the time. This result concurs with the beach analysis above, present observations and past knowledge.

5. *Conclusions*

- The analysis of the beach profiles (2002-2005) indicates that the majority of the beach between Shore Road and the Borough boundary is currently healthy, with all sections showing an upward trend in volume. However, the number of surveys undertaken in this area is over a rather short time period and a larger dataset would be likely to provide a more realistic variation in beach levels, volumes and MHW line position.
- Based on a combination of the different beach surveys and analysis three areas where beach widths can become particularly narrow have been identified, namely Branksome Dene Chine, Branksome chine and Flag Head Chine and are discussed in more detail in chapter 4 of EX5200.

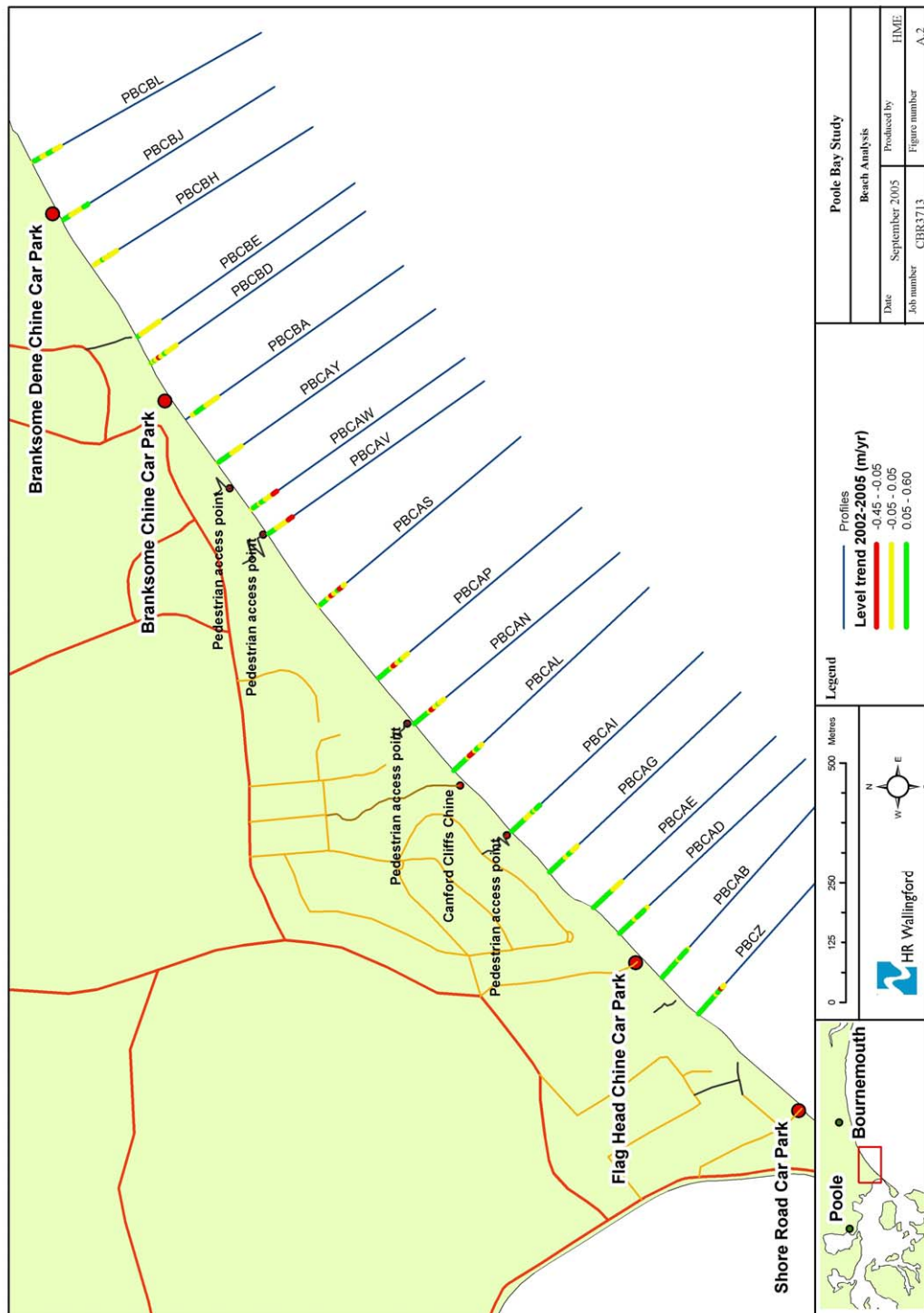


Figure A.1 Changes in beach levels at various fixed locations

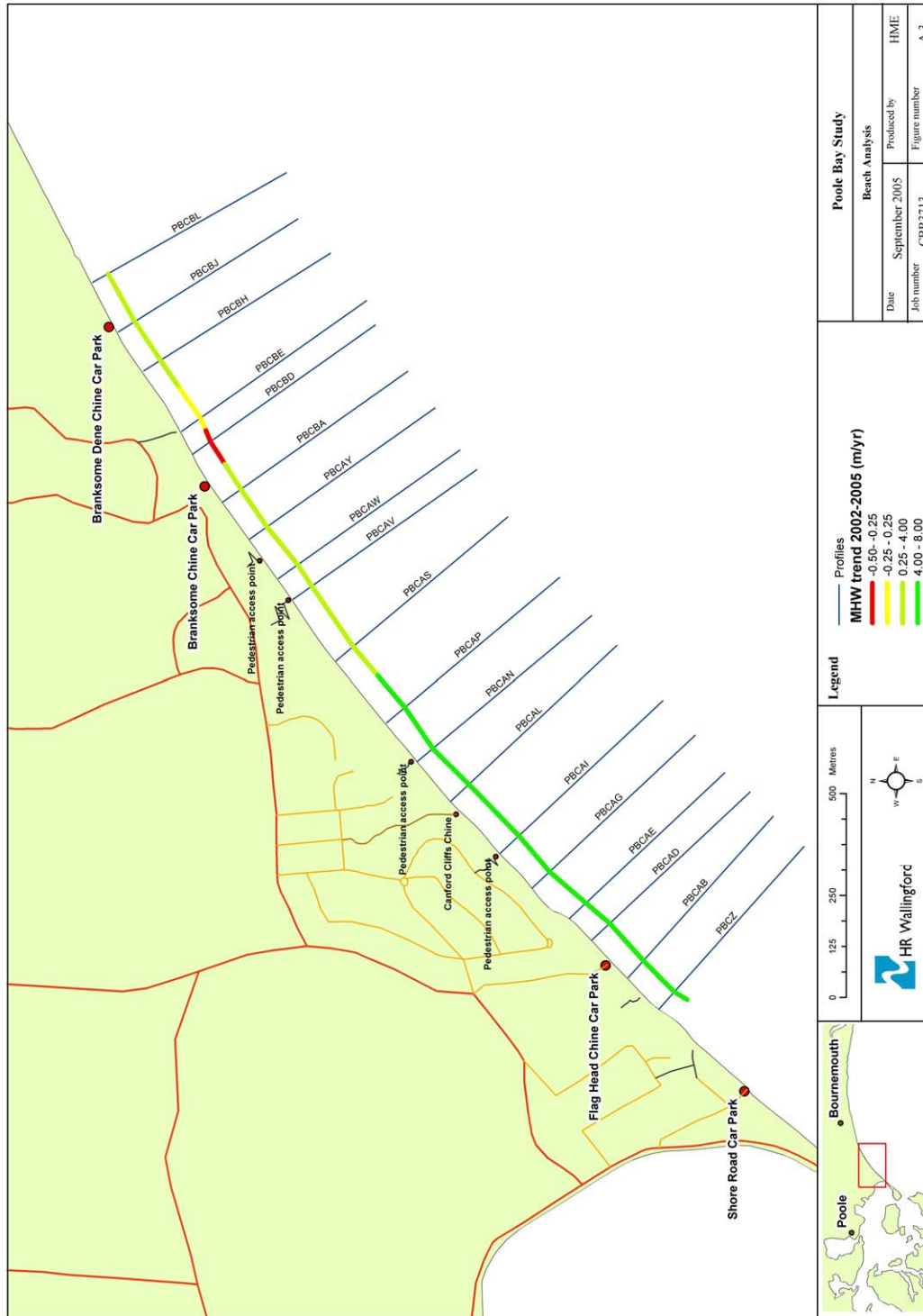


Figure A.2 Changes in the position of the MHW line (+0.6m OND)

