Department for Environment Food & Rural Affairs

Environment Agency

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Beach modelling: Lessons learnt from past scheme performance

Project: SC110004/R2

Flood and Coastal Erosion Risk Management Research and Development Programme

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Our work includes tackling flooding and pollution incidents, reducing industry's impacts on the environment, cleaning up rivers, coastal waters and contaminated land, and improving wildlife habitats.

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Miranda Kavanagh

Director of Evidence

Executive summary

Beach recharge and management, such as re-profiling and recycling, accounts each year for several million pounds of the UK's coastal flood defence capital and maintenance expenditure. Decisions on the development of such schemes are often informed by beach modelling including numerical, physical and empirical approaches.

This document provides the technical detail behind the report on lessons learned from previous schemes that has been developed with the aim of improving understanding in this area. The research has specifically investigated the lessons that can be learned from the actual performance of schemes compared with original model expectations by reviewing case studies for 11 sites, summarised in this report, and drawing on anecdotal information gathered through engagement with industry practitioners.

This document describes the staged research approach undertaken as two distinct phases. It also provides details on the range of modelling tools and techniques available and includes the full case studies from which the lessons learned have been taken and which are summarised in the lessons learned report.

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During the project, consultation was undertaken with the following practitioners as part of initial discussions, meetings and wider workshops held in January and November 2012.

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1 Introduction

1.1 Context

Beach recharge and management, such as re-profiling and recycling, accounts each year for several million pounds of the UK's coastal flood defence capital and maintenance expenditure.

Decisions on development of such schemes are often informed by beach modelling and this guidance has been developed with the aim of improving understanding in this area by looking at past experiences. The research for this project has specifically investigated the lessons that can be learned from the actual performance of schemes compared to original model expectations by reviewing a number of case studies.

1.2 Background

In December 2010, the Environment Agency commissioned Halcrow together with Channel Coastal Observatory to undertake this project, at that time titled 'Improving Modelling Tools for Beach Management through Hindcast Benchmarking' (SC110004). During the project, this title was amended to better reflect the approach and final output. However, the main component of the study remained constant, that is, of 'comparing model predictions against reality'.

This project was not intended to provide an exhaustive examination of, or guidance for, all aspects of modelling or beach design. However, through the use of case studies it has identified potential improvements in approach and should consequently lead to better use of models and tools for beach scheme design and management in the future.

1.3 Project delivery

Delivery of this project was split into three phases.

1.3.1 Phase 1

- Establish which beach models/techniques are being used for beach design and maintenance purposes, understanding the assumptions behind their methodologies, the areas of applicability and the management/design decisions for which they go on to support.
- Design the proposed benchmark tests and identify candidate sites for testing.
- Provide recommendations on the tests to be performed in Phase 2.

1.3.2 Phase 2

- Carry out agreed benchmark tests, hindcasting real observations to provide insight into the predictive capability of models and techniques used for beach design and management.

1.3.3 Phase 3

- Produce advice and recommendations for future beach modelling to assist beach managers.

1.4 Structure of reports

The main output from this project is a document providing guidance for the beach manager or coastal engineer on the application of modelling approaches for beach scheme design or management. This guidance is a separate document to which this technical report provides supporting information, summarising the project approach including, for those interested in the finer details, the full case study assessments.

This technical supporting document is structured as follows:

- Sections 1 to 4 – summary of the work carried out to deliver this project
- Appendix A – assessment of benchmark tests considered during Phase 1
- Appendix B – process of site selection for subsequent analysis
- Appendix C – full comparative analysis case studies
- Appendix D – report on outcome of generic tests

2 Phase 1 work

2.1 Overview of approach taken

Phase 1 of the project was developed through a collaboration of expertise from various organisations.

Baseline reviews of models and tools and modelling/design approaches were conducted by the project team, consulting within their organisations with a large number of colleagues experienced in the fields of beach design, management, monitoring and modelling. Representatives from HR Wallingford were also consulted to help broaden that perspective and to provide valuable input on past schemes and approaches.

Based on these reviews, the core project teams of Halcrow and the Channel Coastal Observatory (CCO) evolved frameworks and criteria for developing and appraising benchmark tests and selecting appropriate sites for their potential application.

A project workshop was held on 19 January 2012 with the Environment Agency, local authorities, other consultants, framework colleagues and academics to engage industry expertise into the process. Current modelling tools and techniques used in beach design and management were presented and limitations discussed as a group. A preliminary list of benchmark tests and potential sites for testing were also discussed.

Feedback from the workshop supplemented the baseline reviews and provided additional information on sites, as well as providing positive feedback on the approaches being advocated for testing. The conclusion of this was to further refine the work of the project team on the proposed approach to Phase 2.

2.2 Categorisation of models and applications of modelling tools and techniques

Phase 1 included examination of the range of approaches available to use in beach design and maintenance planning.

For the purposes of this study, models and techniques for beach design and maintenance were broadly divided into three main categories:

- beach plan-shape prediction techniques
- beach profile (cross-shore) prediction techniques
- data extrapolation techniques using monitoring or historical data

Within each of these, further categorisation was made, for example, differentiating between numerical, empirical and physical models.

Information of each of these was gathered including their applicability, inputs, outputs, assumptions, strengths and limitations, summarised from sources such as the *Beach Management Manual* (CIRIA 2010), relevant studies (for example, Environment Agency 2009, 2010, 2011), and from user experience. Based on further work from Phase 2, that information was updated and incorporated into the main guidance document.

To support the provision of guidance, a fundamental requirement was for this project to establish how models and tools for beach design and management are applied now and what issues are faced by practitioners.

A large number of coastal engineers and modellers engaged in beach design were consulted, together with some of the recipient coastal managers. This knowledge was also captured and incorporated into the main guidance document.

2.3 Identification of benchmark tests

2.3.1 Approach

A key aspect of Phase 1 was to consider and, as appropriate, develop benchmark tests around the application of the range of models and methods. An assessment was made of the benchmark tests that could potentially be applied to each model or technique (see Appendix A). Each was assessed according to:

- the **relative** significance of the model approach/technique in future beach design in relation to sites/areas of spend identified in the Medium Term Plan (MTP), that is, whether that approach is likely to be commonly sought or less so (ranked 1 to 5)
- the ease/difficulties with replicating the original model approach/technique within this study
- the methods that might be employed to carry out tests (irrespective of those difficulties)
- the value that each method might deliver to this particular study, in relation to ability to generate useful outputs against level of complexity and costs to undertake (classified from very high to very low)

2.3.2 Conclusions

Consideration was given to attempting to resurrect previous models or creating new models to re-examine sites with the benefit of new information. However, this was rejected for the following reasons.

- The ability to resurrect previous models will be virtually impossible and software changes mean that some previous models are now obsolete.
- The cost of resurrecting or creating new models of a site would be disproportionately expensive to this study. It was likely that only two or three sites could be replicated and examined within the project budget.
- The relatively unique characteristics of any site meant that the details provided from a site-specific model may not be relevant to another site, reducing the wider value of this effort.

Opportunities to ‘piggy back’ off an existing scheme where models were currently being set up or deployed were also considered, but no candidate sites could be identified.

For these reasons, an approach was established whereby more sites could be examined with a comparative analysis at a broader level but from which the reasons for differences can be identified and lessons provided for future could be derived.

2.4 Site selection

To assess the appropriateness of modelling approaches, schemes which provide a range of different conditions and techniques needed to be identified. Details of the process applied to site selection and the criteria used are set out in Appendix B.

A long list of 90 beach scheme sites was initially examined, considering a range of beach parameters to characterise those sites to inform the selection process. This list was then refined considering a number of further basic criteria and eventually reduced to a short list of 26 sites for more thorough consideration.

One aspect recognised early on in this process as the most critical criteria for the final choice of sites was the availability of, and accessibility to:

- original details on the modelling/design and expected performance
- records of what had actually taken place in regard to beach management and associated activities
- details on actual beach performance such as monitoring data

2.4.1 Candidate sites

Candidate sites for performing benchmark tests covering a range of modelling/design approaches and beach types were identified considering:

- confidence in the required information being available
- complexity of the site and ability to perform the tests
- representativeness of the site (and therefore applicability of conclusions to other sites)

The latter consideration made reference to the MTP and how representative the sites selected might be of anticipated forthcoming schemes. In this respect the approach to identifying candidate sites looked at sand, mixed and shingle beaches, both with and without groynes.

Candidate site selection also established that a sufficiently wide range of modelling/techniques could be covered in the testing.

3 Phase 2 work

3.1 Comparative analysis case studies

3.1.1 Selected sites

Eleven sites were ultimately chosen for comparative analysis. These were:

- Bournemouth
- Folkestone
- Hurst Spit
- Lincshore (Mablethorpe to Skegness)
- Littlestone
- Llandudno North Shore
- Pett (Cliff End to Rye Harbour)
- Prestatyn
- Preston Beach (Weymouth)
- Seaford
- Southend-on-Sea

In addition to the comparative analyses performed by Halcrow and CCO, the team were grateful for contributions made to these from Herrington Consulting and Coastal Engineering UK. Table 3.1 provides an overview of the general information and analysis that can be found in the comparative case studies.

Table 3.1 Summary of information and analysis to be found in the comparative case studies

General information	
Location /scheme name	Location maps
Scheme date	Short description of the scheme
Client	
Approach to modelling and basis of design	
Overall approach to design	Key model runs/outputs
Wave modelling approach and data	Issues for consideration in design
Beach modelling approach and data	
Design/modelling outputs – plans for implementation	
Expected performance	Expected actions
Beach management and performance	
Actual construction	Actual beach management required
Actual beach performance	Actual environmental conditions
Comparative analysis	
Timeline/summary of actual versus predicted	
Potential reasons for differences	Potential reasons for success
Lessons for future beach modelling/design	
Lessons for others to take from this	

3.1.2 Information

A standard template was developed to capture information relevant to the project in a consistent way for each of the 11 selected sites. This template consists of the following sections and typically the information listed below as sub-sections:

Appendix C provides full comparative analysis for each of the 11 sites presented in the template format defined above (though some have been tailored to reflect specific case study comparative analysis requirements). Summaries of each of the full case studies are also provided in Appendix A of the main guidance document.

Data were obtained from a variety of sources, going back to original reports and documents where possible, but also seeking out post-design details on activities and beach performance since. Data quality inevitably varied but was generally sufficient to make a well-informed assessment of the scheme performance.

3.1.3 Comparative analysis

Comparative analysis using the available data for each site was made to identify potential reasons for any differences between actual and predicted beach performance. Aspects examined included:

- different assumptions on construction works/beach management from what actually occurred
  - volumes and locations for recharge/recycling
  - number, size and location of beach structures
  - timing of activities
  - beach material characteristics
- differences in forcing conditions
  - significant differences in wave characteristics, such as prevailing direction, total wave energy, frequency of extremes/large swell or storm events, sequencing
- design/modelling assumptions
  - availability/suitability of data at the time of design
  - appropriateness of modelling/technique(s) used
  - inadequate wider understanding

Through dissecting the test site in this manner, the significance of each aspect was explored and reasons for changes in beach performance concluded. From this independent review of the available data, lessons that could be learned of benefit to others were established.

Wave analysis

Some additional work was carried out specifically to look at a consistent comparison of wave climates across the case study sites. This was based on an assessment of Met Office modelled offshore data, which were transformed to each of the study sites, using previously established transformation models. Data were collected and analysed for the entire duration of the Met Office archive, which extends back to 1988.

A range of systematic analyses were conducted, with data subdivided into pre-scheme design and post-construction datasets for each site. In some instances data were supplemented with alternative modelled data, particularly for those schemes that predate the introduction of the Met Office second generation wave model. Additionally, analysis was conducted of available measured wave data, which included all the regional coastal monitoring programme wave buoy sites. Wave climates were generated for both measured and modelled data for those sites where these are both available.

Independent design and post-construction datasets were analysed and compared for a range of variables. The analysis was completed for both measured and modelled data where possible. Significant wave height probability distribution exceedance plots provide an inter-annual summary of wave climate variability, indicating exceedance thresholds from 10% to 0.05% (a few hours per year). The plots enable a simple year-by-year comparison of the intensity of wave conditions across the entire range of available data.

Storm calendars were generated for each site using both measured and modelled data. These indicate the temporal distribution and intensity of storm events above a defined threshold conditions for each site. These data are particularly useful for the assessment of groups of storm events to identify the most extreme measured events and periods when there has been very little storm wave activity.

Comparison plots were generated for scheme design and scheme construction that indicated percentage distribution of pre- and post-construction significant wave heights. This provides a useful indication of the pre- and post-scheme energy levels, which may assist with the explanation of differences in design stage and post-scheme sediment transport.

Similar analyses based on measured and modelled datasets give an indication of the reliability of the wave modelling method and any bias that may be evident. Similar comparisons are made between the various direction sectors to indicate whether there have been major changes in wave directions between the design and post-construction phases.

3.2 Generic tests

3.2.1 One-line beach plan shape model

In addition to the hindcast analysis, some generic tests were carried out to help inform the development of the best practice guidance.

Although Phase 1 identified that it was not appropriate to recreate a numerical model specifically for any of the candidate sites, it was determined that a non-location specific, one-line beach plan shape model would be established to review the sensitivity of a beach system to differences in key variables within a ‘controlled environment’ including:

- differences in beach material
- changes in wave climate
- impact of changes in beach nourishment (volume and timing)
- differences in scheme type (recycling, recharging, with and without groynes)

Details of this work are presented in Appendix D.

3.2.2 Literature review

Phase 1 identified studies that had made some comparisons between models and actual beach performance, or compared the merits of different assumptions used in the models. These studies were examined further and, where lessons could be extracted from them, those have been incorporated in the main guidance where appropriate.

3.3 Phase 2 workshop

As part of Phase 2, a second workshop was held on 23 November 2012 with the same people who attended the Phase 1 workshop (see section 2.1) invited again. A summary of the main lessons identified from the analysis of the 11 case study sites was presented along with proposals for the content and format of the guidance document.

Feedback from the workshop helped refine the content of the guidance document.

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List of abbreviations

ABMS	Annual Beach Monitoring Surveys
APO	annual probability of occurrence
AOD	above Ordnance Datum
AODN	above Ordnance Datum Newlyn
AONB	Area of Outstanding Natural Beauty
BMP	Beach Management Plan
BPSM	beach plan shape model
CD	Chart Datum
cSAC	candidate Special Area of Conservation
D_{50}	mean sediment grain size diameter
EIA	Environmental Impact Assessment
FEPA	Food and Environment Protection Act
GPS	global positioning system
H_s	significant wave height
MHW	mean high water
MHWS	mean high water springs
MLW	mean low water
mOD	metres Ordnance Datum
OD	Ordnance Datum
ODN	Ordnance Datum Newlyn
SAC	Special Area of Conservation
SMP	Shoreline Management Plan
SNCI	Site of Nature Conservation Interest
SPA	Special Protected Area
SoP	standard of protection
SRCMP	Southeast Regional Coastal Monitoring Programme
SSSI	Site of Special Scientific Interest

Glossary

Term	Definition
Accretion	Accumulation of sediment due to the natural action of waves, currents and wind.
Alarm level / threshold	The level before crisis level/threshold. This is usually a predetermined value where the monitored beach parameter falls to within range of the crisis level, but has not resulted in systematic failure of the function being monitored, for example, recession of a beach crest eroding to within 10 m of an asset, where it has been predetermined that an extreme storm event could result in recession of 5 m. The alarm level in this example is therefore a 5 m buffer. Increased monitoring would be required when an Alarm Level is compromised and intervention undertaken if deemed necessary. Managing alarm levels can be planned in advance.
Barrier beach	A sand or shingle bar above high tide, parallel to the coastline and separated from it by a lagoon.
Beach	A deposit of non-cohesive material (for example, sand, gravel) situated on the interface between dry land and the sea (or other large expanse of water) and actively ‘worked’ by present day hydrodynamic processes (that is, waves, tides and currents) and sometimes by winds.
Beach control structures	Beach control structures are used to inhibit or control the rate of sediment transport along the coastline.
Beach management	The process of managing a beach, whether by monitoring, simple intervention, recycling, recharge, the construction or maintenance of beach control structures or by some combination of these techniques in a way that reflects an acceptable compromise in the light of available finance, between the various coastal defence, nature conservation, public amenity and industrial objectives.
Beach Management Plan (BMP)	A BMP provides a basis for the management of a beach for coastal defence purposes, taking into account coastal processes and the other uses of the beach.
Beach manager	A beach manager seeks to maintain or improve a beach as a natural/recreational resource, or as a means of coastal protection, while providing facilities that meet the needs and aspirations of those who use the beach.
Beach plan shape	The shape of the beach in plan; usually shown as a contour line, combination of contour lines or recognisable features such as beach crest and/or the still water line.
Beach profile	Cross-section perpendicular to the shoreline. The profile can extend seawards from any selected point on the landward side or top of the beach into the nearshore.

Term	Definition
Beach recharge (nourishment)	Artificial process of replenishing a beach with material from another source.
Beach recycling/ re-profiling	The movement of sediment along a beach area, typically from areas of accretion to areas of erosion, and shaping the beach profile to have a desired crest height, width and slope.
Berm	A ridge located to the rear of a beach, just above mean high water. It is marked by a break of slope at the seaward edge.
Bimodal wave period	Related to frequency distribution of waves, for each bimodal wave periods two wave peaks are observed.
Breaching	Failure of the beach head allowing flooding by tidal action.
Breakwater	A structure projecting into the sea that shelters vessels from waves and currents, prevents siltation of navigation channel, protects a shore area or prevents thermal mixing (for example, cooling water intakes). In beach management, breakwaters are generally structures protecting areas from the full effect of breaking waves. Breakwaters may be shore-attached and extended seawards from the beach, or may be detached and sited offshore, generally parallel to the beach, to provide sheltered conditions.
CIRIA	Construction Industry Research and Information Association
Cliffing	The development of almost vertical cliffs, up to 2 m high (although generally less than 1 m) following creation of a new beach slope after beach recharge. The cliffs occur at or above mean high tide, and are a result of mixing different sized sediments and compaction of material by mechanical plant.
Climate change	Long-term changes in climate. The term is generally used for changes resulting from human intervention in atmospheric processes through, for example, the release of greenhouse gases to the atmosphere from burning fossil fuels, the results of which may lead to increased rainfall and sea level rise.
Coastal cell	Coastline unit within which sediment movement is self-contained.
Coastal forcing (forcing factors)	The natural processes that activate coastal hydro- and morphodynamics (for example, winds, waves, tides).
Cohesive sediment	Sediment containing significant proportion of clays, the electromagnetic properties of which cause the sediment to bind together.
Crest	Highest point on a beach face, breakwater or seawall.
Crest level/height	The vertical level of the beach relative to metres Ordnance Datum (mOD).

Term	Definition
Crest width	The horizontal distance measured from the back of the beach to the top edge of the beach face slope – or on a barrier beach the distance between the top of the front slope and rear slope.
Crisis level / threshold	The level at which the function being monitored, such as the stability of the beach and/or any backing structures (seawall/promenade), could be compromised and emergency remedial action becomes necessary, for example, as in the case described under alarm level/threshold above, the beach crest recedes to within 4 m of an asset that requires protection, where it has been predetermined that an extreme event could result in 5 m of recession.
Crenulate bay	Term describing characteristic plan shape of equilibrium beach formed between two fixed headlands.
Cross-shore transport	Movement of material perpendicular to the shore.
Defra	Department for Environment, Food and Rural Affairs
Depth of closure	The 'seaward limit of significant depth change' – it does not refer to an absolute boundary across which there is no cross-shore sediment transport.
Drift-aligned	A coastline that is orientated obliquely to prevailing incident wave fronts.
Drift reversal	A switch of an indigenous direction of littoral transport.
Empirical modelling	Modelling using empirical relationships.
Environment Agency	UK non-departmental government body responsible for delivering integrated environmental management including flood defence, water resources, water quality and pollution control.
Erosion	Wearing away of the land, usually by the action of natural forces.
Flood and Coastal Risk Management	Flood and coastal risk management addresses the scientific and engineering issues of rainfall, run-off, rivers and flood inundation and coastal erosion, as well as the human and socio-economic issues of planning, development and management.
Geomorphology/ morphology	The branch of physical geography/geology which deals with the form of the Earth, the general configuration of its surface, the distribution of the land, water and so on
GIS	Geographical information system
Groyne	Narrow, roughly shore-normal structure built to reduce longshore currents and/or to trap and retain beach material. Most groynes are of timber or rock, and extend from a seawall, or the backshore, well onto the foreshore and rarely even further offshore.

Term	Definition
Groyne bay	The compartment between two groynes.
Hard defence	General term applied to impermeable coastal defence structures of concrete, timber, steel, masonry and so on which reflect a high proportion of incident wave energy.
Joint probability	The probability of two (or more) things occurring together.
Joint Probability Analysis (JPA)	Function specifying the joint distribution of two (or more) variables.
Joint return period	Average period of time between occurrences of a given joint probability event.
Locally generated (wind) waves	Locally generated short period and irregular waves created by the flow of air over water.
Longshore transport	Movement of material parallel to the shore – also referred to as longshore drift.
Mean sea level	Average height of the sea surface over a 19-year period.
Mean high water (MHW)	The average of all high waters observed over a sufficiently long period.
Mean low water (MLW)	The average of all low waters observed over a sufficiently long period.
Met Office	UK Meteorological Office
Monitoring	Systematic recording over time
Nearshore	The zone that extends from the swash zone to the position marking the start of the offshore zone, typically to water depths of about 20 m.
Numerical modelling	Analysis of coastal processes using computational models.
Offshore	The zone beyond the nearshore zone where sediment motion induced by waves alone effectively ceases and where the influence of the seabed on wave action has become small in comparison with the effect of wind.
Overtopping	Water carried over the top of a coastal defence due to wave run-up exceeding the crest height.
Overwashing	The effect of waves overtopping a coastal defence, often carrying sediment landwards which is then lost to the beach system.
Physical modelling	The investigation of coastal processes using a scaled model.
Return period	A statistical measurement denoting the average probability of occurrence of a given event over time.
Rock armour	Wide-graded quarry stone normally bulk-placed as a protective layer to prevent erosion of the seabed and or other slopes by current and/or wave action.

Term	Definition
Scour	Removal of underwater material by waves or currents, especially at the toe of a shore protection structure.
Sea level change	The rise and fall of sea levels throughout time in response to global climate and local tectonic changes.
Seawall	Massive structure built along the shore to prevent erosion and damage by wave action.
Sediment	Particulate matter derived from rock, minerals or bioclastic debris.
Sediment grading	Distribution defined by nominal and extreme limits with regard to size or mass of individual sediment grains.
Sediment transport	The movement of a mass of sedimentary material by the forces of currents and waves. This can be either perpendicular to the shoreline (cross-shore) or parallel to the shoreline (longshore).
Significant wave height, H_s	The average height of the highest of one third of the waves in a given sea state.
Shoreline Management Plan (SMP)	An SMP provides a large-scale assessment of the risks associated with coastal processes and presents a policy framework to manage these risks to people and the developed, historic and natural environment in a sustainable manner.
Standard of protection (SoP)	The level of return period event which the defence is expected to withstand without experiencing significant failure.
Still water level (SWL)	The level that the sea surface would assume in the absence of wind and waves.
Storm surge	A rise in the sea surface on an open coast, resulting from a storm.
Sustainability (in coastal flood and erosion risk management)	The degree to which coastal flood and erosion risk management options avoid tying future generations into inflexible or expensive options for flood defence. This usually includes consideration of other defences and likely developments as well as processes within catchments. It will take account of long-term demand for non-renewable materials.
Swash	The area onshore of the surf zone where the breaking waves are projected up the foreshore.
Swash aligned	A coastline that is orientated parallel to prevailing incident wave fronts.
Swell waves	Remotely wind-generated waves (that is, waves that are generated away from the site). Swell characteristically exhibits a more regular and longer period and has longer crests than locally generated waves.

Term	Definition
Tidal current	The movement of water associated with the rise and fall of the tides.
Tidal range	Vertical difference in high and low water level once decoupled from the water level residuals.
Tide	Periodic rising and falling of large bodies of water resulting from the gravitational attraction of the moon and sun acting on the rotating earth.
Toe level	The level of the lowest part of a structure, generally forming the transition to the underlying ground.
Wave climate	Average condition of the waves at a given place over a period of years, as shown by height, period, direction and so on.
Wave direction	Direction from which a wave approaches.
Wave induced currents	The movement of water driven by breaking waves that create a current travelling in an alongshore direction.
Wave height	The vertical distance between the crest and the trough.
Wave hindcast	In wave prediction, the retrospective forecasting of waves using measured wind information.
Wave period	The time it takes for two successive crests (or troughs) to pass a given point.
Wave refraction	Process by which the direction of approach of a wave changes as it moves into shallow water.
Wave reflection	The part of an incident wave that is returned (reflected) seaward when a wave impinges on a beach, seawall or other reflecting surface.
Wave run-up/ run-down	The upper and lower levels reached by a wave on a beach or coastal structure, relative to still water level.

Appendix A Benchmark tests

This appendix provides a summary of the assessment undertaken to establish benchmark tests during Phase 1 of the project.

A.1 Approach for determining benchmark tests

Potential benchmark tests were identified in terms of:

- the application of the range of models and methods
- relating them to the variation in the parameters that characterise different beaches (described in Appendix B)

The benchmark tests were designed to help provide answers to questions such as:

- Has a beach performed as intended? Is the response similar to expectations?
- To what extent does the management assumed in the design process differ from what happens in reality?
- How different are the actual wave and water levels used in design/modelling relative to those that occurred?
- How significant are those differences in terms of how a beach performs?
- Does a model used provide a realistic representation of the processes simulated?

Taking due account of the practicalities and values of some of these tests, a shortened list of tests for further consideration was established.

The final choice of the tests depended on the availability of sites that were suitable for their application and made best use of the project's resources (budget).

A.2 Potential benchmark tests for each model/approach type

The benchmark tests needed to suit the approach/modelling techniques used as well as the nature of the scheme and the characteristics of each particular site. Some techniques are relatively simple (for example, empirical methods for determining cross-shore profile of shingle beaches) whereas some can be complex (for example, numerical longshore plan shape modelling with structural interaction from groynes).

Potential benchmark tests were initially identified for each approach/technique under the three main descriptors of:

- beach plan shape methods (section A.3.1)
- beach profile (cross-shore) methods (section A.3.2)
- design using measured data (section A.3.3)

Each potential test was then assessed considering:

- the **relative** significance of the model approach/technique in future beach design in relation to sites/areas of spend identified in the Medium Term Plan (MTP), that is, whether that approach is likely to be commonly sought or less so (ranked 1 to 5)
- the ease/difficulties with replicating the original model approach/technique within this study
- the methods that might be employed to carry out tests (irrespective of those difficulties)
- the value that each method might deliver to this particular study, in relation to ability to generate useful outputs against level of complexity and costs to (classified from very high to very low)

The tables in the following sections describe the tests identified and their assessment for this study (taking account of resource available to the project).

A.2.1 Potential benchmark tests for beach plan shape methods

Beach plan shape (longshore)	Ease to replicate in this study	Difficulties	What could be done?	Value to study	Significance to MTP (frequency basis)
Numerical models*	Very hard	Difficult to resurrect old software Difficult to obtain original input data Difficult to replicate site modelled Cost to achieve is prohibitive	Desk-based comparative analysis Review numerical model and design/management report output to provide information on assumptions documented and identify predicted performance.	Very high	1
			Re-run original model (noting potential difficulties).	Very low (due to cost, ability to do so)	
			Use current software to recreate previous model (expensive? data issues?).	Low (due to cost, applicability to other sites)	
			Piggy back on models already set up for another study/scheme (are there any?)	Moderate (but opportunity limited)	
			Set up a simple beach plan model to reproduce site timeline and to test design assumptions (generic)	High	
Empirical methods	Easy	Need design input assumptions and 'as built' parameters	Desk-based comparative analysis Review design/modelling report output to provide information on assumptions documented, what was predicted performance.	Very high	3
			Re-do empirical method for 'as built' scheme.	High	

Physical	Very hard	Cost of replication is prohibitive	Desk-based comparative analysis Review design/physical model report output to provide information on assumptions documented, what was predicted performance.	Very high	5

* If the previous model is recreated, it will be to put in actual conditions to see if the model replicates actual performance. If conditions are recreated with a newer version of the software, the original design will be run as well as actual conditions to ensure that the new software version replicates the new version for the design case.

Conclusions

The following benchmark tests might be considered as the best approaches for sites where plan shape methods have been used.

Numerical plan-shape modelling (ranking 1)

1. Desk-based comparative analysis of model/design predictions with actual performance – for representative sites (different material and different management types) – (value 'very high'). Simple, easy approach as long as information is available.
2. Set up simple beach plan model to test design assumptions – (value 'high'). Relatively easy to set up but more expensive to do.

Plan shape empirical methods (ranking 3)

1. Desk-based comparative analysis of design predictions with actual performance – for range of methods – (value 'very high'). Simple, easy approach as long as information is available.
2. Re-do empirical method for as built scheme – (value 'high'). Easy to set up, may not be required for all sites.

Beach plan shape physical (ranking 5)

1. Desk-based comparative analysis of model/design predictions with actual performance – 1/2 different sites – (value 'very high'). Simple, easy approach as long as information is available.

A.2.2 Potential benchmark tests for beach profile (cross-shore) methods

Beach profile (cross-shore)	Ease to replicate in this study	Difficulties	What could be done?	Value to study	Significance to MTP (frequency basis)
Numerical models*	Moderately difficult	May be problems resurrecting old software versions, age-dependant Original input data may not be available, although would be less than required than for plan shape models	Desk-based comparative analysis Review numerical model and design/management report output to provide information on assumptions documented, what was predicted performance.	Very high	3
			Re-run original model (noting potential difficulties).	Moderate	
			Use current software version to recreate previous model.	Moderate	
Empirical methods	Easy	Need design input assumptions and 'as built' parameters.	Desk-based comparative analysis Review design/modelling report output to provide information on assumptions documented, what was predicted performance.	Very high	2
			Re-do empirical method for 'as built' scheme.	High	
Physical	Very hard	Cost of replication is prohibitive	Desk-based comparative analysis. Review design/physical model report output to provide information on assumptions documented, what was predicted performance	Very high	4

* If the previous model is recreated, it will be to put in actual conditions to see if the model replicates actual performance. If conditions are recreated with a newer version of the software, the original design will be run as well as actual conditions to ensure that the new software version replicates the new version for the design case.

Conclusions

The following benchmark tests might be considered as the best approaches for sites where beach profile methods have been used.

Beach profile numerical modelling (ranking 3)

1. Desk-based comparative analysis of model/design predictions with actual performance – for 1/2 different model applications – (value 'very high'). Simple, easy approach as long as information is available.

Beach profile empirical methods (ranking 2)

1. Desk-based comparative analysis of design predictions with actual performance – for range of representative sites – (value 'very high'). Simple, easy approach as long as information is available.
2. Re-do empirical method for as built scheme – (value 'high'). Easy to set up, may not be required for all sites.

Cross-shore physical (ranking 4)

1. Desk-based comparative analysis of model/design predictions with actual performance – 1/2 different sites – (value 'very high'). Simple, easy approach as long as information is available.

A.2.3 Potential benchmark tests for design using measured data

Monitoring/ historical data	Ease to replicate in this study	Difficulties	What could be done?	Value to study	Significance to MTP (frequency basis)
Monitoring – measured data	Hard	Ability to assess other options/outcomes	Desk-based comparative analysis Review report output and assumptions to provide information and compare with actual.	Very high	2 (ongoing site management) 4 (capital scheme)
			Assess predictive techniques (if used).	High	
Historical data	Easy	Limited value of coarse datasets	Desk-based comparative analysis Review report output and assumptions to provide information and compare with actual.	Very high	5
			Assess predictive techniques (if used).	High	

Conclusions

The following benchmark test could be considered for sites designed or managed using measured data:

Monitoring measured data: management (ranking 2)

1. Desk-based comparative analysis of design predictions with actual performance and assess any predictive techniques – 1/2 sample sites – (value ‘very high’). Simple, easy approach as long as information is available.

A.3 Useful sources of additional information

BRADBURY, A.P. AND MASON, T.E., 2009. An inter-comparison of hindcast and measured wave data: implications for beach recharge design. 11th INTERNATIONAL WORKSHOP ON WAVE HINDCASTING AND Forecasting, Halifax, Canada.

BRADBURY, A.P., MASON T.E., and PICKSLEY D., 2010. *A performance based assessment of design tools and design conditions for a beach management scheme*. In *Coasts, Marine Structures and Breakwaters*: *Adapting to Change*, Proceedings of the 9th International Conference (16-18 September 2009, Edinburgh), ed. N.W.H. Allsop, Volume 2, pp. 338-351. London: Thomas Telford.

VAN WELLEN, E., CHADWICK, A.J. AND MASON, T.E., 2000. A review and assessment of longshore sediment transport equations for coarse-grained beaches. *Coastal Engineering*, 40 (3), 243-275.

Appendix B Site selection

This appendix provides a summary of the approach to site selection.

B.1 Long list of sites

To assess the appropriateness of modelling approaches, schemes which provide a range of different conditions and techniques needed to be identified.

A long list of 90 beach schemes sites was initially examined for later refinement. This was based on the Beach Recharge Inventory (see Figure B.1) compiled during the scoping stage of the *Beach Management Manual* (BMM) (CIRIA 2010), supplemented with sites identified during initial consultation with Halcrow colleagues and the Channel Coastal Observatory.

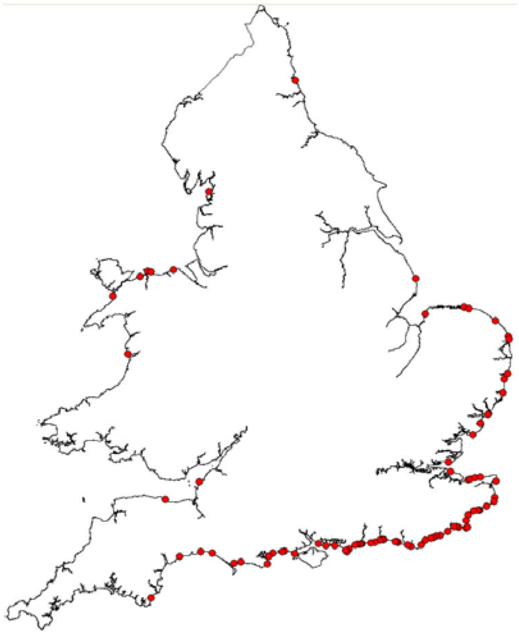


Figure B.1 Beach recharge scheme locations 2007 from the Beach Recharge Inventory (CIRIA 2010)

B.2 Short-listing criteria and selection

The initial long list was used to categorise schemes based around a set of beach parameters (see Figure B.2).

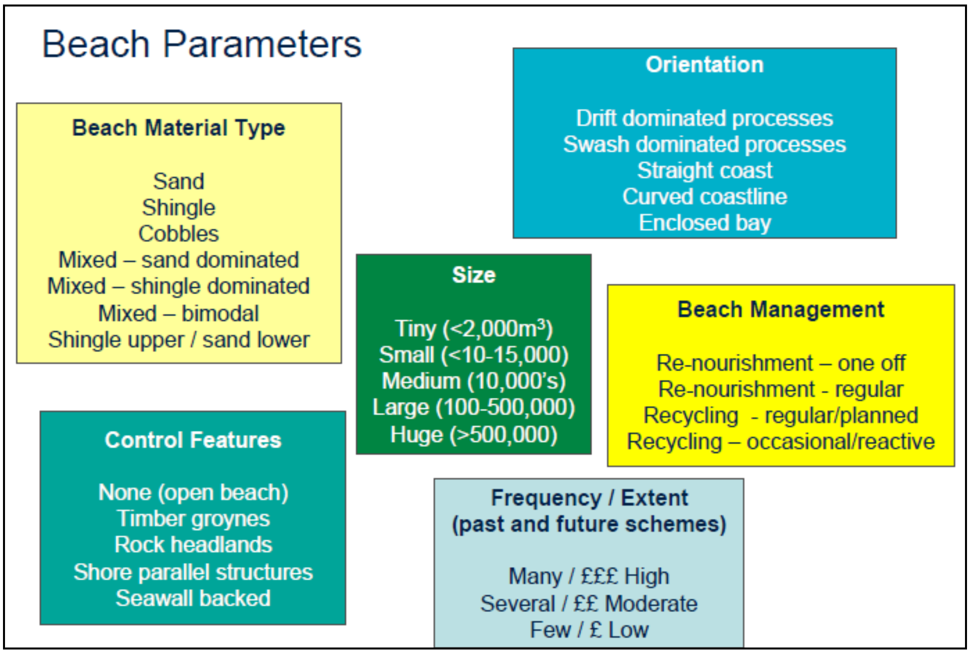


Figure B.2 Beach parameters

That long list was then refined applying basic criteria which included:

- size of scheme (remove sites where a tiny amount of work has been undertaken)
- age of scheme (include primarily schemes undertaken between 5 and 25 years ago)
- remove schemes identified as low likelihood of getting information (many of those which did not provide information to the BMM scoping study)
- open coast (remove schemes in estuaries)
- range of beach types (sought to retain a representative selection of different materials, features and so on)

Consideration was also given to the relative 'frequency' of each factor; that is, the number of beach types fitting certain parameters and how representative these would be of anticipated forthcoming schemes (Figure B.3). This framework provided a method of achieving a balance of schemes of various beach types for detailed examination. It also enabled the focus to be on sites that may be typical of those where significant expenditure (many millions of £) could take place over the next 20 years.

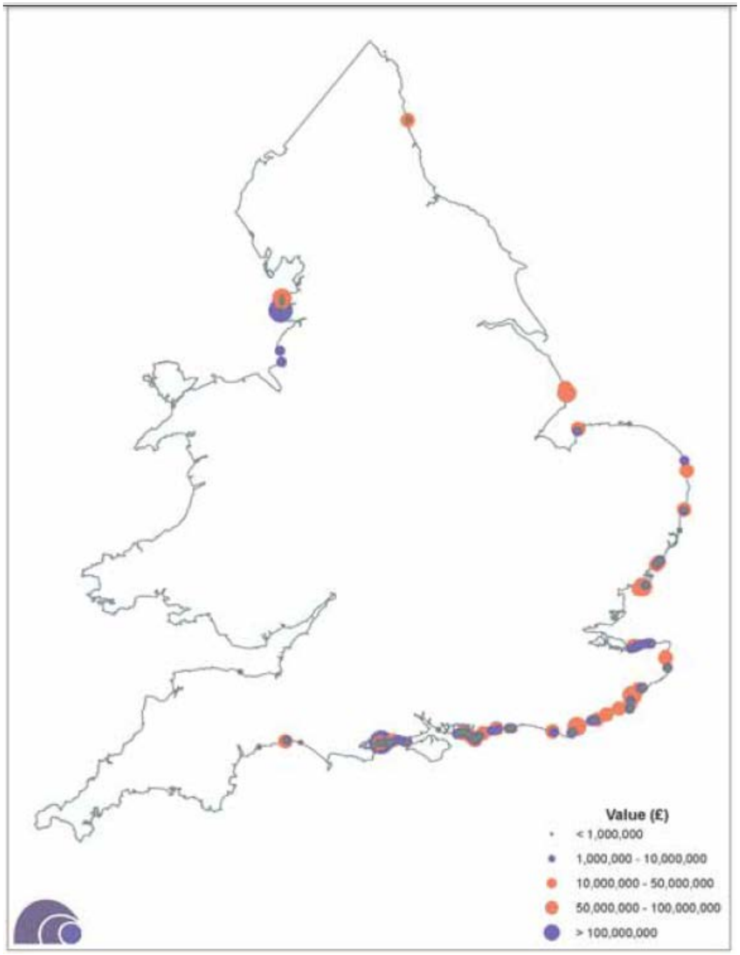


Figure B.3 Distribution of planned investment in beach management in England for period 2009 to 2029 (CIRIA 2010)

Finally, short-listing included a review with workshop attendees which helped identify where there are sufficient records of the actual intervention (for example, material quantities and properties, timing of activities) as well as what was originally designed, or particular modelling/design approaches or known performance issues.

Table B1 provides a summary of the short list of sites identified for possible testing. Where available, the table also includes the type of model/technique/approach used at each site, basic parameters, availability of information, additional information and potential contacts.

Table B.1 Short listed sites for possible testing

Site		Beach plan-shape techniques			Beach profile techniques				Sand		Shingle					Availability of information			Additional Information	Contacts
		Numerical model	Empirical method	Physical model	Numerical model	Empirical method	Physical model	Monitoring for maintenance	Open	Structures	Open	Structures	Recharge scheme size	Backed or barrier	Frontage length	Design / modelling	post construction	Monitoring		
East Coast																				
1	Mablethorpe to Skegness (Lincshore)	X						X	O				H	B	15km+	X	X	X	1994-ongoing scheme Hunstanton/Heacham to Snettisham - HR Wallingford undertook computational modelling of a mixed sand/shingle beach on the eastern flank of The Wash using TELURAY, COSMOS2D and BEACHPLAN. The modelling was used to quantify longshore and cross-shore transport for the existing beach and to assist in establishing the present standard of protection. The models were then used to optimise the design of a proposed beach nourishment scheme. (Reports EX 3809 and 3842 for EA (Anglian Region)) Monitoring 1990s onward	Halcrow Alan Brampton (HR Wallingford)
2	Heacham / Hunstanton									T			L	BB / B	3km	?	?	?	1990 – ongoing scheme EA programme 2011-12: Hunstanton / Heacham Beach Management	Halcrow
3	Clacton / Jaywick	X	X	X								R	L	B	1-2 km		X	X	1986 & 1994 scheme Was re-modelled with additional structures: Physical – HR Wallingford; Numerical – Haskoning 1998 - The sand beach at Clacton/ Jaywick is protected by fish-tail groynes but has recently suffered erosion. HR Wallingford was commissioned to use a physical model to investigate renourishment proposals and the use of rock structures to prevent further migration. (EX3805 plus addendum for EA (Anglian Region). 2003 - A 3d physical model, with a fully mobile sand bed, was used to assess the protection provided by low crest offshore breakwaters at Clacton (itself). The armour stability of the breakwaters was also assessed. Tests were carried out for storm waves from two directions) (EX4658 for Posford Haskoning / Tendring DC). Monitoring from 1990s onward	Halcrow Alan Brampton (HR Wallingford)
4	Southend	X								T			L	B	2.2km	X	X	X	2001-2 scheme BMP EA programme 2011-12: Southend beach management	Halcrow Richard Atkins (?)
South East Coast																				
5	Whitstable / Tankerton			X								T	L / H	B	3.5 km	X	X	X	Whitstable 1989 scheme (T Edwards recommends this site to test) Tankerton 1974-2004 scheme BMP – 1996 CCC Monitoring from 1980s onward, CCO since 2002 Physical modelling - HR Wallingford	Ted Edwards (Canterbury City Council)
6	Folkestone		X									R	L	B	7 km	X	X	X	2004 scheme The performance of the bays has been continually monitored since their construction and the measured plan-shape of each bay has been compared with the theoretical predictions - 2008 project appraisal report EA programme 2011-12: Hythe to Folkestone beach management Folkestone to Rye strategy plan produced (HR Wallingford EX 4142) in 2003(?) CCO monitoring from 2002 onward, also by Shepway Council	Simon Herrington (Herrington Consulting)
7	Hythe		X									R	H		4.6 km	?	X	X	1989-2004 scheme EA programme 2011-12: Hythe to Folkestone beach management CCO monitoring from 2002 onward, also by Shepway Council	Simon Herrington (Herrington Consulting)
8	Littlestone										?	?	?	B	2 km	X	X	X	2006 scheme? Design information and maintenance records available.	Tom Dauben (EA)

Site		Beach plan-shape techniques			Beach profile techniques				Sand		Shingle					Availability of information			Additional Information	Contacts
		Numerical model	Empirical method	Physical model	Numerical model	Empirical method	Physical model	Monitoring for maintenance	Open	Structures	Open	Structures	Recharge scheme size	Backed or barrier	Frontage length	Design / modelling	post construction	Monitoring		
																			Draft BMP Following completion of a major Folkestone to Rye Strategy Study HR Wallingford was further engaged to aid Babtie Brown and Root to develop a detailed scheme for 15 km of flood defence between Littlestone and Dymchurch on the south coast of England. (EX 46760 and EX4728 for Babtie Group and EA (Southern Region)) The definition of the sediment envelope and sediment transport from modelling was a problem (T Dauben) CCO monitoring data since 2002 EA hold 10 years of data	Uwe Dornbusch (EA) Simon Herrington (Herrington Consulting) Alan Brampton (HR Wallingford)
9	Dungeness Power Station	?						X			O			BB	1.7 km	X	X	X	1966 – ongoing scheme BMP – Halcrow Monitoring from 1970s onward, CCO since 2002, also Halcrow HR Wallingford modelling	Halcrow
10	Pett	X										T	L	B	8 km	X	X	X	??? 2000 scheme? BMP – 2009 Halcrow Group Ltd Good comparison with Pevensey EA programme 2011-12: Pett shingle renourishment CCO monitoring 2002 onward	Halcrow
11	Pevensey							X				T	M	B	9 km	?	?	X	1990 – ongoing scheme BMP 2009 – PCDL 1990 - Preliminary estimates of wave conditions on the Sussex coast by HR Wallingford, for coast protection design, were made using available wind data and HINDWAVE 1998 – HR Wallingford provided conditions of service curves for Broadland and Pevensey sea defences, in terms of wave height from the Met Office forecasting model and water level from the Storm Tide Warning System. Take account of existing design sea states, wave measurements, extreme water level predictions, and differences between the 80's and the 90's. 1999 - HR Wallingford was commissioned to supply HINDWAVE time series data (to Halcrow) from an offshore wave model for Pevensey established during an earlier study for the same client. EA programme 2011-12: Pevensey Bay sea defences CCO monitoring from 2002 onward, also Pevensey Sea Defences	Ian Thomas (Pevensey Sea Defences) Jonathan Rogers (Halcrow) Alan Brampton (HR Wallingford)
12	Eastbourne											T	H	B	6.5 km	X	X	X	1983-2004 scheme BMP 2010 – Halcrow EA programme 2011-12: Eastbourne beach management HR Wallingford carried out flume and wave basin physical model testing for Posford Duvivier to help design recharged shingle beach profile and to test groyne layouts EX 2870 Eastbourne Physical Model Study November 1993 T Coates. Early 2000, groynes and recharge at one end. Anecdotal evidence suggests that the scheme didn't work. Material was placed at one end to be moved naturally, but groynes worked too well and material didn't move. The modelling was not right but the design was correct. CCO monitoring from 2002 onward, also Royal Haskoning	Alan Brampton (HR Wallingford)
South Coast																				
13	Seaford	X		X							O		H	B	4.2 km	X	X	X	1987-2007 scheme Terminal groyne at west end BMP 1995/6 1986 - The Sussex Division of Southern Water commissioned a study to investigate various proposals	Andy Bradbury (CCO)

Site		Beach plan-shape techniques			Beach profile techniques				Sand		Shingle					Availability of information			Additional Information	Contacts
		Numerical model	Empirical method	Physical model	Numerical model	Empirical method	Physical model	Monitoring for maintenance	Open	Structures	Open	Structures	Recharge scheme size	Backed or barrier	Frontage length	Design / modelling	post construction	Monitoring		
																			for improving sea defences at Seaford. The scope of HR work included, field data collection &analysis, wave refraction and beach plan-shape numerical; modelling and mobile bed physical modelling. There were several follow-up studies by HR Wallingford analysing beach data and developing/ calibrating longshore drift formulae namely: EX 1768 Seaford Beach Nourishment Performance (1987-1988) EX 1986 Seaford Beach Nourishment Performance December 1989 EX 2116 Seaford Beach Nourishment Performance (1989-90): Interim Report.. EX 2533 Seaford Beach, 1987 - 1991 - Analysis of changes since renourishment, September 1992 Millard T K and Brampton A H, 1996. "The effectiveness of the Seaford Beach Renourishment Programme", Partnership in Coastal Zone Management, Proceedings of Littoral '96, Samara Publishing. EA programme 2011-12: Maintenance shingle recycling and reprofiling CCO monitoring from 2002 onward	
14	Littlehampton / Elmer / Felpham			X								R	M / L	B	4km?	X	X	X	Littlehampton 1993-4 scheme Elmer 1993 scheme Felpham 1999 schemne Mouchel 1995 EA programme 2011-12: Arun to Pagham beach management CCO monitoring from 1992 onward Dominic Reeve was involved in some recent work at Elmer (with Roger Maddrell). There is a wealth of historical data and the scheme has some very interesting	Roger Spencer (Arun Council) Alan Brampton (HR Wallingford)

Site		Beach plan-shape techniques			Beach profile techniques				Sand		Shingle					Availability of information			Additional Information	Contacts
		Numerical model	Empirical method	Physical model	Numerical model	Empirical method	Physical model	Monitoring for maintenance	Open	Structures	Open	Structures	Recharge scheme size	Backed or barrier	Frontage length	Design / modelling	post construction	Monitoring		
																			facets. For example, it was constructed to the then best guidance but it would not be designed in the same way if built now, given what new understanding has developed between now and then. In particular, the fact that there is a change in the coastline alignment within the scheme makes it interesting. Felpham – HR WAllingford carried out a study for Arun DC and the NRA comprising 2-D and 3-D laboratory modelling to design sea wall profiles, assess overtopping rates and then test the proposed beach recharge together with the planned timber, rock and fishtail groynes: Felpham Coastal Defence Study, EX 3145, February 1995. Elmer – HR Wallingford have been involved in 3 reports: Three separate reports: EX 2245 Coastal Defence Works, Elmer, West Sussex, Hydraulic Model Tests (November 1990) (for Arun DC)2-D flume tests to investigate sea wall overtopping. EX2259 Elmer 3-D physical model study (for Arun DC). Field Monitoring of Shingle Beaches at Shoreham and Elmer, West Sussex, Report TR 8, May 1996. (commissioned by MAFF and covering a University of Portsmouth study of the response to wave conditions of beaches near the breakwaters at Elmer and field work undertaken by the University of Southampton on the open (west) beach at Shoreham to measure beach response to waves and the horizontal and vertical sediment movement using aluminium tracer pebbles .	
15	Hayling Island		X									T	H	B / BB	2.5km		X	X	1985, 1997, 2002, 2007,8,9 recharge schemes BMP 1999 East Stoke Dave Harlow PhD of the design profile and volumes The design was actually built (D Harlow) 2004 – HR Wallingford undertook an investigation of	Andy Bradbury (CCO) Clive Moon (Havant

Site		Beach plan-shape techniques			Beach profile techniques				Sand		Shingle					Availability of information			Additional Information	Contacts
		Numerical model	Empirical method	Physical model	Numerical model	Empirical method	Physical model	Monitoring for maintenance	Open	Structures	Open	Structures	Recharge scheme size	Backed or barrier	Frontage length	Design / modelling	post construction	Monitoring		
																			availability and suitability of material for beach nourishment of Eastoke peninsula, Hayling Island. Sources investigated were local offshore sources and Chichester harbour Approach Channel. A target beach profile was designed and the volume of beach recharge material needed was estimated. Possible methods of implementation were reviewed and annual longshore losses were estimated using the local strategy study. EA programme 2011-12: Eastoke, Hayling Island Beach management plan; Eastoke Point, Haying Island coastal defence works. Monitoring from 1990 onward, CCO since 2002	council) Dave Harlow (Bournemouth Council) Marc Bryan (Havant Council)
16	Lee-on-the-Solent											R	L	B	2 km	X	X	X	1998 scheme CCO monitoring from 2002 onward	Halcrow
17	Hurst Spit	X		X								R	L	BB	2.5 km	X	X	X	1996 recharge scheme BMP 1997 CCO monitoring from 1987 onward, also New Forest District Council Y Li & D E Reeve: A stochastic method for predicting average beach shape, Proceedings of the ICE, Maritime Engineering, 162, p97-103, 2009. (doi: 10.1680/maen.2009.162.3.97) - A stochastic version of the equilibrium bay shape applied to New Forest frontage - assessing uncertainties in expected beach position D E Reeve & Y Li: Stochastic description of quasi-static beach behaviour, ASCE J. Waterway, Port, Coastal & Ocean Engineering, 135(4), p144-153, 2009. - A stochastic version of the equilibrium bay shape applied to New Forest frontage - assessing the variations in beach position. HR Wallingford carried out a physical model study of a Coast Protection Scheme, EX 2594 (for New Forest Council) – 2D flume study of proposed improvements to defences at distal end of spit (not particularly relevant (to beach modelling). I have a memory of a wave basin study but cannot find a reference thereto (A Brampton)	Andy Bradbury (CCO) Peter Ferguson (West Dorset District Council) Alan Brampton (HR Wallingford)
18	Bournemouth							X		T			H	B		X	X	X	1975-1990 scheme Recharge has been undertaken periodically over a number of years Bournemouth BC monitoring from 1974 onward, CCO since 2002	Andy Bradbury (CCO) Dave Harlow (Bournemouth Council)
19	Swanage		X							T			M	B	1.25 km	X	X	X	2005-6 scheme CCO monitoring from 2005 onward	Halcrow
West Coast																				
20	Weymouth (Preston Beach)	X		X								R	L	B	1.4 km	X	X	X	1995 – ongoing scheme BMP 2009 HR modelling 1993-4 Post project work Babtie 2001-2 2 different modelling studies done: 1 pre-scheme (HR Wallingford), 1 post scheme (Babtie). Two computational and two physical models were undertaken by HR Wallingford. The numerical models predicted wave conditions and then longshore transport and beach pan-shape development. The laboratory modelling studied the beach profile response and the design of a terminal groyne at the southern (Greenhill) end of the frontage. HR Wallingford report: Preston Beach Road Sea Defence: Hydraulic Model Studies. Final Report, EX 2914, September 1994 for Scott, Wilson Kirkpatrick on behalf of the NRA and Weymouth & Portland BC. CCO monitoring from 2002 onward	Andy Bradbury (CCO) Alan Frampton (Halcrow) Alan Brampton (HR Wallingford)
21	West Bay (East										O		S	B	0.5 km	X	X	X	2007 scheme	Neil Watson

Site		Beach plan-shape techniques			Beach profile techniques			Monitoring for maintenance	Sand		Shingle		Recharge scheme size	Backed or barrier	Frontage length	Availability of information			Additional Information	Contacts
		Numerical model	Empirical method	Physical model	Numerical model	Empirical method	Physical model		Open	Structures	Open	Structures				Design / modelling	post construction	Monitoring		
	Beach)																		BMP East Beach Terminal rock groyne HR Wallingford have undertaken numerous studies, some in the laboratory and some numerical starting in 1979. Those that might be relevant include: EX863 (1979) EX 1301 (1985) EX 2272 (1991) (Analysis of recent beach changes east of the harbour) EX 2524 (1992) (Numerical modelling of beach profiles) EA programme 2011-12: West Bay coast protection Beach management plan PCO monitoring from 2006 onward	(EA) Simon Hills (EA) Alan Frampton (Halcrow) Alan Brampton (HR Wallingford)
22	Slapton								X					B	2-3 km	?	?	X	No active management or scheme CCO/PCO monitoring from 2006 onward A Ruiz de Alegría-Arzaburu, A Pedrozo-Acuña, J M Horrillo-Caraballo, G Masselink & D E Reeve: "Medium-term shoreline predictions on a gravel beach using Canonical Correlation Analysis", Coastal Engineering, 57(3), p290-303, 2010. (doi:10.1016/j.coastaleng.2009.10.014) - Statistical analysis of the changes in beach orientation at Slapton Sands	Gerd Masselink (Plymouth University)
23	Minehead	X		X						R			L	B	2 km	X	X	X	1999 scheme Oblique wave climate and bi-model issues BMP HR Wallingford provided estimates of wave and tidal conditions (between 1992 and 1995) Also, in 1996, 2D flume tests were carried out to determine the optimum beach protection material. PCO monitoring from 2005 onward	Chris Hayes (?) John Pos (URS) Zoe Hutchinson (Mouchel) HR Wallingford – Alan Brampton
24	Llandudno			X						R			L	B	2km	X	X	X	1992 scheme Enclosed bay Post scheme appraisal	Alan Williams (Coastal Engineering UK) David Hall (Denbighshire Council)
25	Kinmel Bay			X					?	?			X			X	X	X	1999 recharge scheme Straight coast No active management undertaken since scheme	Alan Williams (Coastal Engineering UK) David Hall (Denbighshire Council)
26	Prestatyn									R			L	B	4 km	X	X	X	1990-2003 scheme Convex coast No active management undertaken since scheme	Alan Williams (Coastal Engineering UK) David Hall (Denbighshire Council)

Key

Sand/shingle beach type	Recharge scheme size	Backed/barrier beach
O = Open beach	H = Huge (>500,000 m^3)	B = Backed beach
R = Rock structures	L = Large (100,000–500,000 m^3)	BB = Barrier beach
T = Timber structures	M = Medium (10,000–100,000 m^3)	
	S = Small (2,000-10,000m^3)	
CCO = Channel Coastal Observatory, BMP = Beach Management Plan, PCO = Plymouth Coastal Observatory		

B.3 Candidate sites

Candidate sites for performing benchmark tests covering a range of modelling/design approaches and beach types were identified from the short list taking account of:

- likelihood of information being available
- complexity of the site and ability to perform the tests
- representativeness of the site (and therefore applicability of conclusions to other sites)

While the framework outlined above provides a mechanism for selecting a potential range of options, the over-riding criterion for selection of successful benchmark testing sites was the availability of specific information, notably:

- initial design/modelling information
- construction/post construction management information
- forcing conditions information (waves and so on)
- monitoring records

This information is critical to understand:

- the assumptions made and data used
- the initial beach design and predicted management compared with what was actually built
- the actual beach performance
- actual forcing conditions required to carry out the benchmark tests

Table B.2 gives the updated list of candidate sites considered for benchmark testing in Phase 2. From this list a set of 11 'preferred' selected sites were identified.

Table B.2 List of candidate sites

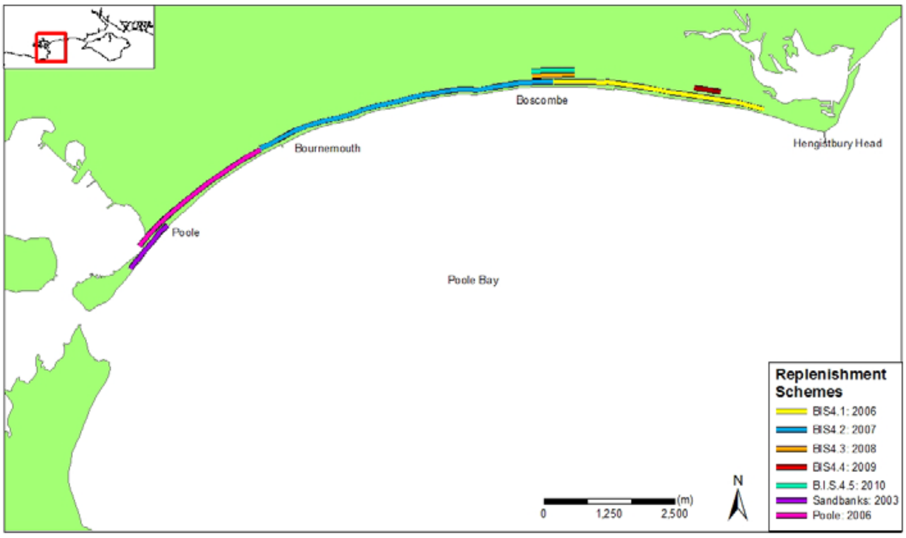
Site		
Lincshore	Pett Levels	Preston (Weymouth)
Heacham/Hunstanton	Eastbourne	Slapton
Clacton/Jaywick	Pevensey	Minehead
Southend	Seaford	Kinmel Bay
Whitstable	Hurst Spit	Llandudno
Folkestone	Bournemouth	Prestatyn
Littlestone		

Appendix C Comparative analysis case studies

Note, that with the exception of pre-and post-scheme wave analysis graphs, unless stated in the captions, all figures contained in these case studies are taken from the sources referenced to the relevant case study.

C.1 Bournemouth

C.1.1 General information

Bournemouth beach management
1974 – present day
Bournemouth Borough Council
 **Figure C.1.1 Location of site**
Background The whole of Poole Bay, on the south coast of England, is fronted by a sandy beach stretching from Sandbanks in the west to Hengistbury Head in the east (Figure C.1.1). The whole section of coastline is actively eroding as the natural supply of cliff material, which once fed the beach, has ceased due to the construction of seawalls and groynes. The seawalls are generally low at approximately 3–4 mOD, and thus the standard of coast protection and flood defence relies heavily on the level and width of the adjacent beach. The seawall structures are generally promenades with a lightweight construction which has been designed on the basis that a healthy beach must be retained fronting the walls in order to avoid seawall failure. To maintain the protection afforded by this defence, the beach is required to be periodically replenished. Extensive and high value developments lie along the cliff tops throughout the frontage. Additionally the frontage provides the basis for a locally important tourism

industry. The purpose of the defences is to safeguard beachfront and cliff-top property and infrastructure.

Bournemouth Borough Council has monitored the beach since July 1974 by surveying beach profiles. This practice continued until April 2002 when it was superseded by the Defra-funded Regional Coastal Monitoring Programme, which has continued to monitor the beach levels to the present day. During this time there have been 24 small- and large-scale beach replenishments along this section of beach, with almost 2 million m^3 of sand used for replenishment to the beaches. The continued need for replenishment was identified earlier in the Poole and Christchurch Bay Shoreline Management Plan (1999). A report by Halcrow (2004) suggests that approximately 3 million m^3 will be required over the next 50 years to maintain beach levels to a sufficient standard.

Losses at Bournemouth are approximately 70,000 m^3 of material annually (1 million m^3 every 13 years). Since the predominant direction of longshore transport in Poole Bay is from west to east, sand gradually feeds the beaches at Southbourne and Hengistbury Head to the east, and beyond into Christchurch Bay (www.poolebay.net). The beach has been replenished in several phases since 1970, with these projects referred to as 'Beach Improvement Schemes' (BIS).

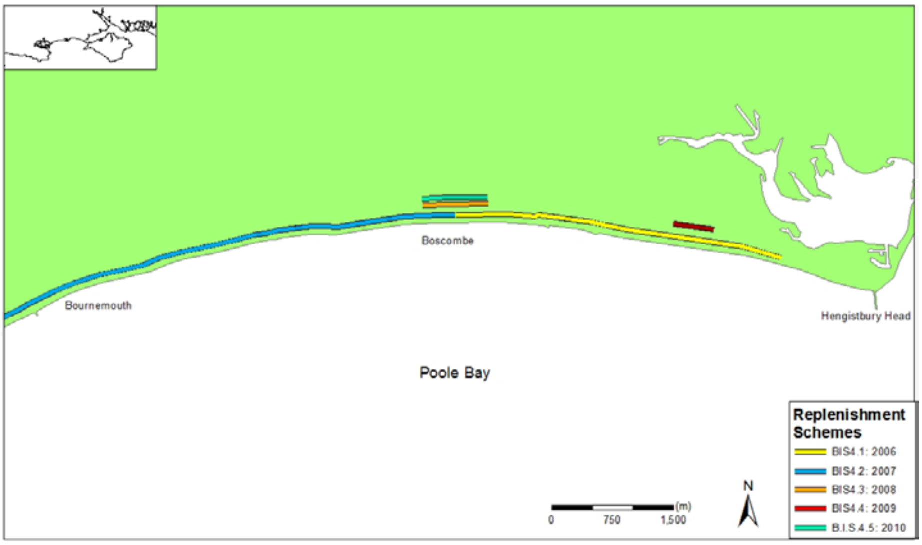


Figure C.1.2 BIS4 replenishment locations

Details of the latest beach replenishment scheme BIS4 (Figure C.1.2) are as follows:

a. BIS4.1 During the winter of 2005-2006, approximately 1.1 million m^3 of sand was dredged from Poole Harbour channels and used to replenish the nearby beaches of Swanage, Poole and Bournemouth.

b. During the winter of 2006-2007 (BIS4.2), 800,000 m^3 of sand was placed between Boscombe and Alum Chine using sand from a licensed dredge area off the Isle of Wight.

c. In March 2008 BIS4.3 took place, adding 70,000 m^3 of material to the beach between Boscombe and Southbourne.

d. In March 2009 BIS4.4 added a further 75,000 m^3 of material to the beach at Southbourne between groynes 50 and 53.

e. BIS4.5 was completed in September 2010, adding an additional 70,000 m^3 of material to the beach at Southbourne between groyne 27 to groyne 33.

C.1.2 Approach to/basis of modelling/design

Overview of approach

The approach to design of beach management at Bournemouth is based entirely on monitoring, observation and empirical relationships between the variables. All the observations have been comprehensively documented over a period of many years. No modelling has been conducted and no reference has been made to hydrodynamic conditions within the design process, although consideration has been given to wave climate observations in context with potential drift directions. Allowance has been made for anticipated changes in sea level and the projected beach volumes reflect this. There is an implicit assumption that the wave climate is not changing significantly with time, since the design reflects the performance of previous schemes since 1974. The design of recent schemes (BIS4) is based on the response of earlier schemes, constructed and monitored since the 1970s.

Design beach cross-sections have been determined empirically on the basis of previous schemes. In previous projects, the design crest was set at or slightly above that likely to be reached by wave run-up. Consequently, no waves overtopped the beach crest and the run-up formed a cliff, up to 2 m high, in the newly placed fill. This stood almost vertically for quite some time and formed a serious hazard to beach users. This proved to be such a problem that the cliffs had to be bulldozed down. Consequently the BIS4 design profile was for a crest level of 2.0 m above Ordnance Datum Newlyn (AODN), considerably lower than the natural beach crest level of about 3.0 m. This meant that the sea immediately overtopped the newly placed material and pushed up a storm beach crest, depositing it in a very ‘natural’ profile – much better than could be achieved by bulldozing.

Crisis thresholds have been determined empirically with the benefit of experience of structure failure when beach levels reached defined low levels during in 1987, resulting in undermining and seawall failure. The crisis threshold has been identified at a beach volume of 1.88 million m^3, which was the volume in 1987 when the seawall failure occurred. Alarm conditions are set at a higher level. The promenade structures at the beach head are of fairly lightweight construction with high foundations at a level of -0.5 ODN, which may become undermined if adequate beach material is not present. This requires a substantial beach in order to avoid undermining during storm events.

The historical performance data used in the design process have been used to project future losses of material. Figure C.1.3 demonstrates rapid initial losses following initial recharge, followed by a slowing of loss with time. The previous approach to replenishment schemes by Bournemouth BC has been one of ‘boom and bust’, whereby a large quantity of material is placed on the beach frontage (for example, 1974) with minimal maintenance until subsequent beach levels become sufficiently depleted to require a further significant replenishment, such as in 1988. BIS1 occurred in 1970 adding 84,000 m^3 to the beach, although no monitoring was started until 1974.

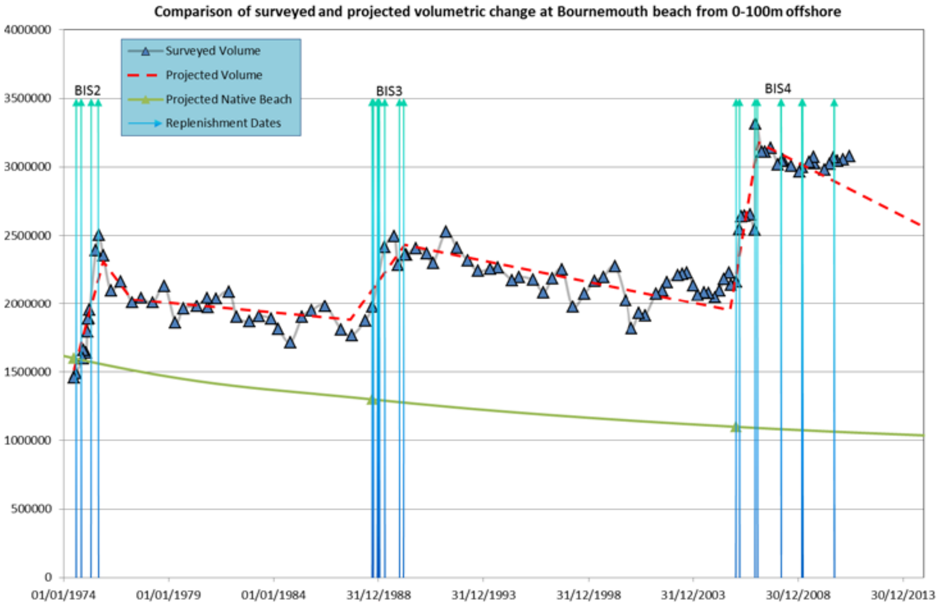


Figure C.1.3 Volumetric change to Bournemouth beach since 1974

A slight variation of this approach was adopted for BIS4.1 in 2005-2006 which was part of a much larger scheme to replenish most of the managed frontage in Poole Bay. Sand dredged from Poole Harbour and its approaches was used to replenish the beaches at Swanage, Poole and finally the Bournemouth frontage. The work was undertaken during the winter months to minimise disturbance to public access to the promenade and beach. The replenishment continued during the winter of 2006-2007 as part of BIS4.2, replenishing the beach between Boscombe and Alum Chine with a supply of sand from a licensed dredge site off the Isle of Wight. Over the following three years from 2008 to 2010, Bournemouth BC attempted to keep the beach at a consistent level and volume by replenishing that which was naturally lost with three further localised schemes BIS4.3, BIS4.4 and BIS4.5.

A permanent tide gauge is located at Bournemouth and this has provided the basis of tidal elevation data since 1996. Design has considered the extreme water levels recorded at this location, which indicates maximum storm surges of about 1 m over the 10-year period.

The sediment particle size grading for beach material was nominally the same for both BIS 2 and BIS 3; this was different for BIS4 which incorporated a range of particle sizes along the frontage. Finer material dredged from Poole Harbour and used in the Poole BC replenishment scheme, naturally filled the groyne bays to the west of the frontage, while coarser material dredged from area 451 was placed towards Southbourne end.

Beach sediment sampling has been conducted annually since 2004 at eight different locations (Figure C.1.4) along the Bournemouth BC frontage from the borough boundary to Hengistbury Head.

Figure C.1.4 Sediment sampling locations along Bournemouth

Three surface sediment samples are recorded at each location – one offshore, one from approximately high water and one from approximately low water. Material is generally coarser higher up the beach towards the promenade, while the fines are washed towards the lower beach and offshore on to the sand bar.

Groynes

The original concrete groynes from the 1970s were replaced with much more effective wooden groynes during the 1980s. Monitoring during BIS3 suggested that spacing of the groynes was not optimal. It also identified that the groyne profile was too flat for the beach geometry, with the seaward end proud and the landward end buried under the beach. This was addressed by steepening the groyne profile to increase the landward height and decrease the seaward end to match the recharge and equilibrium beach slope. During the 1990s the wide spacing was reduced with several new groynes added. In 1995 the groyne field was replaced based on monitoring data to create a standard spacing width. The groynes were buried 4 m deep in to the beach and extended out just beyond the low water mark. At present the 50 wooden groynes along Bournemouth are due to be replaced in the coming years. Rock groynes are built east of Southbourne due to the underlying bed geology containing a significant amount of iron ore, making drilling for wooden piles difficult. However, rock groynes for the whole frontage are unaffordable due to an estimated cost of approximately £1 million per groyne, plus an additional estimated £3,000 a year per groyne in maintenance.

The annual rate of loss is estimated at 70,000 m^3 based on previous monitoring data. Additional allowance for loss of material has been made on the basis of higher losses expected with initial large-scale recharge. A range of design options was considered to optimise scheme performance. These are presented below. The first option presents a solution that requires regular small recharges, which would be undertaken approximately every three years at the crisis level, indicated by the red line in Figure C.1.5. This option was considered too risky in view of previous structure failures and the potential rise in sea level which would increase vulnerability.

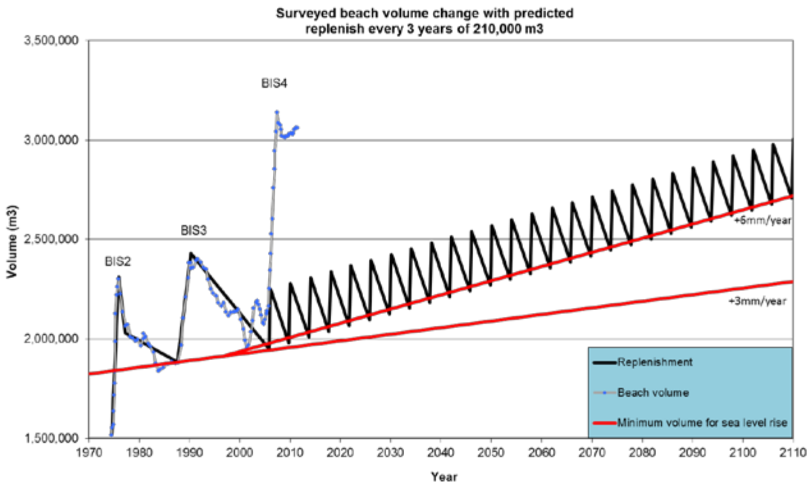


Figure C.1.5 Small-scale regular replenishment option

The second option is similar to the earlier approach, with large-scale schemes every 13 years, with large-scale losses. After a number of epochs it is seen that the standard of service has fallen to a level below the crisis conditions and this too was considered too risky (Figure C.1.6).

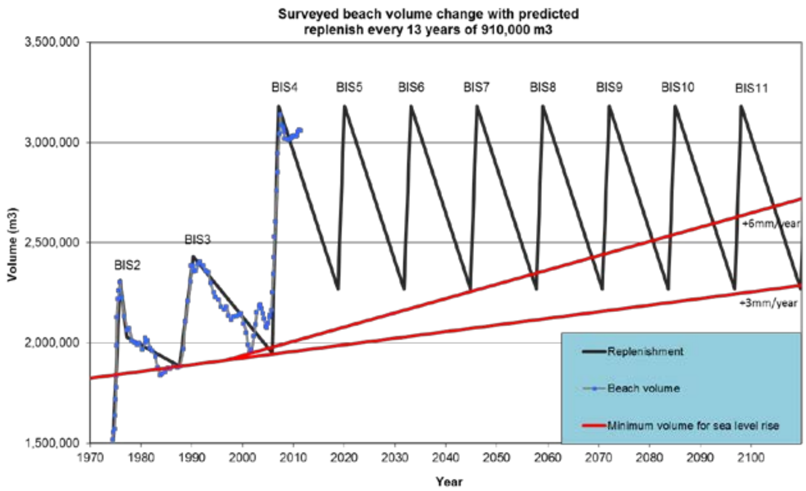


Figure C.1.6 Large-scale boom and bust approach

The third option (Figure C.1.7) makes provision for an initial large-scale recharge, which will provide a very high standard of service initially. Regular top-ups, approximately every three years, enable a high standard of service to be maintained, while keeping pace with sea level rise projections. This approach has been adopted for the BIS scheme, which commenced in late 2005.

C.1.3 Design/modelling outputs – plans for implementation

Projections for scheme performance of the BIS4 scheme were based on the monitored performance of previous beach recharge schemes. A formal beach management plan is not in place for the scheme. The scheme design is expected to provide a dynamic solution to protection of the vulnerable seawalls without risk of undermining or seawall failure. Beach management relies on a comprehensive monitoring programme in conjunction with empirical predictions to provide a decision support system for future intervention. The scheme has a design life of 100 years, during which there will be a requirement to top up the renourishment approximately every three years and to maintain or replace the timber beach-control structures.

The predicted loss from BIS4 was greater than previous measured losses as the fill volume was much greater than previous replenishments, leading to the assumption that losses would at least initially be higher.

Although it has been assumed that the wave climate would remain constant over the course of the BIS4 phase, the scheme has not been designed against defined wave conditions. Long-term allowance was made, however, to allow for increasing sea level rise.

The rate of loss of material for the BIS4 and future phases has been assumed to be at a more rapid rate (910,000 m^3 over 13 years) than for earlier schemes; this primarily provides a factor of safety within the planning process. The long-term plan makes provision for the next major recharge of 210,000 m^3 in 2015.

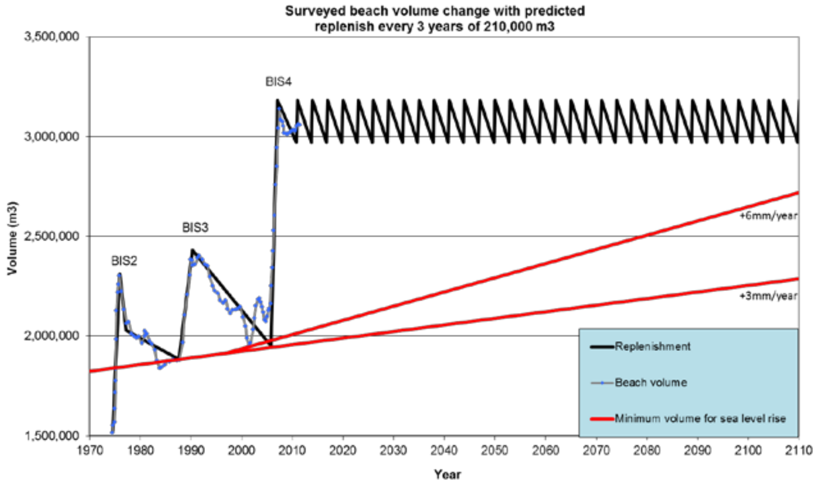


Figure C.1.7 BIS 4 design approach

Several options have been considered for replenishment schemes in the future, though a composite scheme plan may develop in the future. At present it is anticipated that levels will be kept 'topped up' every 3–4 years to maintain the optimum beach.

C.1.4 Beach management and performance

Planform development

Due to the predominant west to east drift within Poole Bay, there is a tendency for sediment to accumulate on the updrift (western) side of groynes and be eroded from the downdrift side. This effect is also evident at the piers, which effectively act as large groynes trapping sediment, particularly towards the upper beach. Some of this material is deposited during wave action, but the majority is due to wind-blown sand creating large 'sand dunes' within the vicinity of the pier (Figure C.1.8).

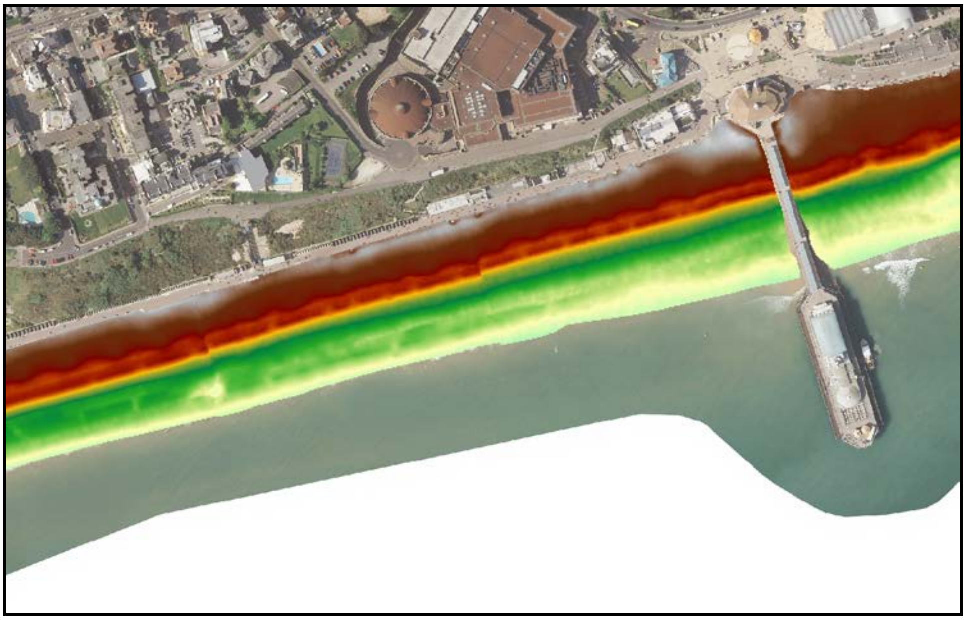


Figure C.1.8 Planform development of beach west of Bournemouth Pier

As a result of the dune development adjacent to the pier, this material was used for the first ever beach recycling event along the Bournemouth frontage. In May 2012, 4,000 m^3 of sand was excavated from this area and placed at the Poole/Bournemouth borough boundary just to the west of groyne 1.

Geotechnical responses:

Since 2006 the tops of the groyne piles have been monitored for movement by measuring the level of the middle of the seaward edge of each pile. The groyne piles were originally levelled with the top of the timber groyne board so that they were not prominent to the eye from distance. However several groynes, most notably between Bournemouth and Boscombe piers, have begun to show signs of vertical movement protruding from the boards. Vertical movement is generally concentrated from pile 5 seawards, with the greatest movement around pile 20 (Figure C.1.9). The greatest movement has been observed at the seaward most pile at groyne 15, approximately 330 m west of Boscombe pier, where a vertical increase of up to 0.56 m has occurred.

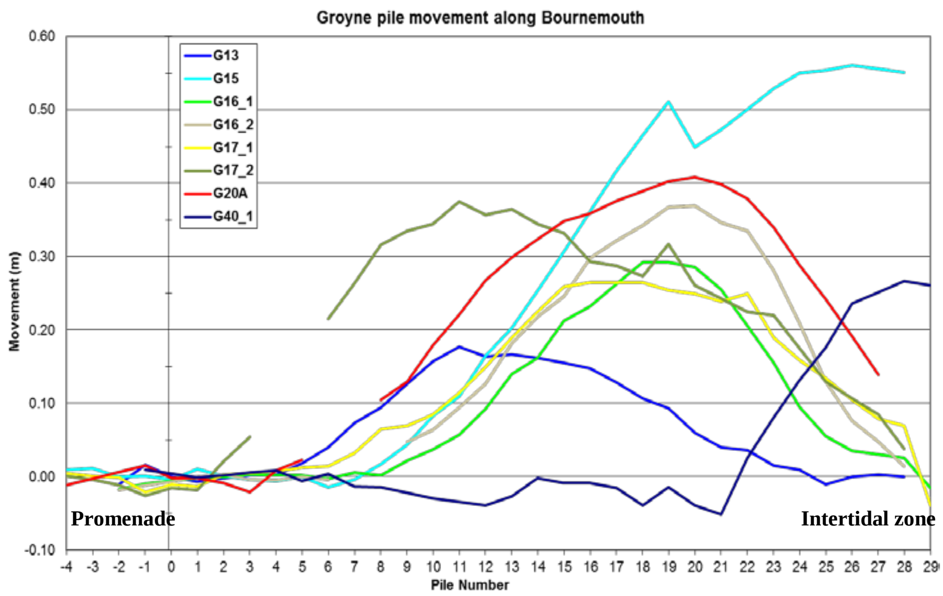


Figure C.1.9 Groyne pile movement along Bournemouth since 2006

Although a wave climate was not used in the design phase, a wave climate has been generated for this investigation to enable assessment of conditions before and after scheme implementation. Extreme wave conditions were determined for events with a range of return periods in deep water. The wave climate was transformed to a suitable nearshore location in about 10–12 m water depth. The pre- and post-construction probability distributions show inter-annual variability of measured wave conditions at the same location from 1988 to 2011 (Figure C.1.10). The measured 10% exceedance level has been comparable with the design stage wave climate (1–1.5 m); there is no evidence of a more severe wave climate with time.

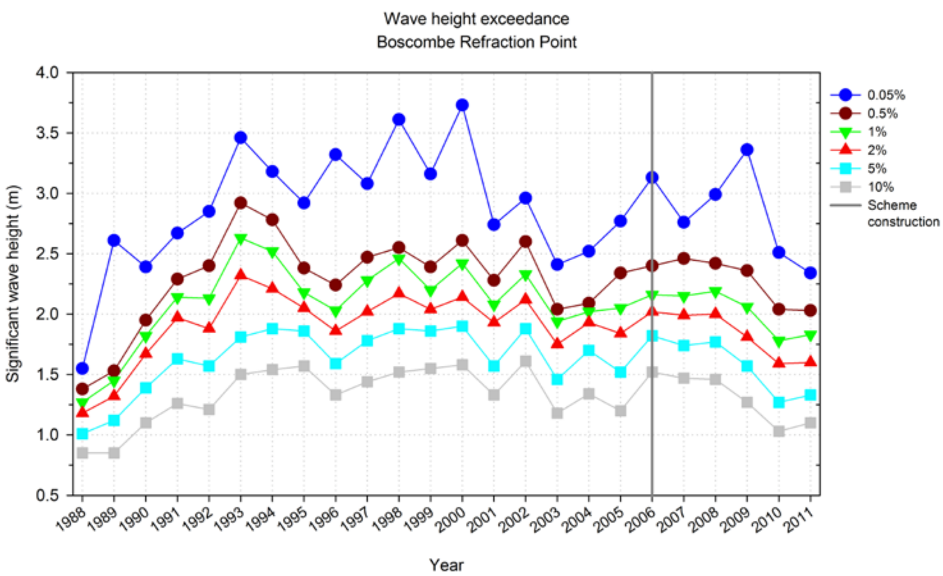


Figure C.1.10 Wave climate timeline for pre- and post-construction

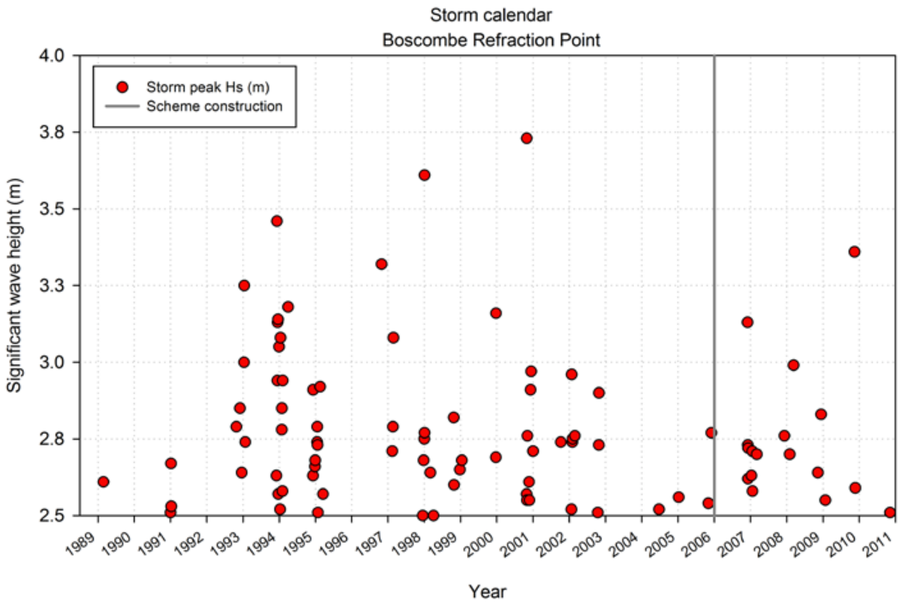


Figure C.1.11 Comparison of hindcast design, pre- and post-construction storms above 2.5 m threshold

Modelled pre-construction stage and post-construction data suggest that the post-construction storm conditions have been of similar intensity and frequency to those during earlier phases. The more severe events over a threshold of 3 m H_s have been less frequent and less intense since the implementation of the BIS4 scheme (Figure C.1.11) than during the previous 15 years. The nominal 1:100 year return period conditions established for the design stage has not been exceeded.

Modelled data distributions of pre-construction stage and post construction comparisons of significant wave heights are shown for the Boscombe wave buoy site (1988-2006 and 2006-2011) (Figure C.1.12). Comparisons show the percentage of wave heights within each height band. The plot shows that hindcast post-construction conditions (2006-2011) were generally less severe than those occurring during the design period. This is likely to have impacted on longshore transport rates and sediment losses, since the differences occur over the more energetic range of conditions.

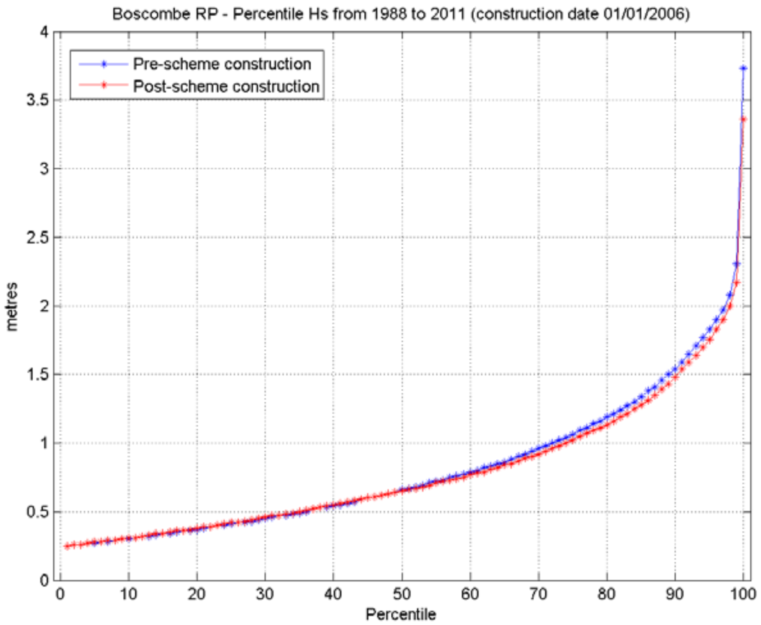


Figure C.1.12 Modelled percentage data distributions of pre-construction stage and post-construction significant wave heights for the Boscombe wave buoy site (1988-2005 and 2006-2011)

While similar comparisons cannot be made between the pre- and post-construction measured and modelled data, synoptic post-construction comparisons of measured and modelled wave data are shown for the Boscombe wave buoy site (2004-2011) (Figure C.1.13). The measured data were compared with transformed data from the Met Office 25 km wave model (2004-2011). Significant differences in wave climate characteristics are evident between the modelled and measured wave conditions. The models typically over predict significant wave height (H_s) conditions when H_s < 2 m. The bias in the Met Office modelled data is quite prominent.

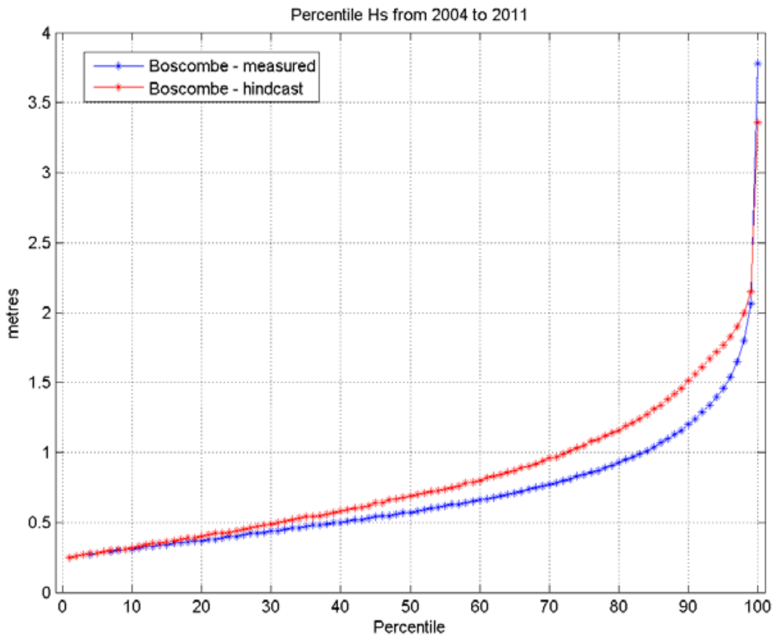


Figure C.1.13 Percentage distribution of modelled and measured post-construction significant wave heights

An event-by-event comparison is made of predicted and measured H_s for individual storms; these all lie within the highest 1% of conditions at the site. A comparison of measured and modelled storm events is shown for events that have occurred since construction. While conditions between 2 and 2.7 m are scattered either side of the perfect correlation line, events above a threshold condition of H_s = 2.7 m (Figure C.1.14) confirm the general observation identified within the bulk statistics, that extreme conditions with measured H_s > 2.7 m are generally under represented by modelling.

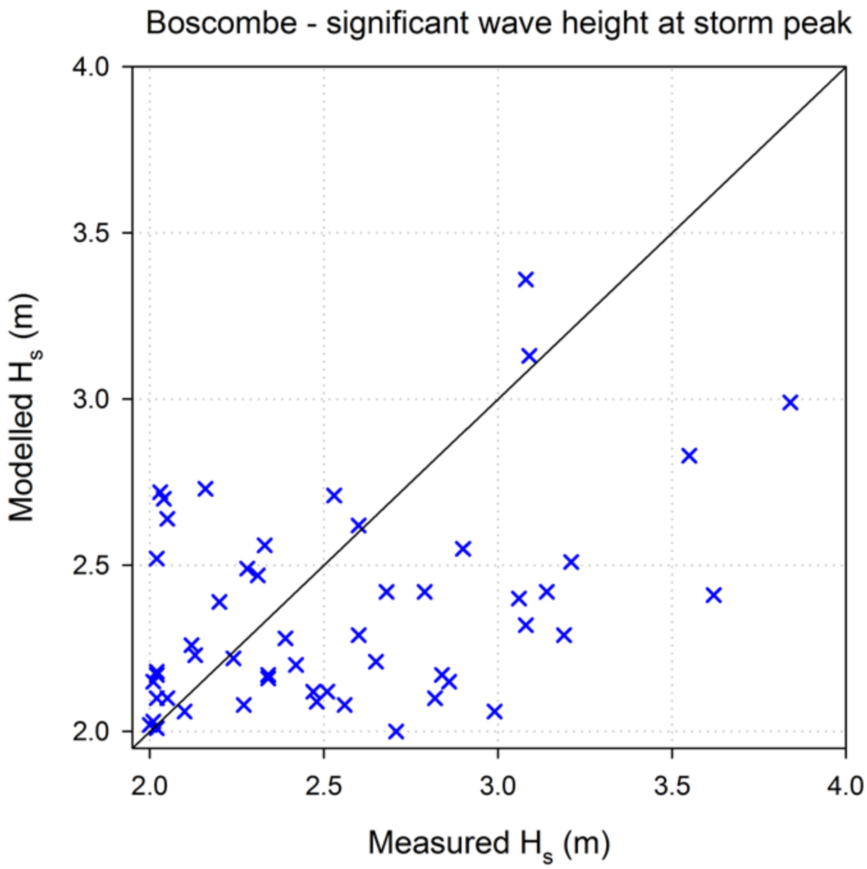


Figure C.1.14 Comparison of modelled and measured storm events above a threshold of 2 m

For the purposes of consistency, the conditions are shown for the standard three-hourly wave records. The three-hourly record is an historical artefact, originally limited by data considerations, but is now widely used. The wave buoys are, however, able to resolve wave conditions on a 30 minute basis. It is clear from records that the measured storm peaks with duration of 30 minutes may be significantly higher than the three-hourly values. While some difference might reasonably be expected, the differences are such that the 30 minute records are likely to result in significantly different beach responses, since the beach can respond rapidly over a 30 minute cycle.

Although Figure C.1.12 suggests that conditions have been generally more severe during the pre-construction phase, translation of these data into a form that is useful for assessing sediment transport variability requires consideration of the combined distribution of both wave height and direction. A comparison of pre- and post-construction percentage distributions of observed wave height and direction is presented in Figure C.1.15. Regrettably the pre-scheme observations cover a period of only 2.5 years.

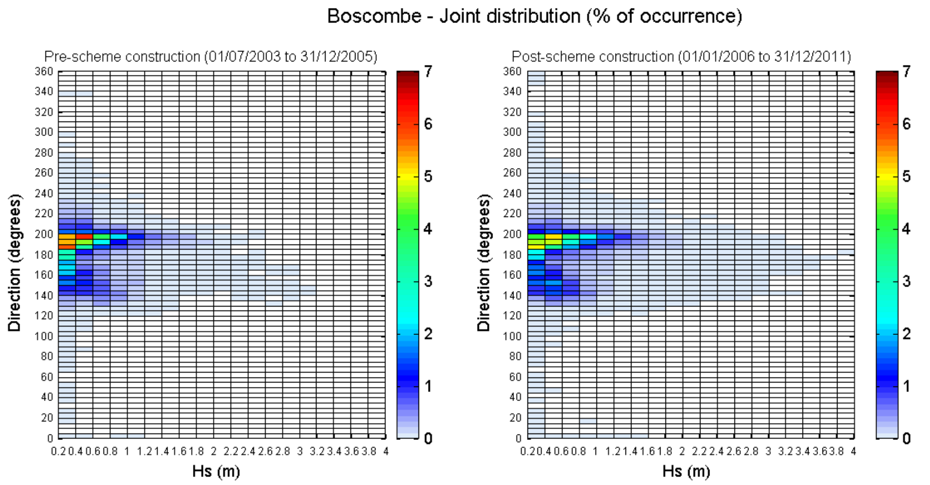


Figure C.1.15 Comparison of pre-construction percentage distributions of observed wave height and direction.

The general shape of the distributions is comparable, but the frequency and proportion of conditions in the 190–200° direction sector is sufficiently different to have a potential effect on sediment transport rates. Based on observations of the beach orientation, and on complementary observations of drift determined by assessment of build-up of material against timber groynes, it is suggested that wave directions with direction >173° will result in easterly drift. The observed data contain a higher proportion of wave data driving material to the east in the pre-scheme datasets, suggesting that drift rates should be proportionally higher over this period.

If the measured and modelled wave data is comparable the graphs (Figures C.1.15 and C.1.16) should show similar patterns. This is not the case. The somewhat curious directional distribution of wave direction and height (Figure C.1.16), in comparison with the observed data, can be explained by the modelling limitations arising from the offshore data that have been transformed to the nearshore site. First, the distribution of wave heights is consistent with previous observations that suggest a bias in the modelling of significant wave height, resulting in more severe modelled wave heights than observations indicate; this is apparently the case for conditions at this site for H_s values below about 2 m and which accounts for a very high proportion (99%) of all conditions at the site. Similarly, the offshore hindcasts present resolution problems for the offshore wave direction. The Met Office 25 km offshore wave model subdivides the wave energy spectrum into 16 direction sectors, each representing a coarse band of 22.5°. However, the wave transformation models spread the directional data over a broader range of directions as a result of refraction. The coarse offshore resolution therefore results in banding of the datasets about the key direction sectors and does not present a true picture of nearshore distributions of direction. This is highly significant to sediment transport modelling. By contrast the resolution of the measured wave data at the nearshore site is much finer (1.5°) and is represented by bins of 5° in Figure C.1.15; this represents a much truer picture of the relative distribution of wave height and direction.

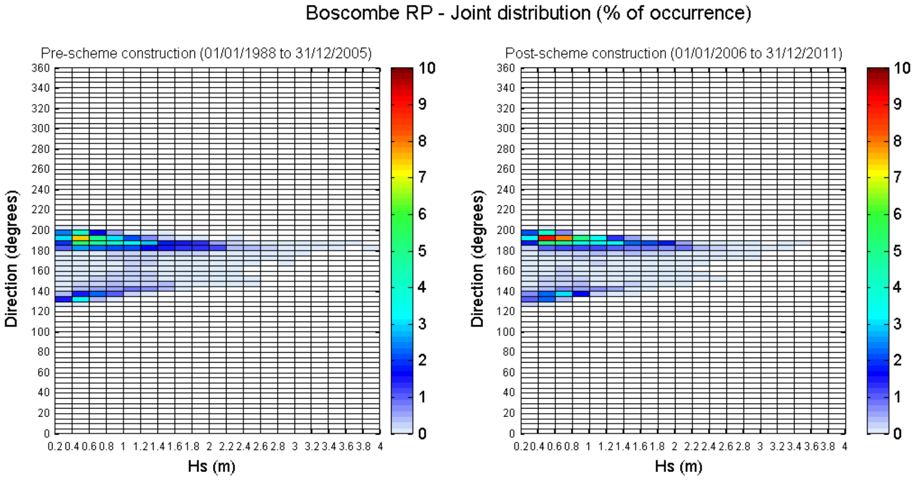


Figure C.1.16 Comparison of pre- and post-construction percentage distributions of modelled wave height and direction.

Despite the coarse directional resolution of the modelled data, the two graphs show comparable trends, although the proportion of data sending material to the east (>173°) is greater post-scheme.

A summary of replenishment activities undertaken at Bournemouth beach frontage from 1970 to the present day is shown in Table C.1.1.

Table C.1.1 Summary of replenishment activities

Scheme	Year	Quantity of sand (m^3)
BIS1	1970	84,000
BIS2	1974-1975	1,400,000
BIS3	1988-1989	1,000,000
BIS4.1	2005-2006	600,000
BIS4.2	2006-2007	898,000 (includes 15,000 m^3 stockpiled for Boscombe surf reef)
BIS4.3	2008	70,000
BIS4.4	2009	70,000
BIS4.5	2010	70,000

As a result of windblown dune development adjacent to the pier, this material was used for the first ever beach recycling event along the Bournemouth frontage. In May 2012, 4,000 m^3 of sand was excavated from this area and placed at the Poole/Bournemouth borough boundary just to the west of groyne 1.

Emergency works have not been necessary as the alarm thresholds have not been reached.

The rate of shingle loss was estimated at 70,000 m^3 during the first year, allowing for initial adjustment losses; this was very close to the actual performance. Timber groynes have been maintained and modified on an incremental basis over a period of time; this approach has enabled the design of structures to be modified. A total of 11 slightly different groyne patterns have been adopted over a number of years.

C.1.5 Comparative analysis

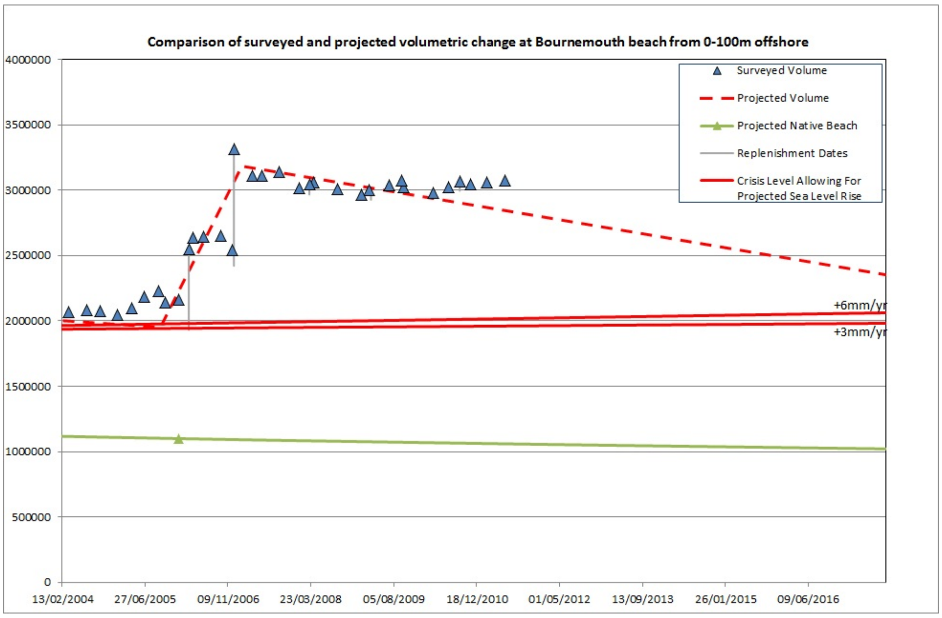


Figure C.1.17 Comparison of surveyed and projected volumetric change at Bournemouth beach from 0–100 m offshore

The beach analysis extends to approximately 100 m offshore, beyond which minimal changes take place. The monitoring includes regular bathymetric surveys and unusually provides a full picture of beach evolution, including the submerged element of the profile. Beach response observations indicate that the volumetric changes to the beach have initially performed as projections suggest (Figure C.1.17). Losses during the past two years have declined and the beach volume seems to have actually increased. It is possible that this reflects the healthy supply of material that is available from beaches to the west at Poole, which was also recharged in 2005-2006 at the same time as the first phase of BIS4.

The BIS4 design profile was designed with a crest level of 2.0 m AODN, considerably lower than the natural beach crest level of about 3.0 m. This meant that the sea immediately overtopped the newly placed material and pushed up a storm beach crest, depositing it in a very 'natural' profile – much better than could be achieved by bulldozing. This approach has also avoided cliffing of the beach, which has occurred following recharges built to higher elevations. This offered a greater degree of safety to the public than crest structures that have been engineered and are artificially steep

The groyne spacing appears to be too wide in places along the beach to minimise loss from some individual groyne compartments. The length of the groynes appears to be suitable, however, to retain the beach sufficient beach material.

While the observation-based design approach has provided a valuable pattern for future designs, this approach is limited to examination of the performance of existing beach configurations and structures. It does not permit assessment of alternative management options, which might provide more economic alternatives. It is considered that modelling a comparison between the performance of timber groynes and rock groynes might provide a valuable alternative.

Experience of previous large-scale recharges suggested that an accelerated rate of loss might be expected immediately following a recharge operation, as the beach seeks to establish an equilibrium position. Such a response is evident following the second phase of the BIS4 scheme when losses of 250,000 m^3 have occurred within a period of six months. Subsequent losses have occurred broadly in line with the projected rate of change, although the rate of loss has reduced since 2008 when the beach volume seems to have stabilised. Smaller interim recharges have had a more limited effect on the rate of loss. Overall the pattern of change appears to be better than the empirically derived projected losses might suggest. This is perhaps a function of the more pessimistic design stage projections of losses relative to earlier schemes.

No unexpected events have occurred during the life of the beach schemes and forcing processes have been reasonably constant over a lengthy period of time. The beach response therefore appears predictable on the basis of historical observations and this approach has provided an appropriate design method for this site.

C.1.6 Lessons for future beach modelling/design

- It is possible to successfully manage a beach scheme over a long period of time without requiring modelling. This is only possible because of knowledge built up over a considerable period of time and maintaining comprehensive records of activities and beach response, and the ongoing application of expertise to analyse and interpret that information effectively.
- Whether some modelling of beach behaviour might have led to a more cost-effective or less effective scheme is impossible to say. However, limits on the ability to use past performance solely to predict future requirements will become increasingly difficult if accelerated climate change starts to alter the wave conditions from those experienced in the past.
- Large-scale renourishments are more likely to experience higher losses, with these occurring early on in the life of the scheme. This needs to be accounted for when undertaking larger campaigns, though needs to be balanced against the potential disadvantages (for example, economic or disruption) of more regular lower volume nourishment activities.
- Constructing a beach to a lower than storm level and allowing nature to build the upper beach profile can be advantageous to avoid cliffing and improve public safety.
- The use of an extensive timeline management log enables a detailed understanding of historical management approaches and the effectiveness of these approaches.

C.1.7 Bibliography

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C.2 Folkestone

C.2.1 General information

Folkestone

2004

Shepway District Council

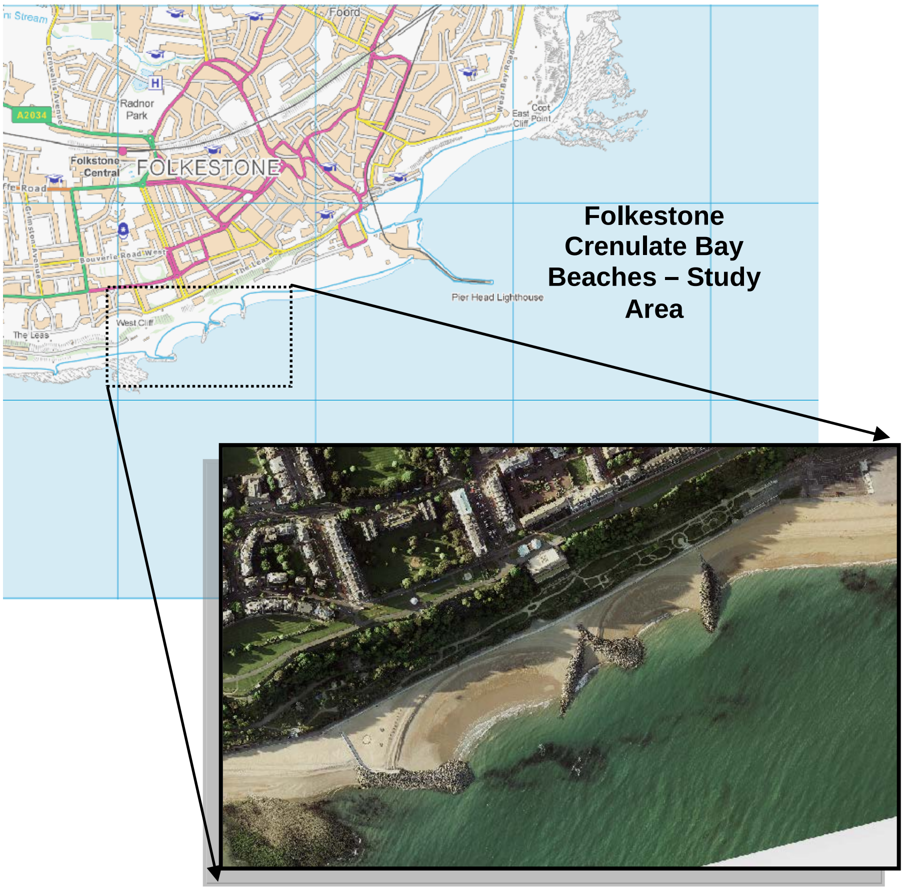

Figure C.2.1 Location of site (image/data courtesy of Channel Coastal Observatory)

Background

The Hythe and Folkestone frontage is located on the south Kent coast and has been defended since the middle of the 19th century (Figure C.2.1). The net littoral drift of shingle is eastwards, but the natural supply from the west has been declining in the recent past. This continued loss in beach volume has caused beach levels in front of the seawalls to drop, and as a result of this 'coastal squeeze', the 7 km of seawalls that protect close to 3,000 residential properties have been subject to considerable wave attack. The frontage has frequently suffered localised flooding and the seawalls, which are in a poor state of repair, have failed on numerous occasions.

In March 2004 work commenced on the Hythe to Folkestone Harbour Coast Protection Scheme, which was funded by the Department for Environment, Food and Rural Affairs (Defra). The scheme was designed to protect a 7 km length of frontage between the coastal town of Hythe and to the east Folkestone Harbour. This frontage can be defined by two distinct characteristics; the western part of the frontage is a continuation of the marine storm gravels that extend from the shingle foreland of Dungeness, while the eastern part of the frontage is where the cliff line meets the coastline – at which point the problem of flooding is replaced by the risk of land slipping and coastal erosion. The following documentation only considers the far eastern section of the frontage, known as Marine Walk.

As part of the 2004 scheme, three new rock structures were constructed in the easternmost 500 m length of the scheme frontage at Marine Walk, forming two static equilibrium bays. These two crenulated bays were constructed using rock headland structures and shingle imported as part of the beach renourishment phase of the scheme. Work on the scheme was completed in September 2004 and the performance of the bays has been continually monitored since their construction.

Supporting background studies

Strategy documents

An initial study was undertaken under a high level strategy in 1996 as a part of the Beachy Head to South Foreland Shoreline Management Plan (SMP), which recommended a policy of 'hold the line'. This policy is continued in the second generation SMP. To facilitate the 'hold the line' policy of the SMP, the Folkestone to Rye Coastal Strategy Study was developed. This was then adopted by Shepway District Council in January 2001. A number of capital schemes have been progressed including the Hythe to Folkestone Harbour Coast Protection Scheme, which was completed in September 2004 (described above).

To take into account the changing risks on the coast, the existing 'Folkestone to Rye Strategy' and the 'Cliff End to Scott's Float Strategy' have since been reviewed and combined to produce a single management strategy. This new strategy, known as the 'Folkestone to Cliff End Flood and Erosion Management Strategy' contains updated information on flood and erosion risk. This strategy has recently been approved, confirming a 'hold the line' policy over the first part of the strategy appraisal period (50 years), with further beach recharge and major sea wall refurbishments to be undertaken during the second part of the strategy period (50–100 years).

Additional reports

In May 2003, Halcrow completed the 'Hythe to Folkestone Harbour Coast Protection Scheme Coastal Processes Report'. This was a study of the coastal processes, undertaken to determine the layout for the (then) proposed coastal protection works along the Hythe to Folkestone Harbour frontage. This study includes an assessment of the wave energy in the study area, marginal wave and water level extremes at 12 nearshore points, and wave and water level joint probability predictions for these 12 points.

Details of the scheme

At the options appraisal stage, many different engineering solutions were considered to reduce the risk of seawall failure and the consequent coastal erosion at Marine Walk. The preferred solution comprised three rock control structures, with 150,000 m^3 of beach recharge between the structures, forming two crenulate bays.

One of the principal influences over the design of the rock control structures at Marine Walk was the identification of the strong uni-directional focus of waves in this region. This presented the ideal conditions for static equilibrium bays and it was this option that was appraised alongside other primary options such as large rock revetments, rock groynes and offshore breakwaters.

The preferred option involved designing the three structures in such a way that they functioned as rock headlands, thus allowing a logarithmic spiral shape to be applied to the design. The logarithmic spiral shape used to describe the equilibrium shoreline that forms as a function of interaction between the angle of wave approach and the controlling headlands is not, however, derived directly from the physical processes that influence the development of the shape itself. The methods used for describing the crenulate beach shape are purely empirical and are based on observations made from many different beaches on many different continents. It is the natural occurrence of these beach bays themselves that leads to some of the greatest uncertainties in the design process.

The natural formations acting as headlands, which influence the formation of the bays, vary from steep-sided cliffs to semi-submerged reefs. It is this potential for diversity in shape and form that makes it difficult to establish the location of the diffraction or control point with respect to the log-spiral beach form.

To refine the design, static equilibrium bay-shape analysis was therefore undertaken and validated using a number of methods. First, the diffraction and refraction effects the structures had on the incident wave climate were analysed using the mathematical wave models. By plotting the wave fronts it was possible to postulate the likely longitudinal alignment of the beach to prolonged wave activity from a specific direction. Further validation was carried out through the examination of crest alignments of stable beaches adjacent to the study area that are exposed to similar wave climate conditions. While there are no fully formed crenulate shape bays on this shoreline, there are lengths of beach, with similar exposure conditions, that are stable and swash aligned. By comparing the orientation of these beaches with the predicted theoretical alignment, it was possible to gain confidence in the values derived from the mathematical models.

By examining the beach plan shapes that had formed in the lee of existing rock structures on the study frontage it was, to some extent, possible to make judgements as to the way in which the new structures would affect wave patterns. This was especially important when determining the crest elevation of the new headland structures because constructing to a level above that which is required would significantly affect the costs. Conversely, too low a crest elevation may not have the desired diffraction effects during higher tidal events.

After studying the performance of the existing ‘Y’ shaped rock groynes and other rock groynes along the Hythe and Folkestone frontage during various states of tide, it was concluded that a crest elevation roughly equivalent to mean high water springs would provide the optimum balance between cost and performance.

The material that was used for the beach renourishment at Marine Walk and throughout the rest of the scheme frontage was dredged from the licensed aggregate extraction source offshore of Hastings. A grading envelope was specified for this material to ensure that the correct balance of course and fine material was delivered to site. The specified D_{50} value (median particle size) of this grading was 15 mm. The beach renourishment was placed to a specified planform and to an initial profile of 1 in 8.