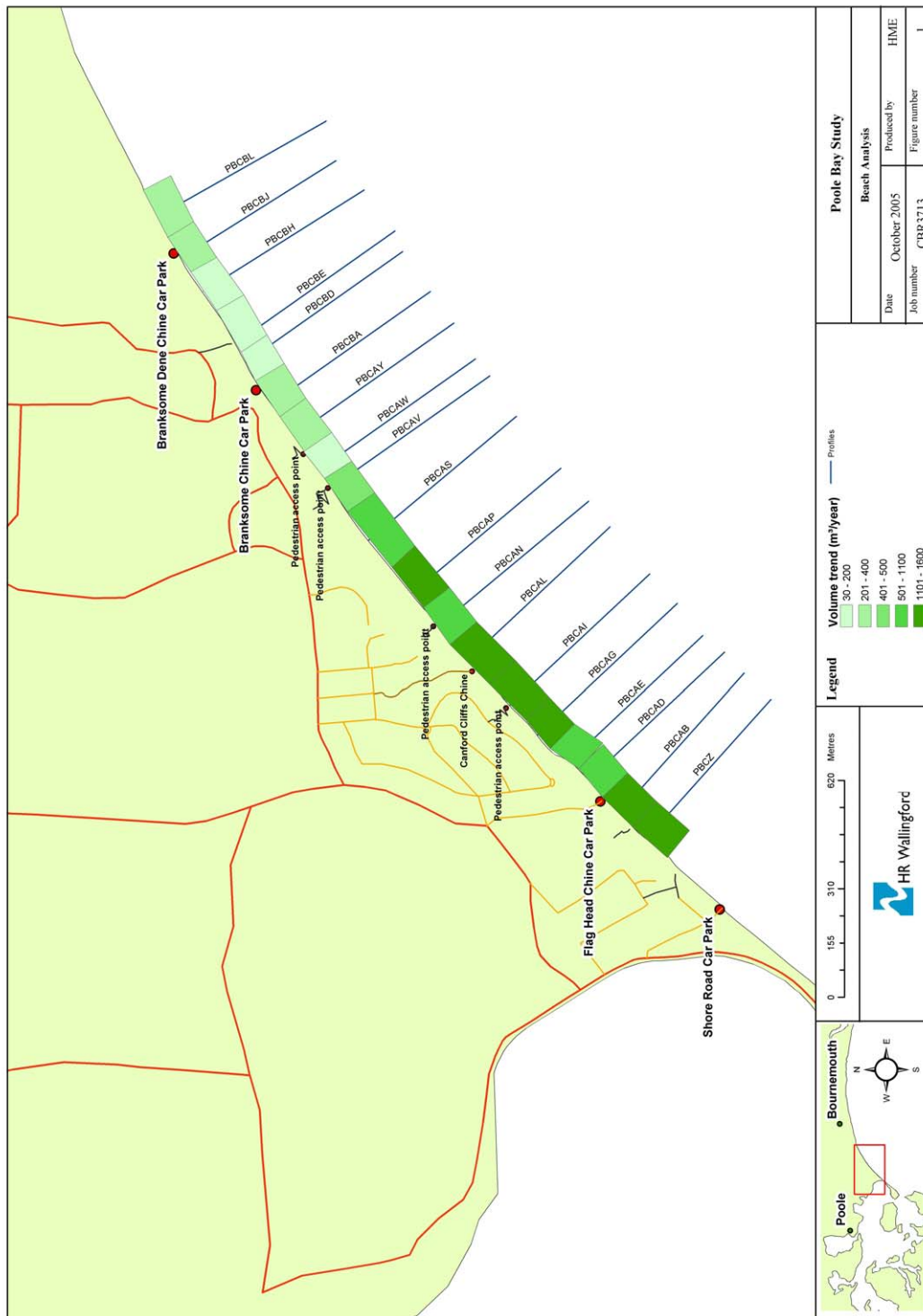


Figure A.3 Changes in sediment volume on the upper part of the beach

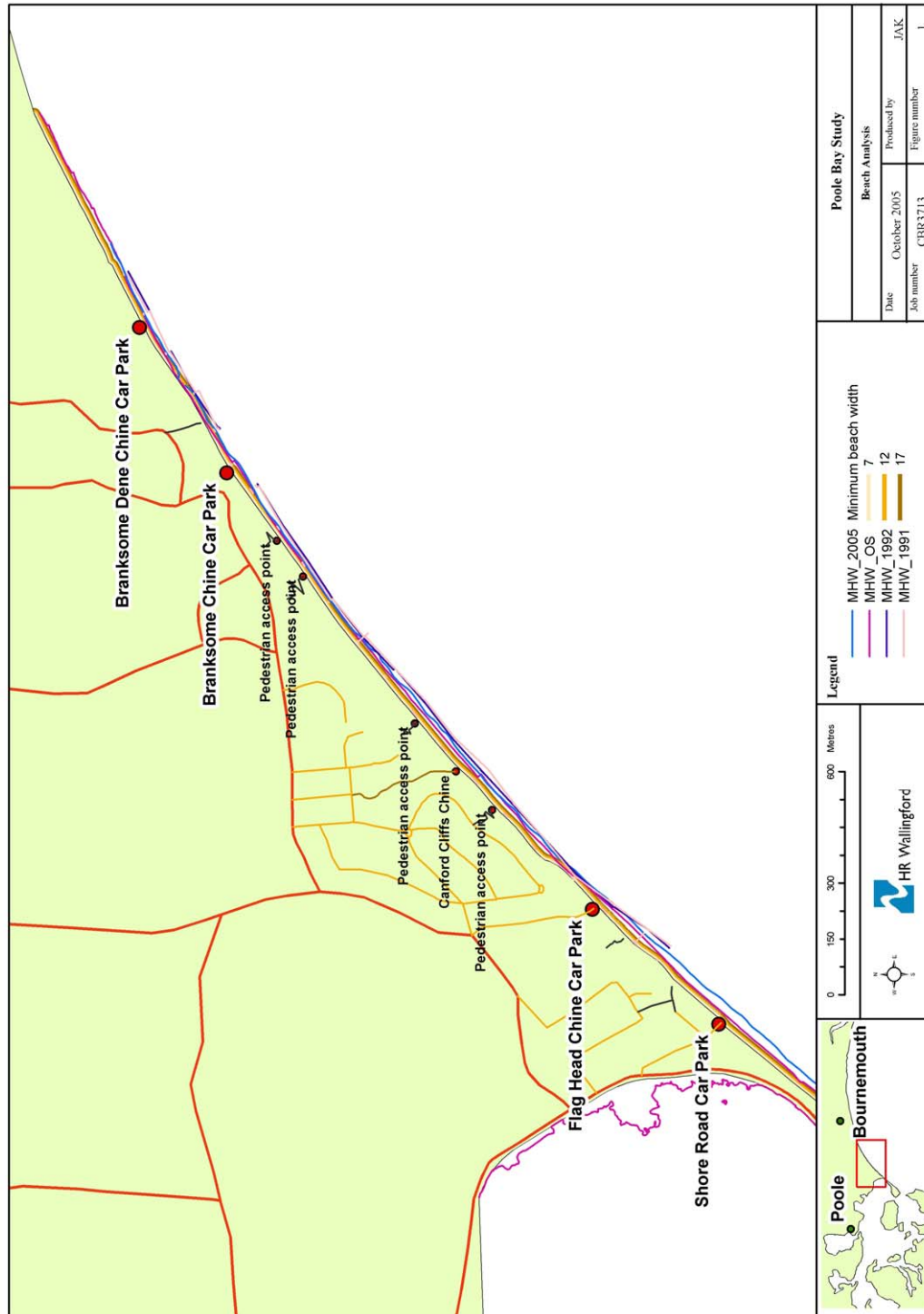


Figure A.4 MHW lines and minimum beach widths

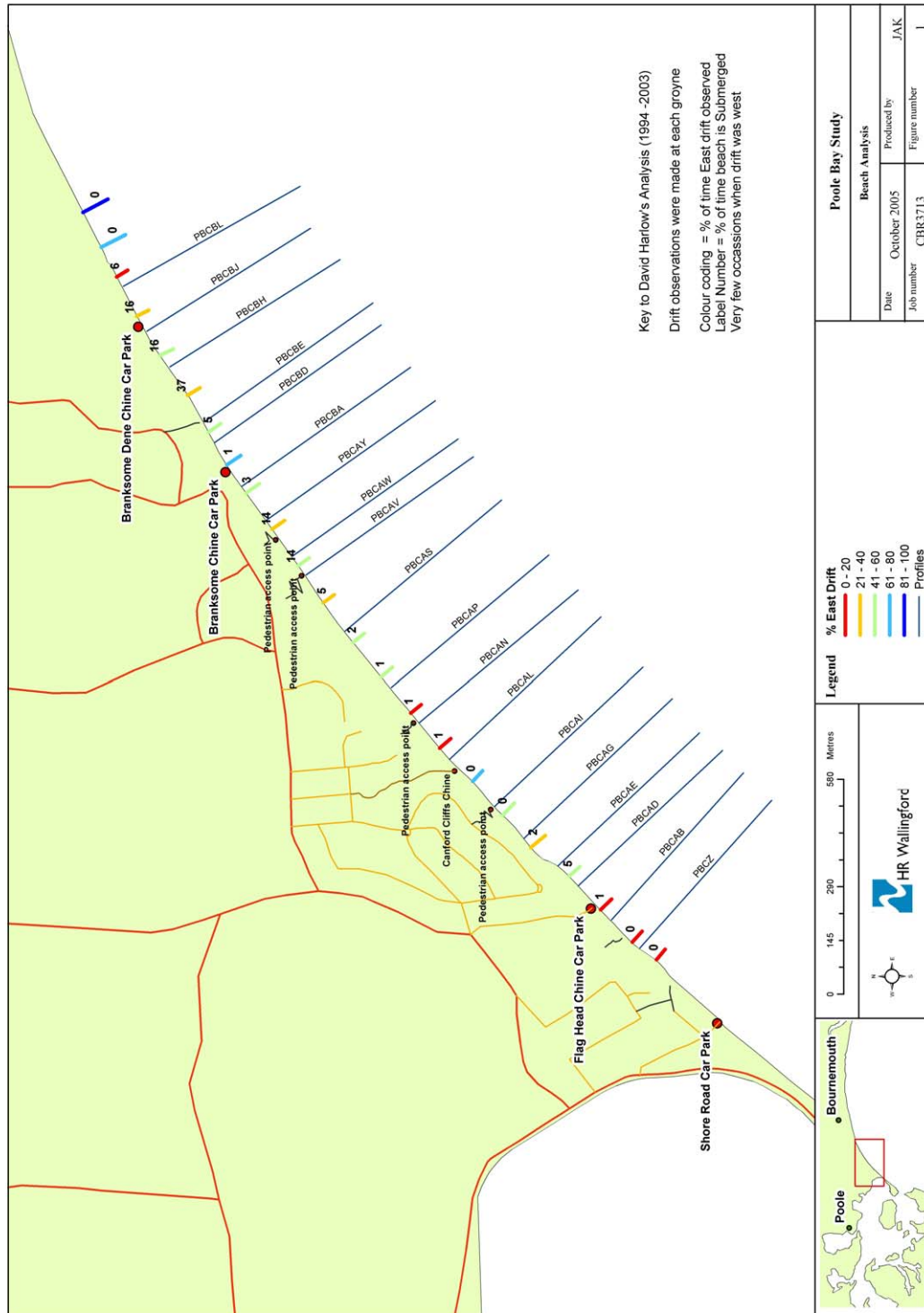


Figure A.5 Direction of Littoral Drift. Courtesy of David Harlow, Bournemouth Borough Council