C.2.2 Approach to modelling and basis of design

The initial coastal process analysis undertaken in 2003 included a series of models, the results of which formed the background information on which the options appraisal process was based. These models included;

- **Nearshore wave climate modelling**: 11 years of wave data were used from the UK Met Office European Wave Model for a location offshore of Folkestone (with a resolution of 35 km). The offshore wave conditions were then transferred to 12 nearshore locations using a wave transformation model and for each location a rose of wave height and wave period was produced.
- **Sediment transport and alongshore beach evolution modelling:** using the wave dataset discussed above, the bay shape formed for different configurations of rock groynes and control structures was assessed. In addition, the alongshore sediment transport rate for different grain sizes and the long-term trends in beach evolution were assessed.

Consequently, at the options appraisal stage many different engineering solutions were considered to reduce the risk of seawall failure and the consequent coastal erosion at Marine Walk. The appraisal of the options examined the technical and economic suitability, as well as the environmental and land use considerations.

With regards to the physical constraints of the site, the strong uni-directional focus of waves in this region was one of the principal influences over the design of the rock control structures at Marine Walk. The other influence was the inshore bathymetry and seabed composition; by capitalising on the area of raised bathymetry caused by the submerged rock outcrop, it was possible to significantly reduce the volumes of rock needed to construct the largest of the headlands. Both of these factors had a strong influence on the evolution of the preferred option which was to use rock structures as headlands to form static equilibrium bays.

At the options appraisal stage, a wave climate study was undertaken as part of a coastal processes study. This used an 11-year record of wave data from the UK Met Office European Wave Model for a location offshore of Folkestone. The offshore wave conditions were then transferred inshore to 12 nearshore locations using a wave transformation model (detailed below) to provide detailed and comprehensive information on the inshore wave climate.

The mathematical wave modelling undertaken was split into two phases. The first phase was to investigate how waves transform as they propagate inshore by applying a wave propagation wave model to the study area. This model is formulated to predict the effects of wave refraction, diffraction, shoaling, bed friction and wave breaking, thus predicting the inshore wave climate for a given range of offshore wave parameters.

The second phase was to investigate the performance and effectiveness of possible coastal defence measures at Marine Walk. To do this a localised version of the wave model propagation model was used. This was similar to the larger model, except it also incorporates wave reflections and is capable of modelling wave penetration in and around coastal structures.

Offshore wave climate information was obtained from the UK Met Office European Wave Model. This is a sophisticated wave model which produces a synthetic climate that includes both locally generated waves and accounts for propagation of swell wave energy. Offshore wave and wind data were obtained from the Met Office at location 51.0°N, 1.54°E. The data cover the period from 12 June 1991 to 6 May 2002.

As part of the Strategic Regional Coastal Monitoring Programme (SRCMP), the Folkestone Datawell directional wave rider Mk III buoy was deployed in July 2003. It is located just over 1 km offshore on the 12 mCD contour and is approximately 2 km due west of the Marine Walk. The data from this buoy have been used in subsequent post-scheme appraisal reports, but as this buoy was deployed after the

Halcrow Coastal Process Report (May 2003) had been completed, the data were unavailable for use in the design process.
Analysis of water level data at Dover for the period from 1 September 1990 to 28 February 2002 was undertaken to obtain 61 harmonic constituents for use in the prediction of astronomical tides and the derivation of surge levels. The surge component was then added to the predicted tidal curve at Folkestone, which was determined on the basis of the main four tidal harmonics contained in the Admiralty Tide Tables.
Sediment transport was modelled as part of the 2003 Coastal Processes Report using a beach plan shape model. This used the 11-year time series of wave data transformed inshore to 12 locations along the study frontage. The model was calibrated using historical beach profile data that have been collected bi-annually by Shepway District Council since the completion of the previous beach renourishment scheme in 1996. While a beach plan shape model was used in the initial stages of option development and to optimise the location and length of the rock groynes constructed elsewhere on the frontage, it was not used to predict planform formation and beach stability along Marine Walk frontage. This was because beach plan shape models were not considered capable of replicating the strong refraction influence of the rock headlands used to form the static equilibrium (crenulated) bays. The design of the static equilibrium or crenulate bay beaches was carried out using the empirical formulae developed by Hsu and Silvester (1989). The procedure for testing the stability of a given bay is based on the fit of the log-spiral shape to the beach planform and was the basis on which the planform of the beaches and the configuration of rock headland control structures at Marine Walk were developed. In order to apply the method outlined by Hsu and Silvester, it was necessary to identify the orthogonal of the predominant or persistent wave direction. This average wave energy direction was derived from the inshore time series of wave data by sorting every wave in the 10-year record into 2° bands. The total wave energy was then determined for each direction band on an annual and total wave record basis. The results of this analysis gave an average wave direction of 186°. Many headland configurations were tested to determine the optimum arrangement of structures that would achieve the required minimum width of beach, using the least volume of material. The performance requirement of the bays was simply to provide adequate width and elevation of beach to dissipate the wave energy so as to prevent the seawall from being undermined or damaged by wave impact. The wave models were used to provide design wave parameters used in the structural design of the rock structures, but in addition, the regional wave model was also used to predict the planform of the beach in the lee of the rock headlands. This was achieved by plotting the wave fronts predicted for the model and comparing these with the results of the empirical methods used to predict the formation of the static equilibrium bays.
Bathymetric data were taken from the Admiralty Charts and supplemented with nearshore data from Fugro EMU Ltd’s environmental monitoring survey undertaken for the ‘Beachy Head to Rye Harbour Strategy Study’ and localised surveys undertaken for Shepway District Council.
• **Natural sediment:** the foreshore along this frontage is predominantly sandy, with

a shingle upper beach. • **Modelled sediment:** a shingle-sized material was modelled using the beach plan shape model (D_{50} = 15 mm). • **Sediment placed**: the material that was used for the beach renourishment at Marine Walk and throughout the rest of the scheme frontage was dredged from the licensed aggregate extraction source offshore of Hastings. A grading envelope was specified for this material to ensure that the correct balance of course and fine material was delivered to site. The specified D_{50} value of this grading was 15 mm. The beach renourishment was placed to the specified planform to an initial profile of 1 in 8. Although this formed a slightly steeper beach than that naturally occurring on the frontage, it was necessary to ensure that the extents of the beach profile remained within the envelope bounded by the rock control structures.
Historically at Marine Walk, a small beach had been held in front of the seawall by a series of steel and timber groynes and two small rock 'Y' shaped groynes. Their effect was limited and the crest of the beach was well below high water level and, as a result, it was the vertical seawall that was subject to the majority of storm energy. It was a combination of the failure of the groyne field and the severe abrasion and damage that was occurring to the seawall itself that gave rise to the high risk of failure and thus the urgency for the works at this location.
The empirical formulae developed by Hsu and Silvester were used to derive the predicted stable beach forms. This analysis was based on the average wave direction predicted by the wave model.
The development of a static equilibrium bay arises through wave sheltering, resulting from the diffraction effects of a headland structure and the refraction effect of the beach in its lee. Where these bays occur naturally, the headland or control point is generally a rocky outcrop or reef; however, in order to recreate such a feature at Marine Walk, it was recognised that man-made headlands would need to be established.

C.2.3 Design/modelling outputs – plans for implementation

The initial volume losses resulting from the wash-out of fines from the renourishment material were estimated based on previous beach renourishment schemes undertaken along the Hythe frontage and other locations in the south-east. Volumetric analysis had shown that losses of around 5% of the initial renourishment volume could be expected to occur within a year of the scheme's completion. Additional material was therefore included in the overall recharge volume to account for these losses. Also, given that material lost from the bays is not likely to return through natural processes, the facility to provide top-up material to the bays was included in the scheme design as part of the annual beach management works that are undertaken immediately west of the headlands. Over the long term it was not anticipated that regular beach management or renourishment would be required within the crenular bays. It was, however, anticipated that beach volumes may need to be topped-up occasionally to retain the design profile.

C.2.4 Beach management and performance

Results of post-scheme inspections Following the placement of approximately 150,000 m^3 of shingle between the rock control structures in July 2004, the beaches were profiled and placed to the specified planform with a 1 in 8 slope. A paper presented to the ICE Coastlines, Structures and Breakwaters conference shortly after the completion of the scheme discussed the initial performance of the beaches based on the first three months of survey data. This concluded that, while there had been an initial realignment from the as-placed form, the bays had appeared to reach an equilibrium shape (Herrington 2005). The movement in alignment shown by the surveys is also supported by regular visual observations, which have confirmed that the beaches are mobile in both plan shape and profile, with small changes in orientation being observed following changes in wave direction. The most significant movement was seen to take place during the first month of the beach's existence, with the majority of this movement considered to have been most likely caused by a storm that occurred some three weeks after the initial placement. This was an event driven by strong southerly winds with an annual probability of exceedance estimated to be in the range of 0.5 to 1.0. Volumetric analysis of the beaches after the initial 20-month period has shown that the actual losses are closer to 6% and that the majority of this loss occurred during the first three months. This reduction in volume has had a significant influence on the position of the mean high water line. One of the unexpected findings was that contrary to what was expected (that is, that these initial volume losses would result in a uniform landward retreat of the beach crest), the majority of the adjustment for this change in volume was taken up at the downdrift (eastern) end of the bays. When the 6% loss of material from the beaches was compared with the volume difference between the predicted and actual planforms, there is good agreement between the two figures. This suggests that the primary reason for the difference between the measured and the theoretical planforms was due to these initial volume losses.
Regular (typically bi-annual) beach recycling takes place on the Hythe to Folkestone frontage, although regular recycling within the crenular bays does not. Some intermittent recycling/beach management has, however, taken place within the crenular bays. This has been in the form of material being transported into the crenular bays from other parts of the frontage where accretion has taken place. These works are described in further detail below.
Analysis of wave data recorded by the Folkestone wave buoy between 2004 and 2011 indicates that measured waves with heights 0.5–1.5 m had a lower percentage occurrence than those predicted by hindcast modelling at the onset of the project design (Figure C.2.2). This is a small difference in wave height and is not considered to have any significant impact on the performance of the scheme. Furthermore, examination of UK Met Office modelled wave data between 1988 and 2011 reveals that large wave events have been much less frequent since completion of the scheme in 2004 than in the period prior to scheme construction where more regular storm events were observed; hence, the actual wave conditions have on the whole been lower than that assumed at the scheme design stage (Figure C.2.3). Again this has not significantly influenced the performance of the scheme. The key parameter in terms of the performance of the crenular beaches is the

average wave direction. From the pre-construction wave modelling, it was determined that the inshore wave climate in this location was uni-directional. This was one of the most important factors influencing the design. The stability of the beach bays since the completion of the scheme has reinforced the initial conclusions that the wave climate is uni-directional at this location. However, the post-scheme directional wave data that have been made available for this project were derived for a location much further offshore than the location used for the scheme design. Consequently it is difficult to draw comparisons.

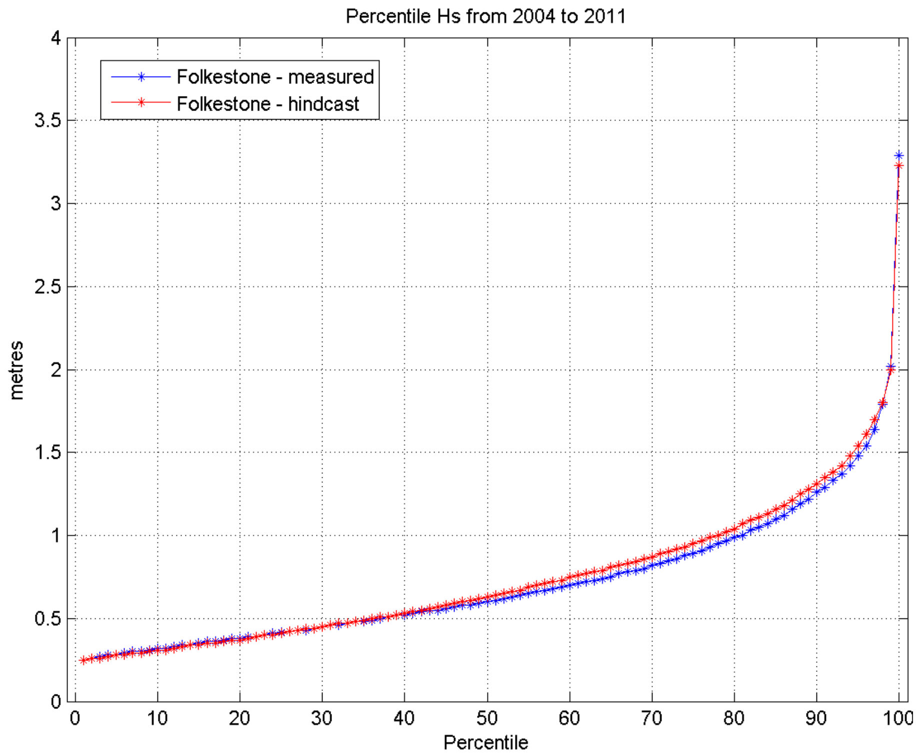

Figure C.2.2 Percentage distribution of pre- and post-construction significant wave heights

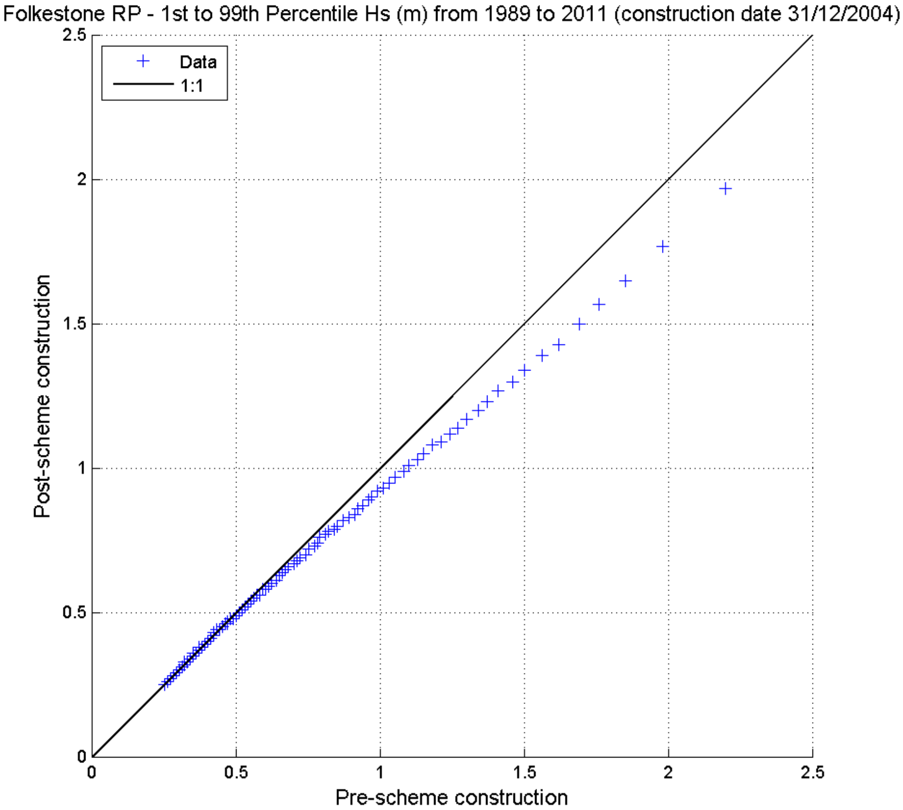

Figure C.2.3 Comparison of pre- and post-construction significant wave heights

C.2.5 Comparative analysis

In the eight years since the completion of the scheme at Folkestone, the beaches have remained stable with only significant reduction in volume being the initial 5% volume loss that occurred within the first year. Some beach management has been undertaking since 2004, however, this generally involved volumes no greater than 10 m^3 per year. These works were primarily for cosmetic/amenity purposes rather than to address beach performance issues.
When the beach planform predicted by the Silvester log-spiral method is compared with that predicted by the mathematical wave model, it can be seen that there is relatively good agreement between the crest line positions predicted by the two methods. The crests of the beaches within the two bays were placed exactly to the theoretical planform with a crest elevation approximately 0.5 m above the elevation of mean high water springs. While observations show that the orientation of the two bays appears to continue to fluctuate depending on the incident wave direction, this fluctuation is limited to about ±2°. This is in good agreement with the variations predicted by the mathematical models used during the design of the scheme.

Assessment of the predicted and actual scheme performance highlights a number of important differences. First, the performance of the two bays is very similar, even though the distance between the control structures of each bay is very different. This suggests that the application of the log-spiral curve used in Silvester's method for predicting the planform of static equilibrium bays is not affected by the size of the bay in relation to the predominant wave climate.

Secondly, the tightness of the theoretical curve at the western end of the bay is not reproduced in practice. This is considered to be most likely due to the transmission of wave energy over and through the rock headland structures, which has the effect of transporting material eastwards along the bay. The static equilibrium bays that occur naturally and replicate the properties of the log spiral curve much more closely are generally formed between rocky outcrops, which do not exhibit these properties.

According to the theory supporting the formation of static equilibrium bays, the section of the curve at the downdrift end of the bay should be aligned to face the averaged wave energy direction. This has occurred, suggesting that the prediction of the inshore wave climate taken from the mathematical model was correct. The fact that the bays have compensated for the loss in volume only at the downdrift ends may therefore be a function of the location of the downdrift control points.

C.2.6 Lessons for future beach modelling/design

The application of crenulate bay theory as an empirical model to design stable beaches has been shown able to deliver a successful and sustainable solution that reflects naturally functioning shoreline features. Good definition of inshore wave direction is critical to that success.

Two other lessons can be learned from the observations of the beaches at Folkestone. First, if a similar approach is adopted elsewhere and the headland control structures are to be constructed from rock armour, then account should be taken of the transmission of wave energy over and through the structures which may affect the plan form locally.

Secondly, the performance of the downdrift control points is susceptible to the structure form. Consequently care should be taken when deriving these points and when designing rock structures to act as control structures.

C.2.7 Bibliography

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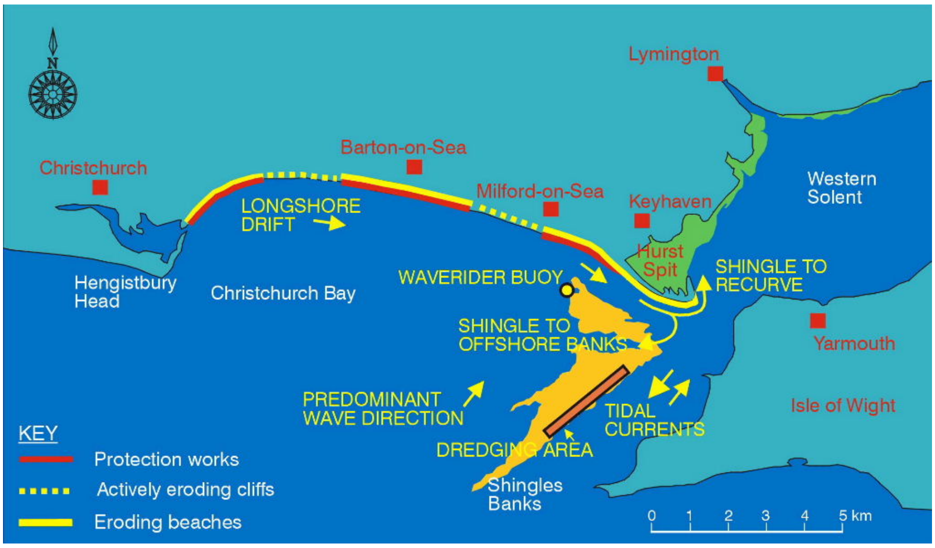
C.3 Hurst

C.3.1 General information

Hurst spit beach management
1996
New Forest District Council
 **Figure C.3.1 Location of site**

Background

Hurst Spit is a barrier beach acting as the sole line of defence to the land in its lee (Figure C.3.1). It is a dynamic structure which can suffer catastrophic failure, leading to extensive damage in a single storm event. Unlike beaches backed by structures or cliffs, the changes caused by storm action on barrier beaches are irreversible processes and leave permanent changes and weaknesses within the defence. The beach performance is extremely sensitive to small changes in geometry, wave and water level conditions.

Hurst Spit is an important environmental feature, which has a number of functions apart from those associated with coast protection and flood defence. These include preservation of an historic monument, protection of the nearby nature reserve which has Site of Special Scientific Interest (SSSI) designation in respect of the salt marsh and shingle plant communities, and also as a breeding and feeding ground for birds. The spit itself is also designated as a morphological SSSI, although some of its value in this respect has been lost due to earlier coast protection work in the area. The stabilisation scheme has been designed to preserve the conservation and aesthetic qualities of the area in as sympathetic a manner as possible, but without compromise to the safety of the design. Figure C.3.2 summarises the risks of not maintaining the defence line.

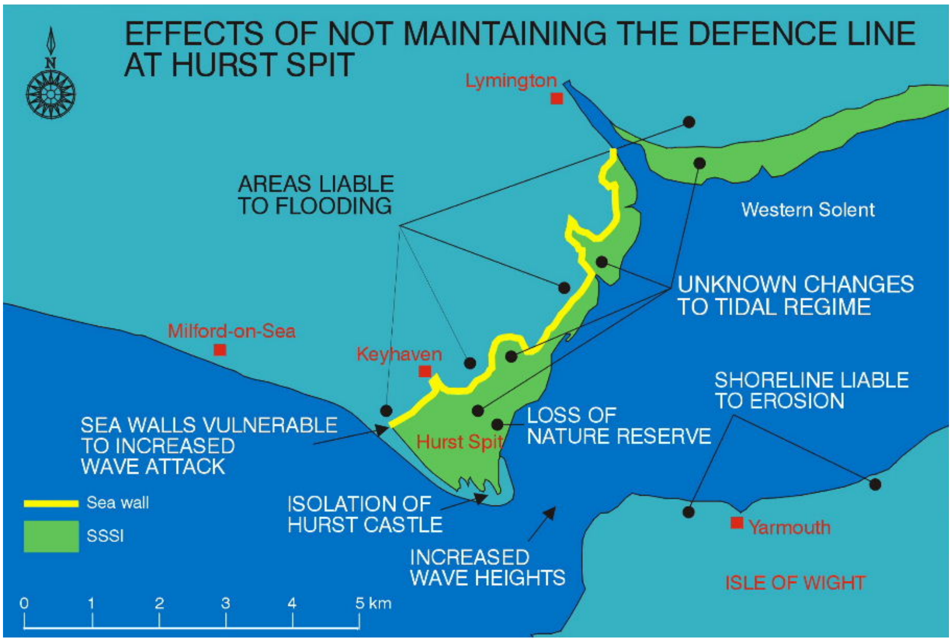


Figure C.3.2 Risks arising from no scheme

C.3.2 Approach to modelling and basis of design

Overview of approach

Probabilistic risk assessment procedures have been applied to a range of management scenarios using a statistical analysis of joint probabilities of wave and water levels. Scheme design was based primarily on three-dimensional (3D) mobile bed physical modelling. Physical modelling was supported by numerical modelling of sediment transport (DRCALC). Note that modelling predates widely available beach plan shape models. Drift calculations were used to calibrate the physical model distortion for sediment transport rates. A wide range of beach geometries were considered, allowing for a range of levels of investment. These have been analysed by sensitivity testing of a wide range of storm events and storm profiles in varying sequences of events. The scheme design allows for the occurrence of all those events which should statistically occur within the design life.

An extensive series of hydraulic model studies were carried out to test the proposed designs and to fine tune designs for maximum cost effectiveness and hydraulic performance. The objectives of the model studies were:

- Identify the various combinations of wave and water level conditions that cause overwashing.
- Determine the rate of loss of shingle from Hurst Spit under storm and 'average' conditions.
- Compare the performance of proposed stabilization measures with the existing Spit.
- Examine the effects of existing structures on shingle transport, such as groynes at Hurst Castle and Milford-on-Sea.
- Identify threshold crest levels and widths to provide alarm levels prior to failure of the shingle barrier.
- Evaluate stability and hydraulic performance of existing and proposed structures.
- Identify a planned maintenance programme following beach recharge.
- Identify the most cost-effective and environmentally acceptable strategy for the maintenance of Hurst Spit.

Analysis of beach profile field data indicated that damage occurs most frequently in severe wave conditions associated with storm surges. A range of water levels including extreme storm surges were considered in combination with storm waves, tidal currents and frequently occurring conditions, in various sequences. Beach responses to these processes were examined by measurement of short-term changes to the beach cross section profile and plan shape. The beach was modelled in a 3D wave basin, at a scale of 1:40, in four segments which were linked together by mathematical modelling to produce an overall picture of the performance of Hurst Spit; this enabled reproduction of the longshore varying wave climate, arising from nearshore shingle banks (Figure C.3.1). Modelling of beach sediment was based on pure shingle sediment with a fine-sediment cut off at 6 mm. The large model scale allowed the sediment response to waves to be reproduced with a high degree of confidence and also allowed rock armour movement to be reproduced and monitored accurately. Changes in alignment of the Spit and effects of sediment control structures, such as groynes at Milford-on-Sea and the terminal bastion at the eastern end of the rock revetment, were also examined.

The test programme was broken down into the following elements:

- mathematical modelling of the nearshore wave climate
- validation of the physical model methodology for shingle barrier beaches
- physical modelling of four overlapping segments of Hurst Spit at a scale of 1:40
- numerical modelling of sediment transport, interactive with the physical model

Various combinations of waves and tides produced alternative design conditions with similar joint probabilities. Each has been considered as a separate design condition, due to the complexity and variation of failure mechanisms that can result in breaching or overwashing of Hurst Spit. Combinations which lie between extreme combinations were also considered.

Offshore time series data were derived from a hindcast driven by Portland wind data for the period 1974-1990. Note the model does not include swell waves. Offshore wave data were transformed using wave refraction models (includes bed friction to account for losses across nearshore bank system). Design data were supplemented following the physical modelling phase with a Met Office 25 km hindcast transformed inshore using a two-dimensional (2D) wave model. Model data were validated with limited measured wave data at a nearshore location.

Pre-scheme wave hindcast data for 17 years (1974–1990) from Portland wind data and locally measured wave data at Milford-on-Sea (1987–1989) were available for calibration. An omnidirectional wave buoy was deployed in 1996, pre-scheme, and replaced with a directional buoy in 2007. The conditions given in Table C.3.1 have a nominal probability of exceedance of 39% during the 50-year design life of the scheme and represent the 1:100 year joint probability return period events, which were used as the design events.

Table C.3.1 Wave conditions

Tide level (m ODN)	Offshore wave conditions		Inshore wave conditions		Probability of exceedance in scheme life
	H_s (m)	T_m (s)	H_s (m)	T_m (s)	
2.27	5.8	8.5	3.8	10.0	0.39
0.87	7.9	9.6	3.6	11.2	0.39

Wave conditions shown are for the most exposed western end of the spit. Energy dissipation provided by the offshore banks (Figure C.3.1) reduces the longshore wave energy significantly from west to east and the nearshore significant wave height falls from 4.1 m to 3.1 m within a distance of just 2.5 km in the design storm. The scheme has been designed to withstand a 1:100 year return period storm, but the difference in design conditions for a 1:50 year event or 1:500 year event is negligible at this site due to the very small range of extremes.

Limited design data were derived from short-term deployments of tide gauges at Lymington and also Bournemouth. Extreme water levels tested were based on those measured during the December 1989 storm, estimated to have a return period of approximately 1:50 years. The lack of certainty of design water levels presented a weakness in the design process.

Design rationale

The plan location of Hurst Spit can be stabilised only if green water overtopping of the beach crest, which results in crest roll back, is prevented. The crest level of the recharged beach must therefore be above the level of maximum run up during the design storm and the beach must be suitably wide to allow the dynamic profile to develop fully within the barrier. Alternatively, the crest width must be both sufficiently high and wide to allow wave events which exceed the crest to deposit entrained sediment at the crest, as opposed to rolling the crest back. The varied wave climate along the length of Hurst Spit results in a differing beach response from west to east. The more severe conditions in the west result in higher wave run up and therefore a higher natural beach crest and wider active foreshore.

Cross-shore response predicted by physical and numerical models

An extensive programme of tests was conducted in a 3D mobile bed physical

model. Design wave conditions, derived from synthetic wave data, provided the basis for physical model testing of the cross-shore beach response. Test conditions were confined to events within the steepness parameter range 0.015 > H_s/L_m > 0.037; these are in line with the suppositions provided by the wave models. Barrier beaches had never previously been tested in mobile bed physical models to examine breaching/overwashing potential, so validation tests were undertaken to compare the model with two recent (1989) storm events, supported by field data. Subsequent testing established the standard of service of the existing beach against overwashing (which was approximately 1:5 years). The primary purpose of the model was to determine the appropriate cross-section of the recharge to avoid overwashing in all but the most extreme conditions, and to identify critical conditions that could be used as a guide to inform the need for intervention during long-term management. A range of cross-sections were tested, with the intention of providing a section that would avoid overwashing during the design conditions. **Beach evolution and longshore transport** Longshore transport tests and beach mathematical models were tested using morphological averaged conditions based on wave climate statistics to determine rates of longshore transport and potential longshore losses. The biggest design challenge was dealing with an historical erosion hot spot at the junction of the downdrift end of a rock revetment and the updrift end of the barrier beach; this zone receives a lower than desirable rate of sediment supply from the west. Several earlier engineering schemes were outflanked, resulting in translation of the hotspot on each occasion that the revetment was extended. The hot spot outflanking problem was approached by designing and testing a nearshore headland breakwater at the updrift junction of the barrier beach with the hard rock armoured revetment; this was designed to re-orientate waves and to develop a stable equilibrium curved planform embayment. The structure was expected to link to the land by a rapidly forming tombolo, subsequently spreading and redirecting wave energy along a broader flatter embayment adjacent to the hot spot zone.
Initial physical model calibration was based on two pre-storm profile surveys derived in September and October 1989. Testing of the pre-scheme (existing) beach was based on topographic and bathymetric surveys of March 1990. A total of 20 beach profile locations, surveyed on a biannual basis and following storm events, dating between 1987 and 1990, were used in the model design.
Grain sizes for physical modelling and numerical sediment transport modelling were based on grading curves derived from a series of sediment samples, captured across the beach profile, at surface and 1 m burial depth. The sediment characteristics are summarised below for each stage of the design and construction process. • **Natural sediment:** Shingle with sand mixed beach, D_{50} ~16 mm but varies widely. Grading envelope used in design (see red envelope, Figure C.3.3). • **Modelled sediment type (size):** D_{50} ~ 16 mm – cut-off at 6 mm in physical model. • **Sediment in final design**: Shingle with sand mixed beach. Grading envelope based on indigenous material (see Figure C.3.3). • **Sediment placement in final design**: Grading envelope based on indigenous

material achieved, based on local offshore supply (see Figure C.3.3).

The design grading envelope (indigenous material) against as built grading samples is shown in Figure C.3.3.

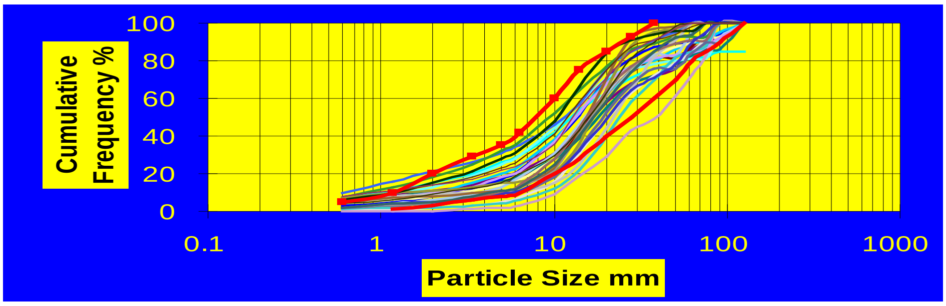


Figure C.3.3 Design grading envelope and as built grading curves

A preliminary application to recycle shingle from the Shingles Banks system in Christchurch Bay consisted of the following elements:

- proving the resource
- engineering options for extracting and placing
- licensing requirements
- environmental impact assessment

A geological exploration programme was developed around the New Forest District Council coastal monitoring programme to investigate the type, distribution and quantity of sediments within the offshore bank system. A reserve of 42 million m^3 of sand and gravel was identified, some of which had an ideal grading for beach recharge. Analysis of sediment transport patterns by interpretation of sidescan, wave and current data demonstrated circulations patterns which included sediment transfer from Hurst Spit to the offshore banks; these showed a net accumulation over a period of 100 years and suggested that the 300,000 m^3 required for recharge of Hurst Spit would be naturally recycled back to the bank system over the duration of the scheme life. The studies suggested that the material quality was considerably better than any of the nearby commercial licensed areas and that the material could be produced at a significantly reduced cost.

A rock breakwater, acting as a headland structure close to the shore, was designed to spread and dissipate wave energy across the weak junction of the rock revetment and barrier beach. The structure is armoured with 6–10 tonne rock at varying slope angles, with a crest at 2.5 m ODN, allowing for overtopping to clear the gap between the breakwater and shoreline during storm conditions. Design for stability of the rock armour presented a considerable problem due to the foreshore geometry and the location of wave breaking on the structure. Difficult ground conditions posed a further design problem with the need for geotextile beneath the breakwater, which is permanently immersed below low water in a minimum water depth of 5 m, at the toe. The plan location of this structure is unconventional being very close to the shoreline.

A wide range of alternative structures were tested in combination with beach recharge, but the only cost-effective solution found was an angled shore detached breakwater, overlapping the shingle spit and the rock revetment. Hydraulically more efficient structures built further from the shoreline could provide a better technical solution, but the very steep approach bathymetry would necessitate construction of a very large and extremely expensive deep water structure. The compromise solution is a structure in shallower water which will require some periodic beach material bypassing due to predicted formation of a tombolo and starvation of sediment to the east. The structure will reduce the wave attack on the junction by dissipation of the waves before they reach the beach. It will also re-orientate waves and drive shingle and sand into the lee of the breakwater, thus providing a beach of finer material under most conditions.

Other alternative designs include a 2 km rock revetment extension, open beach and a recharged beach with a variety of terminal/ headland structures at junction between revetment and barrier beach.

Sediment source

Discussions with aggregate suppliers revealed that none of the licensed areas on the south coast could meet the specification grading. As it appeared unlikely that commercial licensed areas could supply material of the preferred design grading, the design process was reviewed and an alternative design based on finer wider gradings developed. The basis for alternative recharge designs using materials with finer and wider gradings made the assumptions that these materials would have the following effects on the hydraulic performance of the beach.

- The beach will form a dynamic equilibrium slope at a shallower angle for either finer or more widely graded materials than for the indigenous beach grading. This would require a larger quantity of material to form the capital recharge.
- The longshore sediment transport rate would be faster for finer material than for coarse material. Losses from the system would be greater, therefore. This would result in a requirement for more frequent and higher volumes of maintenance to be included in the beach management plan.
- The use of a finer grading or a more widely graded material would reduce the permeability of the beach.
- More widely graded materials would contain a higher proportion of fines, which are likely to be lost from the system at an early stage.

The methodology for the alternative designs was developed on the basis of empirical techniques (Powell 1993) and also empirical examination of nearby beaches with similar gradings. Results of the two methods presented a degree of uncertainty about the expected performance of alternative gradings. Despite these uncertainties, an alternative design based on a finer wider grading was developed and tenderers were invited to price three alternative options:

1) Beach recharge based on the original design grading and supply from the Shingles Banks with material quality at the client's risk
2) Beach recharge based on the original design and supply of the indigenous grading of Hurst Spit, from a commercial source, with material quality at the contractor's risk
3) Beach recharge based on supply of a larger quantity of material to achieve the alternative design, with a wider and finer grading of shingle, from a commercial source, with material quality at the contractor's risk

The most economic option proved to be the preferred design using the Shingles Banks as a source. The unit rate for the supply of shingle from the cheapest commercial source was 154% of the rate for the Shingles Banks. An additional 12.5% of shingle was also required to achieve the alternative design to the Shingles Banks from this commercial source. The real cost of construction from this commercial supply was therefore 173% of the cost of the Shingles Banks supply. The unit rate for alternative 2, which was based on the same quantity and grading as alternative 1, was 189% of the cost of the cheapest Shingles Banks supply.

Hydraulic model tests identified that a crest level of 6.3 m ODN would not be exceeded by green water under any of the combinations of waves and water levels tested, at any location along the spit. Model tests also identified threshold geometry conditions for each profile beyond which the shingle spit would be vulnerable under the design storm (Bradbury and Powell 1992). Variable longshore wave climate arising from nearshore bank system results in variable design conditions along the length of the site.

Cross-shore response

The design beach recharge cross section has a crest level of +7 m ODN, with a crest width of 12 m along much of the length (Figure C.3.4). The design crest reduces further to the east, tapering down to a level of +5 m at Hurst Castle, where the wave climate is less severe. Design crest levels are higher than the maximum predicted run up levels for any of the conditions tested to allow for subsidence of the beach into the salt marsh which lies directly beneath and in the lee of the shingle barrier.

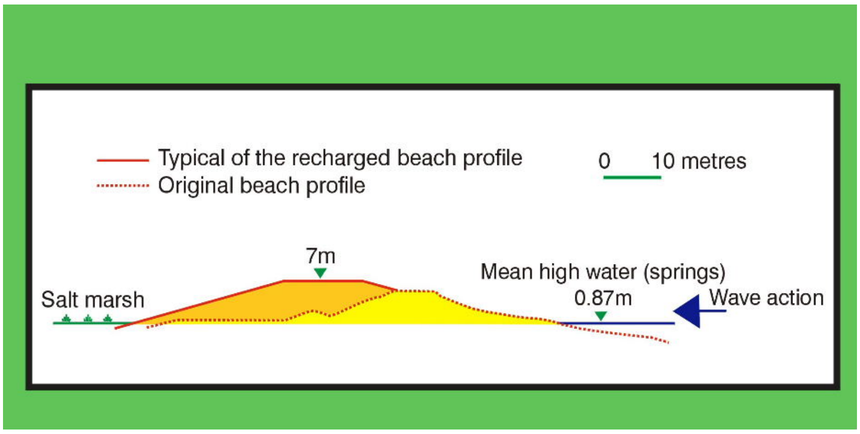


Figure C.3.4 Cross-section of recharge design

A basic empirical framework comprising over 3,000 data points was derived to assist with the prediction of lifecycle management of the beach following recharge and which identified the barrier geometry for critical overwashing threshold conditions (Bradbury 2000). This was based on the results of the physical model tests and supplemented by some limited earlier field observations (supported by synthetic wave data). The extensive test programme provided an empirical framework relating the barrier inertia parameter ($R_c/B_s/H_s^3$) to the wave steepness parameter (H_s/L_m) and with a derived overwashing threshold (Figure C.3.5).

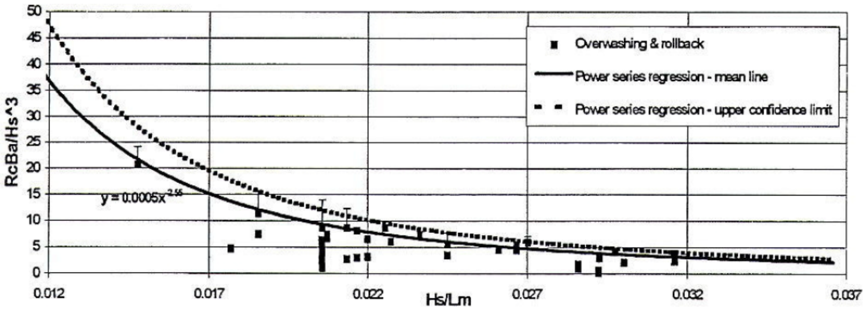


Figure C.3.5 Threshold curves for beach management

Structure design

The plan position of the breakwater, optimised by the model002C was somewhat closer to the shoreline than is usual for headland structures, thus producing a short embayment. Physical model testing suggested that a better hydraulic performance and broader embayment could be produced by construction of the structure further offshore, but at significantly greater cost due to the steeply shelving bathymetry. The balanced solution, which allowed for shallower water construction and some limited shingle bypassing, provided a far more economic lifecycle management solution. Even with the nearshore position of the headland breakwater, the structure toe was in 5 m water depth at low water, making for complex submerged construction.

Longshore transport

Longshore transport tests suggested that the transport rates would be essentially unaltered by the recharge, as the beach recharge is mainly on the lee face of the Spit. The only likely difference is a reduced supply of the shingle to the western end of the Spit due to the construction of the breakwater. Sediment transport rates of 16,000 m^3 per year were estimated.

The main elements of the scheme are as follows and are shown in Figure C.3.6:

- recharge of the barrier beach with 300,000 m^3 suitably graded shingle, based on sediment supply from the nearby Shingles Banks as a source
- construction of a rock revetment around the south western flank of Hurst Castle
- construction of a single nearshore headland rock breakwater at the rock armour/shingle junction
- reconstruction of the existing rock armouring between Milford beach and Cut Bridge

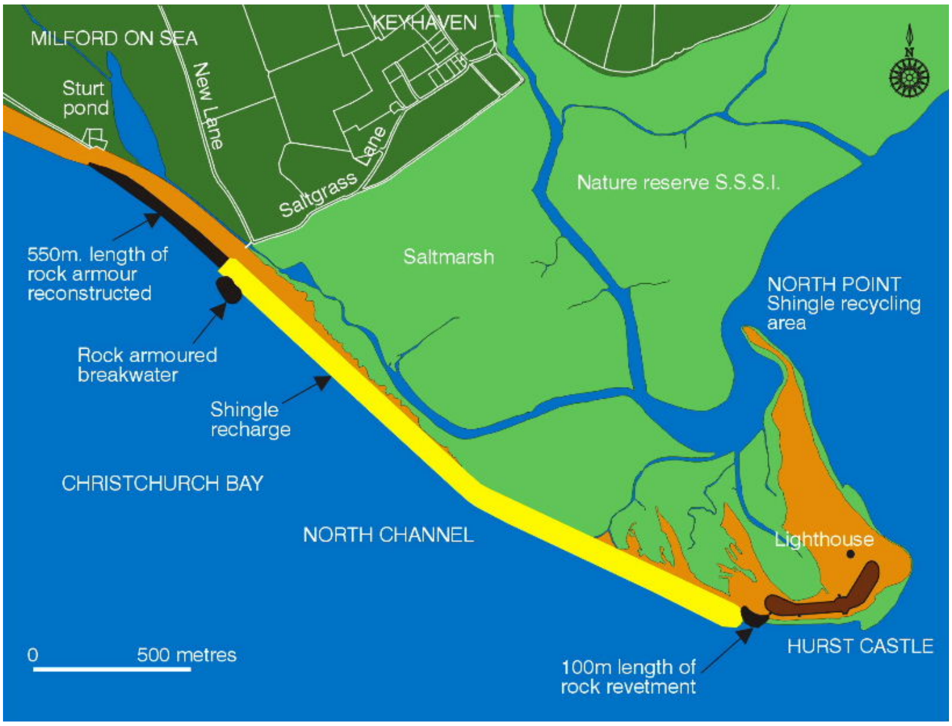


Figure C.3.6 Scheme layout

C.3.3 Design/modelling outputs – plans for implementation

A beach management plan was developed at the design stage and is outlined below.

Outline 50-year programme

On completion of the capital recharge scheme, the beach was expected to withstand the design storm conditions without risk of overwashing or breaching under design conditions. However, the recharge was a dynamic structure which would modify rapidly over time due to both cross-shore and longshore transport processes and it was expected to require maintenance throughout its life. The beach management plan relies on a comprehensive monitoring programme in conjunction with empirical predictive models to provide a decision support system for the maintenance programme. The scheme has a design life of 50 years, during which there will be a requirement to recycle or top up the renourishment and to maintain the rock beach-control structures. Estimates were made to facilitate development of a preliminary programme of recharge maintenance. The programme was to be revised in conjunction with the results of the planned monitoring programme at strategic (five-year) intervals.

No further introduction of additional beach recharge materials were envisaged within the first 10 years of the beach management plan. Planned maintenance work was limited to recycling of material and bypassing of the breakwater in years 1–10. An allowance for major maintenance of the rock structures was included in year 6; this would follow the initial settlements and movements which might be expected during the first few storm seasons. Further maintenance of the beach-control structures was also planned at strategic intervals during the life of the scheme.

The first planned interim recharge was scheduled for year 10, when an estimated 100,000 m^3 of shingle would be required. This would be followed by recharges of 100,000 m^3 at 15-year intervals until year 40. The estimated volumes are based on historical rates of loss due to longshore transport and monitoring. They also assume that the recharge material has a comparable grading with that used in the recent recharge scheme, which was sourced from Shingles Banks area 406.

The size and frequency of the anticipated interim recharges reflects an estimate of economies of scale, in combination with anticipated beach losses and exceedance of the alarm beach cross-section thresholds. A range of alternative strategies were considered, which may result in smaller volumes of interim recharges at more frequent intervals, using alternative delivery methods. Provided that the capital recharge is maintained in accordance with the recommendations given in the beach management plan, emergency action should not be necessary although a contingency plan was prepared.

Detailed five-year programme

Maintenance

A programme of planned maintenance was developed based on the results of both the physical and mathematical model studies, and also on the results of field surveys carried out since 1987. A detailed beach management programme was developed at the design stage.

Threshold levels

A damage threshold or alarm condition was defined at which maintenance is necessary to avoid failure. A simple geometric definition of damage threshold is not appropriate as the damage threshold condition, defined by conditions giving rise to overtopping and resulting in roll back of the crest, varies along the length of the Spit. The wave climate is more severe at the western end of the spit and consequently the alarm or threshold damage level of the crest is higher than at the eastern end where the Spit is more sheltered. The alarm cross-section for the renourished beach was defined in terms of minimum crest elevation and minimum crest width, and also by reference to predictive models of cross-shore transport. This alarm value is reached when the design storm followed in quick succession by a 1:5 year and a 1:1 year storm would result in failure of the bank by crest lowering. The method of determination of this condition is achieved by combined use of the predictive cross-shore empirical models SHINGLE and BREACH.

The maximum run-up levels and alarm levels were defined from the extensive series of physical model tests. The maximum run-up levels recorded by measurement of the level of the run-up berm vary considerably along the length. Values of +6.3 m ODN between profile lines HU6 and HU18 (Figure C.3.10), with a crest width of 6 m and +4.8 m ODN with the same crest width between HU18 and HU20 provide the effective upper limits of wave run-up for the conditions tested. Sections constructed with crest levels in excess of 6.1 m ODN were not overtopped at all during testing and this was expected to provide a safe crest level. The vulnerability of the beach to narrow crest widths was demonstrated in the model. While a crest width of 8 m at a level of 6.1 m ODN provides a very safe situation against breaching in the design storm at the western end of the Spit; the crest width should be maintained at this level to allow for a sequence of storms occurring over a short period of time.

Settlement and shingle loss

The beach recharge was constructed at a higher level than was required to resist overtopping during the design storms. This was necessary to allow for loss of cross-sectional area of the beach recharge by subsidence of the shingle into the very soft substrate. It is difficult to estimate the volume of shingle that might be lost by subsidence during the life of the scheme, since ground conditions vary considerably along the length of the Spit. This is less likely to pose a problem on the section of the Spit which lies above partially compacted salt marsh. The recharge landwards of the beach may pose more of a problem in this sense.

Evidence of rates of settlement by levelling of datum poles suggests that in excess of 0.5 m settlement could be expected during the first year following construction, in addition to the initial settlement caused by the initial loading of the salt marsh. Monitoring of the crest levels following construction was planned to identify those areas that will require maintenance due to settlement. Allowances were made for topping up the crest to the design level during the first two years following construction by recycling of excess materials within the system. Similarly, excess material remaining above the design levels would be trimmed, when appropriate, to provide additional volume at more useful locations.

Routine maintenance requirements

Routine maintenance requirements identified in the original beach management plan included the following elements.

(i) Annual clearance of material from the plugged breakwater gap was planned. The gap between the breakwater and the shingle bank/revetment is only 15 m at the toe. In view of the longshore transport from the west, the breakwater gap was expected to plug with sand and shingle after a fairly short period of time. It was expected, on occasion, to be cleared naturally by wave action, either by overtopping or by waves driving through the gap at high water levels. It was expected that the breakwater would slow the transport rate considerably, consequently requiring some artificial force to drive the material through the gap and on to the main body of the Spit. Some natural bypassing of the toe of the breakwater was expected occur, but this would be a small fraction of the total quantity of material in transport. Material from the anticipated build-up zone will be used to patch the beach approximately 100 m downdrift of the breakwater, at the location where erosion is expected to be greatest and which will provide the supply of material to the downdrift area of the beach. An equilibrium rate of transport was not expected to be reached within less than 2–3 years following construction. Better estimates of long-term maintenance commitments were planned with the aid of the monitoring programme.

(ii) An accumulation of the coarser fraction of material was anticipated at the rock revetment west of Hurst Castle. Excess material was located in the main body of the spit to form a reserve stockpile approximately 400 m from Hurst Castle. This was expected to provide a supply of shingle for recycling to the area between profiles HU7 and HU9 (see Figure C.3.10), which is the area most likely to be vulnerable to erosion. An additional at risk zone was noted between profiles HU19 and HU20. It is likely that annual maintenance of these areas will be required by recycling of material from the stockpiles and accumulation points. Allowance was made to recycle approximately 5,000 m^3 per year.

(iii) Following construction of the rock revetment at Hurst Castle, the rate of shingle transport around the Castle was expected to reduce slowly. The accumulation of shingle at the North Point was also likely to be reduced eventually as the supply of material diminishes. This area will effectively be isolated from the main system. However, wave action from the north-east will continue to drive material along the shingle recurve. As less material will be entering the system from Christchurch Bay, there was a risk that beaches on the northern side of Hurst Castle would be outflanked over a period of years. There was unlikely to be a significant reduction in sediment supply to North Point for at least 10 years if current longshore transport rates are maintained. The longer term problem could be overcome by recycling material between the accumulating North Point and the area immediately to the north of Hurst Castle. Any surplus material could be transported further around onto the main body of the Spit. It was expected that there will be an annual commitment to maintain the river entrance channel and to protect the northern flank of the Castle defences. Recycling from the North Point would still continue when the new rock revetment around the Castle was finally constructed.

Provided that the spit was maintained in accordance with the recommended programme outlined above, emergency works should not be necessary. In the event that the alarm thresholds are reached the following course of action was identified.

1. Identify the extent of damage and calculate the volume necessary to reinstate the beach to the maintained cross section profiles.
2. Examine the possibilities of recycling from existing stockpiles or sections where accumulation has taken place. Make up the shortfall using material from these reserves where possible, using mechanical plant to recycle the material within the site
3. Import the minimum quantity of material to make up any shortfall by road. This option is unlikely to be required, but provides a fall-back position.

The rate of shingle loss was estimated at 40,000 m^3 during the first year, allowing for initial adjustment losses, settlement and subsequently typical longshore transport losses of 16,000 m^3 per year. An average loss rate of 16,000 m^3 per year was projected over the first 10 years, which would theoretically see the crisis value reached.

Allowance for an average 5,000 m^3 per year recycling was made.

Projections suggested that an interim recharge would be required after 10 years and the maintenance programme would be also be reviewed to reflect monitoring (Figure C.3.7). The maintenance programme was due to be reviewed in epochs of five years.

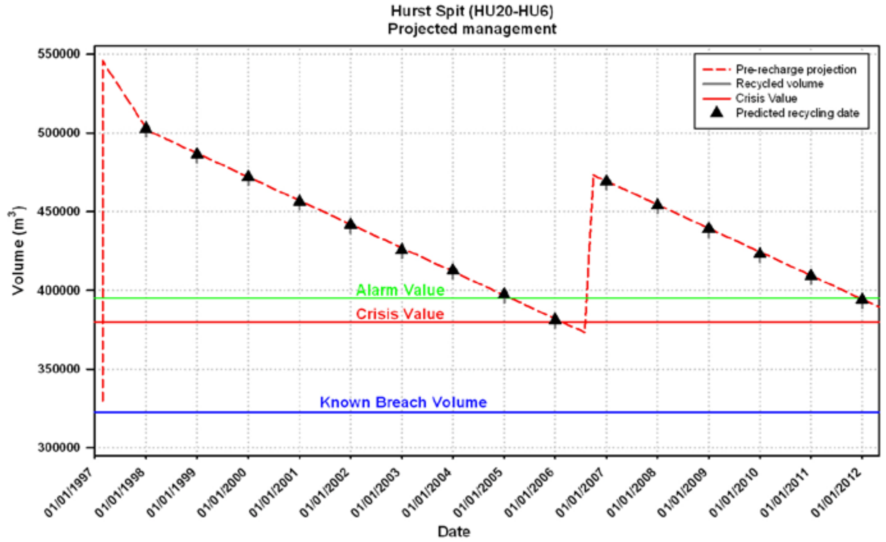


Figure C.3.7 Planned maintenance programme from 1996 to 2012

C.3.4 Beach management and performance

The main elements of the as built scheme are as follows and are shown in Figure C.3.4:

- recharge of the barrier beach with 300,000 m^3 suitably graded shingle
- construction of a rock revetment around the south western flank of the Castle
- construction of a single nearshore headland rock breakwater at the rock armour/shingle junction
- reconstruction of the existing rock armouring between Milford beach and Cut Bridge

The as-built scheme reflects all the geometric and volume details developed at the design stage (FiguresC.3.4 and C.3.6).

The as-built construction geometry at this site was very close to that modelled, thereby making comparison of the performance and the design tools more straightforward. The geometric characteristics of the final design were based closely on the physical model, with additional allowances made for settlement of the beach recharge (1 m), although this scenario was also tested. This might reasonably be expected to impact on cross-shore profile performance in the short term since waves might be able to reach the beach crest less frequently and the beach may consequently become more reflective.

The main difference between the modelling and the as-built construction relates to the grain size distributions of the modelled and the prototype recharge material. Design of the physical model sediments was based on the grading of the indigenous beach material, which also formed the basis of the recharge. The local recharge source was (unusually) able to meet this target grading envelope and a recharge with a D_{50} of about 16 mm was constructed (see Figure C.3.3); this unusually coarse dredged material contained a sand content of about 20%. Physical modelling of the beach was undertaken using lightweight materials (crushed anthracite) designed to simulate the hydraulic performance of shingle. However, the model sediment was scaled to be representative of a shingle grading with a D_{50} of 16 mm, but without the sand content and an effective cut off of material below a grain size of about 6 mm. This is a standard modelling practice, since mixed sediments cannot be modelled effectively at the required scale for 3D wave basin modelling. There is a reasonable expectation therefore that the profile response of the prototype and model recharges might be expected to differ since the model effectively represents a clean shingle, while the prototype represents a mixture of sand and shingle with lower permeability. There is an expectation therefore that the prototype beach might develop a flatter slope and with a lower crest than that achieved in the model.

A series of relatively severe storms have occurred since completion of the works. The beach response, which has been monitored by topographic and hydrographic surveys in parallel with wave and tidal measurements, has been remarkably close to that predicted for the storm events. It is estimated that the pre-storm barrier would have been subject to sluicing overwash leading to barrier breakdown on at least eight occasions within a two-year period on completion of the works. Post-project monitoring is ongoing. This is linked to a planned maintenance programme and an extensive post-project monitoring programme in accordance with the dredging licence requirements.

Profile response

Application of measured post-construction conditions to the design framework provides a marked contrast with the modelled conditions. Most of the post-construction measured wave conditions lie outside of the valid range of the predictive design framework and are generally characterised by steeper wave conditions; this reflects a future research need to extend the validity of the framework. Where wave conditions have occurred within the valid range, the predictive threshold seems to have worked fairly well, although there have been only a limited number of these conditions and few close to the theoretical threshold. The problem in this context is that the wave conditions that were modelled are not representative of those that have been measured. The implication of the differences identified in the comparisons between modelled and measured wave data is that the modelled extreme conditions have been represented by events with lower wave steepness than those which were expected. Much of the monitoring data fall outside of the valid predictive range therefore (Figure C.3.8).

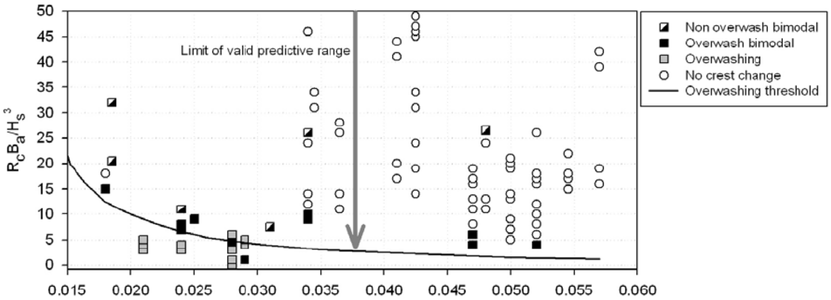


Figure C.3.8 Comparisons between field measurements and the barrier inertia thresholds

The field data have provided the opportunity to populate the dataset with steeper wave conditions. Consequently, an approximation of the threshold can be determined for this site for conditions where $H_s/L_m > 0.037$. The vast majority of the data has resulted in conditions where overwashing has not occurred. The fact that the beach has been maintained at a healthy level has enabled a range of safe conditions to be added to the graph. Much of the field data for the steeper wave conditions lie well above the threshold condition. This is not surprising as the as-built geometry of the beach recharge provided a significant improvement to the standard of service and was well above the anticipated threshold following construction. The subsequent rate of degradation has been fairly slow. As the beach declines in cross-section, it will move closer to the overwashing threshold condition, under design conditions. Extended threshold conditions, where $H_s/L_m >$ 0.037, can only be approximated and reflect only measured conditions.

Planform development

The planform developed following construction is remarkably similar to that developed during physical model testing. It has performed as suggested by the physical model tests during a post-construction monitoring period of more than 16 years. Although the range of model test conditions was limited to an incident wave angle range of about 25°, this has proven to be representative of actual conditions at the site. It is significant to note that the site is not subject to a wide range of incident wave angles due to the limited range of offshore conditions; these are limited due to protection by offshore banks and by the approach bathymetry which results in consistently high refraction of wave energy, and a narrow range of incident wave angles. Incident waves generally arrive within a few degrees of the beach normal.

The recent planform of the beach (Figure C.3.9) has remained relatively stable over a period of several years, with occasional fluctuations arising from severe events. The extension of the dissipation zone and reorientation of waves has had the desired effect of spreading wave energy and reducing losses from the vulnerable junction. The typical plan form shown in Figure C.3.9 was developed at the site within a period of less than one year.

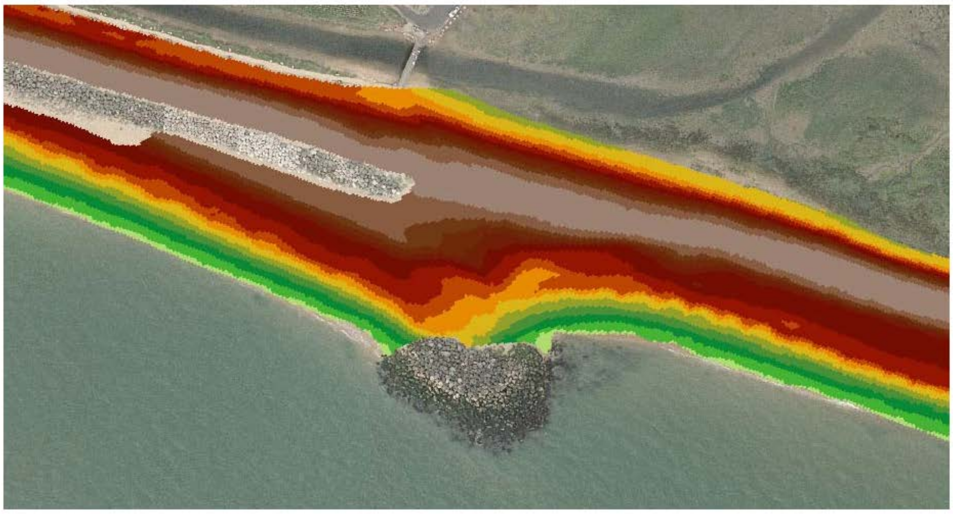


Figure C.3.9 Planform development of beach adjacent to beach 12 years after construction

Long-term beach evolution

Regular beach surveys have tracked progress of the project performance both before and after construction. Surveys are conducted three times per year and also following storm events and maintenance. A design stage allowance of 20% losses was made for the first year following construction, based on a qualitative estimate of expected rapid adjustment to the recharge, including allowance for settlement and longshore losses. The beach performance shows a remarkable fit to the projections for the first year, with losses of approximately 40,000 m^3 (Figures C.3.7 and C.3.17). Subsequently projected losses were based on the average rates arising from the combined results of physical and numerical modelling. This suggested that, following initial losses, the anticipated average rate of loss would be approximately 16,000 m^3per year, primarily through longshore transport.

Losses of about 11,000 m^3per year occurred for a period of about four years, followed by a decline in the loss rate to about 5,000 m^3per year. The average loss over 16 years is about 7,300 m^3per year, or about 45% of the projected rate. Surveys following storm events highlight episodic periods of beach drawdown, when large quantities of material (>40,000 m^3) disappear temporarily below mean low water.

Longshore transport

Since minimal material arrives at the site in longshore transport from the west and the headland breakwater acts as a terminal structure, it is relatively straightforward to determine a coarse approximation of longshore rates by assessing the losses from the beach recharge zone which is downdrift of the headland structure. The orientation of the shoreline, and protection from wave activity from the southeast, means that any sediment transport is virtually always eastwards. Growth of material to the west of the headland structure has been minimal, equating broadly to the original volume of material placed west of the breakwater during the recharge. This confirms the supposition that little sediment was arriving at the site prior to the construction of the scheme and hence the initial problem.

Longshore transport calculations conducted at the design stage suggested faster transport rates than have actually occurred since construction. On average the actual longshore transport rates (1998-2008) have been about 45% of the initial predictions suggested by the modelling (estimated at ~16,000m^3per year).

The rate of loss has reduced subsequently. Measured transport rates from 2004 to 2008 averaged about 5,000 m^3per year and are somewhat lower than the model would suggest. The trend has been for the rate of loss to decline over this period (Figure C.3.15).

Numerical modelling of wave climate suggests that wave energy is variable along the length of the beach recharge and that there should consequently be a variable rate of longshore transport along the beach. This is illustrated in Figure C.3.10 which shows the impact of the offshore banks on the spatial variability of nearshore wave climate for severe events. The general suggestion of longshore variability of wave energy provided by the wave models is supported by clear evidence of variability of longshore transport rates along the length of the beach recharge. This is demonstrated by formation of a gradually accreting ness feature part way along the recharge in the zone where the wave energy reaches a minimum (between profiles HU17 and HU18; Figure C.3.10). Beach profile surveys have identified consistent growth of this feature over a period of 16 years. Erosion is more prominent in the higher energy areas. Although the measured losses of beach material from the recharge do not support the intensity of energy suggested by the wave and sediment transport models, the relative spatial distribution of wave energy seems to be about right, with erosion and accretion occurring at the expected locations. It is suggested that the influence of bed friction in the wave models may be responsible for the overestimate of wave energy at the shoreline.

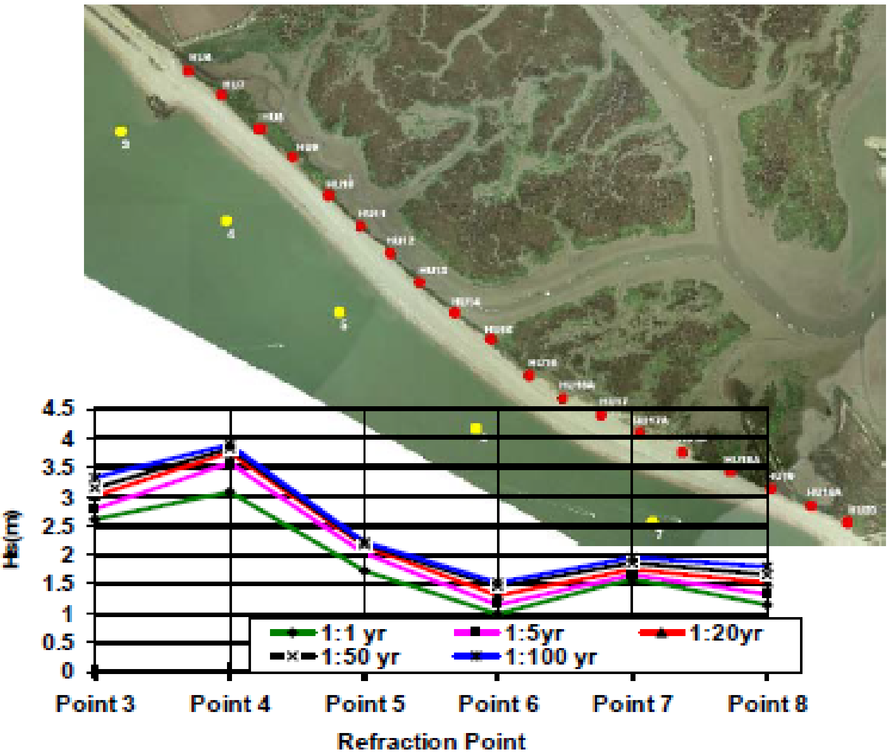


Figure C.3.10 Profile locations and longshore wave climate variability

Beach settlement

A number of settlement plates were distributed throughout the recharge to record the level of post-construction vertical movement due to the loading weight of the recharge material on the unconsolidated marshland. The settlement beacons each consisted of a vertical scaffold pole welded to a 1 m square metal plate, which was buried at the base of the recharge, allowing any vertical movement to be recorded via measurement of the top of the pole. Settlement beacons were placed along the whole length of the site, covering both areas where settlement was expected to be high and zones which were already partially consolidated. The beach recharge was constructed to an elevation 1 m higher than that required to withstand the design hydrodynamic conditions to allow for potential loss of cross-sectional area of the beach by subsidence of the underlying deposits. The tops of the scaffold tubes were subsequently monitored for three years (1997-1999) to identify settlement changes. Settlement caused by the loading of the salt marsh resulted in an average reduction in elevation of 0.24 m over a three-year period. Virtually all the measurable change occurred during the first 12 months following construction. The distribution of change is shown in Figure C.3.11. This suggests a net loss of volume of approximately 11,000 m^3 over the surface area affected compared with the design allowance for potential losses of approximately 45,000 m^3.

More than 0.5 m of settlement has occurred in places, with the greatest changes occurring between profile lines HU14 and HU19; this zone has not previously been subject to loading. The zone between profiles HU18 and HU18a is above a relict shingle and sand recurve (Nicholls 1985) and has been subject to small settlements. As expected, the beach zones that have previously been loaded with sediment have subsided least. The spatial distribution of zones of higher and lower rates are consistent with expectations derived from the design stage testing, although the rates of settlement have generally been lower than anticipated.. Recordings of the settlement beacons ceased in 1999, by which time they had been lost or had deteriorated.

While conservative, the design approach based on the anticipated response geotechnical properties of the underlying material under compression seems to have worked reasonably well. Figure C.3.11 illustrates the relative distribution of cumulative settlement during the course of the monitoring period.

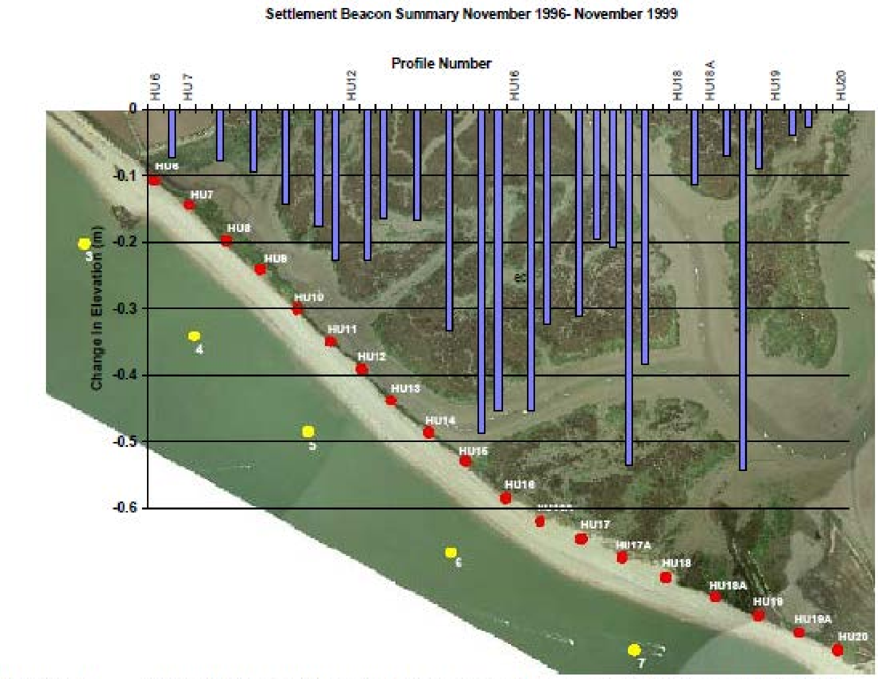


Figure C.3.11 Cumulative settlement of beach recharge over three year period

Allowance was made within the design programme for annual maintenance comprising a combination of crest trimming, recycling and bypassing on average once per year for the first 10 years following recharge. Allowance was made at the design stage for an annual average of 5,000 m^3 of recycling.

The following activities were undertaken during the first 14 years following scheme construction:

Feb 1997	Trimming adjacent to breakwater	4,500 m^3
Feb 1998	Post-storm trimming and recycling	5,600 m^3
Jan 2000	Post-storm trimming and recycling	8,800 m^3
Jan 2001	Trimming and recycling	6,600 m^3
Oct 2001	Recycling from North Point	8,600 m^3
Jan 2003	Trimming and recycling	4,800 m^3
Dec 2004	Recycling from North Point	6,636 m^3
Nov 2005	Post-storm trimming and recycling	7,644 m^3
Mar 2007	Recycling from North Point	5,283 m^3
Mar 2010	Recycling from North Point	4,729 m^3

Modifications to crest elevations have been made within the maintenance programme to reflect the monitoring results and profile response. As the beach crest was constructed to a higher elevation than required to achieve the required morphodynamic performance and the geotechnical response better than expected,

the crest elevation has been trimmed artificially to optimise morphodynamic performance; this has contributed approximately 8,000 m^3 to maintenance activities. This activity is denoted as trimming in the maintenance summary. Annual clearance of material from the plugged breakwater gap was planned. This has not been required due to the performance of the downdrift beach, which has been better than anticipated due to reorientation of the downdrift embayment. The build-up of material updrift of the breakwater has been maintained, with a view to using this only as an emergency supply when other recycling locations are not available for use. This source has been used twice. Threshold levels have been maintained in accordance with the original design criteria, although it would appear that the standard of service of the beach is significantly lower than the original design conditions would suggest. This is due to the bimodal conditions that were not considered as design variables. Small volumes of recycled beach material are shown on each occasion reflecting management practice. The required frequency for maintenance has been less frequent than originally planned, with no maintenance at all conducted in a few years. The average annual maintenance has been very close to that anticipated at the design stage with an average recycled volume of 4,500 m^3per year. Performance has already exceeded design phase expectations for the effective life of the recharge. No maintenance of rock structures has taken place, although allowance was made for this once during the first five years following construction. Recycling from North Point has taken place once every 2–3 years, which is in line with planning. The rate of deposition at this location is highly linear on a year-by-year basis and deposition has continued, which has been balanced by recycling. An accumulation of the coarser fraction of material was anticipated at the rock revetment west of Hurst Castle, but this has not occurred. This may be due to the complexity of the bidirectional wave climate, which may not have been reproduced well in models. Excess material located in the main body of the spit to form a reserve stockpile approximately 400 m from Hurst Castle has been partially used within recycling; this has been assisted by continued deposition and accumulation of shingle in this zone. Recycling has approximated to the annual allowance of 5,000 m^3per year. Emergency works have not been necessary as the alarm thresholds have not been reached. The rate of shingle loss was estimated at 40,000 m^3 during the first year, allowing for initial adjustment losses; this was very close to the actual performance. Settlement and subsequently typical longshore transport losses of 16,000 m^3per year were projected. The lower than projected loss rate of an average 7,500 m^3per year has meant that the first interim recharge (10 years) has not been required. The rate of loss of beach material has reduced since 2007.
Modelled data distributions of design stage and post-construction comparisons of significant wave heights are shown for the Milford-on-Sea wave buoy site (1974-1990 and 1996-2011) (Figure C.3.12). Comparisons show the percentage of wave heights within each height band. The plot shows that hindcast post-construction conditions (1996-2011) were generally more severe than those used for design. This is likely to have impacted on longshore transport rates.

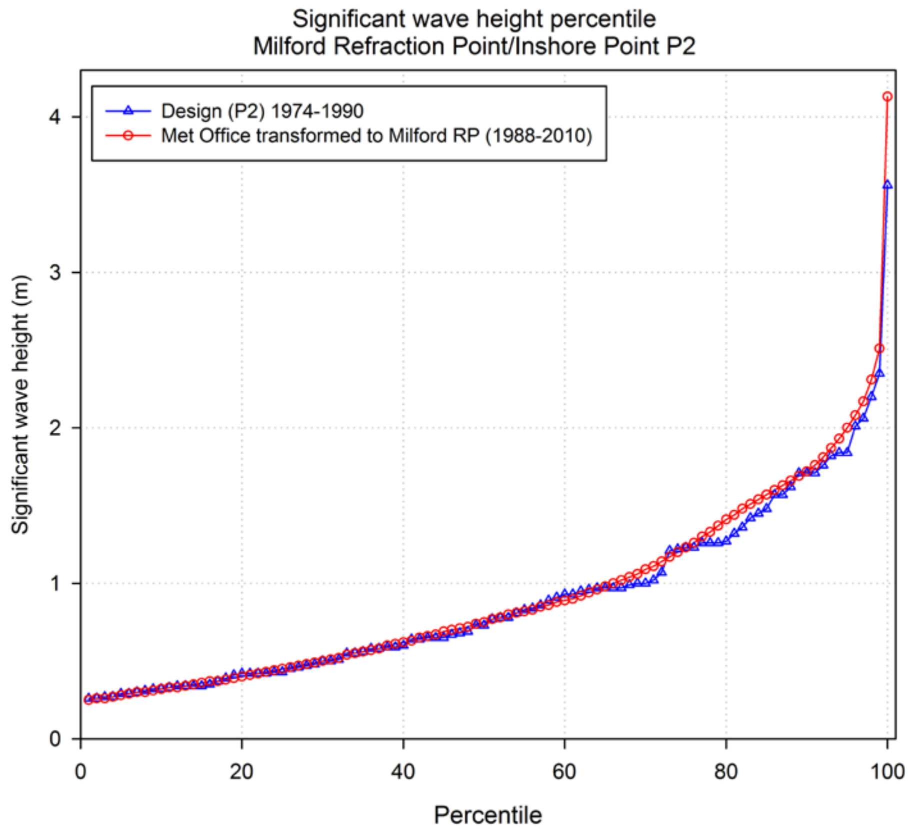


Figure C.3.12 Percentage distribution of pre- and post-construction significant wave heights

While similar comparisons cannot be made between the pre- and post-construction measured and modelled data, synoptic post-construction comparisons of measured and modelled wave data are shown for the Milford-on-Sea wave buoy site (1997-2011) (Figure C.3.13). The measured data were compared with transformed data from the Met Office 25 km wave model (1997-2011). Significant differences in wave climate characteristics are evident between modelled and measured wave conditions. The models typically overpredict significant wave height (H_s) conditions when $H_s < 2$ m.

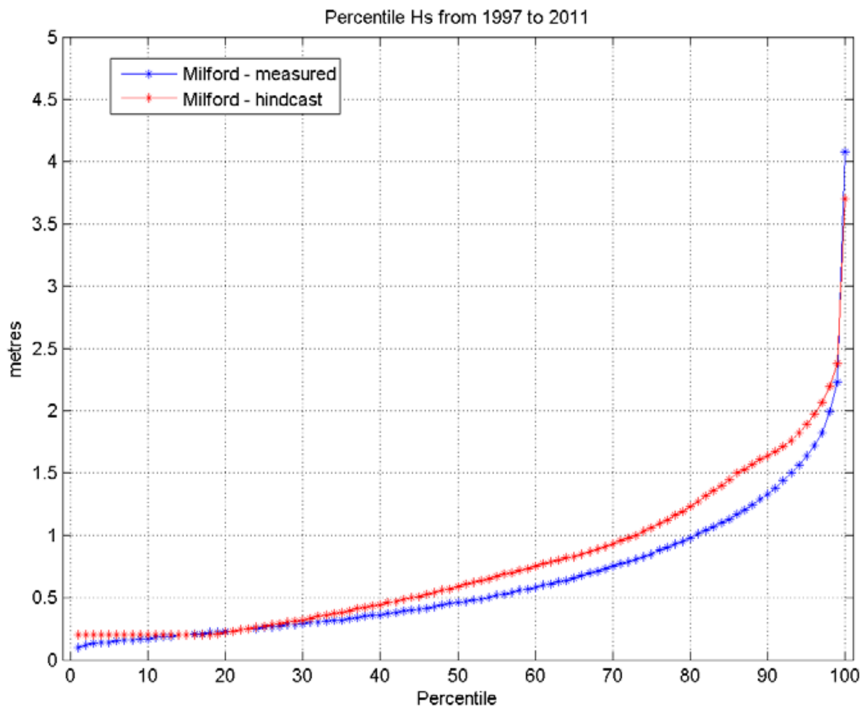


Figure C.3.13 Percentage distribution of modelled and measured post-construction significant wave heights

By contrast, a comparison of wave period (T_z) data suggests that the model typically overpredicts wave period by about 20%, although the data are very widely scattered. These patterns are both typical of systematic differences observed between modelled and measured data at a network of wave buoy sites along the English Channel (Bradbury et al. 2006). This implies wider ranging significance of these observations at many sites where model data have been used for design. Wave periods for design extreme conditions were based on the assumption that wave steepness would be comparable with the more extreme events in the statistical record of hindcast events. The implication of the differences between measured and modelled wave climate is that consistently steeper wave conditions have actually occurred than were expected. The combinations represented in the design phase physical model tests were based therefore on wave height and period combinations with longer wave periods than have actually occurred. The implication on profile response is examined further below. Wave climate statistics produced from field measurements since scheme implementation suggest that the design wave conditions derived from a 16-year hindcast are not representative of the characteristics of storm events from 1997 to 2011 on this basis.

An event-by-event comparison is made of predicted and measured H_s for storm events above a measured threshold condition of 3 m (Figure C.3.14) for events that have occurred since construction. This confirms the general observation identified within the bulk statistics that extreme conditions with measured H_s > 3.5 m are generally under represented by the modelling approach and provides further confirmation that the design conditions used are not representative of a 1:100 year return period event.

Note that there are considerably more frequent measured storm events above the threshold, than modelled. This implies underprediction of these events by the model.

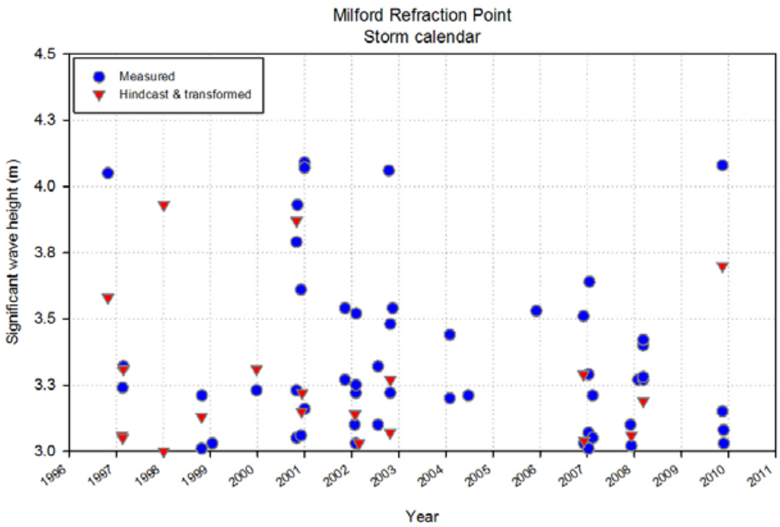


Figure C.3.14 Comparison between measured and modelled post-construction events

In common with many design investigations, the test wave conditions used in the design phase were based on extrapolated extreme offshore wave heights, using a three-parameter Weibull distribution; these were subsequently transformed to the nearshore locations. The measured data suggest that the extrapolated 1:100 year return period event has been exceeded in three consecutive years as demonstrated by the 0.05% exceedance level observations; this implies that the sample period of model data used for scheme design has either been too short, is not representative of more recent conditions, that the model underpredicts extreme conditions or a combination of these. This in turn suggests that the actual design conditions used are representative of conditions that are somewhat more frequently occurring than the desired 1:100 year event, although the shallow water prediction site limits wave heights. The measurements are directly comparable since the wave buoy was co-located at one of the design phase wave prediction points. For the purposes of consistency, the conditions are shown for the standard three-hourly wave records. The three-hourly record is an historical artefact, originally limited by data considerations, but is now widely used. The wave buoys are however able to resolve wave conditions on a 30 minute basis. It is clear from records that the measured storm peaks with duration of 30 minutes may be significantly higher than the three-hourly values. While some difference might reasonably be expected, the differences are such that the 30 minute records are likely to result in significantly different beach responses, since the beach can respond rapidly over a 30 minute cycle.

A further characteristic of the measured wave climate has highlighted that more than 40% of the storm events above the 2.4 m threshold are characterised by spectra with bimodal wave periods. This observation is consistent with similar patterns observed at many sites in the English Channel and has region-wide significance (Mason et al. 2008). The design conditions have made no provision for testing with such conditions and the implication of this is examined further in conjunction with cross-shore beach response below.

C.3.5 Comparative analysis

Observations have demonstrated some significant differences between monitored performance and predictions at the design phase. Many of the differences in performance are interlinked. Overall the scheme has performed better than predicted, despite conditions being significantly more severe than anticipated at the design phase. Best practice design methods have been adopted consistently. In this instance, the under- and over-design elements seem to have cancelled each other out; this is attributed to good luck rather than adequate science.

The monitoring has had a major impact on management of the beach system. It has demonstrated clear differences by comparison with modelled expectations and has provided the basis for modification of maintenance and long-term planning requirements. The monitoring has been particularly valuable for the purposes of evaluation of threshold damage levels and for long-term planning of interim recharge requirements. The main difference has been that the first interim recharge has not been required in line with original projections.

The slowing rate of change of beach volumes is attributed to gradual adjustments of the beach alignment, with the beach becoming more closely swash aligned. The implication of this gradual realignment is that the sediment transport rates have generally reduced. On average, the actual longshore transport rates (1998-2008) have been about 45% of the initial predictions suggested by the modelling (estimated at ~16,000 m^3per year). Although the observed changes are about 45% of the model predictions, this might be considered a reasonable result relative to realistic modelling expectations. Subsequent sample model runs based on one year of measured and modelled time series within a beach plan shape model suggest that use of the measured data reduces the drift potential to about 60% of that provided by the modelled wave data (about 9,000 m^3per year). This observation suggests that the modelling of sediment transport rates may perhaps be more reliable when using measured data, particularly for the first five years following construction when losses averaged about 11,000 m^3per year.

Modelling of wave climate has proven to be significantly different to measured conditions and this is likely to have impacted on sediment transport rates. Post-construction measurements of wave conditions suggest that severe storm conditions have been generally rougher than those suggested at the design stage (Figure C.3.13). This is partially countered by the observation that the wave model overpredicts wave heights for conditions where H_s <2 m.

While the measured wave conditions have been somewhat different to those expected, the beach has performed generally better than predicted. Applications of empirical models of profile response (Powell 1990) and barrier breaching (Bradbury 2000) have been tested at full scale by reference to measured wave conditions. Where conditions have been characterised by similar conditions to those developed in the physical model based empirical frameworks, results have been comparable. The implications of differences in wave climate observations suggest that lower run-up might be expected under most conditions, since the wave period appears to have been overpredicted. Many conditions observed have been outside of the range of the empirical frameworks. The implication is that wave run-up should be lower than modelled and that the as-constructed crest might be higher than is optimal, resulting in a more reflective beach face; this has generally been the case. This is countered however by the impact of bimodal conditions.

Despite the fact that the beach has remained in good condition, overwashing has occurred on a number of occasions, in surprising circumstances relative to the design morphodynamic expectations. Detailed examination of wave climate conditions associated with these events has identified that the wave conditions were characterised by bimodal spectra on each occasion and that a significant proportion of the energy component (20–40%) has typically been in the swell energy range of frequencies. Cross-shore profile responses are not well described in bimodal wave period conditions, which occur regularly. The models generally underpredict wave run-up and crest cut back in such conditions, when simple integrated wave parameters (H_s, T_m) are used. While these observations are not conclusive, it appears that the threshold curves are not valid for prediction of overwashing under bimodal conditions. Broadly similar results have been observed at numerous other sites, on a region-wide basis (Bradbury et al. 2007), which suggests that spectral shape is a key variable that is not normally considered in the design process. The response of the beach under these conditions appears to be worse than conditions defined by spectra with simple shapes. This is consistent with other laboratory observations of profile response (Coates and Hawkes, 1998). Current design guidance does not provide an obvious means of dealing with this design variable, apart from site-specific physical model testing. Ongoing investigations are being conducted to expand the database of measured responses of beaches to bimodal wave periods to determine more appropriate definitions of overwashing threshold conditions.

Monitoring has identified a need for a general review of the scheme standard of service and the need to redefine design conditions by reference to bimodal wave conditions. This means a detailed reassessment of design conditions is required to provide an assessment of the frequency and intensity of bimodal conditions. There is a further need to improve design and assessment methods to make appropriate allowance for such conditions.

Wave climate

- The design significant wave height has been exceeded on numerous occasions since scheme construction in 1996.
- Wave period measurements are more widely scattered than wave modelling indicates, but the measured periods are typically about 20% lower than models indicate. This reduces the wave run-up under typical conditions.
- Wave steepness is subsequently greater than models suggest in most extreme conditions.
- Wave conditions since scheme implementation have not been generally representative of those modelled at the design stage. Modelled H_s is generally higher than measured H_s for all but the most severe events.

Wave conditions have been generally more severe than those tested in design

- A high frequency (>40%) of storm events are represented by wave conditions with bimodal (period) characteristics, not considered at the design stage. This increases run-up and overwashing potential considerably.
- The trends in wave climate characteristics are observed also on a region-wide basis, and have significant implications for design and management at many sites elsewhere in southern England.

Plan shape evolution and sediment transport

- The breakwater has provided the expected stabilising effect as a headland structure.
- Plan shape evolution has been broadly similar to that suggested by the physical modelling process.

- Sediment transport rates have been generally lower than predicted by numerical models. This may reflect the fact that moderate measured wave conditions are generally less severe than modelled conditions. Consequently longshore losses from the system have been lower than design phase predictions suggest. The longshore variability of sediment transport rate has matched that anticipated at the design stage; this is evidenced by a build-up of material in the lower energy zones.
- Gradual changes to the planform orientation of the beach have occurred since scheme construction, which has resulted in swash alignment and a consequent reduction in sediment transport rate and sediment losses.

Cross-shore performance

- Overwashing is underpredicted by the breach prediction model in bimodal wave conditions, but performs well when conditions lie within the limits of the original parametric framework.
- Cross-shore responses have been broadly similar to those modelled, but actual wave conditions have only infrequently matched those tested.
- The predictive framework used to assess alarm conditions has been extended to allow for more typical measured conditions (those with higher wave steepness).
- Adjustments to the empirical framework have not been achieved for bimodal conditions; this requires a more systematic approach to determining the effect of bimodal conditions. Currently the empirical framework will underpredict the possibility of overwashing using standard bulk statistic period variables.
- Broadly similar results have been observed at numerous other sites on a region-wide basis, which suggests that spectral shape is a key variable that is not normally considered in the design process.

Geotechnical performance:

- Measurements of settlement have shown predicted settlement rates to be generally higher than measured; this has provided additional material which can be used for recycling and thereby helping to extend scheme life.

Sediment grading

Although the physical model was designed with material with no sand content, this does not appear to have had an adverse effect on scheme performance. However, beach slopes differ from those modelled and the lower beach slopes are generally flatter than modelling of shingle with no sand fraction might suggest. This appears to be less of a problem for assessment of the upper beach which consists of the coarser fraction of sediments and which performs more in line with the physical model. The sand fraction achieved in the recharge (20%) is unusually small at this site however.

The scheme has performed generally better than might be expected under the actual wave conditions, which have been generally worse than anticipated. The differences between the general overprediction of model wave periods and no recognition of bimodal conditions at the design stage appears to have balanced out, although this is still a problem for long swell events.

The performance of the breakwater has been successful and this reflects the detailed approach to physical modelling. It seems unlikely that numerical approaches to modelling would have produced the level of detail provided by the physical model.

The monitoring programme has provided timely and detailed assessment of performance, and has enabled rephrasing of the next interim recharge at considerable cost saving, resulting from deferred expenditure.

C.3.6 Lessons for future beach modelling/design

Where possible, design wave climates should include, as a minimum, several years of measured wave data to replace or complement numerical hindcasts. The Met Office wave model has been superseded by WAVEWATCH III since 2008; this model appears to reproduce wave heights more reliably on the south coast, with the bias evident in the Met Office model being removed. In order to provide design conditions, appropriately long-term hindcasts will be needed based on this model. This approach will improve the ability to model sediment transport more accurately, since this is strongly dependent on wave height data.

Where wave run-up or overtopping is significant, model wave data should be validated against measured data and adjusted to reflect the measured wave periods. WAVEWATCH III does not appear to reproduce wave periods more reliably than the older Met Office model, although it does provide a better frequency resolution and less scatter. This issue remains problematic and it is recommended that measured data are used to complement synthetic wave data where possible.

Assessments of wave climate should examine the outputs of models and measured data carefully to determine whether bimodal conditions occur at the site. Site-specific tests should be conducted which reflect such conditions to assess the increased risk of overwashing. A probability distribution of bimodal events should be produced to allow assessment of the risks of these conditions. Experience at the Hurst Spit site suggests that regularly occurring bimodal conditions may do more damage than extreme events determined using conventional extremes analysis methods.

Validation of the predictive curves for overwashing provides confidence in this assessment approach, for the range of conditions tested. Extensions to the framework for steeper wave conditions and bimodal conditions may be applicable elsewhere.

The detailed approach to scheme monitoring is summarised in a single plot in Figure C.3.15. This approach to scheme management provides a comprehensive review of scheme performance to date and provides confidence in future projections.

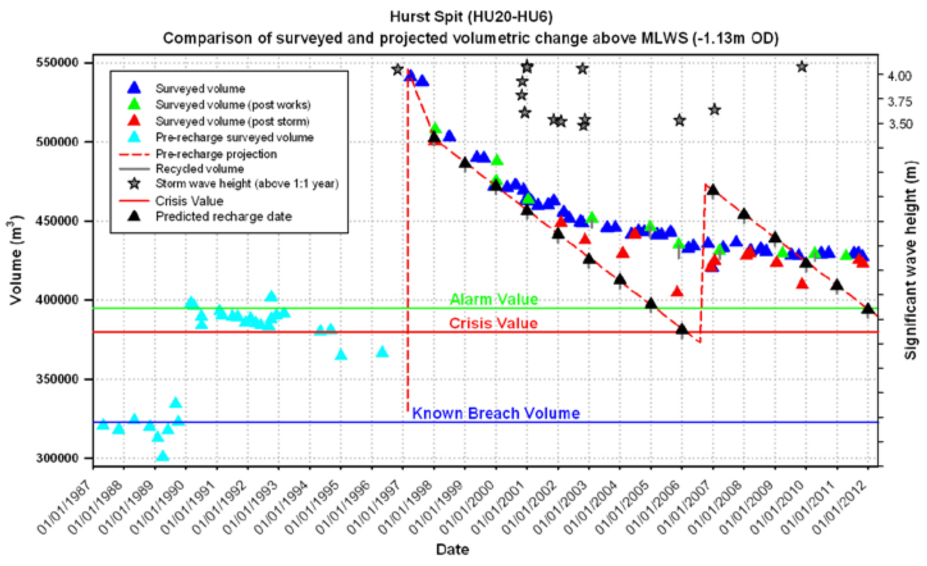


Figure C.3.15 Scheme performance timeline

C.3.7 Bibliography

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C.4 Lincshore

C.4.1 General information

Lincshore (Mablethorpe to Skegness) Beach Nourishment
1994 – ongoing
Environment Agency, Anglian Region

Figure C.4.1 Location of site and beach topographic survey profiles

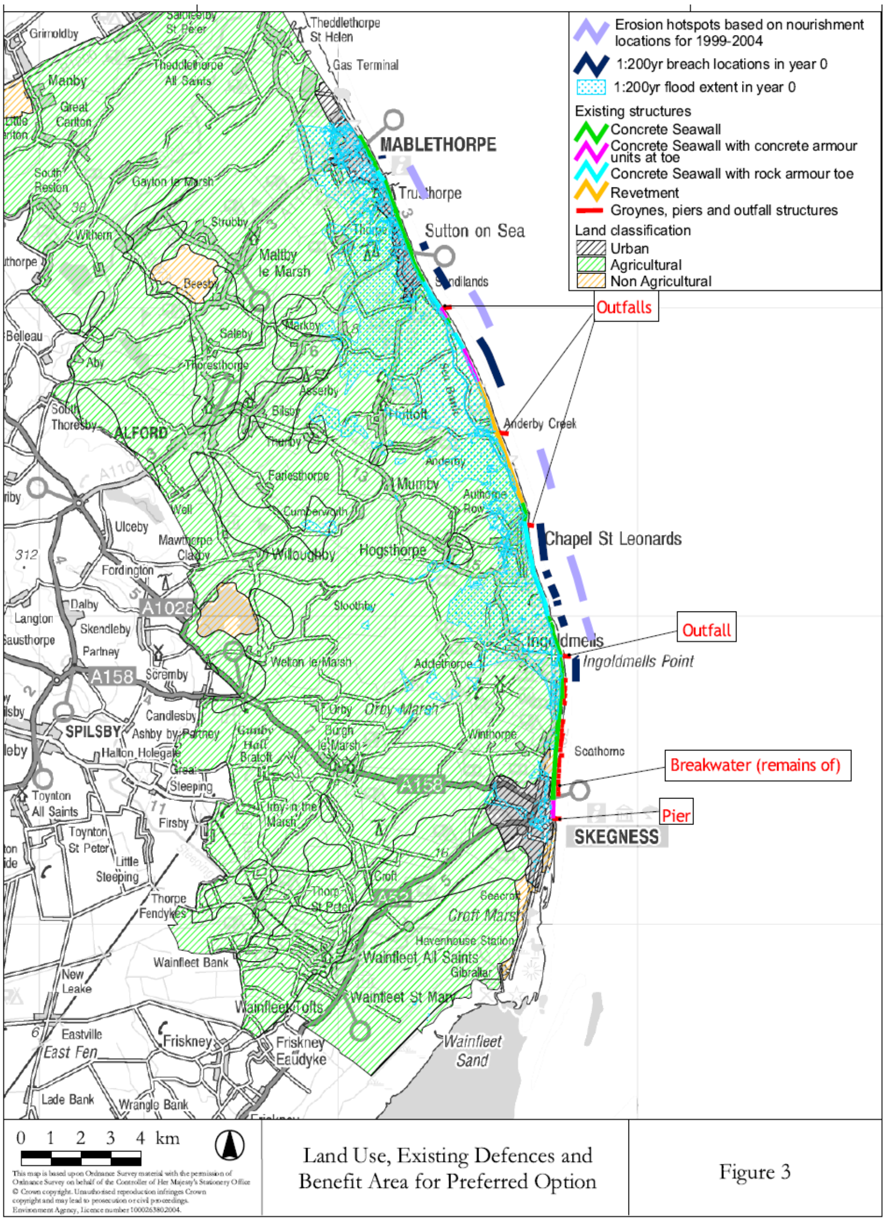

Figure C.4.2 Details of site

Note: map taken from 2004 strategy document but erosion hotspots shown are also relevant for the top-up nourishments undertaken from 2007 to 2012.

Background

The low-lying Lincolnshire coastal flood plain extends up to 15 km inland and over 30 km along the coast (Figure C.4.1). It includes over 35,000 ha of land and 24,400 properties of all types, including the coastal resorts of Mablethorpe and Skegness, with many lying below mean sea level (Figure C.4.2). Coastal flood risk to the area is managed by a system made up of sand dunes, seawalls and a managed beach, much of which has been artificially renourished over the last 18 years.

There was a major breach of the defences on the night of 31 January 1953 when a surge tide broke through in numerous places and 41 people were killed as a result of the flooding. Many defences were rebuilt in the aftermath and have required maintenance, repair and upgrading ever since. In the late 1980s it was recognised that the increasingly higher and larger seawalls and revetments were exacerbating the lowering of the beaches and there was significant exposure and ongoing loss of the underlying clay, compromising the toe of the defences. A different approach was required to deliver a long-term solution.

Key sources of information

- Beach profile data since 1959, records of annual beach nourishment campaigns since 1994, modelling and analysis of coastal processes and beach performance in 1991, 1998, 2004, 2008
- Beach management plans drawn up by Halcrow in 2004 and 2009
- Pre-scheme data included Anglian Coastal Management Study beach monitoring from 1959 to 1992

A long-term strategy defining the approach to deliver the hold-the-line policy through beach nourishment was initially developed in 1991. The strategy was reviewed in 1998 and 2004, and is currently under review again. The strategy reviews and associated scheme approvals have all concurred with the SMP hold-the-line policy and consistently found that the beach nourishment scheme with an open beach is the preferred option on technical, economic, environmental and social criteria.

C.4.2 Approach to modelling and basis of design

Rationale

There have been a number of studies using modelling, surveys and data analysis to assess conditions and scheme requirements since 1991 when the original strategy was developed.

Overview of approach

Modelling has included:

- modelling (hindcasting) of offshore waves, transformation of waves to nearshore with spectral backtracking wave ray model (1987-1991)
- cross-shore numerical modelling of storm beach profile response including sand movement and clay down-cutting (1991)
- sediment transport modelling (longshore transport, cross-shore transport and coastline evolution) (1995)
- surf zone modelling to evaluate cross-shore losses of recharge (1995)
- sediment transport and shoreline evolution with one-line models (1995 and 2003)

The annual beach nourishment campaign relies largely on analysis of the annual post-winter beach surveys. More detailed analyses of the long-term survey records were undertaken in 2003 and 2008.

The 1987 review of groynes on the Lincolnshire coast (Anglian Water 1987) included analysis of wind and wave data from Dowsing and Humber light vessels and wave refraction modelling to determine nearshore wave climates.

Modelling for the original strategy development used hindcast offshore waves which were calibrated against recorded data at Dowsing and transformed to nearshore points using spectral wave back-tracking model. This work was undertaken prior to the strategy development as part of the Anglian Coastal Management Study (Halcrow 1991). Modelling for the strategy review in 2003 used the spectral wave model embedded in SANDS to transform offshore wave data to five inshore points as indicated in Figure C.4.3. These nearshore points were then used as boundary conditions to the local models built into the numerical cross-shore and beach plan shape models.

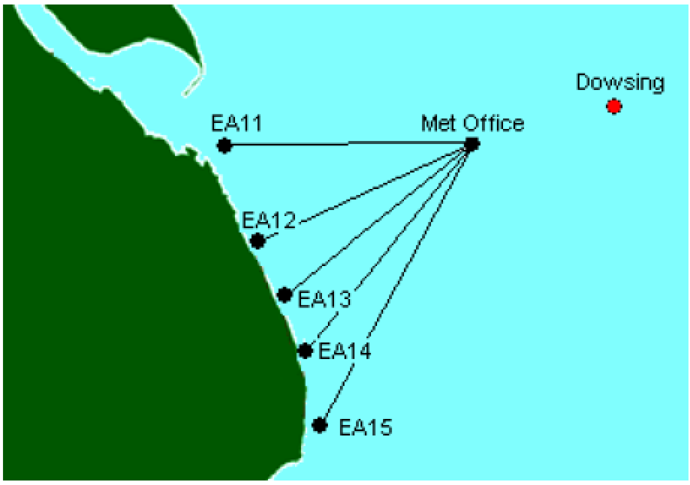

Figure C.4.3 Transformation of waves to Lincolnshire coast for 2003 strategy review

Original modelling in 1991 used offshore hindcast data based on wind data measured at Spurn Point from 1978 to 1987, adjusted to be representative for the Dowsing Light Vessel and calibrated against observed data from Dowsing.

Met Office offshore Met Office European Waters modelled waves for 10 years (1991-2001) were used in the 2003 strategy review.

No measured nearshore wave data were available for calibration at the time of original strategy development or the review in 2003.

Recorded water level data from Immingham and Boygrift from 1991 to 2001 were used in the 2003 strategy review.

Modelling included:

- numerical modelling of long-term beach plan shape evolution
- cross-shore modelling of beach response to storms and down-cutting of clay profile
- cross-shore modelling of the distribution of alongshore sediment transport across the surf zone

Beach profile data from 1959 to early 1980s with monthly cross-sections was used in the Halcrow (1987) groynes review.

Beach and bathymetry profile extension surveys were collected several times each year from 1991 to 1999 as part of the scheme. Since 2000 there have been annual beach surveys in the winter (December and January) to inform nourishment volumes.

There are additional strategic beach profile surveys undertaken biannually (winter and summer) since 1991 as part of the regional strategic monitoring programme, at 1 km intervals.

Bathymetric surveys are carried out every five years as part of the regional strategic monitoring programme.

Baseline conditions

As part of the 1991 strategy study, a detailed sedimentological investigation was commissioned for both the beach (March 1992) and the offshore areas (June 1992) to determine the nature and spatial extent of the existing beach and seabed sediments (EGS 1992). Geophysical surveying was also undertaken to determine the depth of any sand deposits in the offshore areas.

Monitoring of scheme

In response to environmental concerns, periodic (generally three times per year) particle size distribution monitoring has been undertaken since nourishment commenced. These are less extensive, covering only about 10 transects between Mablethorpe and Gibraltar Point and three sample points (upper, mid and sub-tidal zones of the beach) at each transect.

Blott and Pye (2001) undertook an extensive sediment sampling programme at 13 transects, spaced at 500 m intervals between Anderby and Ingoldmells between 1998 and 2000. The samples were taken at 30 m intervals from the sea wall to the spring low tide level at each transect.

Selection criteria: Availability; similarity to natural beaches in area; mobility.

Natural sediment

The 1992 report suggests typical D_{50} 1.74–2.05σ (0.26–0.24 mm) while Pye and Blott suggest 0.2–0.25 mm.

Modelled sediment type (size)

Sand overlying clay, various sizes assessed (0.25–0.65 mm) to inform design quantities and profile.

Sediment in final design

Sand – D_{50} 0.6 mm

Sediment placement approach in final design

Pumped ashore and spread on beach with land-based plant (Figure C.4.4).

Figure C.4.4 Photograph of recharge operation (courtesy of Halcrow).

Prior to the scheme

In 1986 there were around 260 timber groynes of about 50–80 m in length along the Lincolnshire coast. These were constructed after the 1953 floods to try to retain sand on the beaches. The groynes had been maintained until the mid-1980s, but the lowering beaches made them susceptible to damage and required more frequent maintenance. An assessment of the performance of the timber groynes (Anglian Water 1987) concluded that there was little evidence to suggest that the groynes had a marked beneficial effect in retaining sand along the majority of the coast. This assessment was supported by the 1991 strategy and subsequent reviews. In areas of recharge, the groynes have been removed or buried. The remaining groynes on the south of the frontage, where there has not been recharge, are progressively being removed as they deteriorate.

There are four main promontories, or semi-artificial headlands, in the seawall located at Chapel Point, Trusthorpe, Vickers Point and Ingoldmells Point. These locations are vulnerable to wave attack and so protected with rock armour rather than recharged. However, adjacent frontages are erosion 'hot spots' that require frequent nourishment.

With scheme

Options for beach control structures considered in the strategy have been rock groynes, rock reefs or an open beach with annual nourishment of about 350,000 m^3 per year. The reviews of the strategy undertaken so far have all concluded that the open beach solution is preferred on economic, technical and environmental grounds. However, the sustainability and affordability of annual recharge was questioned when the approval for the 2010-2014 works was given and the strategy is currently under review before approval of the next phase.

Initial modelling for 1991 strategy

The initial modelling for the strategy considered the following factors:

- selection of sediment size
- development of design minimum profile/volume
- cross-shore transport
- longshore transport
- storm losses
- analysis of sediment budget
- mixing of existing and renourishment sediment
- effects of short groynes and promontories
- renourishment options

The variability of the natural foreshore was attributed to the relatively fine natural sediment size of 0.12–0.2 mm). The analysis of storm response related to nourishment size indicated significantly less volatility for sediment of 0.5 mm D_{50}, which was subsequently recommended for the nourishment with a 1:25 slope.

Analysis of longshore transport for the natural beach material found a net potential southerly drift at Mablethorpe of about 130,000 m^3 per year. South of Mablethorpe the potential drift increased by 2–3 fold, but due to the denuded state of the beaches, this was not realised. The proposed recharge with coarser material was expected to reduce the maximum net transport rate between Mablethorpe and Skegness to a transport of around 125,000m^3 per year (175,000 m^3 north;

50,000 m^3 south).

No nearshore wave data were available to calibrate the wave models and no actual sediment transport data were available to calibrate or verify the sediment transport calculations.

Modelling studies for phase 2 works (1995)

Cross-shore modelling was used to recommend beach slopes and berm levels for a range of sediment size from D_{50} = 0.25 (1 in 40) to 0.65 mm (1 in 22).

Longshore sediment transport rates from models were used to determine coastline retreat and advance over a four-year assessment period to estimate the required berm crest widths. Transport rates were modelled for a range of sediment sizes (see Figure C.4.5). Accounting for initial losses and design beach slopes options for phase 2 nourishment quantities varied from 16,324,000 m^3 for 0.25 mm sand to 4,311,000 m^3 for 0.6 mm sand. The option taken forward was 0.6 mm sand with an estimated required total volume of 5,107,000 m^3.

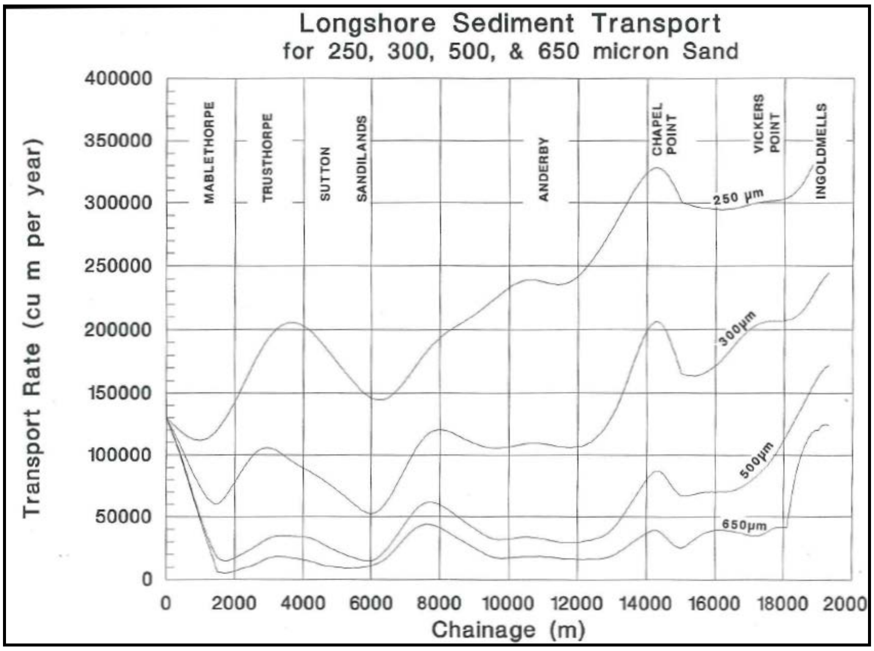

Figure C.4.5 Longshore sediment transport of options for nourishment sand size (Posford Duvivier 1995)

The cross-shore modelling of 0.25 mm sediment (1995) also indicated that around 500,000 m^3 could be stripped from the upper beach of the project frontage during a single storm. This resulted in the selection of coarser 0.6 mm D_{50} sediment in the design to reduce offshore losses. For the coarser sediment, cross-shore shore losses were expected to be negligible and therefore they were not included in the final coastline modelling and the berm widths required were estimated from the longshore transport modelling results.

Modelling for 1998 strategy review

The 1998 strategy review used the same models as the phase 2 works to calculate longshore sediment transport. There was apparently good agreement between the modelling and survey results since 1994, and this was used to justify the calculated transport rates for the prediction of required renourishment works, although there was less agreement for recently renourished frontages. Topographic surveys did apparently show that more erosion was occurring on the upper beaches than predicted and the design profile was reviewed to achieve beach profile equilibrium more quickly.

Modelling for 2004 strategy review

The Interim Review of the scheme in 2001 questioned the dominance of longshore processes and concluded that more modelling was required to assess the 2D transport of sediment, through both cross-shore and longshore processes, rather than focusing primarily on the longshore transport. Numerical cross-shore and beach plan shape modelling was undertaken.

The cross-shore modelling undertaken for the strategy review (2003), together with recorded events, indicated that the beaches are extremely volatile, and that beach levels adjacent to the seawall can vary by up to 3–4 m in a single storm.

Modelling for 2009 Project Appraisal Report

No sediment transport modelling was undertaken for the strategy performance review (2008) or the subsequent Project Appraisal Report (PAR) for the next five years (Halcrow 2008). The analysis undertaken looked primarily at beach performance data, but also included a review of recorded and modelled wave conditions. It was found that the Met Office hindcast offshore wave data underestimated storm waves, particularly for storms from the north. This is considered further for this case study in section C.4.4.

The southern North Sea is particularly susceptible to storm surges. In the North Sea region, positive storm surges normally develop as low pressure systems travel eastwards across the North Sea. The duration of a storm surge is several hours, which is often directly related to the duration of the strongest winds. During a storm surge, the mean sea level rises and higher waves approach the shore, resulting in erosion of material from the beach and dunes, for example, at Gibraltar Point. Typical storm surge height for a 1 in 50 year event has been recorded to be between 1.9 m on the East coast (Flather and Williams 2000) and 2.1 m at Immingham (Lowe and Gregory 1998), with associated maximum wave heights of around 8–14 m and periods of <14s (DoE 1989).

C.4.3 Design/modelling outputs – plans for implementation

Original strategy

The original 1991 strategy allowed for a capital recharge of approximately 7.6 million m^3, which was undertaken between 1994 and 1999, with material placed to the profiles listed in the table below; see Figure C.4.1 for their location. These volumes were based on outputs from the numerical models, although little explanation of the interpretation of the modelling to derive these values was found in the reports examined.

Profile Number (Figure A.2-3&4)	Crest Height (mOD)	Berm Width	Slope
P8	+4.5	10	1:25
P19-P25, P37-P49, P63-P71	+4.5	5	1:25
P9-P18, P50-P62	+4.5	10	1:25
P26-P36, P72-P78	+4.5	20	1:25

Table A.2- 1: Design Berm Widths of Beach

Following the capital nourishment, annual recharge campaigns were planned, to replace annual losses. The original 1991 strategy allowed for the volumes shown in Table C.3.2.

Table C.3.2 Annual recharge volumes

Period	Annual recharge volume (m^3)
1998-2002	67,000
2003-2012	361, 000
2013-2022	327, 000
2023-2032	372, 000
2033-2041	344, 000

It is not clear, however, whether the limited volumes proposed for years 1998 to 2002 were recharge or recycling.

Revised plan from 1998 strategy review

The 1998 strategy review recommended a change in the beach profile design for the upper beach nourishment from a design slope of 1 in 25 to a steeper slope of 1 in 15. This was because analysis of beach response suggested that the addition of recharge material coarser and more poorly sorted than the native material resulted in rapid redistribution and sorting of the sediment and development of a steeper profile than originally predicted, with the actual profile of the recharged material tending to have a 1:10 upper and 1:40 lower slope. The crest berm was kept at +4.5 m OD.

The 1998 review planned for nourishment of about 1.6 million m^3 over the first four years, followed by a reduced annual nourishment of about 155,000m^3 year until 2048.

Revised plan from 2004 strategy review

The 2004 strategy review kept the design crest level at +4.5 m OD and nourishment slope at 1:15, but revised the crest berm widths and expected recharge volumes after extensive analysis of survey data and modelling. A 1.7 million m^3 recharge was planned over three years with subsequent annual recharge averaging 317,000 m^3.

Revised plan from 2009 scheme PAR for 2010 to 2015

Based on analysis of performance, the long-term project renourishment requirements were increased to an average of 341,000 m^3 year with further allowances for future increases to the profile to mitigate future climate change.

The expected actions with regard to renourishment are described above.

No formal beach control structures are required, but the headlands are expected to need refurbishment and improvement as the foreshore drops in front of them in future.

Post-storm activities: the beach management plan includes action and emergency triggers for berm width and also emergency toe levels at which the clay becomes exposed.

C.4.4 Beach management and performance

Details of the scheme

A significant concern regarding management of the coastal defences is the lowering of beaches over the central 20 km long section of the frontage. The sandy beaches are underlain by clay, which is critical to the stability of the sea wall. While the clay is less erodible than the sand it cannot be replaced, so avoidance of erosion and abrasion of the clay is critical to the long-term sustainability of the defences.

The first phase of the strategy involved beach nourishment over a 2 km section to the north of Skegness, completed in August 1995 at a cost of £9 million for placing about 1.5 million m^3 of sand.

Construction of the second phase over the 17 km frontage from Vickers Point to Mablethorpe started in September 1995. This phase had a cost of £42 million and was completed in September 1998. The 6 million m^3 of sand material used for the nourishment was dredged from a commercially licensed source, Area 107, approximately 20 km offshore.

The third phase, from 1999 to 2004, was to renourish the frontage with dredged material to replace the losses due to natural processes. Review of the original strategy documents indicates that these were originally expected to be of the order of 350,000 m^3 per year. However, following review of the strategy in 1998 when the recharge profile was changed from 1:25 slope to 1:15 (as this better matched the natural profile of the recharge), the projected annual losses from 2004 onwards were reduced to be approximately 155,000 m^3 per year (less than half that previously estimated), but following another renourishment of around 1.6 million m^3 over the subsequent four years, at an estimated cost of £17.7 million. The location and quantities of the renourishment campaigns were to be determined each year, based on survey information for the whole frontage. There were, however, funding constraints during this phase and the placement of smaller volumes led to the standard of protection had falling below the 1:200 target in some areas.

The strategy was reviewed in 2003-2004 at the start of phase 4 (2004-2009) and the expected annual requirement was increased to 320,000 m^3 per year. Larger quantities of about 570,000 m^3 in 2005, 2006 and 2007 were required to address the drop in standard of protection during phase 3, with 2.4 million m^3 of sand being placed on the beaches. Maintenance of the beaches following this required nourishment volumes of around 400,000 m^3 in 2008 and 500,000 m^3 in 2009. The cost of this phase was £35.8 million.

In 2008 a strategy performance review was undertaken before preparation of a PAR seeking the approval to the business case for ongoing annual nourishment contract from 2010 to 2015 (phase 5), which planned to place 341,000 m^3 year initially, with increases in future to counteract climate change to 351,000$m^{3.}$per year in 2020 and to 378,000 $m^{3.}$per year from 2060.

Both the capital recharge and the material for renourishment have been dredged with trailing suction hopper dredgers from licensed offshore sites, brought to site and pumped ashore and spread to profile with bulldozers.

As indicated above the beaches are monitored twice each year, so there is now an extensive record of data. Early analysis of the monitoring in 1998, after the initial capital recharge, found that generally the beaches had performed well, with the crest berm remaining stable in the majority of locations. The overall changes in sediment volumes were quite small (±7% of original volume). However, there had been erosion on the upper slope and accretion on the lower slope, such that the placed 1:25 single slope had typically changed to a steeper (1:10) upper slope and flatter (1:40) lower slope. Large volumes of sand had been lost from the promontories, as predicted. Analysis by Blott and Pye (2001) indicated that the coarser sediment was tending to remain on the upper beaches while the finer fraction was migrating seawards, possibly accreting in the intertidal zone. The renourishment placement profile was subsequently adjusted to a 1:15 slope.

A performance review of the strategy undertaken in 2008 used beach volume analysis to demonstrate that there are four main areas of erosion or 'hot spots' along the frontage. The analysis also indicated that, while the strategy recharge campaigns at the hot spots had been sufficient to keep pace with beach losses, there have been periods when beach profiles can revert back to less than the design standard of 1 in 200 years. In effect, this is what triggered the subsequent recharge campaign as a key driver for the scheme is the need to stabilise the beaches to prevent down-cutting of the clay substrate below the sand and gravel beaches to sustain the hard defences at the back of the beaches into the long term.

The review in 2008 also included analysis of wave monitoring data collected offshore from the strategy frontage since 2003. It was found that the offshore wave conditions which were obtained from Met Office model data for the 2003 review may have been underestimated. Hence, sediment transport rates used in the modelling for the strategy review in 2003 may have been too low. It was recommended that further investigation of the standard of protection provided by the design profile was undertaken as part of the scheme PAR development that followed the review in 2009.

The extent of the losses may be partly due to the greater than anticipated alongshore wave energies, as described above, or because beach material had moved into the more stable accretion areas.

The scheme has performed well and maintained a good standard of flood defence for almost 20 years, reduced the erosion of the clay under the beach and protected the seawalls from undermining.

Figure C.4.6 compares the projected required volumes of nourishment with the actual placed volume . Due to beach erosion generally being greater than predicted at each review, the expected required volume has generally increased at each review.

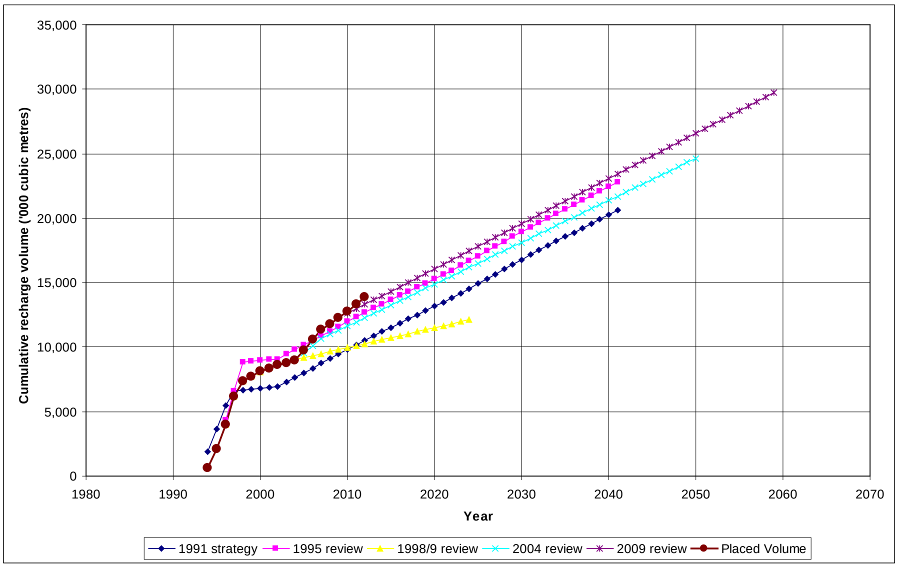

Figure C.4.6 Comparison of predicted to placed cumulative volumes

Note that the 'as placed' volumes from 1994 to 2004 in Figure C.4.6 are based on dredger hopper volumes, whereas from 2005 to 2012, volumes as placed on the beach are used, which may be up to 20% lower than the total volume pumped ashore. However, it is believed that the estimates of the expected required volumes up to the 2004 review were primarily based on uncalibrated modelling, whereas the estimates since 2004 have relied significantly on interpretation of the beach monitoring as well as the modelling.

Analysis of wave data for this case study included transformation of Met Office offshore hindcast data to the nearshore refraction points used in the 2004 strategy review modelling (Figure C4.3). Measured offshore wave data from the Dowsing wave buoy were also transformed to the nearshore points, allowing a comparison of measured to modelled wave data. Figure C.4.7 shows that the wave model is capable of accurately transforming the offshore record to nearshore point EA12. However, Figure C.4.8 shows that, when using synthetic offshore Met Office data, the modelled nearshore waves are significantly underestimated, with the peak 1.9 m measured wave modelled as about 1.5 m. While the Met Office data are now known to not accurately match measured waves, it is the only available long-term data source as the offshore Dowsing buoy has only been in place since 2004 and the nearshore measured wave records are shorter still.

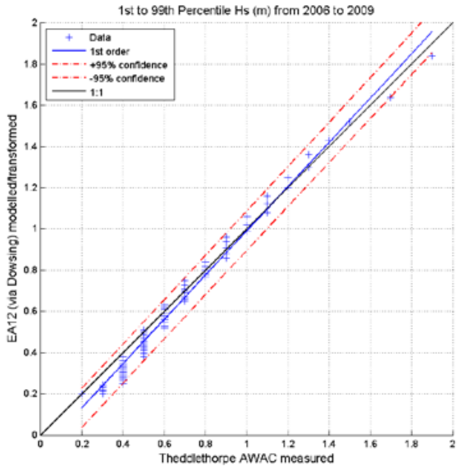

Figure C.4.7 Comparison of modelled to measured waves at EA12/Theddlethorpe using offshore waves taken from the Dowsing wave buoy

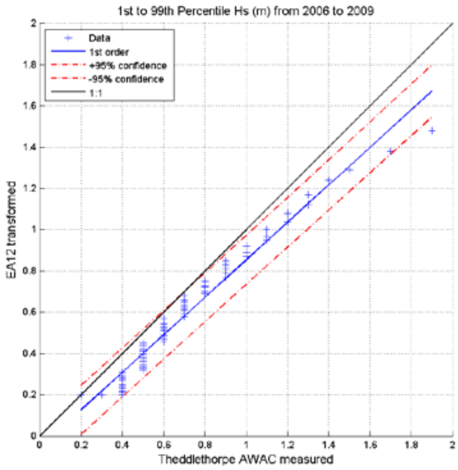

Figure C.4.8 Comparison of modelled to measured waves at EA12 using Met Office hindcast offshore waves

The Met Office offshore wave data from 1988 to present have been used to derive nearshore wave height exceedance plots and storm calendars for the site, and examples are shown in Figures C.4,9 and C.4.10. These appear to indicate that there was a period of relatively low storm intensity between about 1994 and 2008, with larger waves being experienced pre-1994 and since 2008. The changes since 2008 may relate to the change from the 25 km Met Office model to the WaveWatch III model in 2008, but this has not been investigated within the scope of this study.

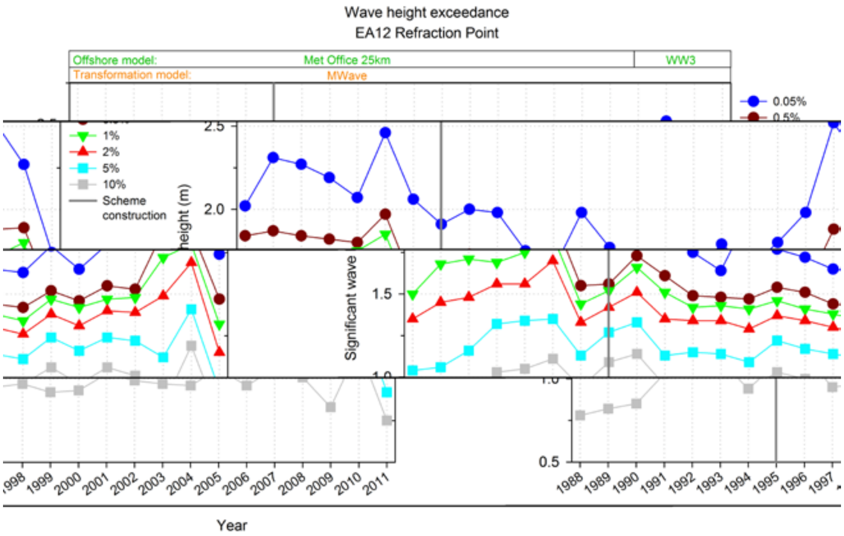

Figure C.4.9 Modelled wave height exceedance plot for location EA12

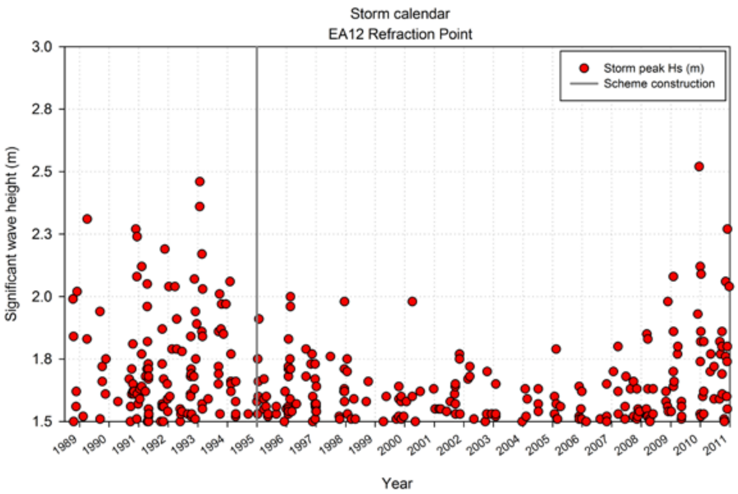

Figure C.4.10 Modelled storm calendar at location EA12

During a review of the scheme performance, Halcrow (2008) measured and modelled waves were reviewed alongside beach volume changes to attempt to correlate erosion with wave energy. Due to the ongoing annual beach management, this was only found to be possible for the beaches to the north and south of the nourished frontage. Figures C.4.11 and C.4.12 present the results for Donna Nook to Theddlethorpe at the north and Ingoldmells to Gibraltar Point respectively.

Figure C.4.11 appears to indicate a general correlation between accretion and years of relatively high wave energy for the coast to the north of the recharge site. However, the volume changes in 2004 and 2005 do not follow the trend.

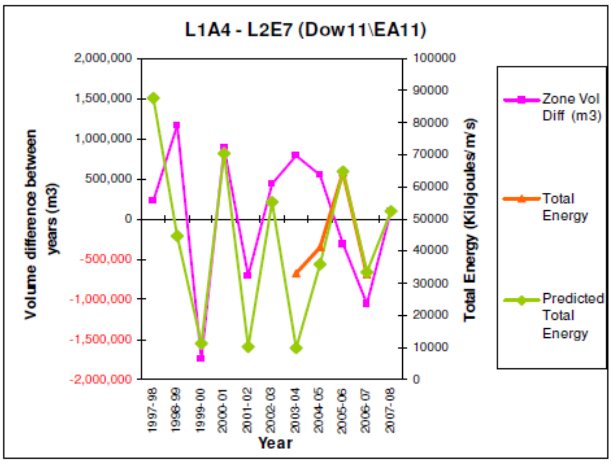

Figure C.4.11 Zone 1 Donna Nook to Theddlethorpe St Helen (Environment Agency profiles L1A4 to L2E7, north of Mablethorpe): annual volume changes and total annual storm wave energy for 1997 to 2008

For Zone 6, to the south of the recharge site, FigureC.4.12 appears to show that in years of higher wave energy there appears to be erosion and when there is less wave energy there is accretion.

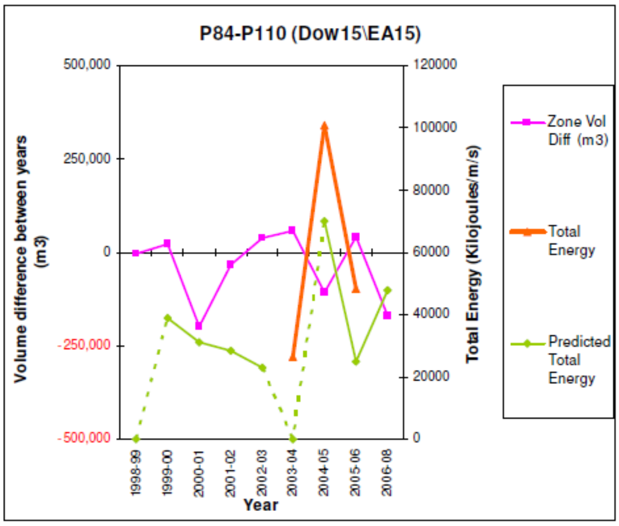

Figure C.4.12 Zone 6 Ingoldmells to Gibraltar Point (profiles P85 to P110): annual volume changes and total annual storm wave energy for 1998 to 2008

C.4.5 Comparative analysis

Figure C.4.6 shows a comparison of estimated long-term nourishment requirements to the timeline of actual cumulative recharge. The cumulative volume of sand placed on the beaches to date is greater than the expected requirement up to the 2022.

At each review of the strategy, the estimated future nourishment requirements have gone up with the exception of the 1998 review when steeper beach profiles were introduced. The 1998 long-term estimates were significantly less than the original strategy and all other estimates. It is possible that offshore losses occurred, which may have been underestimated in the 1998 review.

The storm calendar in Figure C.4.10 indicates that the actual environmental conditions may have been less severe for most of the time since scheme initiation than may have been estimated in 1994. However, the original wave modelling did not use Met Office synthetic offshore wave data (only three years of data would have been available) as a long-term hindcast was used. The original wave data used were not available for consideration in this case study.

Analysis of wave monitoring data collected offshore and at nearshore points along the strategy frontage indicates that the Met Office offshore wave data used in the 2003-2004 strategy review may have underestimated wave conditions, particularly omitting larger storms from the north. This means that the 2003 modelling may have underestimated sediment losses through longshore transport towards the south. However, the original wave modelling for the 1991 strategy was calibrated against observations at Dowsing and should not have had this issue. As the sediment modelling in 2003 was calibrated by cross-comparison to both the earlier modelling and actual site performance, the calibration of the longshore drift modelling appears to have compensated for the inadequacy of the offshore wave data used.

The underestimation of annual losses and hence recharge requirements may be at least partly due to lack of calibration data for the sediment transport models. No modelling has taken place since 2003, which was before the first measured wave data were available. There are now records of measured offshore and nearshore waves, regular beach monitoring and beach nourishment data available, which could be used to develop calibrated models in the future.

The actual beach management uses survey data alongside model predictions to determine actual annual recharge requirements.

The flexible approach of using measured data to update and improve modelled estimates has allowed the beach management plan and future cost estimates to be updated as the scheme has progressed.

C.4.6 Lessons for future beach modelling/design

The lessons that can be drawn from this study which may of benefit to further schemes include the following.

- The modelling should consider both the native (pre-erosion) sediment and options for available sediment from recharge sources under consideration.
- Met Office model wave data may underestimate actual conditions, and therefore adjustment or calibration of the data should be considered before use.
- When uncalibrated models are used to derive long-term requirements for beach recharge, suitable contingency factors should be included in deriving final estimates or the models should be revisited as better data becomes available.
- Regular review of the performance and updating of the beach management plan as additional data becomes available is important, especially for large schemes where beach response is highly volatile.
- As longer term monitoring datasets become available they can provide a more reliable means to predict and plan future beach performance, but ahead of those data existing comprehensive and wide ranging modelling can be critical for assessing and selecting the most appropriate beach management approach.
- The objectives of all modelling exercises and how they relate to one another need to be clearly documented. Furthermore, the links between the model findings and subsequent design/implementation need to be explicitly documented.

C.4.7 Bibliography

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C.5 Littlestone

C.5.1 General information

Littlestone Scheme Improvement Works
2002-2004
Environment Agency

Figure C.5.1 Location of site

Background

Littlestone-on-Sea is located on the south coast of England between Folkestone and Rye and approximately 10 km north of Dungeness (Figure C.5.1). The Littlestone frontage is approximately 3.8 km long, the majority of which is protected by a concrete sea wall. The low-lying hinterland is part of the Romney and Walland Marshes, which form an extensive flood risk area covering well over 50% of Shepway Council's district. There is very little contemporary sediment feed into the Littlestone frontage and therefore the shoreline along this section of coastline is subject to erosion. The net littoral drift of shingle is northwards, but the direction of sediment drift is extremely sensitive to small changes in wave direction, meaning there is potential for periodic drift reversal along the study frontage.

The Littlestone Sea Defence Scheme was constructed between April and September 2003. These works included 260,000 m^3 of beach renourishment, strengthening and raising of the concrete seawall, construction of a new promenade and construction of a terminal rock groyne. The original scheme was designed to protect the town of Littlestone from coastal flooding for events having a return period up to and including 1 in 200 years.

As part of the detailed design of the Littlestone scheme, a numerical modelling study of the coastal processes along the Littlestone frontage was undertaken. The objectives of this study were to:

- provide wave and water level data to be used in the scheme design
- assess the performance of scheme options
- optimise design details for the preferred scheme option

Supporting background studies

Strategy reports

The ‘Littlestone to St Mary's Bay Sea Defences Beach Management Plan’ was prepared in 2006 following completion of the capital scheme. The plan covers the initial performance of the scheme and also includes a summary of the coastal process analysis that had been undertaken up to that date. It also examines the performance requirements of the sea defences as well as the vulnerability of the original seawall to breaching and overtopping. The main output from this report is the establishment of storm event thresholds for different sections of the frontage, and the beach crest widths and elevations required to provide the required standard of protection against failure.

To take into account the changing risks on the coast, the existing ‘Folkestone to Rye Strategy’ and the ‘Cliff End to Scott’s Float Strategy’ have since been reviewed and combined to produce a single management strategy. This new strategy, known as the ‘Folkestone to Cliff End Flood and Erosion Management Strategy’ contains updated information on flood and erosion risk. This strategy has recently been approved confirming beach recharge and major sea wall refurbishments to be undertaken during the first part of the strategy appraisal period (50 years), with a ′sustain′ policy for the second part of the strategy period (50–100 years) in line with the second generation SMP.

Additional reports

In 2009 Herrington Consulting Ltd was commissioned by the Environment Agency to examine available data for the Littlestone area and to provide a hypothesis on the processes acting on the frontage. The Littlestone Report on Options for Scheme Improvement Works (2009) investigates various options for improving the performance of the existing scheme as well as assessing any potential environmental impacts.

Details of the coastal defence scheme:

Prior to the implementation of the existing scheme, the land, properties and road to the rear of the seawall was exposed to frequent overtopping during the winter months. The original sea defences were formed from a concrete seawall that had a crest elevation of approximately 6.3m ODN; however, this was in a poor state of repair and was too low to provide an appropriate standard of protection.

The ‘Folkestone to Rye Coastal Defence Strategy Study’ (HR Wallingford, 2001) identified the Littlestone frontage as one of its ‘priority action’ frontages and recommended that the existing seawall be improved and the shingle beach renourished. This strategic option was developed further, through the Project Appraisal Report (PAR) stage by the Environment Agency’s consultants and between 2002 and late 2004, the Littlestone Sea Defence Scheme was constructed in three phases.

These works consisted of improvements to, and raising of, the concrete seawall to 7 m ODN, and a capital renourishment of approximately 260,000 m^3 of shingle. This allowed for placing the nourished beach with a slope of 1:7 and a minimum crest width of 10 m. The raised seawall provides the enhanced level of protection against overtopping and incorporates steel flood gates at access points.

The 260,000 m^3 of shingle, dredged from the Hastings Bank licensed site, was

delivered to the Littlestone frontage via pipeline and over-bow pumping. The shingle was bulldozed to form the design beach profile. The full standard of protection is provided by the combination of the shingle beach and the sea wall. The sediment transport modelling undertaken as part of the detailed design phase of the project identified that the net annual sediment transport direction is south–north. On the basis of this information, the scheme also included construction of a terminal groyne at the northern end of the frontage. The purpose of the terminal groyne was to prevent shingle migrating north and on to the St Mary's Bay frontage, where it would be not be easily recovered. Furthermore, this scheme design would allow for future recycling of the shingle that accretes against the terminal rock groyne back across the frontage. The analysis undertaken at the scheme design stage suggested that approximately 5,000 m^3 of material would need to be moved annually from north to south along the frontage.

C.5.2 Approach to modelling and basis of design

Numerical modelling was undertaken to: • provide wave and water level data to be used in the scheme design • assess the performance of scheme options • optimise design details for the preferred scheme option The numerical modelling included joint probability analysis of wave and water level extremes, tidal current modelling, wave overtopping, beach plan shape and alongshore drift modelling, and beach profile cross-shore storm beach response modelling.
No information on the specific wind and wave data used in the modelling process was available at the time of this appraisal.
The joint probabilities for the extreme wave and water level conditions were determined using a simplified approach as outlined in the first edition of the *Beach Management Manual* published by CIRIA in 1996.
No information on the water level data used in the modelling process was available at the time of this appraisal.
A one-line sediment transport model was used but the specific model applied for the original study is not known.
The beach profile data that were available at the modelling stage came from the Strategic Regional Coastal Monitoring Programme. Additional data from the Environment Agency's Annual Beach Monitoring Surveys (ABMS) was used for periods pre-dating the Strategic Regional Coastal Monitoring Programme.
The study area lies within the Dungeness Special Area of Conservation (SAC), the Dungeness to Pett Level Special Protection Area (SPA) and falls partly within the Dungeness Romney Marsh and Rye Bay SSSI. Consequently the coastal defence scheme development for this frontage needed to look carefully at the potential impact of beach recharge on these environmental designations. In the event of a breach in the frontage, the number of properties at risk is around 200. This number will increase overtime with sea level rise. The direction of sediment drift is extremely sensitive to small changes in wave

direction in the study area, meaning there is potential for periodic drift reversal along the frontage.

- **Selection criteria**: It is important to avoid introduction of inappropriate or non-indigenous beach material as it could have adverse environmental impacts on the study area. It was also considered to be essential to avoid disturbance to vegetated shingle as a result of beach recycling operations and burying of established vegetation with renourishment material.
- **Natural sediment:** The foreshore along this frontage is predominantly sandy, with a shingle upper beach. The original D_{50} value of the shingle along the Littlestone frontage is not known and was not published in the design documents/reports. However, given that it was transported there by natural processes it will have been well sorted, that is, its grading will have been relatively narrow. Consequently, it is likely that this material would have had a D_{50} value similar to other natural shingle beaches in this area, which is in the region of 15–17 mm.
- **Modelled sediment**: Shingle
- **Sediment in final design**: Shingle with grading based on analysis of the existing beach.
- **Sediment placed**: The grading curves for the as-placed material show that the D_{50} value of the renourished material was approximately 12 mm. Visual inspections undertaken since the placement of the material, however, suggest that some of the shingle has a significantly lower D_{50} value and is also much wider graded than would normally be specified for a shingle beach.

Prior to the scheme

There were long wooden groynes at 50–200 m spacing along the frontage. The lower ends of the groynes, where they extend onto the sandy foreshore, were in a semi-derelict state. They appeared to have little impact on the condition of the lower foreshore, with no beach level differential evident across them.

Modelled structures

The modelling of the defence scheme considered only the rock groyne at Jesson outfall.

C.5.3 Design/modelling outputs – plans for implementation

The design profile of the renourished beach had a beach crest at +6 m ODN and 10 m crest width along the frontage except for at the Jesson Outfall where the crest width at installation was 7 m. The design beach profile at all locations was 1 in 7.

The scheme was designed to incorporate beach management and the three trigger levels for intervention were identified in the design documents as follows. Thresholds for intervention are summarised in the table below (see also (Figure C.5.2).

- **Installation:** This is the position upon completion of the construction works and will provide the base case for the consideration of beach movement. No intervention will be required.
- **Minimum design:** When this threshold is reached there may be a need for recycling of material to this part of the frontage. Beach movements can fluctuate rapidly depending upon conditions and at this stage a watch will be kept on the

beach with the potential for material to be recycled during the annual recycling exercise.

- **Urgent:** This will identify where the beach width has been reduced to an extent where the design standard of defence is not being met. This should initiate immediate corrective action to restore defence standards.

Thresholds for the five sections of the frontage were defined as shown in Table C.5.1.

Table C.5.1 Frontage thresholds

Unit ID	Description	Installation	Minimum design	Urgent
18/1		10 m berm	7.5 m berm	4 m berm
18/2	Jesson Outfall	7 m berm	5.5 m berm	2 m berm
18/3	Golf Course	10 m berm	7.5 m berm	4 m berm
18/4	Pirate Springs	10 m berm	7.5 m berm	4 m berm
18/5	South of Terminal Groyne	10 m berm	7.5 m berm	4 m berm

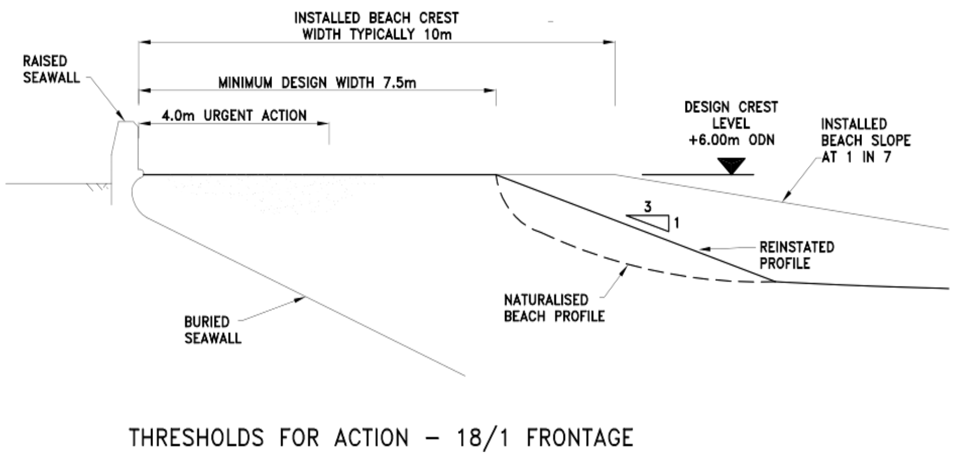


Figure C.5.2 Littlestone to St Mary's Bay Sea Defences Beach Management Plan, 2006 (courtesy of Jacobs Babtie)

The design analysis of coastal processes predicted the potential for annual recycling of approximately 5,000 m^3 of material from areas of deposition to areas of erosion. Specifically, it was anticipated that this net drift would be in a northerly direction and thus would accumulate at the terminal rock groynes and close to the Jesson outfall.

C.5.4 Beach management and performance

Following completion of the scheme two key issues were readily apparent. The first was that material was moving in a southerly direction (opposite to the predicted direction); the impact of this was most evident at the RNLI and Varne Boat Club slipways. Here the shingle had built up and overtopped the two slipways causing operational issues for the RNLI and fishing club. The build-up of shingle in the vicinity of the RNLI slipway continues to cause the outfall to the New Romney Main Sewer to become blocked at times. This is a fundamental issue in terms of scheme performance and the way in which the beach along this frontage is managed over the longer term.

As well as the inconvenience caused to the RNLI and the boat club, the migration of material southwards has fundamental impacts on beach management because of the environmental designations. It is not possible to undertake beach recycling works in areas south of the slipways and therefore any material bypassing this point was effectively lost from the system. Given that there is no significant feed into the system, such losses will result in an overall reduction in beach volume and consequently a drop in the standard of protection provided.

In addition to managing the southerly component of sediment transport along the frontage, the Environment Agency has also experienced problems in undertaking beach recycling operations using the material from the northern extents of the frontage. The original scheme was designed on the basis that approximately 5,000 m^3 of material would be recycled from north to south each year. However, because of the geometry and design of the terminal groynes, there is a limit to the volume of material that can be stored at this location before it bypasses the groyne and heads north onto the St Mary's Bay frontage. Furthermore, the groyne does not include any barrier that is permeable to shingle-sized sediment. Consequently, the shingle is free to flow through the voids between the rock armourstone resulting in little recent accretion at the terminal rock groyne.

In 2005 and between 2009 and 2012, beach recycling works were undertaken along the Littlestone frontage. Each year, between 3,000 and 21,800 m^3 of mixed sand and shingle was taken from the frontage immediately in front of the Varne Boat Club and RNLI station and deposited along the foreshore fronting the Littlestone Golf Course. Following the recycling, any excess shingle was removed with an excavator and the beach re-profiled to a 1:7 gradient with a 10–15 m crest width to ensure a maximum level of protection is maintained. Figure C.5.3 shows an example of beach monitoring data in front of the golf course.

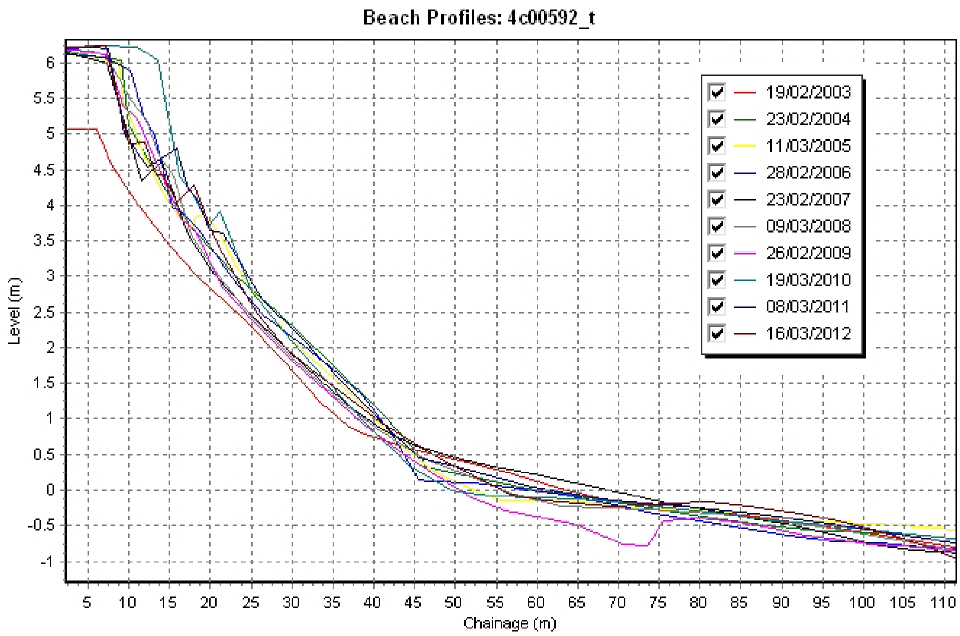


Figure C.5.3 Example of beach profile monitoring data from Littlestone Golf Course

Wave conditions

Statistical offshore wave data for the study area are provided by the Folkestone Directional WaveRider buoy, which is maintained by the Channel Coast Observatory. Analysis of the data available shows that, for the majority of the scheme's life, there have been two predominant wave directions; these being around 100° and 195°. The largest waves are from the south-southwest direction; however, the most frequently occuring waves are from around 100° and have a significant wave height of between 0.5 and 1.5 m (Figure C.5.4).

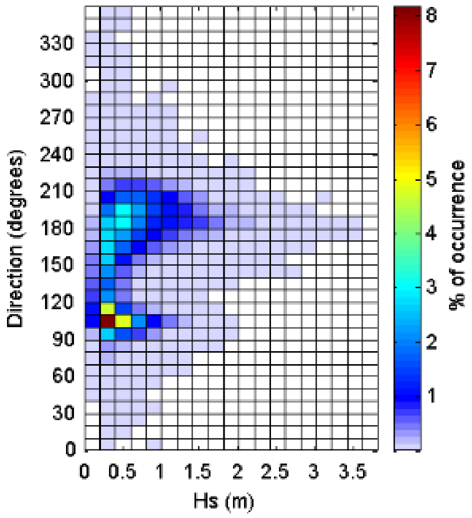


Figure C.5.4 Joint distribution plot of wave height and direction at Folkestone (2003-2010)

Due to the sheltering effect of the Dungeness Peninsular and the orientation of the Littlestone shoreline, the directional window for waves that would result in a northerly transport of sediment is limited to between 100° to 160°. From examination of the the offshore wave data recorded by the Folkestone wave buoy, it can be seen that there is a significant proportion of wave energy present within this directional sector. For material to be moved in a southerly direction, it is necessary for waves to approach the shoreline from an angle of less than 100°. When this is compared with the directional spread of waves recorded by the Folkestone wave buoy, it can be seen that there is only a very small percentage of wave energy within this sector.

The results of the numerical modelling undertaken as part of the detailed design for the scheme show that, for the analysis period (1971-1998), approximately 25% of the overall modelled wave climate in this location was from between 55° and 100°. This is less than from the 100° to 160° sector, and is why the predictions from the sediment transport models show a net northerly transport of material.

Further analysis based on UK Met Office modelled data between 1989 and 2011 provided by Halcrow show that there have been very few large storm events (>1.8 m) since completion of the scheme in 2006. This trend becomes drastically apparent when compared with the first half of the wave record (1988-2001) where storm events occured much more frequently and with significantly greater peak wave heights of up to 2.1 m (Figure C.5.5).

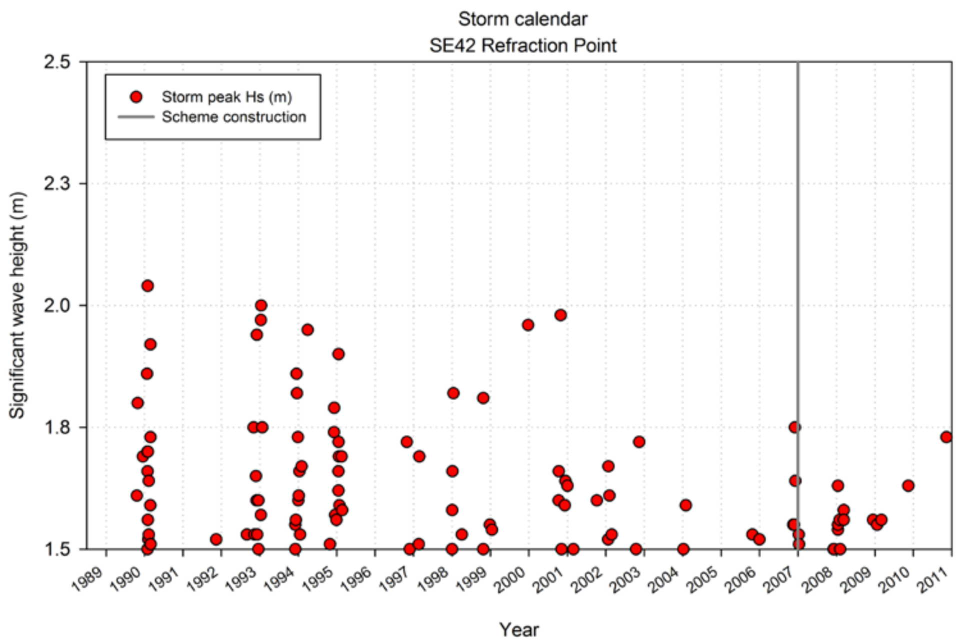


Figure C.5.5 Storm events at location SE42

Directional data for these storm events are not known, although anecdotal evidnece suggests that many of the larger events experienced before the completion of the scheme were southerly storms. These would have contributed significantly towards the northerly sediment transport component. The reduction in large southerly storm events may well be a contributing factor to the net southerly transport experienced post-scheme.

The plots in Figure C.5.6 illustrate the comparison in significant wave height percentile pre-scheme and post-scheme. The greater occurrence of larger wave heights (0.5–1.5 m) prior to scheme construction indicates that the sediment transport modelling may have overestimated sediment transport rates post-construction.

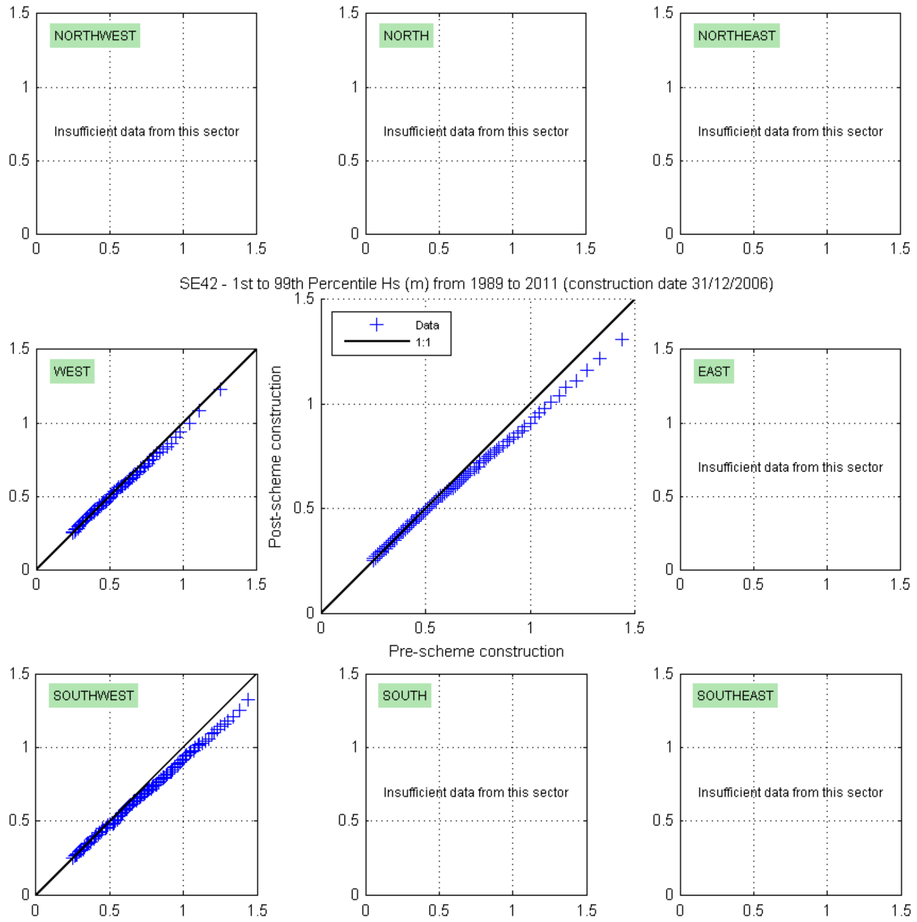


Figure C.5.6 Comparison of pre- and post-scheme wave climate

C.5.5 Comparative analysis

- **2005:** Following completion of the scheme, recycling works involving 5,000 m³ material were carried out in 2005.
- **2006:** Summer topographic surveys indicated that of the 17 beach profiles in this area, four were below set critical levels, and a further nine were showing an erosive trend over time. Shingle re-profiling was carried out in autumn 2006 along approximately 2.5 km of foreshore between Jesson outfall to Littlestone water tower.
- **2007:** A reduction in crest width throughout much of the shingle bank along the frontage continued to be observed during monitoring surveys. The erosion was partly offset by accretion of material at the toe and foreshore level. However, with the continued reduction in crest widths for much of the frontage the number of profiles approaching crises level inevitably increased. No beach management work was undertaken in 2007.
- **2008:** Beach monitoring surveys indicated that losses sustained over the past year had resulted in beach volumes returning to levels prior to 2003, when the beach recharge was undertaken. The combined effect of volume loss and beach level lowering resulted in beach levels being far below design standards. Recommendations were made to investigate future provision of coastal protection and the improvement of existing standards. No beach management work was

undertaken in 2008.

- **2009:** 21,800 m^3 of replenishment material was bought from Cemex and an additional 7,000 m^3 taken from the Varne Boat Club at the life boat station and deposited along the length of foreshore fronting the golf course as shown in Figure C.5.7. Post-recharge surveys indicated that the accumulation of shingle from the recharge works resulted in the seaward advancement of the upper crest of profiles in this location by up to 5 m.
- **2010:** Following a reduction in crest height of up to 2 m in some places along the Littlestone frontage, beach recharge works were undertaken in the spring; 8,600 m^3 of mixed sand and shingle material was taken from the Varne Boat Club at the life boat station and deposited along the length of foreshore fronting the golf course. After the replenishment the beach was re-profiled to a 1:7 gradient with a 10–15 m crest width.
- **2011:** In the spring, 3,000 m^3 of sand/shingle was taken from the Varne Boat Club at the life boat station and deposited along the foreshore, followed by re-profiling of the beach slope to a 1:7 gradient with a 10–15 m crest width (awaiting results of analysis from the 2012 beach monitoring surveys).
- **2012:** Beach recycling and re-profiling works were undertaken in the spring, which saw 7,500 m^3 of mixed sand and shingle material taken from the Varne Boat Club at the Life Boat station and deposited along the foreshore as shown in Table C.5.2 (results of analysis from the 2012 beach monitoring surveys were not available at the time of writing).

Table C.5.2 Comparison of predicted and actual scheme performance

Year	Predicted scheme performance	Actual scheme performance
2005	No management works expected	5,000 m^3 of beach recycling
2006	Recycling of ~5,000 m^3 from terminal groyne to southern end of frontage	Erosion recorded, but recycling not carried out because of insufficient accretion at terminal groynes.
2007	Recycling of ~5,000 m^3 from terminal groyne to southern end of frontage	Further erosion recorded, but recycling not carried out because of insufficient accretion at terminal groyne and environmental restrictions on moving material from the southern end of frontage.
2008	Recycling of ~5,000 m^3 from terminal groyne to southern end of frontage	Beach levels reaching critical thresholds along much of the frontage. Still no recycling due to lack of material at terminal groyne.
2009	Recycling of ~5,000 m^3 from terminal groyne to southern end of frontage	21,800 m^3 of replenishment material imported to frontage. Further 7,000 m^3 of material recycled from Varne Boat Club.
2010	Recycling of ~5,000 m^3 from terminal groyne to	8,600 m^3 of material recycled from the Varne Boat Club

	southern end of frontage	
2011	Recycling of ~5,000 m^3 from terminal groyne to southern end of frontage	3,000 m^3 of material recycled from the Varne Boat Club.
2012	Recycling of ~5,000 m^3 from terminal groyne to southern end of frontage	7,500 m^3 of material recycled from the Varne Boat Club.

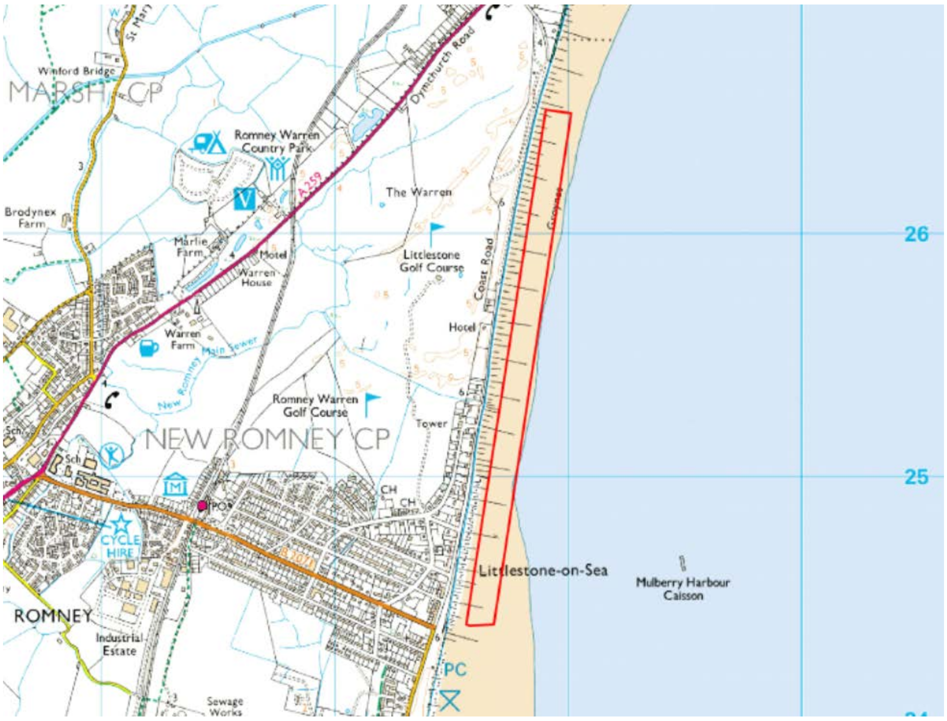


Figure C.5.7 Location of replenishment deposition site

The numerical modelling study indicated that net littoral drift along the Littlestone frontage is from south to north, and therefore the original scheme design was based on this assumption. Consequently, the only control structures included in the original scheme was the rock groyne at the Jesson outfall, designed to prevent material leaving the frontage and heading north onto the St Mary's Bay frontage.

While the majority of shingle movement on this frontage is from south to north, it is also evident that there is also a strong southerly component to the sediment transport regime. This has resulted in the build-up of shingle against and over the RNLI and boat club slipways, and as a consequence causes operational issues for both organisations. While this could have serious ramifications in terms of the speed and safety of the lifeboat deployment, the movement of material southwards across the frontage boundary will also result in a net loss of shingle from the study area.

Furthermore, because it was anticipated that net drift would be in a northerly direction, it was also assumed that material would accumulate at the terminal rock groyne near the Jesson outfall. However, as the groyne does not include any barrier impermeable to shingle-sized sediment, the shingle is free to flow through the voids between the rock armourstone resulting in very little accretion at the groyne. This has created a further problem for the Environment Agency in undertaking the original proposed beach recycling operations at this location.

Since the scheme was completed, there has been a significant loss of sediment from the beach fronting the seawall and this has resulted in a significant reduction in the standard of protection provided by the scheme. The Environment Agency has been undertaking ongoing beach management during this interim period to mitigate the impact of these losses, but a more sustainable and robust solution is sought.

Differences between predicted and observed scheme performance along the Littlestone frontage are due to a combination of the introduction of additional material and the smaller sediment size of this material. A prolonged period of northerly storms and lack of large southerly events has also contributed to the strong southerly component to the sediment transport along the frontage.

While the general intention of the capital renourishment undertaken in 2003 was to nourish the beach with sediment of a similar size and grading to that which already existed on the beach, it is evident that the as-placed material had a significantly lower D_{50} value and had a significantly wider grading. Consequently, the smaller material within the newly placed beach is mobilised under lower wave energy conditions. This in turn means that the point along the frontage at which shingle ceased to be influenced by the wave action has moved south. The introduction of smaller sized sediment has increased its mobility, while the overall increase in material volume along the frontage has also resulted in an increase in the surface area of material exposed to wave action. Therefore the rate of southerly sediment transport in the vicinity of the RNLI slipway has been increased.

During high energy northerly events, a significant volume of material is transported south along the frontage. As there have been no significant westerly storms in the period since the nourishment to offset the southerly movement of shingle, material has built up against the RNLI slipway, which has acted as a groyne structure resulting in accretion. Given the shallower water at the southern end of the study frontage, inshore wave energy is less than that experienced on the slightly deeper and more exposed northern half of the frontage. Consequently, material that has been moved onto the southern half of the frontage is less likely to be moved back in a northerly direction.

In summary the two main reasons for the difference between expected performance and actual performance are as follows.

- The smaller and wider graded sediment used for beach renourishment comparred to the assumed values used for the sediment transport modelling.
- The frontage is sensitive to wave direction and the modelled wave dataset possibly did not reflect the conditions that have been experienced post-construction.

C.5.6 Lessons for future beach modelling/design

The scheme design was based on the premise that the net sediment transport direction was south–north and therefore the terminal groyne at the northern boundary would prevent losses from the frontage. Actual sediment transport regime along this frontage is more complex. While the net transport direction is important in general terms, what is critical is the fact that the consequences of both the southerly and northerly components of this regime result in a different outcome to that which was predicted using the net transport direction alone.

Lessons to take forward from this are as follows.

- For frontages that are potentially sensitive to changes in sediment transport direction, apply sensitivity tests to the directional wave data used in the model.

This can provide an envelope of outcomes from which beach management options and the potential extent of variability/flexibility can be better determined.

- In considering sensitivity, consider also other schemes being examined in the vicinity (for example, wave analysis for the Folkestone scheme was undertaken at a similar time but was there any cross-referencing?)
- When modelling sediment transport using a single sized (D_{50}) value, it is necessary to understand that most as-dredged material will be relatively wide-graded. This may result in a natural sorting of material with finer sediments being transported under more frequent, but lower energy events and larger sediments only being transported under higher wave energy events. The behaviour of mixed sand/shingle beaches is complex and not always well replicated by numerical sediment transport models, but again scenario testing considering a range of sediment sizes can help to better inform the designer of potential variability in the outcome and build that into the management planning.
- Consequently, there needs to be an element of engineering judgement applied to the results of the model. Validation of predictions is not always possible but reference to site inspections, monitoring data and local knowledge is important to consider in providing confidence or raising questions regarding beach response.

C.6 Llandudno North Shore

C.6.1 General information

Llandudno North Shore
Phase 1: 1996-1997 Phase 2: 2000
Conwy County Borough Council
  **Figure C.6.1 Location of site**
Background The requirement for coast protection in Llandudno is linked closely to its development as a holiday resort and as a residential area. The first coastal defences were constructed between 1894 and 1906 to provide amenity (a new promenade) and reduce the threat of breaching of the natural defences, which comprised an upper

shingle bank and lower sand foreshore – the product of erosion of the shoreline and the remnants of nearshore deposits of glacial material left by the ice sheets.

The frontage is effectively a pocket beach some 2 km in length, located between two large rock headlands – the Ormes – which provide control on exposure conditions and beach movement (Figure C.6.1). The earliest coastal defences date from the late 19th century, with the present promenade and stepped concrete revetment constructed in 1937-1938, following a severe storm. At this time a series of cross-shore timber groynes were also constructed. These works were extended in the 1960s.

This technical history shows that a trend of erosion occurred across the Llandudno frontage for most of the 20th century and that successive authorities tried to arrest this erosion by building sea defences as soon as developed land was threatened. Unfortunately, these measures contributed further to the erosion process to such a point that, prior to the implementation of the current scheme, the present defences were in danger of imminent failure.

Desk studies carried out in the early 1980s showed that the natural beach head equilibrium line lay behind the existing sea wall at the western end of North Shore. If the sea wall were to fail in this area with no repair then natural shoreline recession to this boundary would take place with the concomitant loss of over £15 million (1990s prices) of property and infrastructure. Further eastward at Craig-y-Don, the level of defence was lower and the hinterland falls in level moving to landward making this frontage vulnerable to flooding especially if sea levels were to rise. (This area was seriously flooded by the storm events of February 1990.) After consideration of the study results and public consultation, a preferred management approach comprising artificial beach replenishment (with or without control structures) was identified.

Details of the scheme

The scheme was implemented in two phases. The first phase, constructed in 1996-1997 across the central section (1,400 m) of the frontage, consisted of the importation of approximately 60,000 tonnes of material to recharge the beach levels in front of the defences, as well as carrying out repair works to the existing defences, reconstruction of the promenade (which was in poor condition), provision of an improved flood wall at the rear of the promenade and construction of a new slipway access and low level terminal groyne at the eastern boundary of the recharge. A terminal rock groyne was also installed at the western boundary of this phase of the works to prevent westerly drift of the imported beach material into the adjacent section.

The second phase of the works, constructed in 2000, involved extension of the recharge westerly for a further 460 m, importing a further 20,000 tonnes of recharge, together with improvements to the promenade and flood wall, as carried out in phase 1. Furthermore the existing slipway towards the western end of the frontage was enlarged and extended, providing improved access to the lower sand beach for the public and emergency services and providing a permanent terminal groyne. The temporary rock structure was removed at this time.

Key sources of information

- British Maritime Technology (BMT), *Aberconwy Coastal Study Parts I to V*, 1987.
- Aberconwy Borough Council, *Llandudno Coastal Works – Engineer's Statement*, November 1987.
- HR Wallingford, *North Shore, Llandudno: Second Opinion on Proposed Coast Protection Scheme*, Report EX 2633, October 1992.
- HR Wallingford, *North Shore, Llandudno: Mobile bed physical model study of proposed developments*, Report EX 2754, April 1993.

- HR Wallingford, *Appendix to North Shore, Llandudno: Mobile bed physical model study of proposed developments*, Report EX 2754, July 1993.
- Shoreline Management Partnership, *North Shore Coastal Works*, April 1994.
- Coastal Engineering UK Ltd, *Conwy Beach Management Plan (Draft),* March 2009.

C.6.2 Approach to modelling and basis of design

Rationale

Desk study and empirical design approaches had been used in relation to similar works introduced in Penrhyn Bay (1989-1990) and Llandudno West Shore (1991-1992). For this scheme, increased local scrutiny and consultation required a more detailed modelling approach to be taken to inform option development.

Overview of approach

The initial scheme study and development consisted of wave refraction modelling to provide inshore wave conditions and empirically based assessments to define preliminary details. A combination of numerical modelling (further wave refraction, empirical sediment transport calculations was used initially, followed by development of a 3D physical model to examine various beach recharge/control structure arrangements compared with behaviour under existing conditions.

A backward ray tracking wave refraction model was used in preliminary studies to provide inshore wave climate at two locations.

For detailed assessments, a wind-wave hindcast model was used to provide the offshore wave climate in Liverpool Bay. Wave conditions were transferred inshore to single central location using a wave refraction model.

A total of 20 years' recorded wind data from Squires Gate Airport, Blackpool, were used as input for the hindcast model.

No recorded water level data were available locally.

Extreme water levels for site were based on predicted levels for Liverpool and Llandudno and extremes for Liverpool (method of Lennon, 1963).

Cross-shore methods (Powell 1990) were used to inform the definition of an initial cross-shore profile.

A 1:70 scale physical model was developed covering the westerly half of frontage, approximately 1.4 km in length.

Local council historic beach profiles were available at three locations:

- Clonmel St: 15 surveys between 1978 and 1990 + two historic profiles (1900 and 1937)
- Carmen Sylva Rd: 13 surveys between 1978 and 1990
- Boating Pool, Craig-Y-Don: 13 surveys between 1978 and 1990

Selection criteria

Extensive research of available local glacial and quarried sources was carried out. The requirement was for material to match or be coarser than the existing sand/shingle mix for performance and environmental reasons. Sand was unacceptable due to environmental impacts (wind-blown sand and so on) and vulnerability to movement.

- Natural sediment: upper beach shingle; lower beach sand.
- Modelled sediment type (size): shingle (D_{50} = 40 mm)
- Sediment in final design: coarse shingle/cobble (D_{50} = 60–80 mm)
- Sediment placed: coarse shingle/cobble (glacial origin)

Prior to the scheme

A series of wooden groynes was introduced as part of the 1930s scheme. These had either been removed or were largely dilapidated and ineffective by the time scheme introduced in the 1990s. Any remaining groynes were removed as part of the scheme.

Options considered

- Single terminal rock fishtail breakwater/groyne at Pier location (west end)
- Single terminal rock fishtail breakwater/groyne at Pier location, with amended length of easterly arm (±35 m)
- Single terminal rock fishtail breakwater/groyne at Pier location, with trunk length shortened by 15%, 25% and 50%
- Single terminal fishtail breakwater/groyne at Pier location, with landward flank to trunk and east arm replaced with vertical harbour wall
- Single terminal rock fishtail breakwater/groyne at Pier location, with north arm removed
- Beach recharge only
- Beach recharge and land reclamation at west end of frontage

The physical modelling examined the following conditions:

- annual average, 1 in 1 year, 1 in 10 year and 1 in 50 year storm conditions from a northerly direction
- annual average, 1 in 1 year, 1in 10 year and 1 in 50 year storm conditions from a north easterly direction

The scheme proposals at the western end of the frontage required extensive public consultation in order to satisfy local businesses, the general public and the Welsh Office. The scheme was therefore split into two phases that allowed beach recharge and promenade improvement works to be carried out in the centre of the frontage to alleviate the primary flood risk, while discussions and consultation continued with regard to the works to be carried out at the western end. In the event, the Welsh Office would only fund the beach recharge works and no control structures at the western end were constructed.

The profile identified for the recharged beach superimposed on the pre scheme profile is shown in Figure C.6.2.

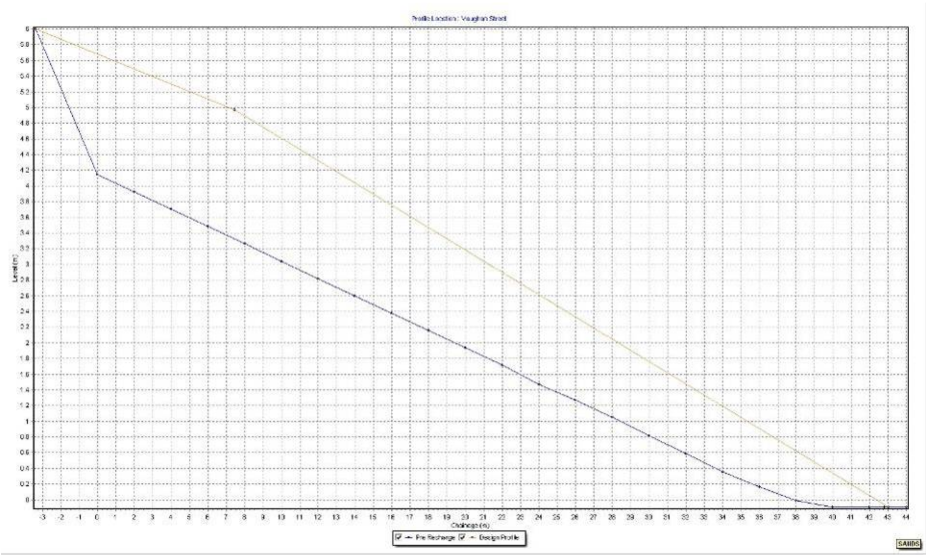


Figure C.6.2 Design and pre-scheme beach profiles

There was public concern regarding impacts on amenity of shingle/cobble recharge. There was also concern about maintenance of access across beach for emergency services.

The beach recharge was placed to a uniform profile consisting of a 10 m wide berm sloping to seaward from +5.0 m AOD at the interface with the existing defences to +4.5 m AOD. From here the beach sloped at a constant 1 in 9 gradient until it met the existing beach profile.

C.6.3 Design/modelling outputs – plans for implementation

The modelling provided the following outputs in terms of scheme performance:

- average drift rates per year at four locations across the frontage – one of these is located within the phase 1 frontage, two are located within the phase 2 frontage and the other was located at the far west end in the section between the Pier and the new slipway
- drift rate increase factors for each of the storm conditions modelled
- comparisons of overtopping performance for each of the storm conditions modelled

The sediment drift rates across the frontage vary with position due to the changing orientation of the shoreline relative to the predominant wave conditions. Annual drift rates expected due to normal conditions vary between 200 and 20,000 m^3 per year, dependant on location with the higher values anticipated to occur over a short length (~250 m) mid-way along the phase 2 length. Drift rate factors under storm conditions could potentially increase rates by 2–4 times and 5–12 times for 1 in 10 year and 1 in 50 year storm conditions, respectively.

The modelling identified the following requirements.

- Material should be recycled from east to west following storm conditions and more regularly once alarm conditions are reached
- Alarm beach crest levels should be set at a level higher than mean high water springs (MHWS).
- Predicted transport rates should be used to define beach management requirements.

C.6.4 Beach management and performance

All beach material provided in phase 1 was obtained from Cefn Grainog near Penygroes, between Caernarfon and Porthmadog (see Figure C.6.3). Material was transported by road to Penrhyn Quay at Bangor, where it was transferred to a bottom dumping vessel, for transport to site and offloading at high water. For the phase 2 works, due to the lower quantities, material was imported to site by road. During low water periods, material was placed to the required profile by dozers.

Modelling did not consider:

- two-phased approach to implementation
- impacts of permanent and temporary terminal groynes
- beach behaviour across the whole of the recharged frontage or the whole bay

The scheme implemented did not include recharge at the far western extremity of the frontage (between the Pier and the new slipway) to avoid impacts on RNLI access.

The D_{50} of material used in the scheme was coarser than that used in the modelling.

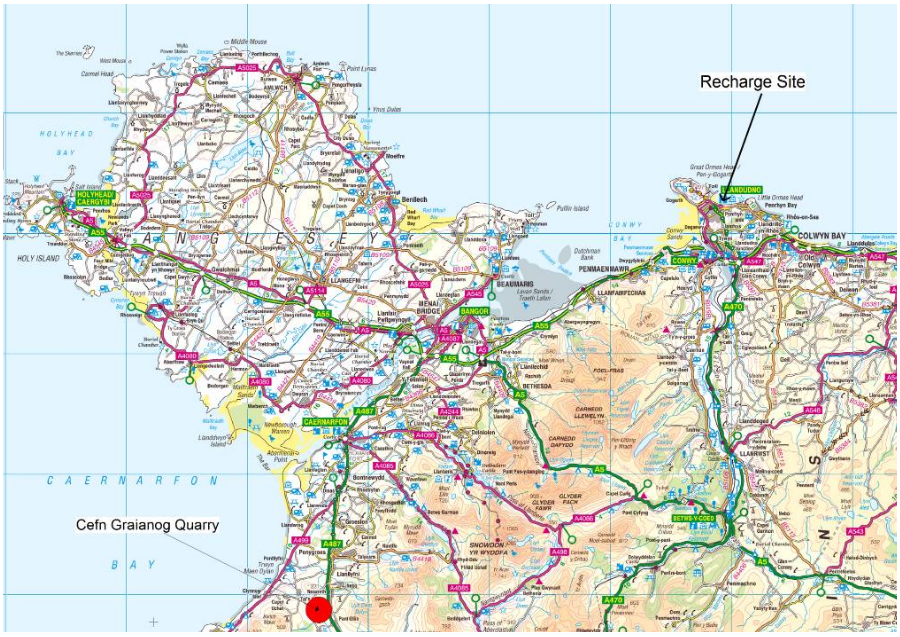


Figure C.6.3 Source of recharge material

Bi-annual topographic monitoring of the foreshore across the whole of the North Shore frontage, including the sections that were not recharged, to the east and at the far west end, commenced in 1997. Post-storm surveys were carried out in February 2005, April 2007 and April 2010. This monitoring includes both the upper shingle/cobble beach and the lower sand beach.

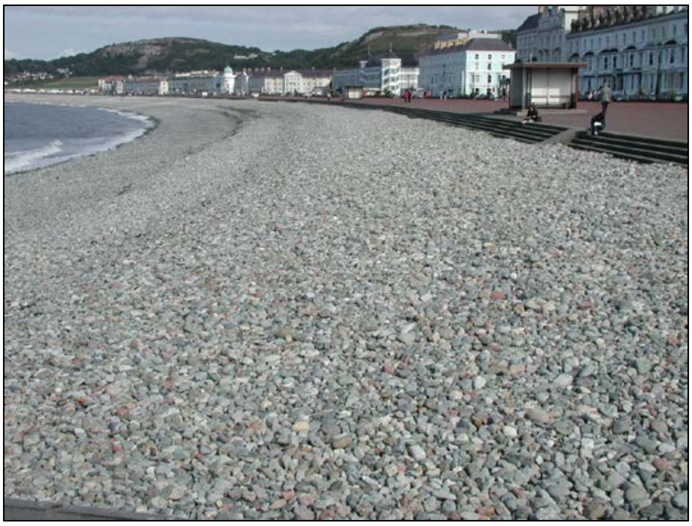
Overall since the present monitoring regime commenced in 1997, following completion of the first phase, approximately 40,000 m^3 of material had been lost overall from the frontage, notwithstanding that 12,500 m^3 was added as part of the second phase of works between Trevor St and Vaughan St in 2000.

Generally material is lost from the upper sections of the beach and transported easterly along the frontage or material is drawn offshore, some of it into the areas below low water mark. Cyclical behaviour is observed between surveys with losses followed by gains, indicating that material that is drawn offshore during storms can be returned but generally only to the lower sections of the foreshore.

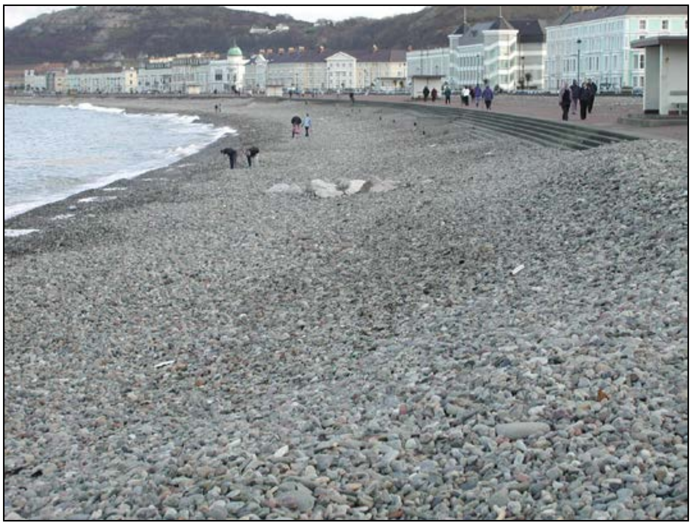
Overall, the area in front and immediately west of the Craig-Y-Don boating pool is in equilibrium with volumes today similar to what they were in autumn 1997. To the west beach volumes are lower, while to the east volumes are higher. Material can, however, move bi-directionally and, prior to the first phase recharge, the frontage to the east of the Craig-y-Don pool lost approximately 40,000 m^3, primarily as a result of a north-easterly storm in 1996.

The permanent terminal groyne at the eastern end of the recharged section of frontage appears to providing a beach retention function by controlling upper beach drift at this end, but the gains on the eastern side indicate that material is bypassing the structure lower down the beach and feeding the frontage to the east.

Under normal conditions the design beach profile is generally maintained (Figure C.6.4A). Under storm conditions, however, the profile deforms with the top of the slope at the seaward edge of the crest steepening and the slope below slackening slightly, as predicted by the Powell model. Storms also cause complete destruction of the crest and beach drawdown (Figure C.6.4B) with some material being thrown up and deposited on the promenade (Figure C.6.4C).



A



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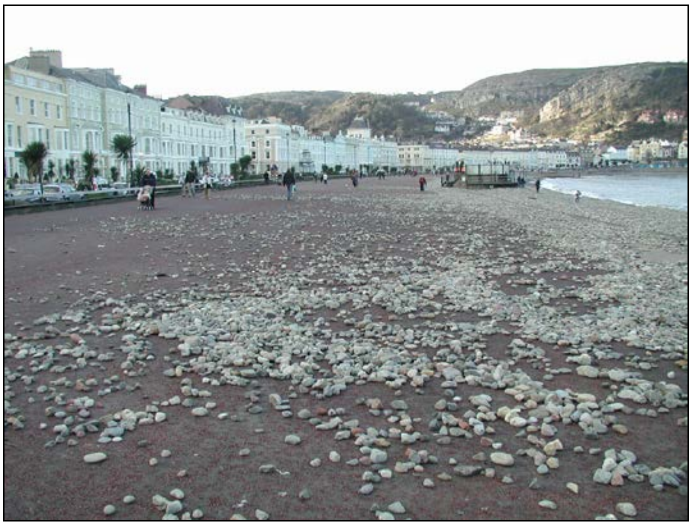

C

Figure C.6.4 Beach profile variations

Beach management carried out since the scheme was implemented has been fourfold:

- retrieval of material moved longshore, but only within the length recharged
- retrieval of material moved on/offshore
- retrieval of material thrown up onto promenade
- re-profiling where beach has steepened due to storm action

Typically beach management has been carried out on an annual basis in April–May, following the winter period.

At times material has been moved on/offshore and upper beach shingle has become mixed with lower beach finer sediments, requiring sieving using a riddle bucket, before being returned to the upper beach. This has primarily occurred at the western end of the frontage, where there is a greater extent of lower beach.

In March 2010, approximately 3,500 tonnes of additional cobble was imported and placed across the phase 2 length.

Figures C.6.5 to C.6.9 present wave condition data from pre- and post-construction. The wave conditions are for a location offshore from the Little Orme headland, at the east end of the bay.

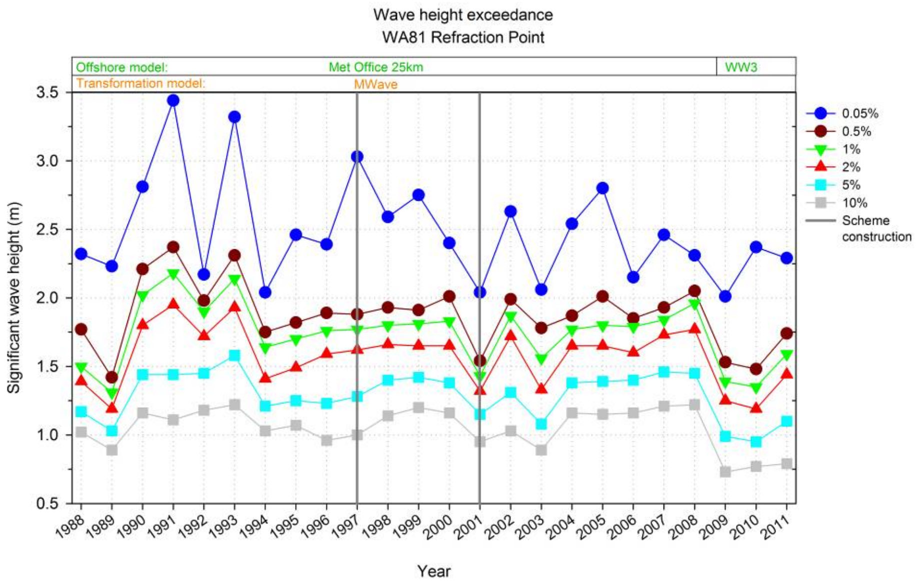

Figure C.6.5 Wave climate pre- and post- construction at point WA81

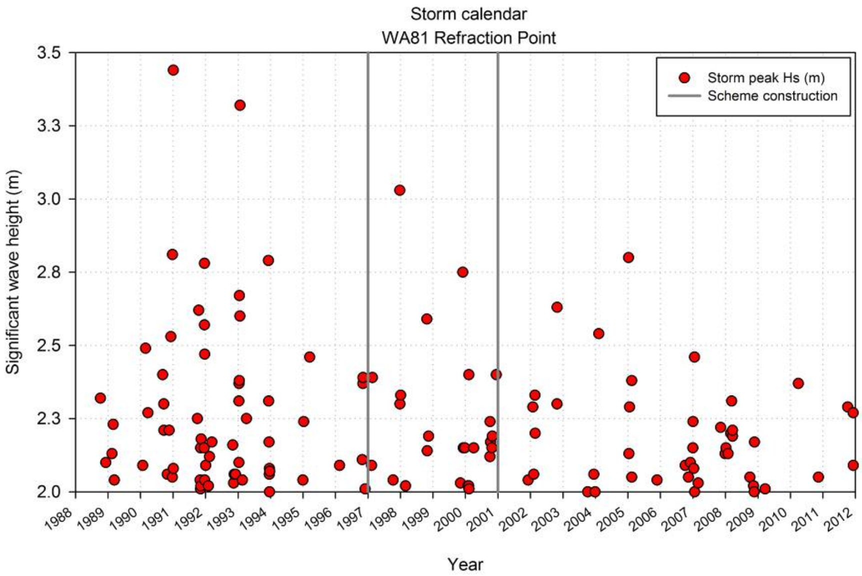

Figure C.6.6 Storm events at point WA81

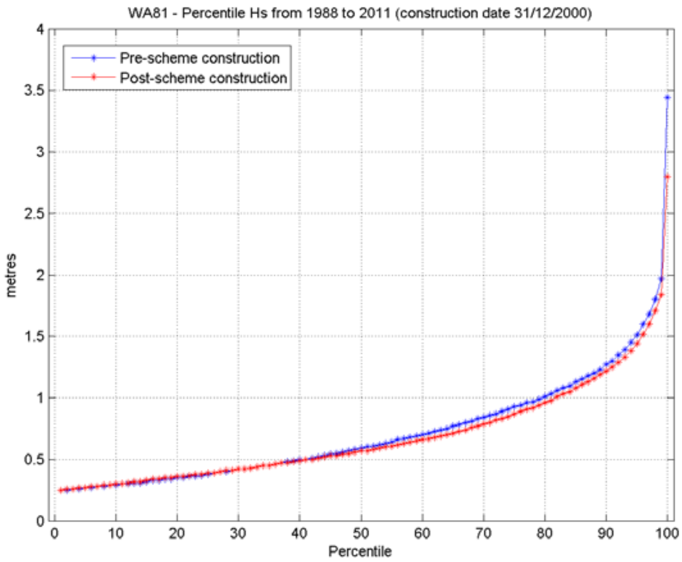

Figure C.6.7 Percentage distribution of pre- and post-construction significant wave heights

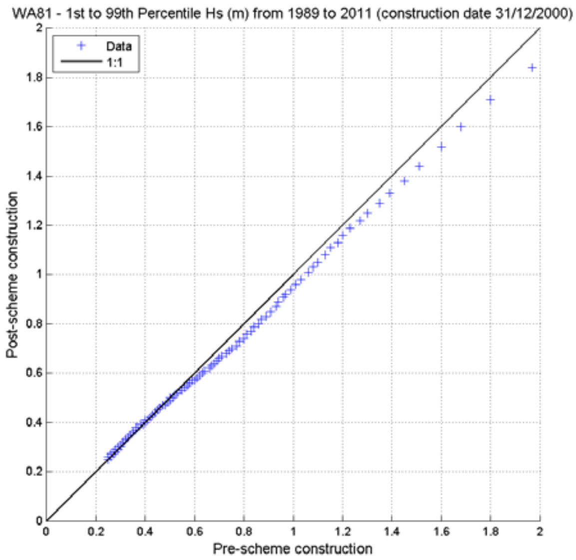

Figure C.6.8 Comparison of pre- and post-construction significant wave heights

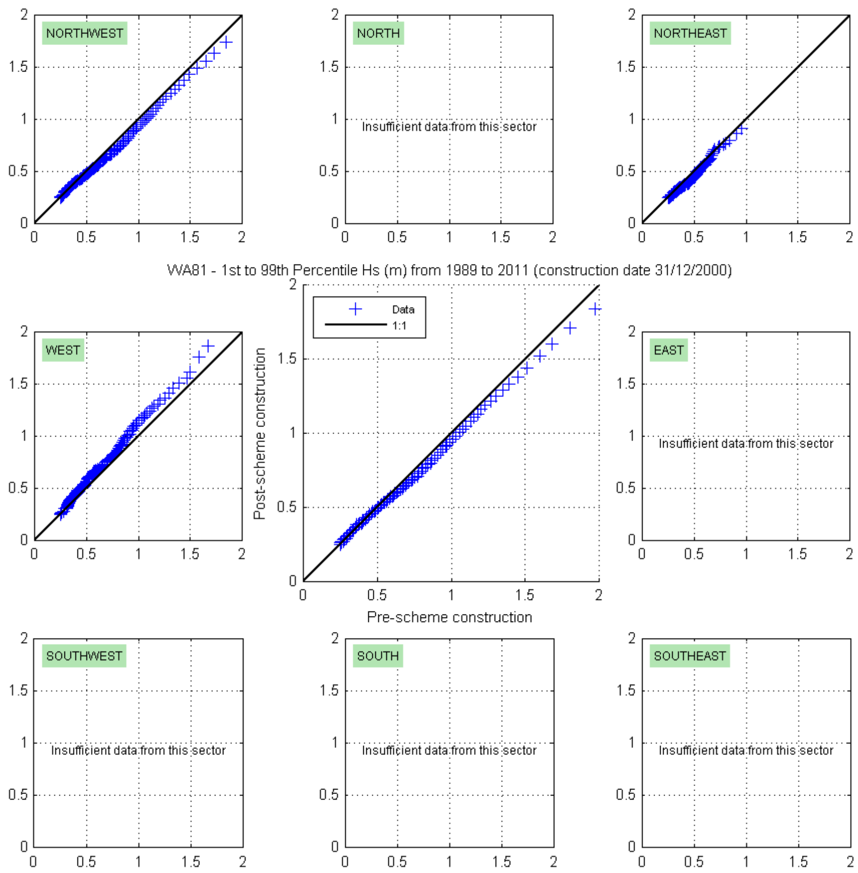

Figure C.6.9 Comparison of pre- and post-construction wave climate

Wave conditions vary across the Bay due to the shelter provided by the headland at the west end of the bay (the Great Orme). With the recharged frontage being at the western end of the bay, these conditions are not necessarily representative of conditions applying across the recharge frontage. Notwithstanding, these modelled conditions show that there was a period (1990-1994) prior to the first phase of scheme implementation when storm activity was greater and overall that conditions have been generally less severe since scheme completion than in the same time period preceding the completion of phase 2.

Also the data show that post-scheme wave heights in excess of 1 m have been more prevalent from westerly directions but less prevalent from the north-westerly sector and from all directions overall. For north-easterlies there is little difference pre- and post-scheme.

The inshore wave climate used in the original modelling, as shown in Figure C.6.10, was located at a point mid-way between the Pier and Craigside, on the –5 m contour relative to Chart Datum (CD), which is 3.85 m below Ordnance Datum at Llandudno, approximately 300 m from the shoreline. The crucial point is that, due to the shelter of the Great Orme, the direct impact of westerly waves is significantly limited, unlike the data presented above.

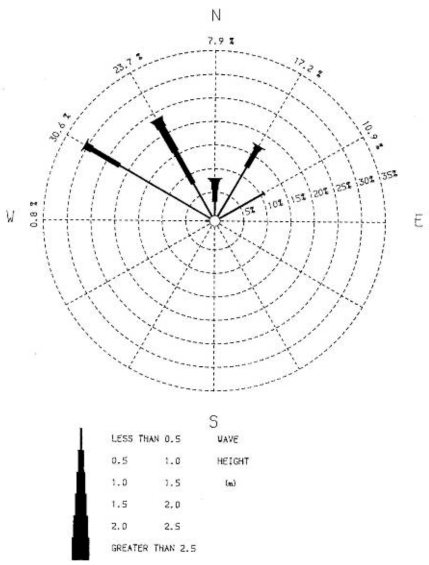

Figure C.6.10 Inshore wave climate used in original design modelling

C.6.5 Comparative analysis

Based on the results of the monitoring surveys up to and including 2008, the frontage has been losing material at a rate of nearly 3,000 m^3 per year, as shown in Figure C.6.11.

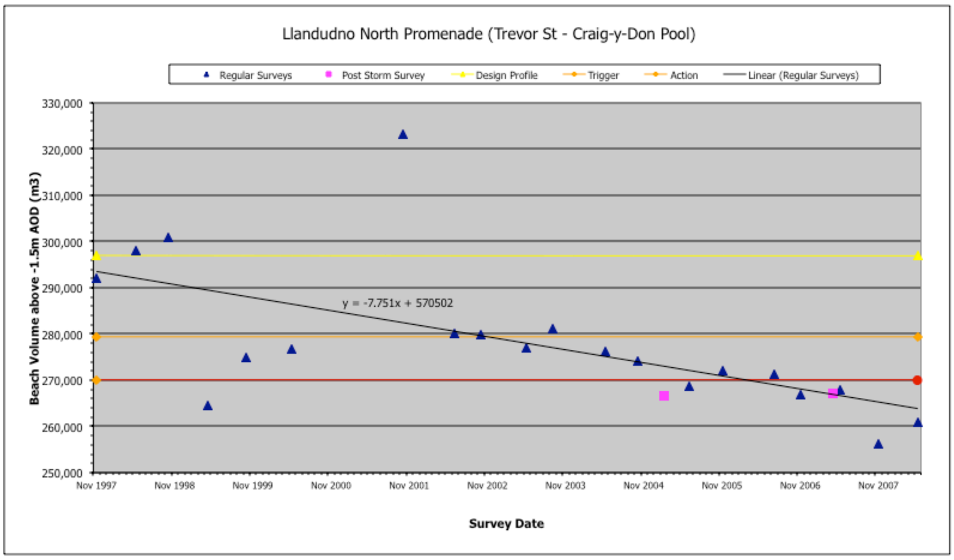

Figure C.6.11 Monitoring of scheme post-construction (to 2008)

Typical changes in the profile of the beach across this section are shown in the time series in Figure C.6.12, with the outer red line indicating the profile approximately 18 months after recharge had taken place.

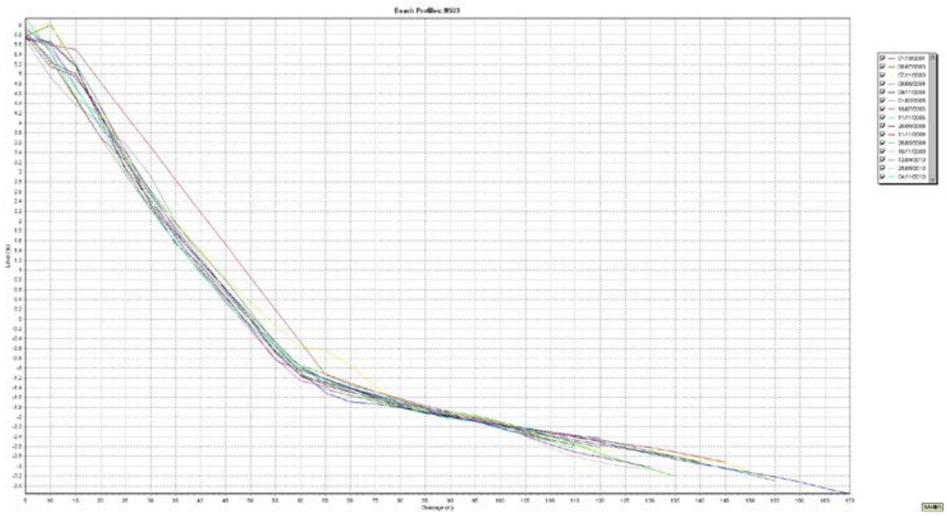

Figure C.6.12 Typical beach profiles since construction (to 2008)

Conversely the frontage at the eastern end of the bay has on average gained material at a rate of approximately 2,500 m^3 per year, as shown in Figure C.6.13.

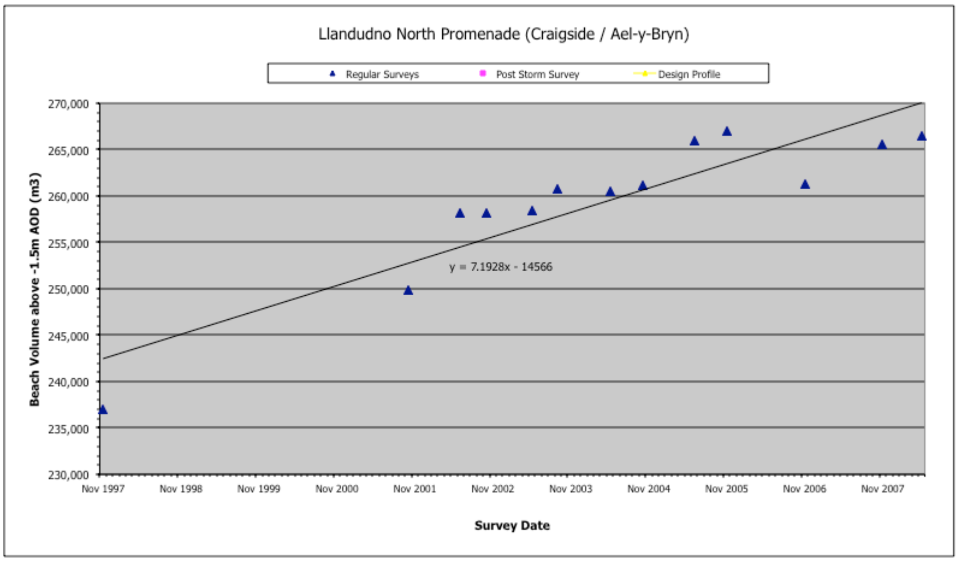

Figure C.6.13 Accretion trend at eastern end of the bay

This monitoring suggests that the majority of movement has been longshore, with about 15–20% being moved offshore and potentially lost from the system.

The losses identified have been within the range of drift rates predicted by the modelling undertaken. However, it is likely that the coarser material used in the scheme compared with that modelled will have contributed to drift rates and losses being lower than would otherwise have been the case.

Based on the environmental data provided to support the case studies, it would be expected that:

- easterly drift would be greater
- on/offshore movement (not predicted by the modelling) would be less
- westerly drift would be greater than those expected pre-scheme

In this case, however, the environmental data can only be considered as indicative of conditions as they provide different conditions to those directly influencing the recharge. In all schemes, actual post-scheme wave climates **will not** be the same as those used in the modelling; however, without a comparison climate at the same location as used for the modelling, the influence of differences in (pre- and post-scheme) wave climates on scheme performance cannot be identified.

Overall, the gross and net movement of sediment that has occurred following scheme implementation has been within the limits identified by the model, although no material has to date been recycled from the east end of the frontage.

The model also identified the phase 2 length as being an area where the greatest beach depletion would occur, which has been the case.

Based on the above the following potential reasons for differences between the modelled and actual performance have been identified:

- differences in wave climate pre- and post-construction (directional and height/period)
- different sediment size used in the works compared with that used in the model
- modelling did not identify on/offshore movement
- modelling did not consider beach performance on a whole recharge frontage or even bay wide scale, reducing performance comparisons that can be made
- the use of physical rather than numerical modelling to determine complex hydrodynamic interactions applying, notwithstanding that cross-shore interaction was not accurately modelled
- lack of accurate definition of boundary conditions relevant to the frontage in question

C.6.6 Lessons for future beach modelling/design

The key lessons to be learnt from this scheme are as follows.

- Modelling should be considered as one of a range of tools to inform scheme definition for beach recharge schemes.
- A thorough understanding of process behaviour and likely scheme behaviour backed up by empirical calculation and judgement is essential.
- It is important to identify appropriate boundary conditions for modelling and post-scheme evaluation.
- Where appropriate, modelling may need to consider behaviour over a wider basis than just potential scheme limits, which was not the case here.
- If possible, modelling should consider a range of potential sediment sizes.
- As far as possible, modelling should seek to replicate potential future conditions or ranges of conditions against which actual scheme performance can be assessed.
- Ideally, modelling should provide sufficient information that can, in association with post-scheme monitoring, provide the basis for scheme performance evaluation and be used to inform future beach management requirements.

C.6.7 Bibliography

- Powell, K.A., 1990. *Predicting Short-term Profile Response for Shingle Beaches*. Report SR219. Wallingford: Hydraulics Research Limited.

C.7 Pett

C.7.1 General information

Pett (Cliff End to Rye Harbour) Sea Defences

2004 – ongoing

Environment Agency, Southern Region

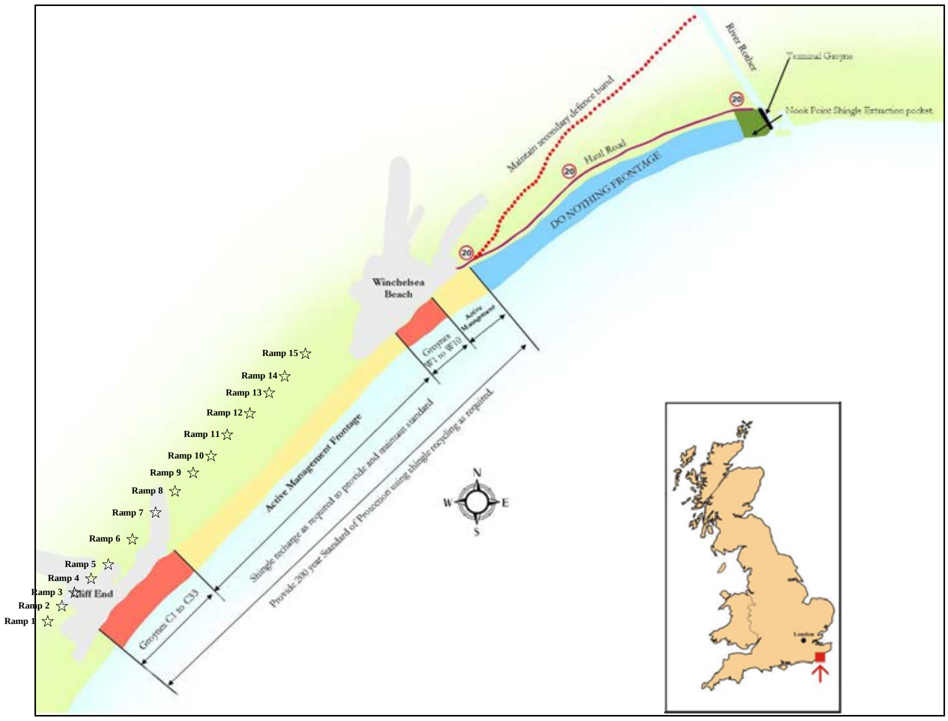

Figure C.7.1 Scheme plan (ramp positions are approximate)

Background

An approved 50-year flood defence scheme is currently being implemented to provide a 200 year standard of protection for the frontage. The scheme comprises an upgrade of the existing flood and coastal erosion defences and improvements to the current flood defence management system. This scheme includes the construction of groynes at Cliff End and Winchelsea Beach, stabilisation of the shingle extraction pocket (Nook Point), shingle recycling, construction of a secondary defence flood bund and the construction of a gabion wall to protect the Haul Road which runs from Nook Point to Winchelsea Beach.

The frontage is located on the East Sussex coast, to the east of Hastings. 'Pett Frontage' is the term that has been adopted to describe the coastline between Cliff End and Winchelsea Beach and in front of the low-lying hinterland of Pett Level, but the Project Appraisal Report (PAR) for the scheme also considers the stretch of frontage to the west, up to Rye Harbour Arm (also referred to as the Rye Terminal Groyne) (see Figure C.7.1). This gives a total frontage of about 8 km in length.

The beach comprises a broad sandy foreshore overlain by a steeper shingle upper beach, located above approximately mean sea level. The shingle barrier, sloping revetment and timber groynes protect a large area of low-lying land from sea flooding, including approximately 390 properties and 3,000 ha, and an area of freshwater wetland of international conservation importance.

There is a net west to east transport of shingle, but limited input of shingle from the west meaning that, without intervention, beaches along the frontage would gradually become denuded of shingle over time, with the shingle accumulating at the eastern end of the frontage against the western harbour arm at Rye Harbour.

This stretch of coastline has a long history of shoreline change; early maps show that the high water line at Cliff End receded 100 m between 1907 and 1943 and there have been a number of flooding events. It was not until 1933 that any measures were taken to stabilise the coast (Halcrow 2001), following a breach in the shingle ridge just to the east of Cliff End. To prevent further breaching, two lines of timber breastwork were constructed. After the Second World War, harbour works were constructed at Hastings, updrift of the frontage, which further restricted the natural supply of shingle to the frontage. To combat the resulting gradual loss of beach material, the Pett seawall was constructed between 1947 and 1952. Since then, movement of shingle from the west half of the frontage has been compensated for by recycling material from Nook Point. Additional timber groynes were also constructed along the frontage in an attempt to maintain the beach in front of the seawall.

Prior to the Pett Frontage Sea Defences Scheme, these groynes had fallen into a deteriorated state and there were increased losses of shingle occurring at Pett, resulting in an increased risk of flooding. It was recognised that the maintenance regimes were not sufficient to keep pace with the deterioration of the existing defences and two significant flood events in 1990 and 1998 confirmed that the standard of protection was at an unacceptable level.

A long-term strategy to deliver the hold-the-line policy identified by the 2002 Shoreline Management Plan (British Maritime Technology and others) was initially developed in 1998 as part of the Cliff End to Scots Float Sluice Strategy Plan. This report concluded that the preferred strategy for the Pett frontage was to improve the standard of protection to a 200-year level. The strategy was revised in 1999 and the preferred option for the Pett Level frontage was developed into a preferred scheme design, but considerable changes were made to the original option due to environmental concerns. From the detailed appraisals, it was concluded that defence options involving holding a shingle beach in front of the seawall were the only viable methods for implementing the preferred strategy. A PAR was prepared for the Pett Frontage Sea Defences Scheme in April 2001, updated in March 2002 and submitted for approval in 2002.

Details of the scheme

The principle of the scheme is to increase the standard of protection afforded by the flood defences by increasing the width of the shingle beach to reduce overtopping rates and to dissipate more wave energy, therefore reducing the risk of damage to the seawall structure, which is at the end of its design life.

The 1:200 standard of defence is dependent on maintaining a minimum crest height, crest width and beach slope. At Cliff End and Winchelsea Beach, this is achieved through a groyne field and maintained shingle beach. Beach levels here have been increased to the 1:200 standard of defence through beach recycling using shingle from Nook Point, an extraction pocket located adjacent to the Rye Harbour Arm. This may be topped up on an annual basis through shingle recycling from Nook Point. Between the Cliff End and Winchelsea Beach groyne fields, the 1:200 standard of defence is to be achieved through an active management programme of shingle recycling activities.

The ongoing scheme has so far involved the construction of 43 groynes (33 along the Cliff End frontage and 10 along the Winchelsea Beach frontage), stabilisation of the shingle extraction pocket, shingle recycling, construction of a secondary defence flood bund and the construction of a gabion wall to protect the Haul Road. The aim of the proposed scheme was to undertake a capital scheme gradually over a period of eight years to minimise the potential for environmental impact. After the 1:200 standard of defence was achieved, it was intended that the new beach levels should be maintained by annual recycling of smaller volumes of material that would naturally accumulate in the extraction pocket at Nook Point.

It was estimated that the 'extraction pocket' would enable the removal of approximately 90,000 m^3 of shingle in the first year. It was intended that further shingle would then only be taken from the area once it had accreted within the pocket, with estimates of shingle recycling quantities provided in the PAR. The PAR did acknowledge that the recycling values were based on mathematical modelling and were therefore approximate. Consequently, it was recognised that there was a level of uncertainty in the timeframe for implementing the scheme; if less material was available from the extraction pocket, then it could take more than eight years to reach the 1 in 200 year standard of defence, but if more material became available, the implementation could be achieved sooner. As part of the scheme modelling, a sensitivity test was undertaken to assess the annual variability in drift rates at the extraction pocket groyne. This found that for the period appraised (1989 to 1996) drift rates varied from 30,000 m^3 to 73,000 m^3.

Alongside the capital works, the beach is currently maintained by the Environment Agency Operations Team through regular beach maintenance operations involving emergency reactive recycling and re-profiling of beach sediment from the collection pocket at Nook Point to wherever it is required to restore beach crest level, width and slope.

Between Winchelsea Beach (where the secondary flood defence bund reaches the haul road) and Nook Point, the approved strategy is 'Do Nothing'. The beach along this frontage is expected to develop a natural profile in response to wave activity and no maintenance operations are undertaken to rebuild the beach crest or maintain structures. The only operations that are permitted are those that relate to maintaining the integrity of the Haul Road, which is relied upon for future management of the entire frontage.

Supporting background studies

Strategy and scheme development

- The Cliff End to Scots Float Sluice Strategy Plan was initially produced by Halcrow in 1998 and consisted of two volumes (Volume 1 – Strategy Report; Volume 2 – Study Reports).
- The Strategy Report (Volume 1) was revised and reissued in 1999, with additional study reports produced.
- The preferred option for the Pett Frontage was developed into a detailed scheme design, but with considerable changes from the original option. This is reported in the Pett Frontage Sea Defences – Scheme Design Report (Halcrow 1999).
- A Project Appraisal Report (PAR) was produced in April 2001 for Defra approval based on the preferred scheme design.
- The scheme design for compartment A was revised and an updated Strategy PAR submitted to Defra. Alongside this, a PAR was prepared for the compartment A

scheme, known as the Pett Frontage Sea Defences. This document was published April 2001, updated in March 2002 and September 2002, and submitted to Defra in September 2002 for approval.

- An additional document was submitted to Defra to deal with outstanding issues, further to consultation with Defra, the Environment Agency and Halcrow.
- An Update Addendum (December 2002) was added to the Cliff End to Scots Float Sluice Strategy Plan to summarise changes made over the course of the scheme development.
- Various internal calculation files have been produced in support of the scheme design.

Additional studies

Beachy Head to Rye Harbour Coastal Process and Resource Study: sediment budget for Cuckmere Point to Copt Point, Folkestone (Halcrow 2000)

This study was undertaken between the scheme studies and production of the scheme PAR. It undertook new modelling, using the existing beach plan shape model set up for the strategy, but with new offshore wave data and transformed inshore data.

- Wave and wind data were obtained from the UK Met Office European Wave Model for location 50.75°N, 0.74°E. A 10-year dataset was obtained, covering the period from January 1989 to December 1999.
- A suite of 2D numerical wave models was used to predict wave climate near the shore for a given offshore wave height, period and direction, then used to transform the offshore time series obtained from the Met Office to 14 inshore points between Cliff End and the western harbour arm at Rye. These data were then used as input to a numerical one-line beach plan shape model.
- The beach plan shape model from the strategy was recalibrated using the new inshore wave data.
- A numerical 2D cross-shore beach model was used to assess potential sediment transport on the lower sandy beach for defined ‘storm’ and ‘typical’ wave conditions.

Scheme implementation

- Annual beach monitoring surveys for the Pett Frontage Sea Defence scheme have been undertaken since 2004. These consist of a detailed walkover survey between Cliff End and the western harbour arm at Rye Harbour, known as Nook Point. The latest reports also present results of beach volume analysis using measured beach profile data from the ongoing beach monitoring programme conducted as part of the Southeast Strategic Regional Coastal Monitoring Programme (SRCMP).
- A Beach Management Plan (Halcrow 2009) outlines the operational management of the scheme. This is due to be reviewed shortly.

C.7.2 Approach to modelling and basis of design

Rationale

Modelling was undertaken in support of the original strategy, with further extensive modelling carried out as part of the scheme development and final scheme design.

The beach is a composite shingle and sand beach, but the focus of the modelling carried out in support of the scheme has been on the upper shingle beach, which is a key component of the coastal defence system. Numerical modelling of sediment transport has therefore concentrated on wave-driven processes. Current measurements carried out by Hydrographic Survey Ltd as part of the strategy plan, and subsequent tidal flow modelling, confirmed that current velocities along this shoreline are not sufficient to move sediments larger than sand. The strategy and subsequent studies looked at the lower sandy beach and all concur with the conclusion that this beach zone is generally stable, with little net trends of change evident. The rationale behind the modelling during the design stage was to design a scheme that provided a minimum beach width throughout the entire model run. Therefore a 'design line' was defined, which was the absolute minimum beach width required to provide a 1 in 200 year standard of protection. **Overview of approach** Modelling has included the following. Strategy Plan (1998-1999) • Wave transformation modelling using refraction and shoaling model for 15 inshore refraction points • Cross-shore modelling using 2D numerical model to assess storm beach response and also the potential for sand transport across the lower sandy foreshore • Parametric shingle cross-shore model to assess beach stability under a range of storm conditions • Alongshore transport of shingle using one-line model beach plan shape model • Beach evolution using one-line beach plan shape model to assess various strategic options Scheme design (1999) • Adapted previous beach plan shape model and reduced grid spacing to improve sensitivity. Used this to predict beach evolution for a range of different scheme options. The same wave data was used as for the strategy.
Modelling for the original strategy development used hindcast waves, which were transformed inshore using refraction and shoaling model. The program was run in the 'back-tracking' mode to provide a more accurate and detailed definition of the wave climate. Waves were tracked in a range of direction between 50° and 230° at 1° intervals from each of the refraction points. Fifteen refraction points were used: 13 between Cliff End and the western harbour arm at Rye, at approximately the +4.0 m CD contour. For each refraction point, 15 wave periods were run for five water levels. The water levels were chosen so as not to include periods where the water level was below the toe of the shingle. Using the output from the wave refraction model, a SANDS database was used for the wave transformation modelling. This was used rather than 2D wave model because the seabed was considered by fairly uniform. These nearshore wave data were also used in the design option modelling.

Wave and wave data The original modelling for the strategy used a nine-year continuous record of three-hourly offshore time series data recorded at Dungeness between 1989 and 1997. For the wave transformation modelling the wave period had to be changed from T_z to T_p; for this it was assumed $T_p = 1.27T_z$. **Water level data** For the original strategy modelling, water level data were available from the Broomhill to Dungeness Study and comprised time series hourly data at Dover Harbour for 1973 to 1990. This time series data were not, however, used in the beach plan shape model (BPSM) because there was only a very small period where the wave and water level data overlapped. The model therefore used predicted time series water level data.
Cross-shore modelling For 10 locations along the frontage, a 'typical' beach profile identified from the Environment Agency's ABMS data was run through a parametric shingle model to assess storm profile responses for a range of wave heights and water levels. An adaptation of the Van Der Meer's methodology was used to predict overtopping volumes at the beach crest, using the storm profiles obtained from the parametric shingle analysis. Using methodology in HR Report SR261 (Owen and Steele 1993), minimum allowable crest widths and heights to achieve the defined standard of protection were calculated. Using this methodology it was possible to optimise the berm width (corresponding the beach crest width) until overtopping volumes were within pre-defined acceptable limits. **Beach plan shape modelling** The minimum beach widths determined from the cross-shore modelling were used in the beach plan shape model to investigate a range of schemes for implementation of each of the viable defence options that, over a 50-year design life, would provide a standard of service of 1 in 200 years. The mean high water line was used as the beach contour. The model was set up and calibrated for the strategy plan to look at management options. For the design options stage, the same input wave data was used, but the model was modified to make it more sensitive and to allow more accurate positioning of the groynes. For the design option modelling the spacing was reduced from 25 m to 10 m for the majority of the frontage. The minimum beach crest widths required to ensure a 200-year standard of service, established through the cross-shore modelling, were used to define a 'design line' for the beach crest. During the beach plan shape modelling exercise, the various schemes were developed such that the modelled contour did not pass behind the design line. This was achieved by varying the groyne positions and recycling quantities and locations. The initial model runs assumed that an infinite volume of shingle would be available for this. Later model runs refined this by calculating the amount of shingle that could be available for recycling at the end of the model run for each year.
Beach profile data used in the strategy were ABMS Environment Agency data for the period 1978 to 1997. ABMS is a regional programme that produces photogrammetric beach profiles from annual aerial surveys.

The current management of the beach is informed by beach profile data collected as part of the strategic regional coastal monitoring project. Since summer 2003, the beach has been surveyed three times a year using land-based GPS techniques. 'Designated profiles', that is, those that are monitored three times per year, are spaced at between 100 and 500 m apart. In 2012, a laser scanning technique was used, but the output from this is not yet available.

Baseline conditions

As part of the strategy study, beach samples were collected at kilometre intervals between Cliff End and the western harbour arm at Rye for five cross-shore locations:

- shingle crest
- high water mark
- shingle/sand interface
- 10 m seawards of the interface
- 0 m OD contour

A noticeable gradation in shingle size was determined, with large material found at the western harbour arm. This was explained by the fact the high energy waves from the west moved larger particles eastwards, while lower energy waves from other directions are not able to move these back eastwards. Comparison with earlier data (Balfour Maunsell 1993) showed that the ranges of sediment size had apparently increased. This is likely to be due to the recycling activities.

Monitoring of scheme

The scheme involves recycling using locally derived shingle extracted from Nook Point and therefore there is not a requirement to specifically monitor sediment size. There are onsite observations of the material available from the pocket and it has been noted that, when limited shingle accumulates within the pocket, the quality of material for recycling deteriorates and is more sand-rich. Under the requirements of the Environmental Impact Assessment (EIA) for the Pett Frontage scheme, no foreign material is to be imported to this frontage (from either onshore or offshore sources).

Selection criteria

- Natural sediment*:* typical shingle size D_{50} 16 mm, although the Maintenance Manual (updated February 2009) specifies that the material extracted from the pocket should have a minimum D_{50} value of 20 mm.
- Modelled sediment type (size): only the shingle-sized sediment was modelled. There is some discrepancy in the reports as to whether 12 or 13 mm D_{50} was used in the modelling, but the sediment size used for the detailed option modelling was based on the best calibration results. As a sensitivity test, the sediment size was reduced to 8 mm; this was found to cause a 12% increase in the predicted sediment drift rates.
- Sediment in final design: *s*hingle, with lower sandy beach – the scheme uses native shingle extracted from the downdrift accumulation area at Nook Point.
- Sediment placement approach in final design: material is extracted from the extraction pocket at Nook Point, transported along the coast via the Haul Road and placed at various locations along the shoreline. Shingle recycling is carried out annually with volumes dependent on need and availability. Beach recycling and re-profiling is typically carried out using a tracked bulldozer and a hydraulic

excavator.

Prior to the scheme

Prior to the scheme, the main form of protection was the Pett seawall and fronting shingle beach. The 2002 scheme PAR concluded that the Pett seawall had reached the end of its design life and, without a healthy beach in front of it, was unable to withstand prolonged wave impact. Due to inadequate natural sediment supplies from the west, the shingle beach was only **maintained** by extracting shingle from adjacent to Rye Harbour arm to replenish localised sites of erosion along the frontage. While the beach was partially held in place by a series of timber groynes, the majority of these were dilapidated or too low to prevent the loss of beach material under storm conditions.

Scheme design

The original scheme proposed a phased construction of 31 timber groynes at Cliff End and up to 17 timber groynes at Winchelsea Beach, and their subsequent maintenance and replacement as necessary. However, it was planned that the construction of the seven eastern-most groynes at Winchelsea Beach would only be implemented if downdrift erosion was found to occur following construction of the initial 10 groynes, as these are located within the SAC. These groynes have not yet been constructed.

Model calibration

During the calibration of the beach plan shape model, it was found to be necessary to allow a volume of material into the model at the updrift boundary at the western end of the Cliff End seawall. Without this shingle input, significant erosion was experienced; however, this was not in agreement with observations from aerial photographs, which indicated that shingle had accreted over several years in the small bay updrift of the Cliff End seawall, suggesting that there is some material entering the frontage from the west.

To gain an understanding of the volumes of material entering the system at the updrift end, the model was run with a ‘stable point’ boundary condition. This allows material to enter and leave the system at a rate that is sufficient to maintain the equilibrium position of the boundary condition. The volume required to maintain equilibrium was found to be approximately 19,000 m^3 and this volume was eventually used in future project runs. Sensitivity tests were carried out with input feeds of 19,000, 10,000, 5,000 and 0 m^3 at the updrift boundary condition.

The results showed that, when the rate of feed was reduced at Cliff End, the beach line receded in areas where there are no groynes, with the amount of recession proportional to the volume of material fed into the system. When zero feed was input to the model, the most significant change was in the areas where there were no groynes, that is, downdrift of the cliffs up to the first groynes and at the downdrift end of the groyne field at Winchelsea Beach. The model showed that the impact occurred after around 3–5 years.

This sensitivity test was run again with the final scheme configuration in the model. Conversely, it was found that the impact of changing the input at Cliff End had only a localised impact and do not affect the scheme performance as a whole.

Scheme design

Once calibrated, a wide range of modelling runs were undertaken, responding to the changes in the detailed scheme design in response to economic and environmental constraints.

Originally, the idea was that the capital scheme would be completed in a 12-month period. However, mitigation measures necessary to minimise the environmental impact of the scheme resulted in the scheme construction period being extended over eight years to minimise the impact of the extraction operation. An early decision was also made to construct groynes in front of the villages only, with active management along the intervening stretch and between Winchelsea Beach and the Do Nothing frontage. The impact of this was that higher recycling rates were found to be required compared with an earlier scheme that involved building more groynes along the frontage.

In developing options and the detailed scheme design, extensive beach plan shape modelling was carried out which looked at a wide range of groyne and recycling combinations, both with and without the environmental restrictions on the extracted volumes.

In designing the final scheme, various groyne configurations, lengths and spacings were considered within the beach plan modelling. The lengths were determined based primarily upon the minimum width of beach crest that was acceptable (based on the shingle cross-shore modelling – see above). The spacing was determined by considering how the volume of material within the groyne bay could move around as a result of the various modelled storm conditions. Modelling also looked at the impact of timing of groyne construction and various runs were undertaken to look at building groynes gradually. It was also concluded from the model runs that the eight original proposed groynes at the eastern limit of Winchelsea Beach, appeared to have a minimal effect on the development of the shoreline. As these are located in the SPA/SAC, it was decided that the need to construct these groynes would be informed by future monitoring.

Modelling studies also looked at a range of different recycling volumes and locations, including whether recycling should start at Winchelsea Beach or Cliff End; from this, it was concluded that a problem would arise at Cliff End if Winchelsea Beach was recharged first.

Once the groyne locations and lengths were decided, the recommended recycling quantities were refined. Initially it was assumed within the model that extraction from Nook Beach would take place over one month, but this was then changed to two months. The later model runs tried to account for the fact that shingle can only be taken from Nook Beach once it accreted in the pocket due to natural longshore drift. The volume of material available at the end of year 1 and so on (for feed during the following year) was therefore estimated using the BPSM results.

A sensitivity test was carried out for the final scheme design to look at the impact of using only 45,000 m^3 recycling per year, once the 1 in 200 year standard was achieved throughout the frontage (that is, after year 8). This is approximately the same volume as the pre-scheme recycling. The model results suggested that this was an overestimate of the need and that a volume less than 45,000 m^3 would be required annually.

An additional sensitivity text was carried out to examine the effects of using wave data from years containing lesser and greater directional energy. The conclusion from the results was that, for the data available, no significant variations were apparent in beach alignment between years.

- The presence of the cSAC and SPA has had a significant influence on the development of scheme options. The area is the subject of international environmental conservation designations and therefore there were major environmental constraints on the design and implementation of the scheme. These can be summarised as follows:

- o SSSI – need to avoid disturbance to birds, vegetated shingle areas and the petrified forest and shingle ridges
- o Ramsar – need to avoid disturbance to birds
- o SPA – need to avoid disturbance to birds
- o cSAC – need to avoid disturbance to vegetated shingle areas

An Environmental Statement identified the following mitigation measures:

- o restricting the size of the area to be adopted for shingle extraction to minimise the effects on vegetated shingle
- o scheduling construction activities to avoid the bird-nesting season

- Of significance in the development of any sustainable scheme for this frontage was the assumptions made regarding the volume of material entering the system around the cliffs at Cliff End (see section above). The Scheme Design Final Report (2009) states that the quantities of recycling were increased by 20,000 m^3 to allow for the possibility of the cessation of material entering the system from around the cliffs, although it is not clear whether this was carried through to the final scheme design.
- The preferred source for shingle was concluded to be Nook Beach but, due to environmental constraints (see above), this was restricted to a limited area: 220 m in longshore length and 60 m wide. Estimations of available shingle were calculated by shifting landwards the existing profile by 60 m and estimating the sand-shingle interface to be around 0 m OD. Borehole data suggested that pure shingle stopped at around 7–8 m below the surface, which concurred with this. It was therefore assumed that the ‘extraction pocket’ would enable the removal of approximately 90,000 m^3 of shingle in the first year, with further shingle only being taken from the area once it had accreted within the pocket. This calculation was carried out in 2000 and presumably used beach profile data for this year; the scheme itself did not start for another four years.
- The estimated shingle recycling quantities were based on mathematical modelling of beach response and calculations of the additional volume required to maintain the design beach. It was made clear in the scheme PAR that these volumes were approximate and would depend upon the material available from the extraction pocket. Consequently, it was concluded that there was a level of uncertainty in the timeframe for implementing the scheme. If less material were available from the extraction pocket, then it would take more than eight years to reach the 1 in 200 year standard of defence, but if more material were available, the implementation could be achieved sooner.
- Beach plan shape modelling for the strategy suggested that the average longshore shingle transport was 30,000–45,000 m^3 per year, with groynes. This concurs with the pre-scheme recycling operation, which involved transport of between 20,000 and 50,000 m^3 per year, but is less than the potential shingle transport rates derived from the subsequent Beachy Head to Rye Harbour Coastal Process and Resource Study (Halcrow 2000). The beach plan shape modelling from both studies did, however, indicate that, for the wave data used, there was significant variation year to year in drift rates.

C.7.3 Design/modelling outputs – plans for implementation

The principle of the scheme was to increase the standard of protection by increasing the width of the shingle beach so that it acts more efficiently at breaking the incident waves, thereby reducing the risk of damage to the seawall structure, which is at the end of its design life.

The key components of the preferred scheme are as follows:

- modifications to Rye Harbour Arm to maintain its capacity to retain accreting material
- temporary construction of extraction pocket groyne at Nook Beach
- capital recycling of shingle on the frontage taken from extraction pocket
- initial maintenance of existing timber groynes at Cliff End until the final design beach width is achieved
- purchase of land for the secondary defences and habitat compensation
- refurbishment, upgrading and realignment of the secondary defences
- construction of timber groynes at Winchelsea Beach
- annual recycling of shingle
- annual maintenance of groynes
- monitoring of the beach profiles and quantities recycled
- alterations to the slipway at Cliff End and the outfall on the Pett frontage
- possible construction of 57 timber groynes between Cliff End and Winchelsea Beach
- monitoring the impacts of the scheme on the environment

The scheme originally consisted of a phased capital recycling programme over an eight-year period, followed by 42 years of annual shingle recycling. It was, however, recognised in the scheme design report that the scheme would need to flexible to take account of both the availability of shingle from the extraction pocket at Nook Beach and the uncertainty regarding the volume of shingle entering the system at Cliff End.

The design anticipated that once the 200-year standard of protection was consistently achieved by building out the beach to the required width, recycling would still be required, in the order of 30,000–50,000 m^3per year.

Future re-profiling works would also be required to ensure that the beach material is not steeply sloped and that the beach is profiled to the design 1:7 slope from the knee pile/edge of crest, and the crest levels are maintained at +6.5 mOD (+4.5 mOD between Cliff End groynes C1 to C9). The design crest width, for the 1 in 200 standard of protection, was defined as 15.0 m, but a minimum crest width was defined as 6.0 along the frontage.

The need for additional groynes between Cliff End and Winchelsea was to be informed by monitoring. The monitoring of beach performance and recycled quantities was outlined in the Project Appraisal Report prepared by Halcrow in 2002. The original works schedule is shown in Table C.7.1.

Table C.7.1 Original scheme programme of works

Year	Beach recycling		New works	Maintenance of existing groynes		
	Volume (m^3)	Location		Cliff End	Winchelsea Beach	Pett Level
1	90,000	Thin band from Cliff End to ramp 12	Extraction pocket groyne and reconstruction of part of harbour arm	yes	yes	yes
2	60,000	Cliff End: ramp 1/2 to 4	31 new groynes at Cliff End		yes	yes
3	54,000	Cliff End: ramp 1 to 3/4	Secondary defences		yes	yes
4	51,700	Winchelsea Beach: ramp 14/15 to 15/16	Nine new groynes at Winchelsea Beach			yes
5	50,000	Ramps 10 to 14	Further eight groynes at Winchelsea Beach if required			yes
6	50,000	Ramps 10 to 14				yes
7	50,000	Ramps 10 to 14				yes
8–50	Up to 50,000	Ramps 2, 3, 4, 9 to 13				

The Beach Management Plan (Halcrow 2009), produced in year 6 of the scheme, set out the trigger levels shown in Table C.7.2.

Table C.7.2 Action and emergency trigger levels set by the BMP (2009)

Location	Action triggers	Emergency triggers
Cliff End Sea Wall, Groynes C1 to C9	Beach profile falls below the design level (crest width 15 m, crest height 4.5 m OD)	• Exposure of the revetment. • Excessive cliffing - beach slope falls significantly below 1:7 slope • Crest width falls below 6.0 m • Beach differentials across groynes exceed allowable differentials: 2 m at groyne inner end, 3 m in central section and 1 m at outer end
Pett Sea Wall, Groynes C10 to Winchelsea W10	Beach profile falls below the design level (crest width 15 m, crest height 6.5 m OD)	As above
Winchelsea, Groyne W10 to start of Do	Beach profile falls below the design level (crest width	As above

Nothing frontage	15 m, crest height 6.5 m OD)	
Do Nothing frontage and Nook Point	None	None

For the PAR, losses of shingle from the system were estimated to be less than 5,000 m^3per year, through material being thrown over the back of the terminal groyne; and material being drawn down past the toe of the terminal groyne.

The Pett Frontage Sea Defences Maintenance Manual Update (February 2009) specified that the material from the pocket groyne should have a minimum D_{50} value of 20 mm and that the assumption was for annual volumes of between 30,000 and 50,000 m^3 to be extracted as required.

C.7.4 Beach management and performance

The scheme commenced in 2003-2004. The original plan was to place 90,000 m^3 of shingle, extracted from Nook Point, as a narrow band between Cliff End and ramp 12 (which is midway along Pett Level. However, only 30,000 m^3 recycling was undertaken between ramps 1 and 2, at the western end of Cliff End. The Beach Management Plan reports that this was ‘due to circumstances beyond the Environment Agency’s control’; it is likely that this was at least partially due to not enough shingle being available from the extraction pocket.

Subsequent recycling has also been below the recommended amount (see Table C.7.3) and the 2011 beach monitoring report notes that at this time there was a net deficit of around 207,000 m^3 of shingle recycling from Nook Beach (including year 8, 2010-2011) in comparison with the PAR.

Table C.7.3 Proposed scheme compared to actual work to date

Year	Proposed recycling (m^3)	Actual recycling (m^3)	Variation from proposed works
1 (2003-2004)	90,000 as a thin band from Cliff End to ramp 12.	30,000 at Cliff End (ramp 1 to 2)	
2 (2004-2005)	60,000 at Cliff End (ramp 1/2 and 4)	40,000 between groynes 10 and 28 5,000 at Winchelsea Beach	Instead of 31 new groynes at Cliff End, only groynes C10–C28 constructed and these were only planked to upper waling.
3 (2005-2006)	54,000 at Cliff End (ramp 1 and 3/4)	35,000 between groynes 1 and 28	Construction of groynes C1–C9 at Cliff End. Groynes C11–C17 planked to full height.
4 (2006-2007)	52,000 at Winchelsea Beach (ramp 14/15 to 15/16)	None	Proposed 9 new groynes at Winchelsea Beach not constructed Reduction of pocket

			groyne level
5 (2007- 2008)	50,000 (ramps 10 to 14)	36,500 in total: • 33,000 groynes W1–W10 at Winchelsea Beach • 3,500 at ramp 7 to deal with localised erosion	Groynes W1 to W10 constructed at Winchelsea Beach, but not fully planked
6 (2008- 2009)	50,000 (ramps 10 to 14)	42,890 in total: • 5,400 at Cliff End (groynes C1–C7) • 6,760 between groynes C29 and C33 • 17,430 between groynes W1 and W10. • 13,300 at ramp 7 to deal with localised erosion	Construction of transition groyne field at Cliff End (groynes 29–33), originally proposed for year 2.
7 (2009- 2010)	50,000 (ramps 10 to 14)	29,200 in total: • 9,900 at Cliff End • 19,300 ramps 5–9	20,500 shingle moved from ramp 14 to ramps 4, 6, 7, 9 and 9
8 (2010- 2011)	30,000–50,000 (ramps 2, 3, 4, 9–13)	31,000 ramps 5–8, C1–C5 and C29–C33	
9–50	30,000–50,000m (ramps 2, 3, 4, 9–13)		Groyne at pocket groyne has not been removed.

In addition to the recorded volumes above, emergency works have been carried out a various locations along the frontage, but particularly in the vicinity of ramps 6 and 7. Re-profiling of the beach is also carried out to ensure that the beach is not too steeply sloped and that the beach is profiled to the design 1:7 slope from the edge of the crest, and crest level are maintained at +6.5m OD (or 4.5m OD between Cliff End, groynes C1 to C9).

Beach monitoring reports have been produced each year since 2005, which provide a very good record of how the beach has behaved since the scheme commenced. At the time of writing the 2012 report was in preparation.

At the western end of the frontage, between groynes C1 and C6 at Cliff End, loss of material has been reported since 2008. The groynes were constructed in year 3 (2005-2006). Beach lowering in the mid to upper sections of the beach has meant that recycling has been required here, which was not originally planned. The latest report (2011) recorded that the problem here was continuing and at this time, the beach was below design standards. It was estimated that around 7,500 m^3 recycling was required at this location. The 2011 monitoring report anticipated that an ongoing beach management commitment would be needed at this location in the future.

Along the Cliff End frontage, only 18 of the originally planned 31 groynes were constructed in year 2 and another 10 in year 3, up to groyne C28. By year 4 (2006-2007) there had been erosion of the beach to the east of groyne C28. This prompted the construction of transitional groynes C29 to C33 in year 6 (2008-2009). The groyne bays were filled with shingle at the time of construction of the groynes. Since

construction of groynes C29 to C33, there has been no further problem and the 2011 report stated there was no evidence of scour downdrift of groyne C33 and that the beach was performing well and as designed. Between 2009 and 2011 the overall beach volume remained fairly stable (less than 5% change). However, continued recycling is required to replace the sediment that is naturally transported eastwards. Along the Active Management frontage, the intention was to manage this through recycling activities rather than through construction of new groynes; the intention was that existing groynes would be maintained. In general, the monitoring reports record that the majority of the frontage has remained relatively healthy, with sufficient crest width and level as a result of the recycling operations. Extensive recycling has, however, been necessary around ramps 6 and 7, with additional emergency works necessary following storms. The 2011 monitoring report states that this small section of beach is traditionally volatile, but noted that this section of the frontage was less than the design 1 in 200 and required further recycling. Prior the construction of groynes W1 to W10 at Winchelsea, there were problems of shingle loss and beach narrowing along the Winchelsea Beach frontage. Since construction of groynes in 5 (2007-2008) (a year later than planned), monitoring reports suggest that the scheme performing as planned along this section of beach, with generally healthy beach widths and slopes. There was a loss in beach level between 2010 and 2011, but the overall standard of protection remains in excess of the 1 in 200 design profile. The final groyne, W10, is only partially planked and this is sufficient to support the 1 in 200 year standard of protection, while allowing shingle to be moved eastwards. Along the Do Nothing frontage, the crest width varies but no maintenance activities are undertaken here to re-profile the beach or maintain structures. The only operations permitted are those that relate to maintaining the integrity of the Haul Road. Monitoring data for this stretch indicate periods of both accretion and erosion as sediment is naturally moved alongshore from the replenished areas and then on to feed the extraction pocket. This is a section where erosion has been an issue, as indicated in the modelling. The 2011 beach monitoring report concluded that, with the exception of the frontage between groynes C1 and C5 at Cliff End, the 1 in 200 year standard of protection had now been achieved along the lengths of frontage protected by the groynes, that is, between groynes C6 and C33 at Cliff End, and groynes W1 and W10 at Winchelsea Beach. The frontage between W10 and the start of the Do Nothing frontage is also currently stable at a 1:200 standard of protection. In 2011, the Active Management frontage between groyne C33 and groyne W1 was in a better condition than in previous years, and between ramp 10 and groyne W1, the beach was providing a standard of protection of 1:200. To the west, the beach slope is too steep and is prone to cliffing and erosion during storms. The monitoring report concluded that the standard of protection is improving year on year.
Modelling of the scheme was based on Met Office data for the period 1989 to 1997. For the modelling of the final design options, a ‘typical’ year, 1991, was used for the model runs for year 1 of the design option. Analysis, as part of this project, of UK Met Office modelled wave data between 1995 and 2011 (see Figure C.7.2) shows there have been slightly fewer large storm events since the scheme began in 2004. This may explain why slightly less recycling has been sufficient to maintain a reasonably healthy beach along the majority of the frontage, despite the initial recycling in year 1 being much less than planned.

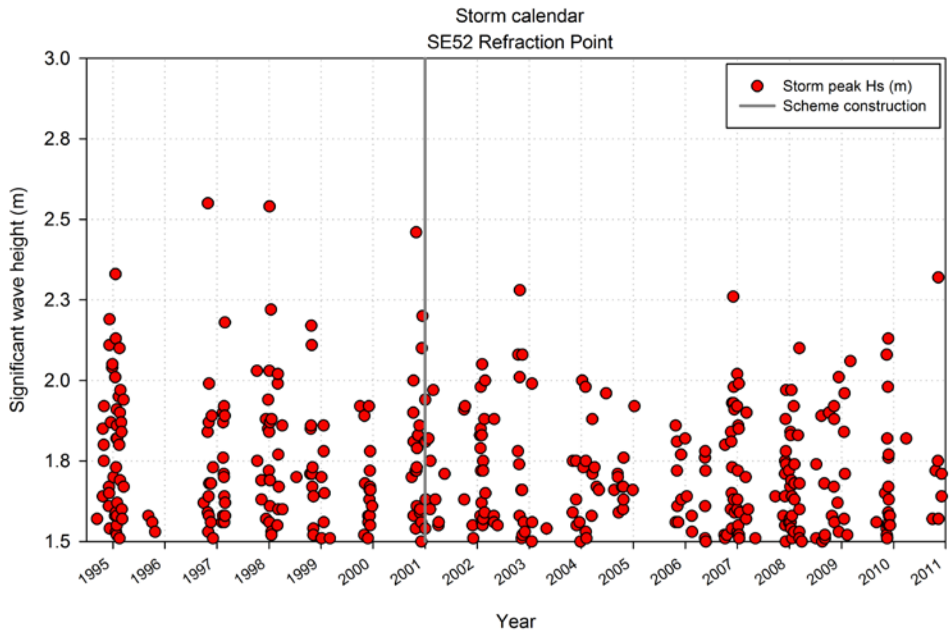

Figure C.7.2 Storm calendar for Met Office offshore wave data, pre- and post-scheme

Note: the scheme PAR was produced in 2001, but the scheme did not commence until 2004.

The plots in Figure C.7.3 show the comparison in significant wave height percentile pre-scheme and post-scheme planning. The greatest difference is for waves from the southern sector, which is the key wave direction driving the west to east transport of shingle. The data show that there was a greater occurrence of larger waves (>1 m) prior to 2001. From this is may be inferred that the modelling would have overestimated, rather than underestimated, the rates of sediment transport and therefore the recycling requirements.

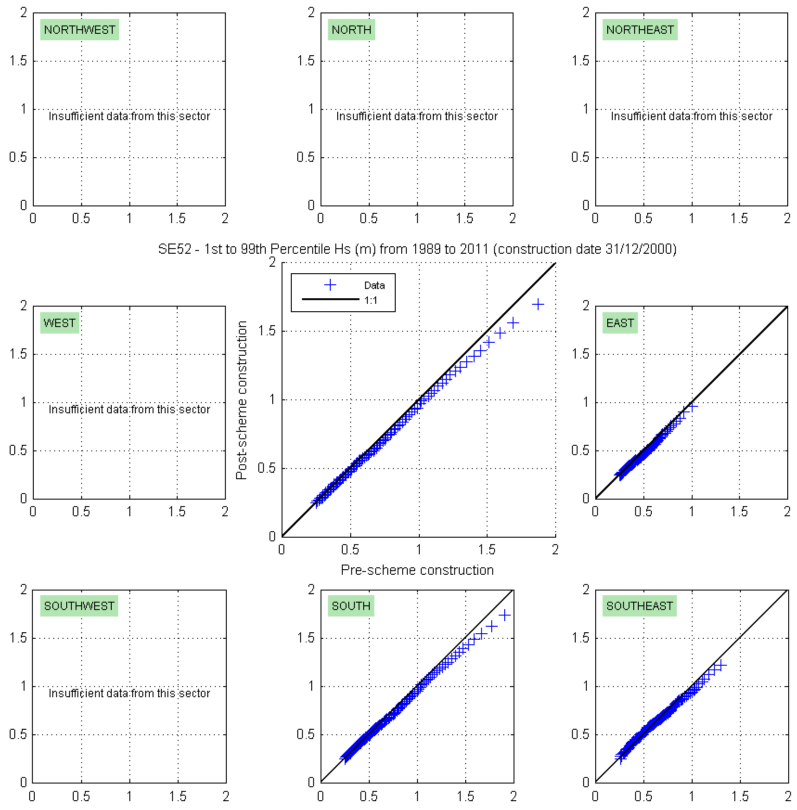

Figure C.7.3 Significant wave height percentiles

C.7.5 Comparative analysis

Table C.7.4 compares the actual works undertaken up to and including year 8 with the works proposed in the PAR. Figure C.7.4 provides an illustration of the differences. This shows that, in years 1 to 3, recycling was much less than planned but in similar locations. Some additional recycling was required along the Winchelsea Beach frontage. From year 5 onwards, the pattern of actual recycling differs from that proposed by the PAR, with the planned recycling centred around the middle section of the Active Management frontage, but actual recycling carried out further to the west, between ramps 6 and 7. Monitoring reports suggest this recycling has been a reactive response to areas of localised erosion where the crest width has reduced significantly rather than planned recycling, as set out in the scheme schedule of works.

As previously mentioned, there has also been a need for additional recycling, not originally anticipated, at the westernmost stretch of frontage between groynes C1 and C5 at Cliff End.

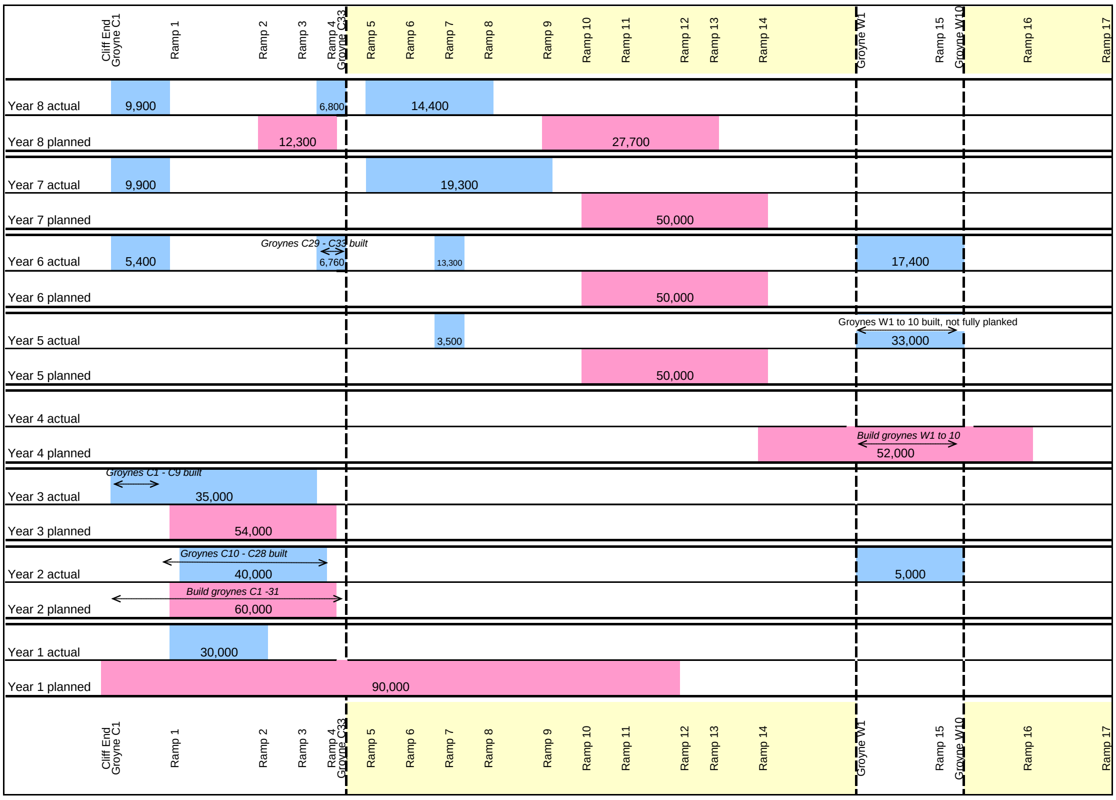

Figure C.7.4 Actual recycling and groyne construction compared with the proposed works (yellow highlight indicates the Active Management frontages)

Table C.7.4 from the 2011 beach monitoring report shows volumes extracted from the extraction pocket and the change in volume of the material in the extraction pocket. This indicates that, over each 12 month period, around 30,000m^3 of shingle is moved eastwards alongshore by natural processes but that this does vary, probably due to both prevailing conditions and changes in the beach management, that is, groyne construction or change in the efficiency of the groynes.

Table C.7.4 Shingle movements for the scheme up to 2011

Year	Summary	Shingle movements at Nook Point extraction pocket
1 (2003-2004)	30,000 m^3 of shingle recycled from Nook Point to area around ramps 1 and 2	Total recycled: 30,000 m^3 Change in pocket: unknown Balance: unknown
2 (2004-2005)	C10 to C28 are constructed but not fully planked; consequently, material is retained and the beach further east is starved of material. This is demonstrated by ramps 4–9 seeing a loss of material. Ramp 7 losses of approximately 8,000 m^3 Recycling at Winchelsea Beach A large loss at the pocket due to	Total recycled: 45,000 m^3 Change in pocket: -16,000 m^3 Balance: 29,000 m^3

	extraction for recycling to Cliff End.	
3 (2005-2006)	More groyne construction and recycling at Cliff End. Previous groynes fully planked so more material retained and depletion continues east of the new groynes with 18,000 m^3 lost since year 2. Significant accretion at ramp 14 and east of Winchelsea Pocket volume increased since year 2.	Total recycled: 35,000 m^3 Change in pocket: +1,700 m^3 Balance: 36,700 m^3
4 (200-2007)	No recycling events this year. Continued loss east to Cliff End due to increased efficiency of groynes. Continued accretion at ramp 14, significant loss east of W10 and accretion to a similar order at Lifeboat House. Losses over the remainder of the frontage, except at the extraction pocket which increased by 10,775 m^3.	Total recycled: 0 m^3 Change in pocket: +10,775 m^3 Balance: 10,775 m^3
5 (2007-2008)	Increase at Cliff End. Increase at ramp 7 relating to storm repairs. No recycling at Cliff End and marked reversal of material loss between ramps 5 and 9. Construction of groynes W1–10 accompanied by a large recycling event and loss east of W10 due to new groynes. Rolling accretion from Lifeboat House towards pocket. Pocket down nearly 12,000 m^3	Total recycled: 39,500 m^3 Change in pocket: -11,700 m^3 Balance: 28,700 m^3
6 (2008-2009)	Beach recycling at Cliff End, W1–10 and ramp 7. Large material increase east of W10, perhaps due to groynes bays being full and material being able to move eastwards. Accretion east of Lifeboat House and at extraction pocket of around 20,000 m^3.	Total recycled: 42,890 m^3 Change in pocket: +8,500 m^3 Balance: 51,390 m^3
7 (2009-2010)	9,900m^3 recycled to Cliff End and a large emergency recycling event from ramp 14. Ramp 14 lost 20,500 m^3 and was down 4,000 m^3	Total recycled: 29,200 m^3 Change in pocket: +3,800 m^3 Balance: 33,000 m^3

	from previous year, so without extraction would have been up by 16,500 m^3. Possibly rolling accretion seen at ramps 5–9 in year 4.	
8 (2010-2011)	Material recycled along western section, moderate increases east of groyne 33 to ramp 14, losses from W1–W10. Substantial loss immediately west of Lifeboat House. Remainder of beach fairly static, although accretional trend up to the extraction pocket.	Total recycled: 31,700 m^3 Change in pocket: -7,200 m^3 Balance: 24,500 m^3

Standard of protection

The original scheme intended that by year 8, a 1 in 200 year standard of protection would be achieved along the whole frontage. The 2011 beach monitoring report concluded that, apart from between groynes C1 and C5 at Cliff End, the 1 in 200 year standard of protection had now been achieved along the lengths of frontage protected by the groynes, that is, between groynes C6 and C33 at Cliff End, and groynes W1 and W10, at Winchelsea Beach. The frontage between W10 and the start of the Do Nothing frontage was also currently stable at a 1 in 200 year standard of protection. The key area where the required standard of protection has not been achieved is along the Active Management frontage between Cliff End and Winchelsea Beach.

Given the significant difference between the proposed recycling and the actual recycling volumes, the beach has performed better than might have been expected. The various model runs, with varying quantities of recycling and groyne construction, tended to indicate that the best results were achieved from intensive beach recycling in the first few years following groyne construction and then a gradual reduction, with the problem of downdrift erosion tending to be a short-term issue. However, early model runs suggested that under the preferred option in which groyne construction and recycling was focused on the villages rather than along the whole frontage, it took longer for the required standard of protection to be reached along the Active Management frontages.

Erosion between ramps 6 and 7

A key area of current concern is between ramps 6 and 7 along the Active Management frontage. Here the principle of the scheme was to maintain the beach through undertaking recycling rather than construct new groynes. This section of beach is however a pinch point and an area that has required significant intervention, particularly following storms. It is suggested in the reports that a possible reason for the volatility at this location is that it is coincident with a slight change in direction of the shoreline to the north and the beach monitoring reports suggest that this frontage has been volatile in the past. This area was not identified in the modelling as a particular risk area for the majority of modelling runs for the final scheme. The modelling did, however, indicate that the area between ramp 4 and ramp 8 was sensitive to both the amount of shingle entering the beach system at Cliff End and the length of groynes, that is, longer groynes seemed to cause a greater issue.

The 2011 monitoring report stated that Halcrow was planning to undertake a high level appraisal of a new groyne field versus active management to address the issues here; it is understood that this report in currently in preparation.

Sediment transport rates

The 2011 beach monitoring report calculated that, from beach profile analysis, approximately 30,000 m^3 of shingle is transported along the beach in a 12-month period. This compares with the strategy BPSM model which suggested that the average longshore shingle transport was 30,000–45,000 m^3 per year, with groynes.

The detailed scheme modelling does appear to have overestimated the amount of drift (and accumulation of material in the pocket), but this could also be due to a number of factors such as:

- natural variations in drift
- how the model is able to replicate groyne efficiency
- initial recycling volume used
- assumptions made regarding input at Cliff End

Change in schedule of works

A key reason for the difference in scheme performance from anticipated is the fact that the schedule of works, and in particular volumes of recycling, changed from that originally planned. Although this was a risk recognised by the PAR, it was not specifically considered in the modelling. Through the design process the original aim was for a large capital recycling and subsequent maintenance of the beach, but due to constraints on the source of recycling, the initial recycling was minimised as much as possible, with the beach build-up programme spread over eight years rather than one.

The eventual volume placed on the beach in year 1 was 60,000 m^3 less than planned. Subsequent recycling has also been less than originally planned. This is thought to be, at least partially, due to the limited availability of shingle within the extraction pocket.

The construction of structures has also varied from that planned, with construction of groynes delayed. The modelling indicated the importance of groynes to prevent downdrift cutting and this was experienced downdrift of groynes constructed at Cliff End, resulting in the need for transitional groynes. At Winchelsea, there were also problems of shingle loss and beach narrowing prior to groynes being constructed, a year later than planned. Attempts were made to minimise this impact by gradually building up the groynes and this approach was modelled as a possible option. The model runs concluded that, by undertaking a gradual construction, the recycling requirement could be reduced to around 40,000 m^3, which is still more than actually undertaken during the scheme.

Accumulation of sediment in extraction pocket

Due to environmental constraints, recycling has to be restricted to a small zone at Nook Beach. Shingle can only be taken from Nook Beach once it has accreted in the pocket due to natural longshore drift.

Calculations of beach material for the initial beach recycling in year 1 were based on beach monitoring data from 2000. The scheme did not, however, commence until 2004, which may explain why only 30,000 m^3 was recycled in year 1 rather than the 90,000 m^3 proposed by the PAR.

Beach monitoring data since the scheme have been used to approximate net shingle movements along the front based on volumes extracted from the extraction pocket and the change in volume of the material in the extraction pocket. These data indicate that approximately 30,000 m^3 of shingle is moving across the beach in a 12- month period.

In some years, such as following construction of new groynes, the volume of material reaching the extraction pocket is down, as occurred in year 4 and this is probably due to the increased efficiency of the retaining structures along the beach and the accretion of material at ramp 14. The modelling seems to have overestimated the amount of shingle available in the pocket, which might be due to slightly higher rates of transport being predicted by the model, groynes behaving more efficiently than anticipated from the modelling, or the difference in recycling volume starting from year 1. Accretion of material within the extraction pocket was an area of uncertainty highlighted in the PAR and a key reason why a flexibility approach, informed by monitoring, was advocated.

Sediment input at Cliff End

Historical evidence suggested that, at this end of the frontage, the beaches tended to remain relatively stable. When modelling the options, an artificial feed of sediment to fed into this boundary to replicate this stability. It was however acknowledged that this was a large uncertainty and that the model was also very sensitive to changes made to this boundary condition. Since 2008, there have been issues with beach lowering along the section of shoreline between groynes C1 and C6. It has been suggested in the modelling reports that this may be due to a reduction in sediment received from further west, as a result of the Fairlight Cove scheme. The Fairlight Cove scheme was constructed in 2006-2007 and includes a rock bund placed at the toe of a landslip from the sea. Although the cliffs protected by the rock bund are composed of clay and therefore would not have contributed any shingle to the beaches, it is possible that the bund is interrupting some shingle, or even some fines, being transported alongshore, which is then having an impact on Cliff End beaches. Another possibility is that that new groynes are currently interrupting some occasional westward shingle transport that originally took place, but which was not picked up in the modelling.

It is possible that this is not a long-term issue and there is anecdotal information (further to discussion with the current Halcrow project manager) that some shingle is starting to be moved around the headland at Cliff End. This is an area that is being carefully monitored at present. The deficit in shingle is estimate to be around 7,000–8,000 m^3.

Wave climate and wave modelling

Figures C.7.2 and C.7.3 suggest that there have been slightly fewer large storms since the scheme, which may suggest that volumes of shingle transport and therefore recycling requirements may have been overestimated in the modelling.

There were also differences in the sediment transport rates estimated by the modelling compared with the later modelling carried out by the Beachy Head to Rye Harbour Coastal Process and Resource Study (Halcrow 2000). The strategy BPSM modelling suggested longshore transport rates of between 30,000 and 45,000 m^3per year, whereas the Beachy Head to Rye Harbour Coastal Process and Resource Study (which used a different wave model to transform a longer offshore time series inshore) determined average sediment transport rates to be between 20,000 and 25,000 m^3per year eastwards. This difference was attributed to the different wave models used. However, the beach monitoring results mentioned above do support a transport rate of around 30,000 m^3per year.

Design of the scheme involved extensive modelling and a large number of options were investigated, partly due to the change in scheme design necessary as a result of the environmental constraints imposed on shingle extraction. The modelling files reviewed suggest that much care was taken to replicate the real-life situation as accurately as possible, although it was acknowledged that it was not possible to take into account the reactive nature of recycling.

The PAR recommended that management of the frontage should be adaptive to take account of uncertainty in the model with regard to the variability in annual drift rates and the uncertainty regarding sediment feed at Cliff End.

Modelling, combined with engineering judgement, did indicate that construction of groynes were likely to cause downdrift impacts along this frontage. The scheme's success was therefore very much dependent on adequate recycling to ensure that groyne bays were filled and therefore sediment transport was not being totally inhibited. A review of the model runs also reveals that modelling showed that a large initial recycling was important to the success of the scheme.

Calibration of the model found that the model was most successful using a smaller sediment size than the average D_{50} of the existing sediment. Although this could be an issue on beaches replenished with dredged material, it is acceptable on this beach, where native, downdrift material was the source of sediment.

C.7.6 Lessons for future beach modelling/design

The timing of scheme construction relative to completion of modelling studies can have an impact on the predicted behaviour of the scheme, in this case because the initial recycling volume was so dependent on the available of material in the source area and potentially as baseline conditions may have altered in the interim period. Where a scheme does rely on such accurate information, it is important to ensure that the most up-to-date information is incorporated prior to construction, with the impacts of any change fully considered.

The extensive modelling of this coast led to a greater understanding of the processes and beach response. So although the scheme did not follow the proposed plan of works, the model runs provided a large amount of information to inform decisions on how to respond to the change in scheme. This information should continue to be used by coastal managers.

Modelling can indicate where the uncertainties lay and the potential impacts of these uncertainties on potential beach behaviour. Here, this appears to have led to a more flexible scheme being developed, with a heavy emphasis on monitoring. The final scheme design also allowed for additional groynes to be incorporated should monitoring support their requirement. This type of flexible approach is advocated where there are a number of uncertainties to be accommodated.

Calibration of the model was most successful when adopting a smaller sediment size in the model than the actual material on the beach. Although this would be an issue on beaches replenished with dredged sediment, it would be acceptable where native sediment is the source of recycling material.

There is debate about the suitability of one-line beach plan shape models for use on shingle beach, but the application here appears to have been successful. This is probably due to the fact that the beach model could actually be calibrated successfully.

C.7.7 Bibliography

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C.8 Prestatyn

C.8.1 General information

Prestatyn
1993
Rhuddlan Borough Council (now Denbighshire County Council)

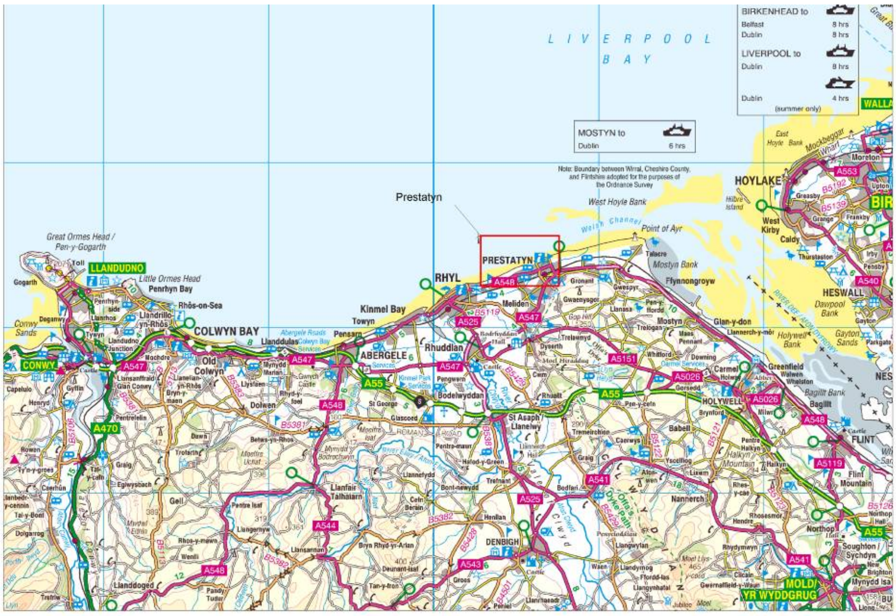

Figure C.8.1 Location of site

Background:

Up until the mid-20th century, the coastal frontage of Prestatyn (Figure C.8.1) was undefended and consisted primarily of a natural dune belt.

In 1951 the then neighbouring local authority to the west, Rhyl Urban District Council, constructed new coastal defence works immediately west of the Rhyl/Prestatyn boundary. At the time, the then Prestatyn Council Engineer suggested, prophetically, that ‘interference by artificial construction with natural coast formation will have incalculable repercussions on neighbouring coasts and will in the long term create more problems than it solves’.

Over the next 20 years the dunes across the Prestatyn frontage were progressively protected due to erosion of the shoreline and groynes were introduced and subsequently modified in an attempt to control beach levels across the frontage.

In 1978, a report on coast protection across the Prestatyn frontage identified beach loss across the Prestatyn frontage as being due to:

- normal cyclical coast erosion
- provision of artificial coast protection works
- retention of material by groyning to the west at Abergele and Rhyl

This report suggested that the shortening of the groynes should help beach levels across the frontage but that some beach recharge might be required.

Pursuant to studies carried out by Dobbie & Partners in the mid-1980s, a programme of works to replace life-expired sections and/or strengthen other sections of the linear defences was carried out between 1986 and 1991. In addition a programme of replacement of the original timber groynes with rock groynes and some localised regarding of the beach were carried out.

Details of the scheme

In 1992-1993 a contract was let to increase the level of the rock groynes and to import beach recharge from ‘licensed dredging areas’ to improve beach levels between the groynes. Material used in the scheme was placed across the beach between rock groyne no. 13 which is approximately 350 m east of the old Rhyl/Prestatyn boundary and rock groyne no. 3 at the east end of the Prestatyn Central Beach defences at Barkby Beach. The work was carried out during the period January to May 1993.

Key sources of information

- HR Wallingford, 1986. Wave Modelling Report EX 1369.
- CH Dobbie, 1987. Coastal Investigation Barkby Beach to Splash Point Phase 2.

C.8.2 Approach to modelling and basis of design

Rationale

No access to design and/or engineer’s reports was available during preparation of this case study. It is believed that desk study and empirically based design approaches were used, with modelling and data collection carried out to provide design parameters.

Overview of approach

Modelling was used to establish wave conditions.

Offshore wave conditions were established by:

- a wave rider buoy deployed approximately 10 km offshore from Prestatyn between February 1985 and May 1986

- numerical wave hindcasting of hourly wind data for the period 1979 to 1984 to put the year of measured data into a long-term context

Six years of recorded wind data from Squires Gate Airport, Blackpool, were used as input for HINDWAVE model.

Two tidal current field investigations involving tracking of floats were carried out in February 1985 and June 1986 before and after modifications to the then timber groyne field. Note these data were not directly relevant to the recharge scheme as the groynes were subsequently replaced by rock structures.

No recorded water level were available locally.

No modelling was undertaken. The scheme design is understood to have been based on empirical methods and engineering judgement.

Inter-tidal profiles at 10 locations were obtained by photogrammetry from vertical aerial photography in November 1983; July 1984; April 1985; November 1985 and June 1986.

Selection criteria

Suitable material, with a D_{50} greater than or equal to the existing beach material, was available nearby.

- Natural sediment: Sand
- Modelled sediment type (size): N/A
- Sediment in final design: Sand
- Sediment placed: Offshore dredged sand

No records of actual grading of material are contemporarily available. Known data summarised in Figure C.8.2.

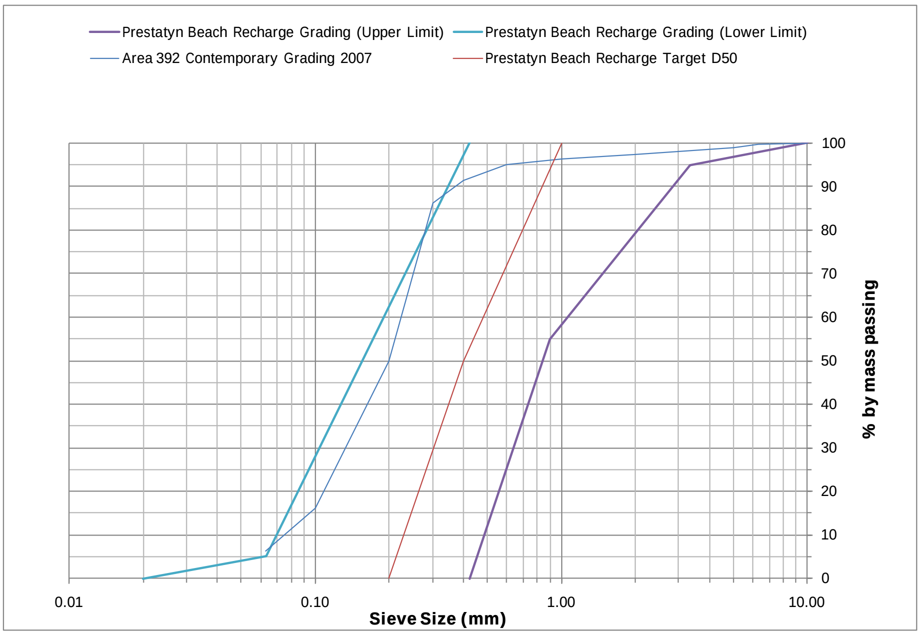

Figure C.8.2 Sediment grading target and achieved

Prior to the scheme

A total of 13 low-level rock groynes across the frontage had replaced earlier timber structures.

Options considered

No options were considered.

The profile defined for the recharged beach, superimposed on the pre-scheme profile, is shown in Figure C.8.3.

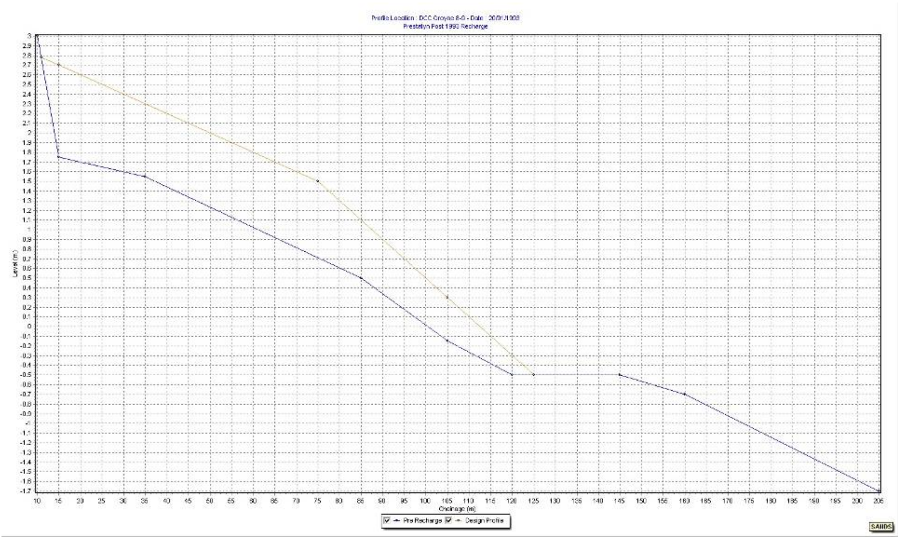

Figure C.8.3 Design and pre-scheme beach profiles

The design profile for the beach was:

- a crest level of 2.7 m OD at the toe of the defences
- a slope of 1 in 50 over the first 60 m to a level of 1.5 m OD
- a slope of 1 in 25 from 1.5 m OD until the profile met the existing beach

C.8.3 Design/modelling outputs – plans for implementation

- Beach movement was to be largely controlled by groyne structures, but with some transference of material expected between bays.
- No specific topping up arrangements were identified.
- Beach monitoring was to be carried out to verify changes in profile.
- Beach management was to be carried out as necessary.

The material used in the Prestatyn Beach Recharge scheme in 1993 came from the Hilbre Swash (or alternately named 'West Hoyle' offshore licensed dredging area (Areas 392 and 393) (Figure C.8.4).

Figure C.8.4 Source of recharge material

The original contract was for the importation of 110,000 m^3 but following receipt of tenders, which were significantly lower than the estimate, the amount was doubled and the scheme records identify that approximately 210,000 m^3 of beach recharge material was placed.

The material used was dredged from the seabed by commercial plant and transferred to a bottom dumping barge approximately 1 km from the site. Material was dumped directly onto the beach from the barge, above low water, during suitable tide conditions. The material was the pushed up the beach to the required profile using conventional land-based plant, that is, bulldozers.

Following the recharge operation, post-completion monitoring of the beach profile was carried out in the centre of each groyne bay from 1993 to 1998. From November 2002 to the present day (2012), topographic plan surveys of the beach have been carried out to the frontages to either side – Rhyl Golf Links to the west and Gronant and Talacre Dunes to the east.

Changes in beach profile have been analysed using the SANDS (Shoreline and Nearshore Data System) developed by Halcrow. The results are provided in Table C.8.1.

Key sources of information

- Coastal Engineering UK Ltd, 2005-2010. Denbighshire Annual Local Process Monitoring Reports.
- Coastal Engineering UK Ltd, 2011. Prestatyn and Talacre – Review of Beach Feeding Schemes.

Table C.8.1 Changes in beach profile

Profile Reference / Pre 1998	Area Change 1993-1998 (m^2)	Av. Area Change 1993-1998 (m^2)	Vol. Change 1993-1998 (m^3)	Area Change 2002-2008 (m^2)	Av. Area Change 2002-2008 (m^2)	Vol. Change 2002-2008 (m^3)	Area Change 1993-2008 (m^2)	Av. Area Change 1993-2008 (m^2)	Vol. Change 1993-2008 (m^3)
Groyne 13									
		41.87	6700		-7.27	-1163		32.78	5245
Groyne 12-13	41.87			-7.27			32.78		
		47.36	14207		8.98	2696		67.36	20208
Groyne 11-12	52.84			25.24			101.94		
		47.95	11986		23.28	5820		82.74	20685
Groyne 10-11	43.06			21.32			63.54		
		24.75	8662		14.51	5079		50.73	17756
Groyne 9-10	6.44			7.70			37.92		
		-4.56	-1368		4.01	1203		15.18	4552
Groyne 8-9	-15.56			0.32			-7.57		
		-30.42	-9126		1.84	551		-20.12	-6036
Groyne 7-8	-45.28			3.35			-32.67		
		-41.58	-12473		-2.70	-810		-23.19	-6957
Groyne 6-7	-37.87			-8.75			-13.71		
		-47.11	-14133		-7.08	-2124		-14.86	-4456
Groyne 5-6	-56.35			-5.41			-16.00		
		-43.37	-13010		-0.27	-81		-2.80	-840
Groyne 4-5	-30.39			4.87			10.40		
		-25.84	-7751		2.00	599		-6.49	-1946
Groyne 3-4	-21.29			-0.88			-23.38		
		-21.29	-3151		-0.88	-130		-23.38	-3460
Groyne 3									
		Gr 9-13	**40,186**		**Gr 9-13**	**13,634**		**Gr 9-13**	**68,446**
	Total(s)	**Gr 3-9**	**-59,644**	**Total(s)**	**Gr 3-9**	**-1,995**	**Total(s)**	**Gr 3-9**	**-23,695**
		Gr3-13	**-19,458**		**Gr3-13**	**11,639**		**Gr3-13**	**44,751**

Key points arising from examination of these data are as follows.

- Overall across the frontage, beach volumes at present are greater than when the recycling was completed
- Within that overall trend, different behaviour has taken place with the following applying:
  - o Across the western part of the frontage beach volumes have risen, although behaviour is cyclical.
  - o Across the eastern part of the frontage) beach volumes have fallen, although behaviour is also cyclical.
- Examination of the trends shows that, in the first five years after the recharge (1993-1998), the pattern of behaviour identified above was established.
- More recently (2002-2008), however, the losses have reduced and the majority of the frontage is showing an accretive trend.
- Overall, however, when the recharged profile is included with the more recent data (1993-2008) the trend is still one of gains over the western half and losses over the eastern half

No beach management has been carried out since the scheme was implemented, apart from local recycling of wind-blown sand back onto the beach.

Figures C.8.5 to C.8.9 present wave condition data from pre- and post-construction. The conditions shown are for a location approximately 1 km from the shoreline in shallower water.

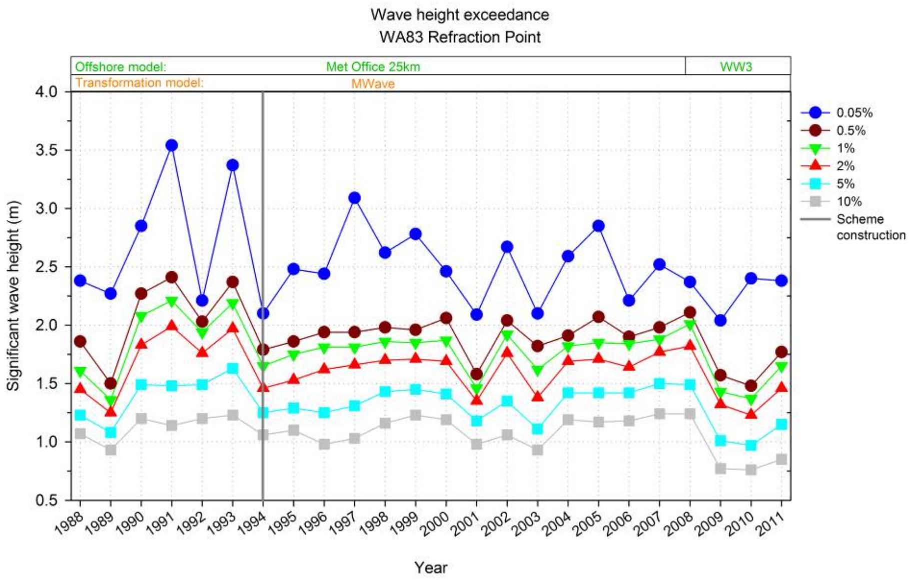

Figure C.8.5 Wave climate pre- and post- construction at point WA83

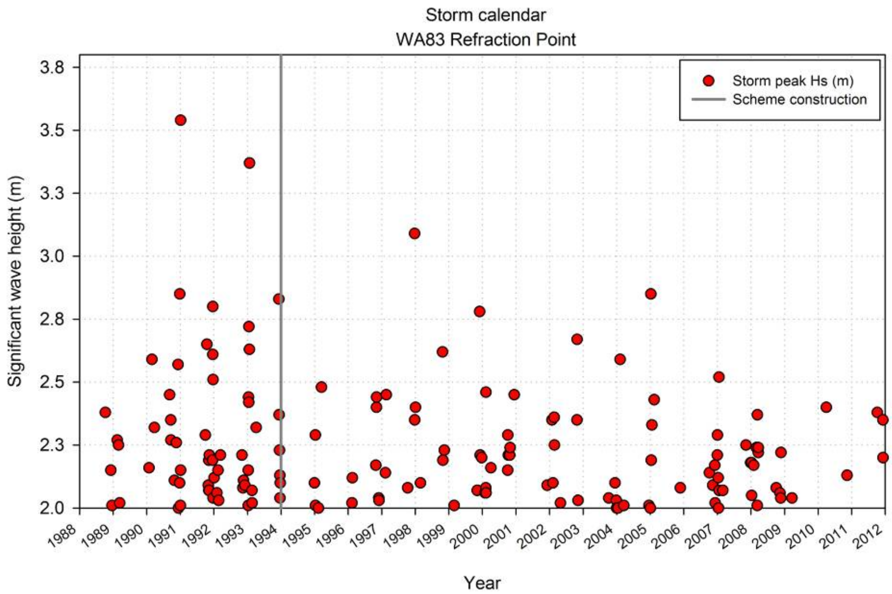

Figure C.8.6 Storm events at point WA83

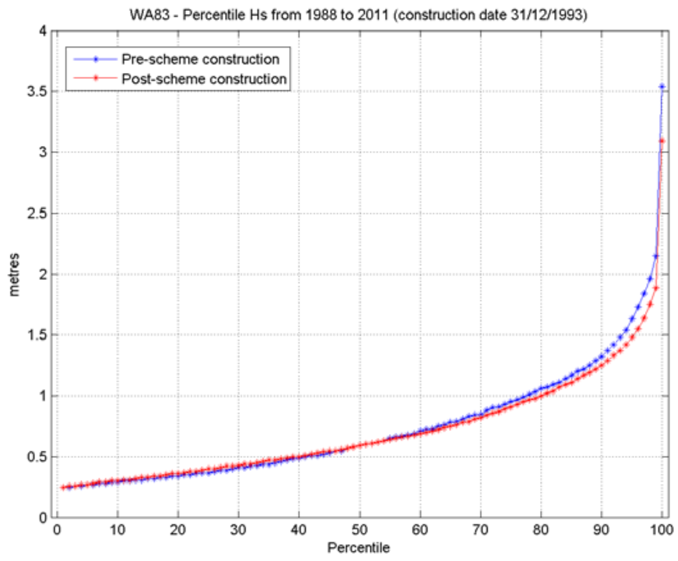

Figure C.8.7 Percentage distribution of pre- and post-construction significant wave heights

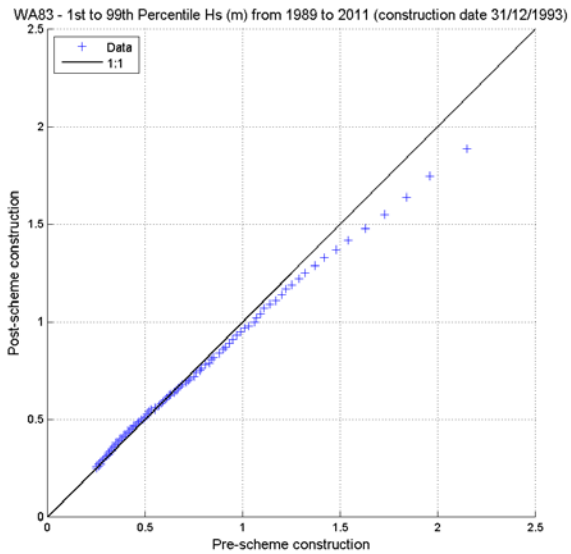

Figure C.8.8 Comparison of pre- and post-construction significant wave heights

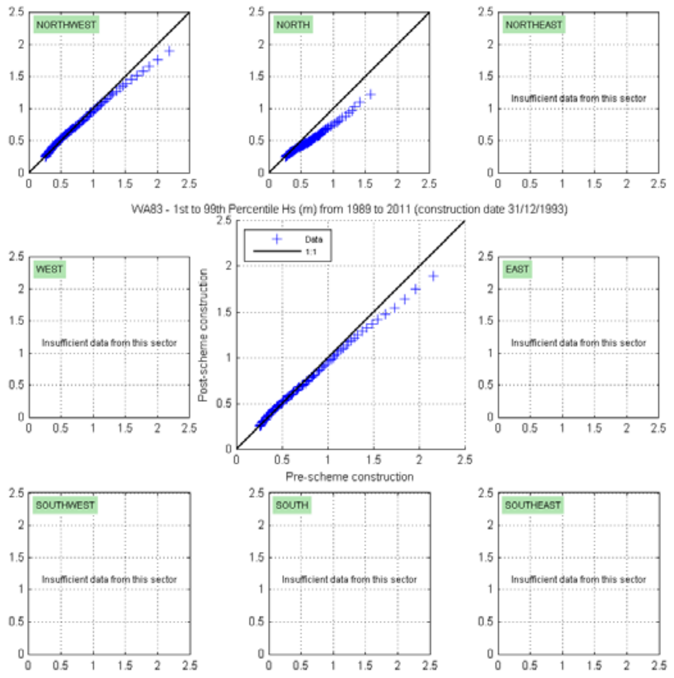

Figure C.8.9 Comparison of pre- and post- construction wave climate

The wave height exceedances identified from recorded and hindcast conditions pre-scheme were greater than the values identified from the modelled wave climates identified above. However, the wave conditions available pre-scheme were for a location approximately 10 km offshore.

Pre-scheme wave heights generally in excess of 1 m were more prevalent from all directions, indicating that the beach has been subject to less wave energy than occurred immediately prior to scheme implementation. However, without specific pre-scheme performance predictions, it is not possible to identify the impacts of the difference in conditions on scheme performance.

C.8.5 Comparative analysis

No comparative data were available.

The combination of rock groynes and beach recharge has acted to stabilise beach levels across the frontage, with the rock groynes playing a major role in the observed behaviour.

Overall the behaviour of the beach has been as expected, although there are no predictions of behaviour against which comparisons can be made. Pre-scheme calculations identified that there was a potential net drift deficit of approximately 60,000 m^3 between material entering and leaving the frontage. The scheme has turned this potential deficit into an average net gain of approximately 3,000 m^3 per year.

There is a difference in performance with the updrift (western) half of the frontage gaining material (≈4,500 m^3 per year on average), while the downdrift (eastern) half of the frontage has lost material (≈1,500 m^3 per year on average). This is believed to have been due to the westerly groynes intercepting drift immediately post-scheme and preventing material from moving further easterly. Analysis of the most recent data (2002-2008) suggests that ongoing drift is gradually being reinstated with only a small net loss over this period taking place across the easterly half of the frontage.

Across the downdrift frontage immediately east of the scheme limits, examination of data provided three important observations.

- Immediately downdrift there are two further rock groynes where the beach was not nourished. The general effect on behaviour in this section has been neutral with the groynes stabilising levels across this section.
- Downdrift of the final rock groyne, over a distance of approximately 1 km, beach volumes have reduced, suggesting that the scheme has caused starvation in this area.
- Beyond the 1 km limit, drift mechanisms appear to have re-established and accretion is taking place

The key to scheme performance has been the control on beach behaviour exerted by the rock groyne control structures. The structures have not blocked all the drift, but have acted to maintain improved beach levels across the frontage while allowing natural process behaviour to be maintained.

C.8.6 Lessons for future beach modelling/design

The scheme was carried out without any detailed modelling of scheme behaviour and/or performance, using a solid background and knowledge of local process behaviour allied with inputs from experienced staff who had a good understanding of how the scheme was likely to behave. Without detailed modelling of the beach behaviour it is possible that the scheme design may have been more conservative.

The key lessons to be learnt from this scheme are as follows.

- In many aspects, specific local knowledge and experience can be equally important as detailed modelling, although a thorough understanding of process behaviour and likely scheme impacts backed up by empirical calculation and judgement are essential.
- Detailed and in some cases expensive modelling may, however, not always be necessary.
- Consideration of all available design tools, including modelling, is important at the outset to ensure that the design is based on the best possible understanding.

C.9 Preston Beach

C.9.1 General information

Preston Beach (Weymouth)
1995-1996
Environment Agency South West Region, with Weymouth & Portland Borough Council and Dorset County Council

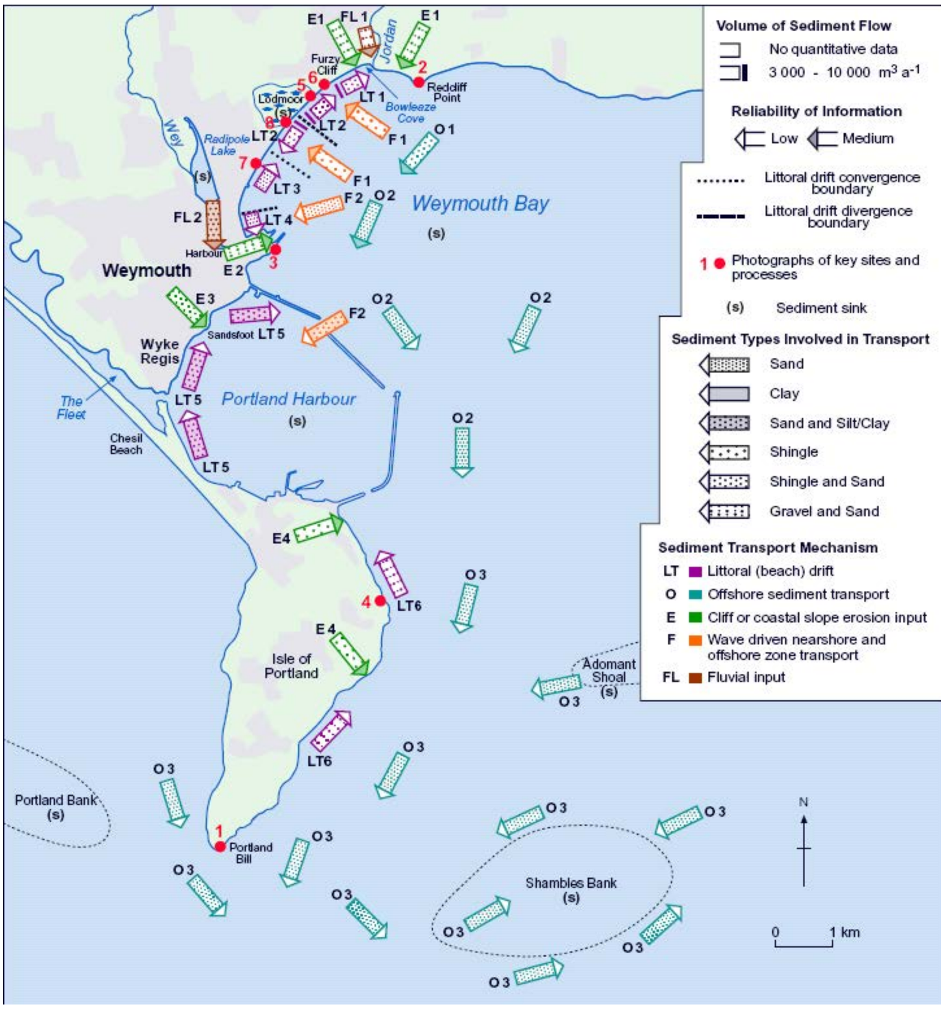

Figure C.9.1 Location of site

Preston Beach is located towards the northern end of Weymouth Bay in Dorset and is a popular amenity destination for a range of activities ranging from water sports to walking (Figure C.9.1). The 1.4 km long beach is orientated from north–east to south–west and is adjacent to the A353 Preston Beach Road, a major highway route for the town of Weymouth. Preston Beach has no environmental designations but is adjacent to a number of other environmentally designated sites including:

- Lodmoor SSSI and Royal Society for the Protection of Birds (RSPB) reserve
- Overcombe Site of Nature Conservation Interest (SNCI)
- South Dorset Coast SSSI
- Isle of Portland to Studland Cliffs SAC
- Dorset & East Devon World Heritage Site (the 'Jurassic Coast')

Preston Beach forms part of a complex sediment transport system within the wider Weymouth Bay that extends from Redcliff Point in the north to (at least) the Weymouth Harbour entrance in the south (Figure C.9.1). It is also afforded some protection from wind-driven waves from the south and west by the presence of the Portland Harbour breakwaters. The exact nature of how this sediment transport system works is subject to some debate and there are a number of uncertainties that remain to be answered.

There is a long history of flooding and erosion problems at Preston and an equally long history of man-made intervention. For example, in the late 19th century, a road that followed the beach had to be set back 60 feet, but it was already being overwhelmed again by the end of the century. Groynes erected to stop shingle drift had 'wasted away' by 1883 following which many thousands of tons of Portland Stone, in the form of large blocks, were placed along the shore to provide protection. Subsequent works saw the blocks replaced by a retaining wall and the addition of further groynes. After the Second World War, patchwork repairs were undertaken to the wall, but there was frequent overtopping leading to large volumes of shingle being deposited onto the A353 road, a road whose importance as a major route into Weymouth had grown significantly during the last century.

After a particularly fierce storm in 1989 a section of the sea wall collapsed. This focused attention on the need for a more efficacious solution and the investigations that form the basis of today's scheme commenced. In addition to transport chaos, overtopping of the old retaining wall could have caused considerable damage to 30 properties, a major SSSI and RSPB nature reserve and some significant tourist attractions. It could have also led to a significant pollution incident had flood water reached a nearby municipal landfill. Figure C.9.2 shows the areas at risk from no scheme being implemented.

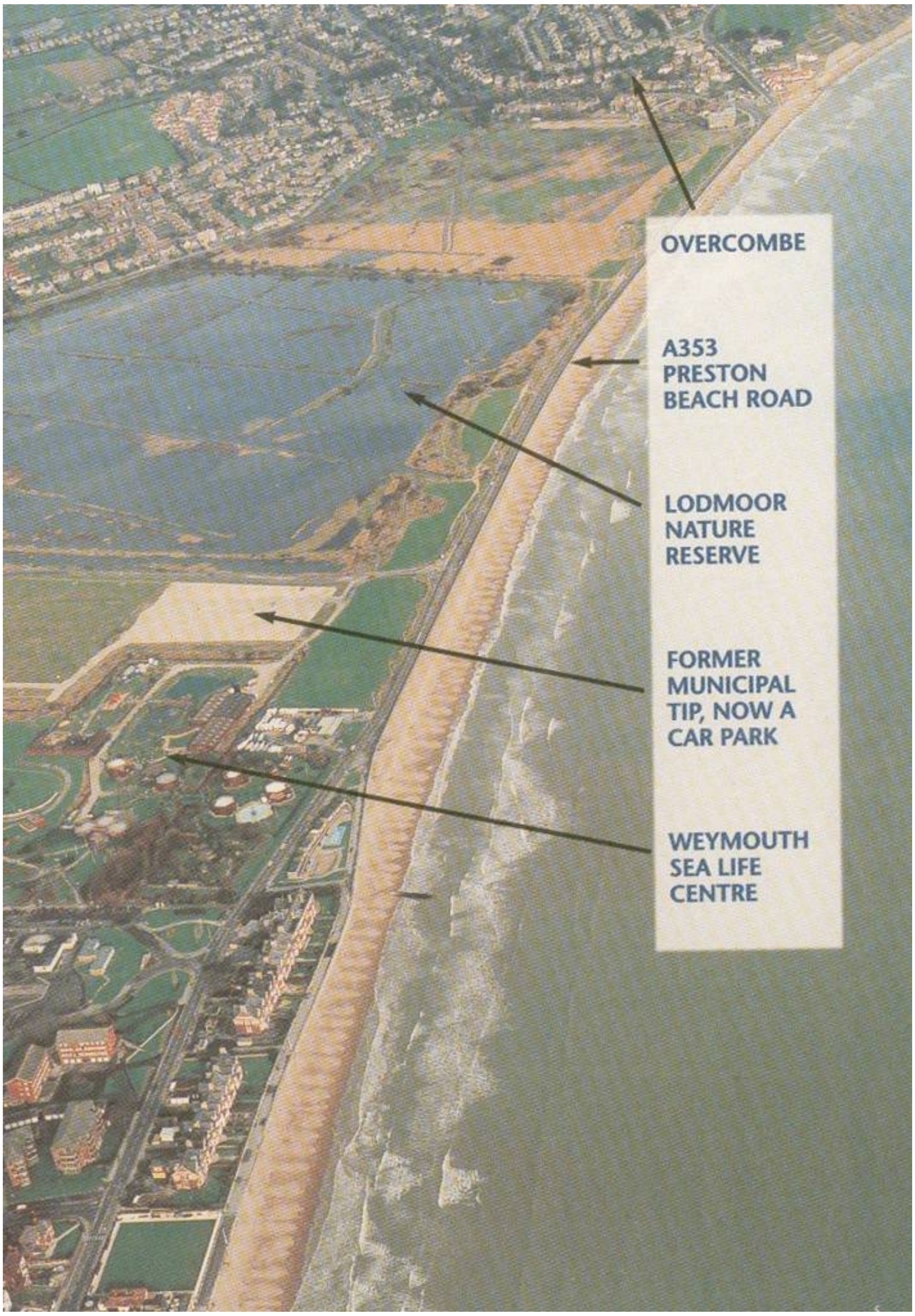

Figure C.9.2 Risks arising from no scheme

The objective of the scheme was to manage the risk of coastal flooding and erosion to assets on the low-lying land inshore which include:

- 86 residential properties
- the A353 Preston Beach Road
- infrastructure
- a municipal landfill
- environmentally designated areas such as Overcombe SNCI and Lodmoor SSSI and RSPB reserve

The beach is adjacent to the A353 Preston Beach Road (a major highway route for the town of Weymouth) and the Lodmoor SSSI and RSPB nature reserve. Preston Beach itself has no environmental designations but it is adjacent to a number of designated features:

- South Dorset Coast SSSI, which includes geological exposures and maritime cliff and slope habitats at Furzy Cliff, immediately north of Preston Beach
- Isle of Portland to Studland Cliffs SAC, which includes Furzy Cliff
- Dorset & East Devon World Heritage Site (the 'Jurassic Coast') which also includes Furzy Cliff
- Dorset Area of Outstanding Natural Beauty (AONB)
- Lodmoor SSSI and RSPB nature reserve and Overcombe SNCI, located immediately on the landward side of the road adjacent to the beach and protected by the beach from tidal inundation
- Weymouth Bay is identified as being a 'Sensitive Marine Area'

Weymouth Bay is also within both the Eastern Channel Marine Natural Area and the South Dorset Coast Maritime Natural Area, both of which designations extend seawards from the mean low water mark.

C.9.2 Approach to/basis of modelling/design

Overview of approach

Scheme design was based primarily on 3D and 2D mobile bed physical modelling, which were supported by numerical modelling of sediment transport (Beachplan). Drift calculations were used to calibrate the physical model sediment transport rates. A range of beach geometries were considered, allowing for a range of levels of investment and allowable levels of overtopping. These have been analysed by sensitivity testing of a range of storm events and storm profiles. The scheme design allows for the occurrence of all those events which should statistically occur within the design life of 50 years. A series of hydraulic model studies were carried out to test the proposed designs and to fine tune designs for maximum cost effectiveness and hydraulic performance. The objectives of the model studies were to:

- identify the standard of service of the pre-scheme system
- determine the optimal combined beach geometry and structure configuration to provide protection against overtopping in 1:100 year joint probability return period of wave and water levels
- determine the anticipated long-term plan-shape evolution and longshore transport rates
- compare the performance of proposed stabilisation measures with the existing situation
- examine the effects of proposed terminal structures on shingle transport
- estimate long-term maintenance commitments required to maintain the required standard of service
- identify threshold crest levels and widths to provide alarm levels prior to unacceptable overtopping and failure of the seawall
- identify a planned maintenance programme following beach recharge

Analysis of beach profile field data indicated that damage occurs most frequently in severe wave conditions associated with storm surges. A range of water levels including extreme storm surges were considered in combination with storm waves, and frequently occurring conditions, in various sequences. Beach responses to these processes were examined by measurement of short-term changes to the beach cross-section profile and plan shape. The beach was modelled in a 3D wave basin at a scale of 1:50 and also in a wave flume at a scale of 1:25. Modelling of beach sediment was based on pure shingle sediment with a fine sediment cut-off at 6 mm.

The large model scale allowed the sediment response to waves to be reproduced with a high degree of confidence and also allowed rock armour movement to be reproduced and monitored accurately. Changes in alignment of the beach and effects of sediment control structures, such as a terminal groynes, were also examined. The test programme was broken down into the following elements:

- mathematical modelling of the offshore and nearshore wave climate
- validation of the physical model methodology against pre scheme layout
- physical modelling alternative cross section and plan layouts of proposed scheme
- numerical modelling of sediment transport, interactive with the physical model

Design data were derived from long-term deployments of tide gauges at Weymouth. Extreme water levels tested were based on those measured during the 1989 storm, estimated to have a return period of approximately 1:50 years.

Initial modelling was based on a single survey of the beach undertaken prior to the modelling, around 1994. No regular beach monitoring data were available to calibrate the sediment transport in the plan shape model.

Various combinations of waves and tides produced alternative design conditions with similar joint probabilities. Each has been considered as a separate design condition, due to the complexity and variation of failure. Combinations which lie between extreme combinations were also considered.

Offshore time series data were derived from a hindcast using Hindwave, driven by Portland wind data for the period 1974-1990. Note the model does not include swell waves. Offshore wave data were transformed using refraction modelling to two locations at either end of the study area. Subsequent post-scheme construction modelling (2002) was based on the Met Office 25 km hindcast model transformed inshore using a 2D wave model.

Design rationale

Interactive numerical and physical model tests were used to design the scheme. The crest level of the recharged beach must therefore be above the level of maximum run-up during the design storm and the beach must be suitably wide to allow the dynamic profile to develop fully within it. This beach configuration must be maintained throughout the course of the scheme. The varied wave climate along the length of the site results in a differing beach response from west to east. The more severe conditions in the west result in higher wave run-up and therefore a higher natural beach crest and wider active foreshore.

Cross-shore response predicted by physical models

An extensive programme of tests was conducted in a 2D flume and a 3D mobile bed physical model. Design wave conditions, derived from synthetic wave data, provided the basis for physical model testing of the cross-shore beach response.

The primary purpose of the model was to determine the appropriate cross-section of the recharge to:

- avoid overtopping in all but the most extreme conditions
- identify critical conditions that could be used as a guide to inform the need for intervention during long-term management

A range of cross sections were tested with the intention of providing a section that would avoid overtopping during the design conditions.

Beach evolution and longshore transport

A one-line beach plan shape model was used to assess the beach plan shape. The model was based on a recent beach survey, but no data were available to calibrate the sediment transport rates. The beach is orientated from north–east to south–west with a typical shore normal angle of 135°N, Waves arriving within direction sectors >135° should drive sediment towards the northeast, while those arriving from direction sectors <135° should drive material to the south. The lack of availability of beach monitoring data to calibrate the numerical model may have had a significant effect in the final outcome of the model.

Longshore transport tests and beach mathematical models were tested using morphological averaged conditions based on wave climate statistics to determine rates of longshore transport and potential longshore losses.

This suggests that beach transport will occur in both directions, with a small net transport typically to the south-west. The small volume suggested by the modelling indicates that the sediment transport rates are low. The frequent drift reversals indicate that the beach alignment is close to an equilibrium shape relative to incident wave conditions. The wave climate data do not include swell wave conditions, which will include conditions primarily from the west and south-west. Such conditions might reasonably be expected to drive sediment towards the north-east, since they will have originated from the south-west.

Grain sizes for physical modelling and numerical sediment transport modelling were based on grading curves derived from a series of sediment samples captured across the beach profile at surface. The sediment characteristics are summarised below for each stage of the design and construction process.

- Natural sediment: shingle with sand mixed beach, D_{50} ~13 mm but varies widely; grading envelope used in design
- Modelled sediment type (size): D_{50} 13 mm (scaled – cut off at 6 mm in physical model)
- Sediment in final design: shingle with sand mixed beach; grading envelope based on indigenous material (see Figure C.9.3)
- Sediment placement in final design: grading envelope modified based on indigenous material with reduced D_{50} and increased sand content achieved, based on local offshore supply (not shown)

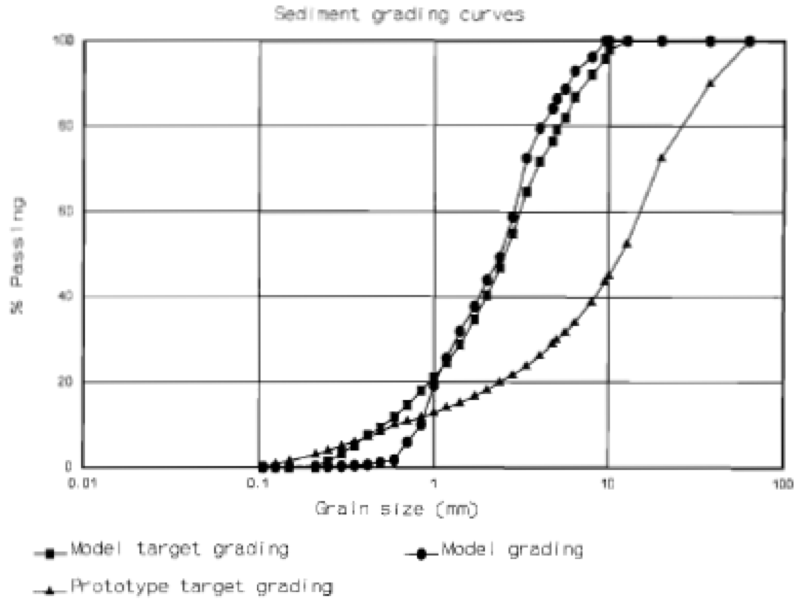

Figure C.9.3 Design and model sediment gradings

A rock groyne acting as a terminal structure was designed to capture material anticipated from the north-east. The structure is armoured with 3–6 tonne rock at varying slope angles, with a crest at 2.5 m ODN.

Two alternative structures were tested in combination with beach recharge of varying lengths:

- recharge of the barrier beach with 214,000 m^3 suitably graded shingle (based on sediment supply from an offshore dredging area)
- construction of a rock revetment
- construction of a single terminal groyne
- a 1.4 km long embankment seawall, promenade and revetment

Cross-shore response

Hydraulic model tests identified that a beach with a 25 m wide beach crest, at a level of 3.5 m ODN, and with an initial seaward slope of 7.5:1 would not be exceeded by green water under any of the combinations of waves and water levels tested, although shingle might be expected on the promenade in the most extreme conditions. The required crest width varied from 25 m at the north-eastern (Overcombe) end of the frontage to 15m at the south-western (Greenhill) end of the frontage. Design profiles (Figure C.9.4) were based on beach recharge with sediment of similar grading to the indigenous beach material. Model tests also identified threshold geometry conditions for each profile beyond which the beach would be vulnerable under the design storm. Variable longshore wave climate arising from nearshore bank system results in variable design conditions along the length of the site.

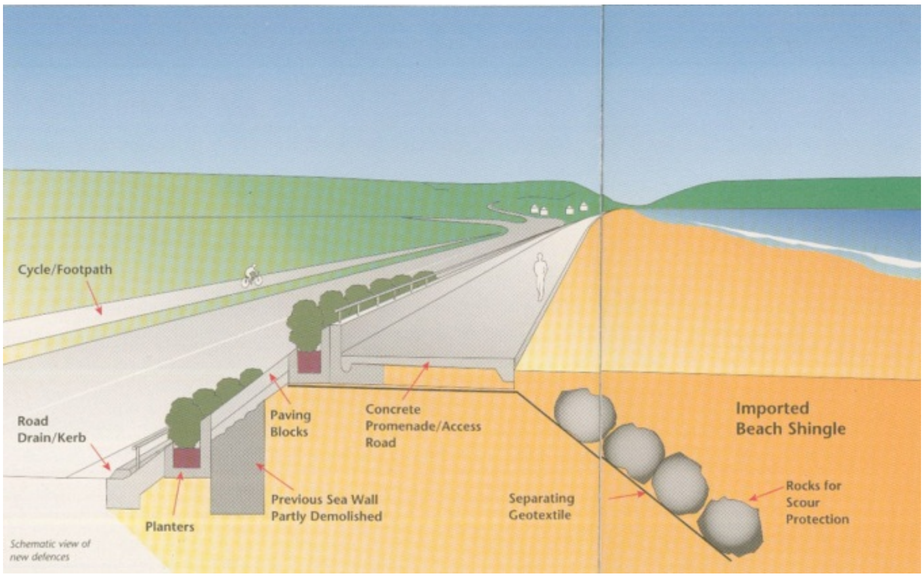

Figure C.9.4 Schematic of the 1995-1996 design constructed at Preston Beach

Longshore transport

Longshore transport tests suggested that the transport rates would be slightly higher with the recharge solution than the existing beach. Modelling noted that use of a finer or more widely graded recharge material might increase the sediment transport rate. Net average sediment transport rates of 2,900 m^3per year were estimated towards the south. The plan optimised position of the groyne produced a short embayment.

Structure design

The seawall, promenade and beach were designed to act together to manage the risk of coastal flooding and erosion along the frontage. Should the crest level of the beach drop below +3.3 m OD in front of the promenade, a rock revetment would be exposed. The revetment, made up of a single layer of rock armour, has been designed to provide short-term protection to the toe of the promenade in advance of the beach being recovered by recycling and re-profiling. The rock revetment is not designed to withstand direct wave action and, if it is exposed, there is a significant risk of the rock armour being displaced and the promenade being undermined which could lead to progressive failure of the defence.

Discussions with aggregate suppliers revealed that none of the licensed areas on the south coast could meet the specification grading, at reasonable cost. As it appeared unlikely that commercial licensed areas could supply material of the preferred design grading, the design process was reviewed and an alternative design based on finer wider gradings considered. The basis for alternative recharge designs using materials with finer and wider gradings made the assumptions that these materials would have the following effects on the hydraulic performance of the beach.

- The beach will form a dynamic equilibrium slope at a shallower angle for either finer or more widely graded materials than for the indigenous beach grading. This would require a larger quantity of material to form the capital recharge.
- The longshore sediment transport rate would be faster for finer material than for coarse material. Losses from the system would therefore be greater. This would result in a requirement for more frequent and higher volumes of maintenance to be included in the beach management plan.
- The use of a finer grading or a more widely graded material would reduce the permeability of the beach
- More widely graded materials would contain a higher proportion of fines, which are likely to be lost from the system at an early stage.

These assumptions were not tested in either physical or numerical models.

C.9.3 Design/modelling outputs – plans for implementation

The scheme design life was 50 years (to 2045) and provides a standard of protection with an annual probability of occurrence (APO) of 1% (1 in 100 year return period) against overtopping and an APO of 0.2% (1 in 500 year return period) against breaching. This standard of protection is provided by the beach, seawall and promenade and is maintained provided that a suitable beach profile is retained along the frontage. A preliminary beach management plan was developed at the design stage and is outlined below.

Outline 50-year programme

On completion of the capital recharge scheme, the beach was expected to withstand the design storm conditions without risk of overtopping or breaching under design conditions. The recharge was, however, a dynamic structure which would was expected to modify over time due to both cross-shore and longshore transport processes and it was anticipated that occasional maintenance might be required once every 7–10 years throughout the scheme life.

The beach management plan relies on an understanding of performance derived from a simple monitoring programme in conjunction with pre-defined alarm conditions to provide a decision support system for the maintenance programme. The scheme has a design life of 50 years, during which there will be a requirement to recycle or top up the recharge and to maintain the rock beach-control structures. Estimates were made to facilitate development of a preliminary programme of recharge maintenance. The programme was to be revised in conjunction with the results of the planned monitoring programme at strategic (five-year) intervals.

No further introduction of additional beach recharge materials was envisaged within the 50 years of the beach management plan. Planned maintenance work was limited to recycling of material on average once every 7–10 years. An allowance for maintenance of the rock structures was included; this would follow the initial settlements and movements which might be expected during the first few storm seasons. Further maintenance of the beach-control structures was also planned at strategic intervals during the life of the scheme. The estimated volumes are based on estimates derived from beach plan shape modelling.

Detailed five-year programme

Maintenance

A programme of planned maintenance was developed based on the results of the physical and mathematical model studies. These suggested that recycling might be required (from the south-east end of the beach about once in five years.) An allowance for an average £10,000 per year expenditure was allowed within the benefit–cost assessment for this activity.

Model studies suggested that the new recharge might be expected to achieve an equilibrium plan shape within the first two years**.** The rate of shingle loss was estimated at 2.900 m^3 per year, allowing for initial adjustment losses, and subsequent typical longshore transport losses. An average loss rate of 2,900 m^3per years was projected over the first 10 years. Allowance for an average 2,000 m^3per year recycling was made, assuming that 10,000 m^3 transport occurred over a five-year period (Figure C.9.5).

Projections suggested that an interim recharge would be required after 50 years and the maintenance programme would be also reviewed to reflect monitoring. The maintenance programme was due to be reviewed in epochs of five years.

Threshold levels

Damage threshold and alarm conditions were defined at which beach maintenance is necessary to avoid breaching failure (emergency). This is taken as the point at which the stability of the promenade and seawall could be compromised. The alarm condition is defined at the beach condition required to achieve an acceptable level of overtopping (alarm); these have both been defined with the aid of physical model studies and relate to the 1 in 100 year (1%) APO.

The maintenance works should aim to ensure the beach along its length has a crest level of at least +3.5 m ODN, with a minimum crest width of 15 m, and beach slope no steeper than 1:7.5.

Beach recycling works are to be carried out as required throughout the year. The need for these works is to be triggered by the action levels (crest width <15 m or crest level <+3.5 m OD) and emergency levels (crest width <10 m or crest level <+3.3 m OD).

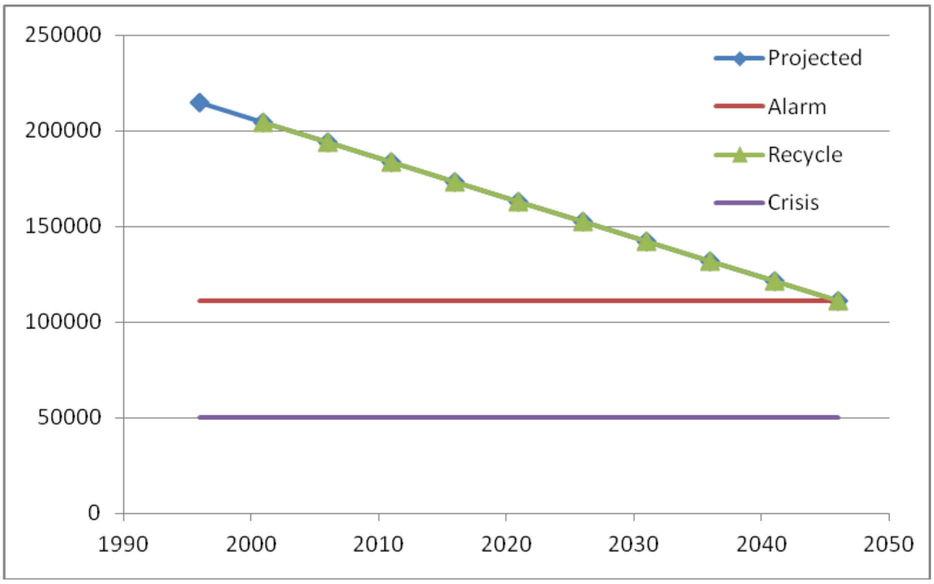

Figure C.9.5 Planned maintenance programme from 1996 to 2046

C.9.4 Beach management and performance

Works for the £6.3 million capital scheme, started in January 1995 and were completed in July 1996 (Figure C.9.6). A 1.4 km long embankment seawall and promenade were constructed. Approximately 214,000 m^3 of dredged material were supplied from offshore and then spread by bulldozers over the 1,400 m length of the beach. Beach recharge was placed adjacent to the seawall and promenade to reduce the volume of overtopping onto the promenade, seawall and onto the A353 during storms.

The dredged material was won from the nearby Needles (Isle of Wight) aggregate dredging area. The material was pumped onto the beach and, when dry, spread to a nominal crest on average 20 m wide, with a height of +3.5 m ODN and a seaward slope of 1 in 7. The retaining wall was rebuilt with the addition of a 700 mm high upstand sea wall (to prevent overtopping and shingle being deposited on the road) and a concrete promenade was built on the seaward side of the upstand.

A terminal rock groyne comprising 6,200 tonnes of 1–8 tonne rock was constructed at the southern end of the scheme at Greenhill. Modelling showed that beach material would drift south, so the terminal groyne was included to catch this material. It was assessed that the material so caught would need to be redistributed across the beach every 7–10 years.

A single layer of rock armour was installed along the seaward edge of the promenade. If the beach experiences drawdown during a storm event (below 3.3 m OD) and the rock armour is subject to direct wave action it will be displaced, which could undermine the promenade, leading to progressive failure of the defences.

Figure C.9.6 As-built beach cross-section

The as-built scheme reflects all the geometric and volume details developed at the design stage, thereby making comparison of the performance and the design tools more straightforward. The geometric characteristics of the final design were based closely on the physical model.

The design grading envelope (indigenous material) against as-built grading samples is shown in Figure C.9.7.

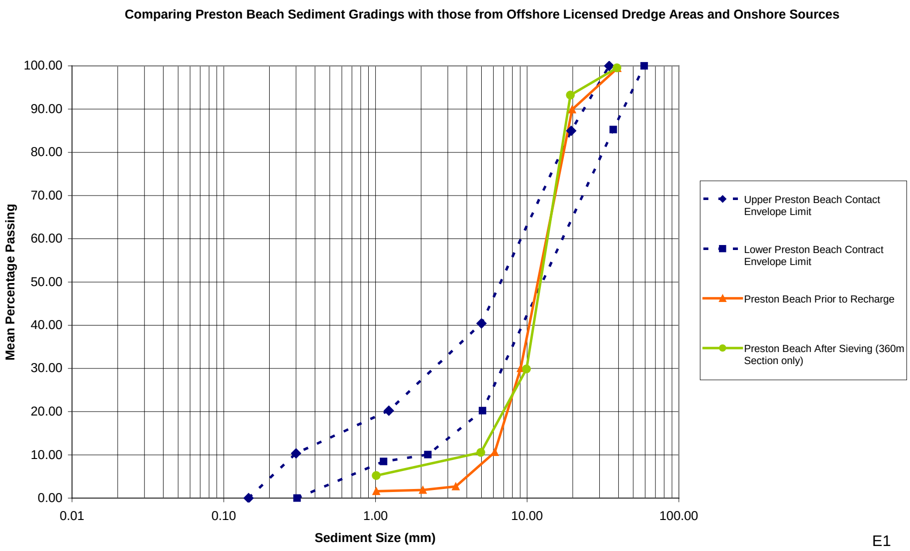

Figure C.9.7 Design, as-built and sieved grading envelopes

Evidence from the as-built surveys indicates that the beach crest was built to a slightly higher elevation than designed. The dredged beach recharge material had a D_{50} of 11 mm and a sand content of less than 45%. However, the possibility of increasing the D_{50} to 13–15 mm was looked into. Such material, which would have increased the new beach's ability to absorb wave energy, could have been sourced from the Owers Bank on the eastern side of the Isle of Wight. However, greater dredging costs and transport costs would have added £2 million to the project and so the option was not pursued. The main difference between the modelling and the as-built construction relates to the grain size distributions of the modelled and the prototype recharge material. Design of the physical model sediments was based on the grading of the indigenous beach material, which also formed the basis of the recharge design. The local recharge source was unable to meet this target grading envelope and a recharge with a D_{50} of about 11 mm was constructed; this dredged material had a sand content of about 45%.

Physical modelling of the beach was undertaken using lightweight materials (crushed anthracite) designed to simulate the hydraulic performance of shingle. However, the model sediment was scaled to be representative of a shingle grading with a D_{50} of 15 mm, but without the sand content and an effective cut-off of material below a grain size of about 6 mm. This is a standard modelling practice, since mixed sediments cannot be modelled effectively at the selected scale for either 3D wave basin or 2D wave flume modelling. There is a reasonable expectation therefore that the profile response of the prototype and model recharges might be expected to differ, since the model effectively represents a clean shingle while the prototype represents a mixture of sand and shingle, with lower permeability. Academic studies of mixed beach performance suggest that mixed beaches perform similarly to sand beaches when the sand content reaches about 40%, although there is limited published guidance to quantify this difference. There is an expectation therefore that the prototype beach might theoretically develop a flatter slope and with a lower crest than that achieved in the model.

Regular beach surveys have tracked progress of the project performance since construction. The monitoring programme has been developed further since 2002 and again in 2006 on introduction of the regional coastal monitoring programme. Surveys are now conducted three times per year, and also following storm events and maintenance. The seaward extent of surveys has been increased since 2002, so data are not strictly comparable prior to this date. Earlier data from 1996-2002 have been extrapolated seawards, based on the lower beach slope, to enable extension of the dataset.

Profile response

Following construction, it was soon found that a crest width of 25 m along the north-eastern part of the frontage (from Overcombe, south-west of profile 5g00297) for about 600 m) was unsustainable due to insufficient understanding of the general wave climate, drift reversals and location of drift divide along this section, thought likely to in part be a direct result of the lack of available data to calibrate the original model (Figure C.9.9). Rapid reductions in the crest width occurred within 12 months, which reduced the crest width by up to 13 m at some locations An inordinate amount of maintenance was required to maintain the designed berm widths along this part of the frontage. Evidence provided by the Environment Agency suggested that a crest width of only 15 m could be sustained along the north-eastern part of the frontage.

The beach is generally stable now except under southerly waves, where material is transported to the north-east in front of Furzy Cliff. This material is then lost to the recharged beach unless brought back through intervention. Such intervention occurs on average 2–3 times per year. The field data indicate rapid changes to the cross-shore profiles and formation of steep scarps at the upper beach. There is evidence of steep cliffs at the beach crest along much of the frontage. Initial monitoring did not extend throughout the whole of the coastal cell to the north-east of the works, but there is clear photographic evidence of a build-up of material in this zone.

There is clear evidence of regular onshore offshore exchanges of beach material, demonstrated by a series of detailed surveys that have been turned into terrain models and which show transfer of material from the upper to lower beach and vice versa.

The frequency and distribution of storms has been similar before and after the works. Several notable storm events have occurred during which the crest has been cut back and new crest berms formed reaching a maximum level of 4.2 m OD. A notable event occurred in March 2008 (Figure C.9.8) and demonstrates the large-scale cutback of the beach crest in a single storm event. The beach response, which has been monitored by topographic surveys in parallel with wave hindcasts, has been remarkably close to that predicted for storm events with those characteristics.

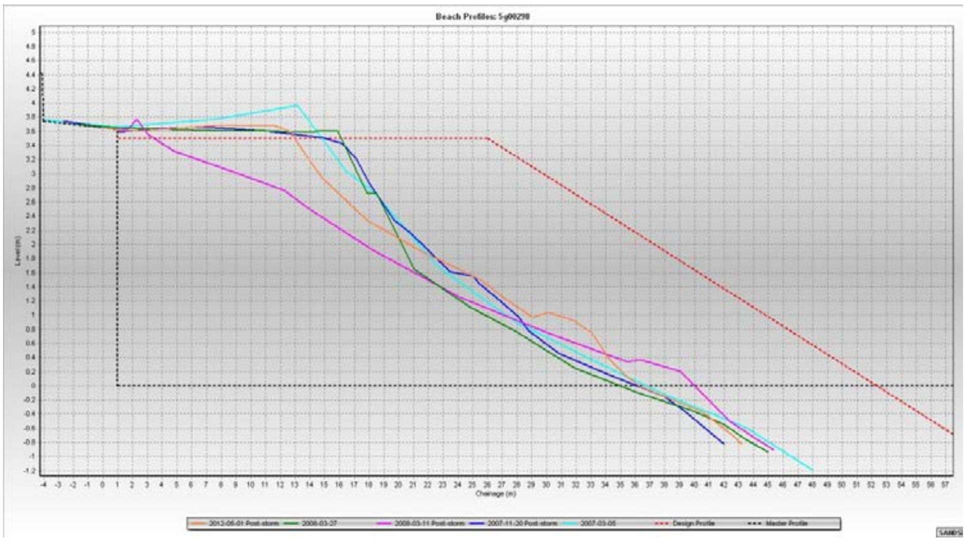

Figure C.9.8 Typical post-storm profile response

Figure C.9.9 Plan layout of monitoring profiles

Planform development

The planform developed following construction indicates that there has been accumulation of material to the north east of the recharge site and rapid erosion at the northern end of the recharge site. There is clear evidence that there has been significant net transport direction towards the north for the whole of the period following recharge. A net build-up of sediment to the north of the beach recharge scheme has been evident on each survey following the recharge.

The plan form of the southern end of the recharge has remained fairly stable. There is limited evidence of south-westerly movement of material, although a slight increase in beach volume to the south of the terminal groyne suggests some limited bypassing of the groyne in a south-westerly direction. Limited realignment of the beach is also evident to the south-west of the site with signs of a build-up, adjacent to the terminal groyne, soon after beach recharge. This zone has subsequently stabilised and has remained unchanged during the past 10 years, supporting the design hypothesis that beach restructuring would occur soon after the capital recharge. The northern parts of the recharge area have undergone rapid erosion. This evidence suggests that material is moving both to the south-west and the north-east, and suggests a drift divide at the site over this period. The recent planform of the beach adjacent to the terminal groyne is shown in Figure C.9.10; this has remained relatively stable over a period of several years, with occasional fluctuations. The typical plan form shown was developed at the site within a period of less than one year.

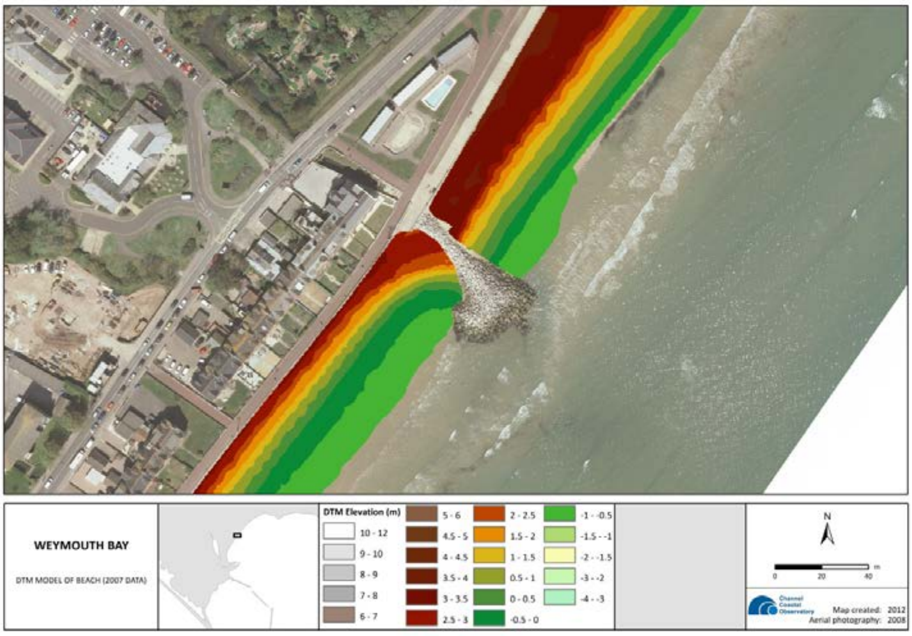

Figure C.9.10 Planform development of beach adjacent to terminal groyne 12 years after construction

Long-term beach evolution

Initial losses were rapid (20,000 m^3) during the first 12 months. The net volume of the recharged beach volume had declined by about 40,000 m^3 over the period 1995-2003; this does not include for any recycling which would make the situation considerably worse. Average net losses of about 2,400 m^3per year have occurred from 1996-2003. A decline in the loss rate has occurred since 2003, since when the beach volume has stabilised. Subsequently, changes to net beach volume within the recharged beach have been maintained at the same volume with the aid of recycling.

Longshore transport

Minimal material arrives at the site in longshore transport from the south-west. The headland groyne acts as a terminal structure, capturing drift from both south-west and north-east. Minimal changes to beach volume have occurred adjacent to the terminal groyne structure since 1996. No recycling has taken place in this zone.

Beach profile evidence indicates little long-term change adjacent to the groyne, although there is evidence of some limited net transport towards the south-west between 1995 and 2003 during which a build-up of approximately 7,000 m^3 occurred within the zone 300 m to the north-east and adjacent to the terminal groyne. This equates to an average accumulation of about 850 m^3per year over an eight-year period. It is entirely possible that the changes in this zone have occurred more quickly, but there are no data to support this. This suggests that the net transport to the south-west has been significantly lower than the projected 2,000 m^3per year. The observations of beach performance at the southern end of the beach are consistent with the modelled projections, which suggest virtually no change.

It is relatively straightforward to determine a coarse approximation of longshore rates by assessment of losses from the beach recharge zone, which is (according to the beach monitoring data) north-east of the headland structure. The overall recharged beach volume has remained roughly constant at 175,000 m^3 since 2006.

Beach performance has only been monitored outside of the recharge zone since 2006. Six years' data are now available that includes the Furzy Cliff section of beach. This coincides with the collection of sediment recycling logs. The graphs show minimal net change in volume over a period of six years.

Assuming that this is a closed sediment cell, the net drift can be calculated simply by assessing the annual recycled volumes, which result in no net change in total beach volume. This indicates a net drift towards the northeast of approximately 5,000–9,400 m^{3}per year (based on recycling logs). Comparison of the recharge and downdrift zones (Figure C.9.11) indicates an approximate symmetry, which balances the total beach volume. The total beach volume shows some fluctuation over this period, which reflects ephemeral movement of beach volume beneath MLWS under storm conditions (Figure C9.12), but there has been a net loss of only 2,500 m^3 over a period of six years. Regrettably recycling logs are not available for all years, although anecdotal evidence suggests a similar volume is recycled each year, and has been since the scheme construction. There is clear evidence also of significant erosion of the zone south-west of Overcombe during this period, since this is the zone that has received recycled material, yet which has remained stable in volume. The profile data present a varying picture of change depending on whether the surveys were conducted prior to or following recycling operations. The fact that there do not appear to be losses from the southern section of the beach and that small accumulations have occurred here suggest that the modelling reproduces the drift rates and directions appropriately in this area. It is suggested that the drift rate is variable along the length of the beach and that the modelling might benefit from additional wave prediction points as model input.

There has been an approximate mass balance of the recharged beach with the Furzy Cliff section since 2006. It is not possible to determine a mass balance with the Furzy Cliff section before this, however, since monitoring of this zone only began in 2006.

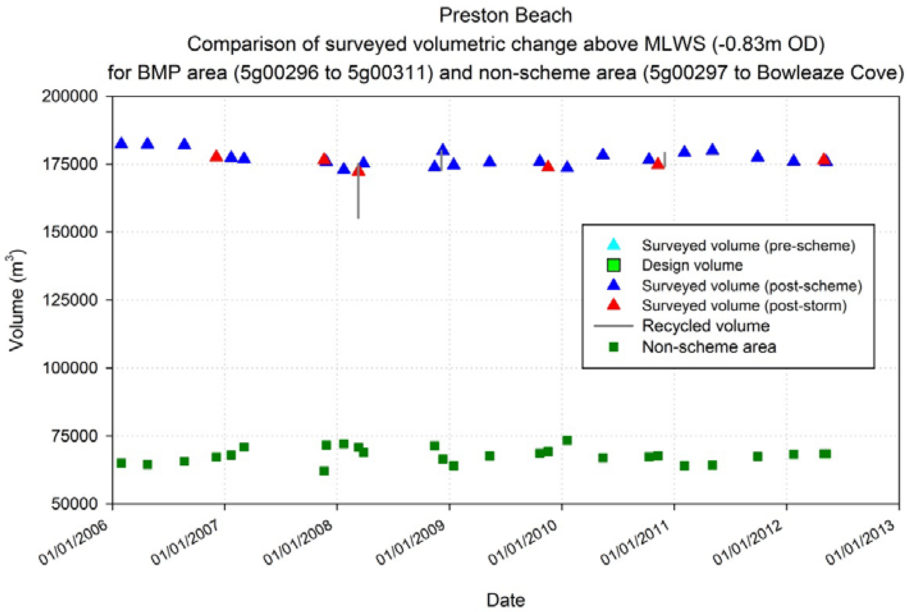

Figure C.9.11 Comparison of beach volumes within recharged zone and adjacent recycling source zone to north-east

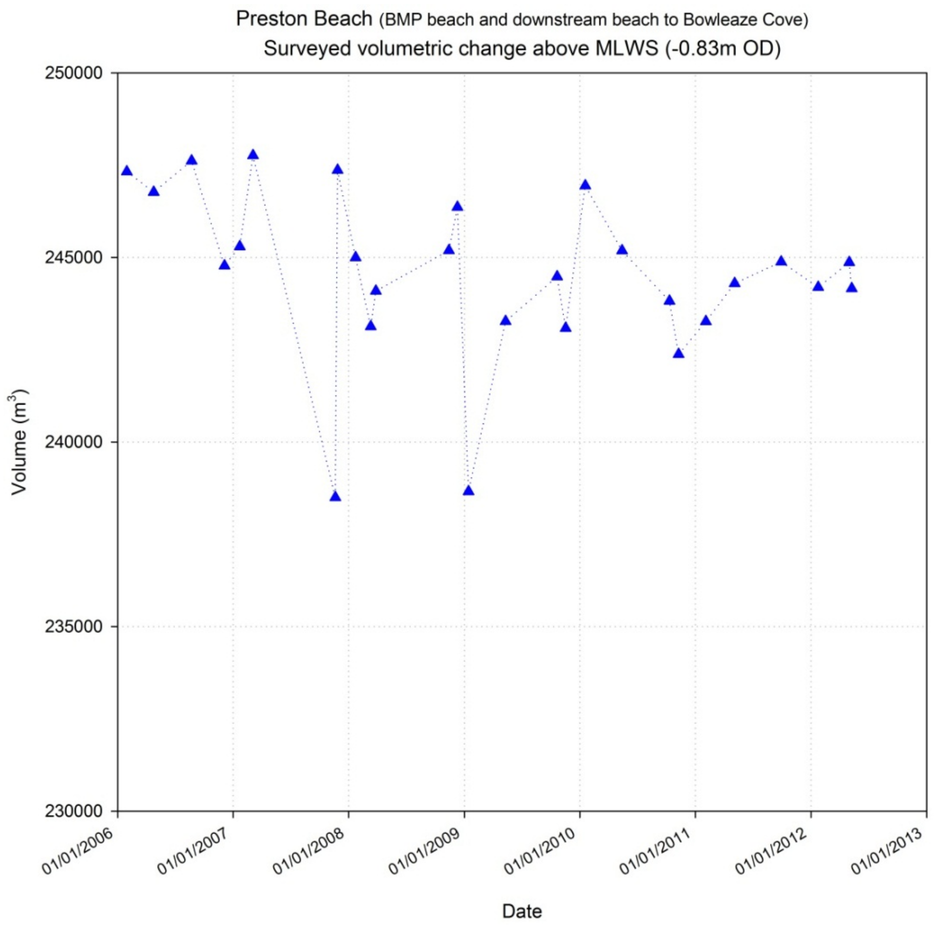

Figure C.9.12 Total beach volumes fluctuations within recharged zone and adjacent recycling source zone to north-east

Allowance was made within the design for a maintenance programme consisting of a combination of crest trimming, recycling and re-profiling on average once every 5–7 years following recharge. Allowance was made at the design stage for a total of 10,000 m^3 of recycling once every five years from adjacent to the terminal groyne where material was expected to build up. An allowance was made in the benefit–cost assessment, which included an average annual expenditure of £10,000 for recycling.

Considerably more regular recycling has been required and this has generally been from different areas of accumulation to those expected, to the north-east of the recharge site at Furzy Cliff (Figure C.9.13). It is estimated that the total recycled volume has been 2.5–5 times greater than originally anticipated. No recycling has been taken from the terminal groyne due to the primarily net opposite direction of drift.

Records of actual maintenance are not available for the whole period since construction. It has been reported that recycling has been undertaken each year since construction and that the volume of material recycled has been 'similar' each year. Recycling logs have now been introduced and these data provide some very valuable information over the past few years (Table C.9.1).

Table C.9.1 Recycling volumes

Date	Volume (m^3)
2008-2009	7,200
2009-2010	9,380
2010-2011	5,200

Figure C.9.13 Location of maintenance activities

Modifications to crest elevations have been made within the maintenance programme to reflect the monitoring results and profile response. It has not possible to restore the 1995-1996 design crest width 25 m at Overcombe since it is quickly eroded.

Experience suggests that a width of 15 m in this area is sustainable, though only with significant maintenance. Based on the latest beach management plan review, which has included review of the latest beach profile monitoring data and discussion with the Environment Agency's Operations Team, it has been determined that the trigger levels for Preston Beach should modified be as follows:

- Action level = crest width along any part of the beach falls below 13 m
- Emergency level = crest width along any part of the beach falls below 10 m

The action crest width level has been reduced from the original design level of 15 m to 13 m in this latest revision, as 15 m can only be achieved with significant intervention which is unsustainable. This move from a 15 m to a 13 m crest width action level has implications for flood warning procedures as it reduces the standard of protection provided for public safety, although structural safety limits are maintained at required levels. Both of these crest width trigger levels assume the crest level is maintained at +3.5 m OD. The promenade rests on top of the beach and the base level of this is at a level of +3.3 m OD. Should the beach level fall below the level of the base of the promenade, the underlying single layer of rock armour and geotextile would be exposed to direct wave action resulting in a significant risk of undermining of the promenade and progressive failure of the defence. Given this, two further action and emergency levels are defined:

- Action level = crest level along any part of the beach falls below +3.5 m OD
- Emergency level = crest level along any part of the beach falls below +3.3 m OD

Threshold levels have been maintained in accordance with the original design criteria, although it would appear that the standard of service of the beach is significantly lower than the original design conditions would suggest. It has not been possible to hold the desired plan shape and hence beach width, as designed.

The 'design' slope of 1 in 7 is often altered by storms, with mini cliffs being created in the shingle. These cliffs undermine the beaches ability to offset the impacts of storm events. In January 2001, the beach maintenance operations involved screening a 360 m length of Preston Beach, near to the Overcombe end of the beach, to remove finer grained material less than 5mm in diameter (Figure C.9.13). The purpose of this was to improve the porosity and so performance of the beach. While there is no confirmed data related to the sieving exercise undertaken in 2001, independent observations suggest removal of some 18,000 tonnes of beach material and a cost of £140,000. The volumetric surveys indicate minimal change in volume over this period, which suggests that the material removed was generally from within the interstices of the gravel and that the sieving operation has improved the porosity and permeability of the beach, as expected. The Environment Agency believes that this trial has been successful and anecdotal evidence suggests that, since this work was undertaken, the section of beach that was regraded has performed much better.

Figure C.9.14 Location of wave prediction and measurement points

Design phase extreme wave conditions were determined for events with a range of return periods in deep water. The wave climate was transformed to suitable nearshore locations in about 7 m water depth at MHWS (points 1 and 2 – see Figure C.9.14). These are compared in Table C.9.2 with statistics for the wave rider buoy site and Met Office transformed data from 1988-2011, derived after construction. Note that the wave buoy is in significantly deeper water.

Table C.9.2 Extreme wave conditions

	Return period Hs (m) 1:1 year	Return period Hs (m) 1:10 year	Return period Hs (m) 1:100 year
Point 1 (5 m CD)	2.53	3.25	3.96*
Point 2 (5 m CD)	2.86	3.78	4.70*
SW 15	2.07	2.39	2.68
Wave rider (10 m CD) 5 year deployment	2.68	3.03	Not determined

Note: * Breaking wave conditions may limit wave heights.

Pre- and post-construction probability distributions, based on transformed Met Office model data, show considerable inter-annual variability of measured wave conditions at point SW15 from 1999 to 2008 (Figure C.9.15). Note that these conditions were derived following design but relate to some of the design period data, which indicate that these wave conditions were generally less severe than have been typical over the past 20 years.

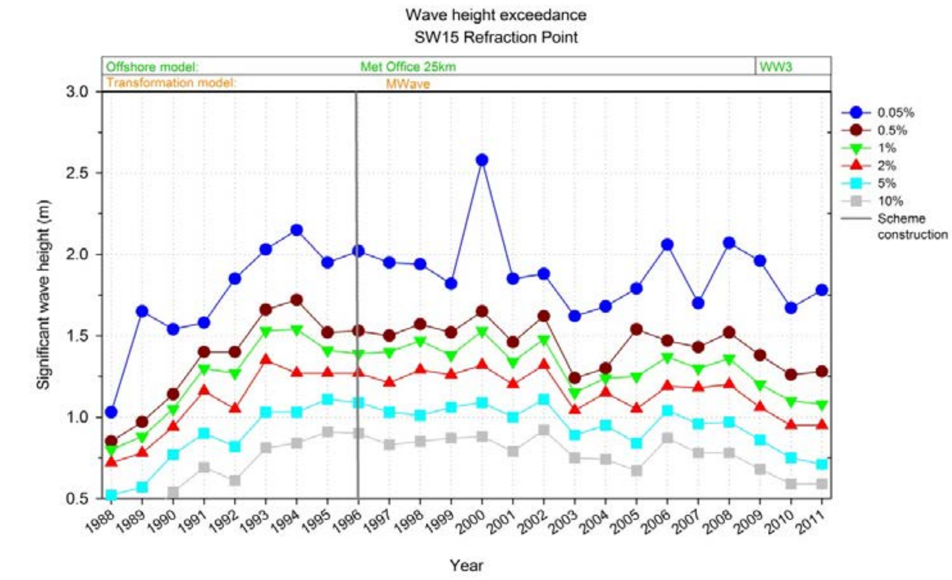

Figure C.9.15 Modelled wave climate timeline for pre- and post-construction

The percentage scatter distributions of significant wave height and direction pre- and post-scheme (Figure C.9.16) indicate very similar distributions of both direction and wave height over both periods. A very high proportion of the wave energy approaches the shoreline at an angle greater than the beach azimuth of 135°, suggesting that sediment transport should be predominantly towards the northeast. It is noted that all conditions with Hs >1 m occur at an angle >135°.

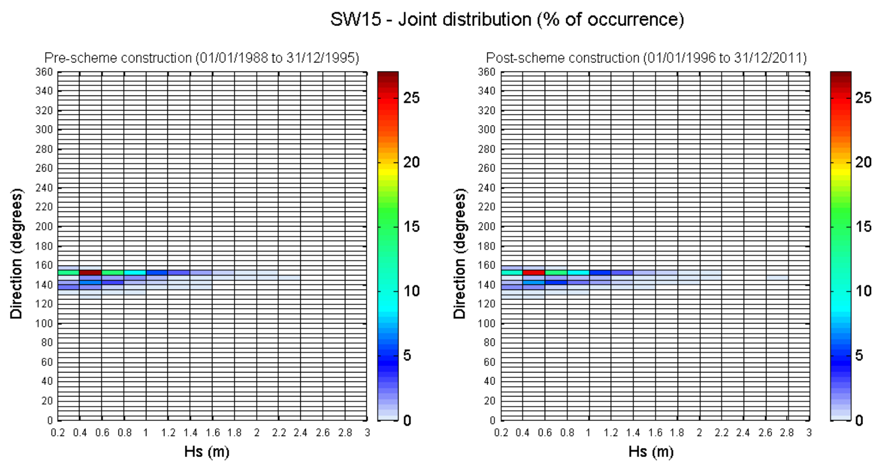

Figure C.9.16 Distributions of significant wave height and direction pre- and post-scheme

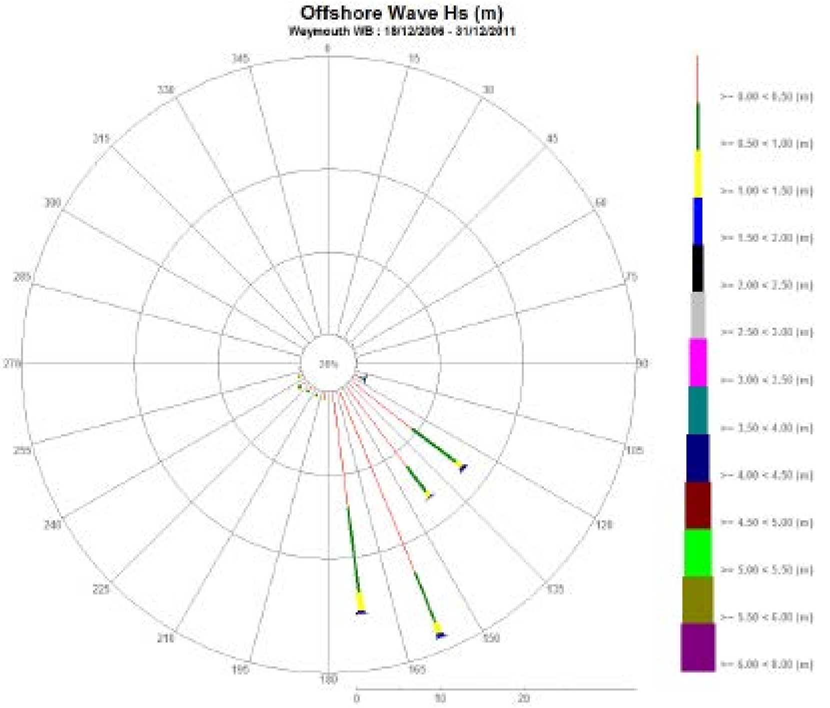

Figure C.9.17 Directional distribution of measured wave data 2006-2011

Measured wave data similarly indicate that a high proportion approaches the shoreline from a greater angle than the beach azimuth of 135° (Figure C.9.17). Regrettably the data does not extend back earlier than 2006. The distribution is shown in scatter form in Figure C.9.18, which resolves direction more finely (10° bins). This also demonstrates that some moderately energetic conditions occur also for directions less than 135°, but that a great proportion of the energy lies within the >135° sector. The location of the buoy is in deeper water than the design shallow water sites.

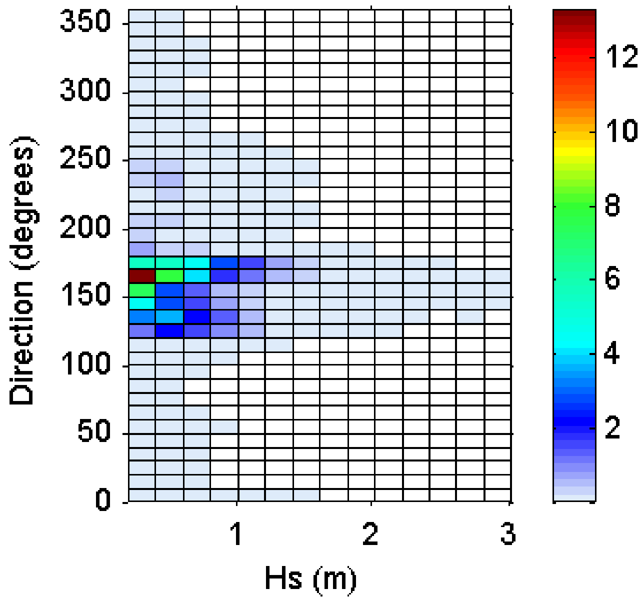

Figure C.9.18 Directional distribution of measured wave data 2006-2011

Modelled design stage and post-construction data suggest that the intensity of pre- and post-construction storm conditions have been comparable (Figure C.9.19). One storm event in 2001 stands out as being more severe than any other event. Note that the comparison is made at a location in similar water depths to, but not at, the wave prediction sites used in the design. Measured data at the deeper water buoy site are also added for comparison, where conditions are significantly more energetic.

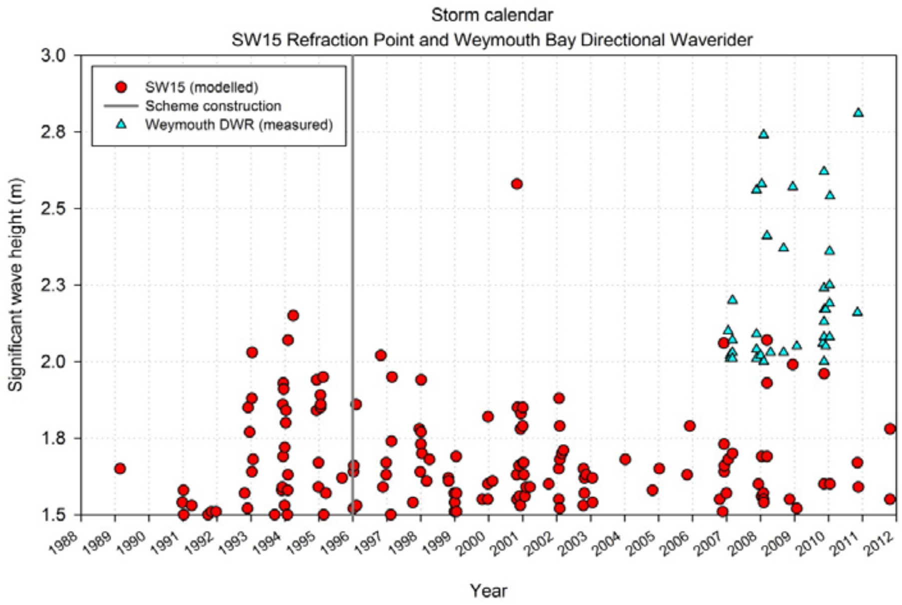

Figure C.9.19 Hindcast design, pre- and post-construction storms (5 m CD) above 1.5 m threshold and measured post-construction storms (10 m CD)

Measured exceedance probabilities indicate a very constant year-to-year distribution of significant wave heights since 2007 (Figure C.9.20).

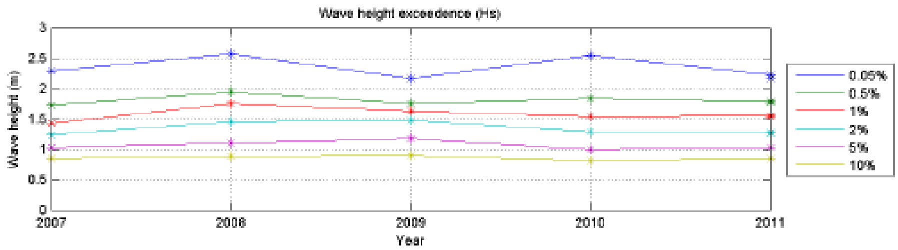

Figure C.9.20 Measured wave climate timeline since 2007

Modelled data distributions of design stage and post-construction significant wave heights are compared for 1974-1990 and 1988-2011 (Figure C.9.21). The sites are not precisely co-located but are sufficiently close to enable a reasonable comparison to be made. Comparisons show the percentage of wave heights within each height band. The plot for all data shows that hindcast post construction conditions (1988-2011) were generally more energetic than those used for design of the beach plan modelling. The stepped distribution of the design data reflects the coarsely binned representation of conditions available from the design data. The more interesting comparisons occur within the directional data. A greater proportion of the more energetic design conditions (those where H_s >0.5 m) lie within the 135–165° sector (than the 105–135° sector), in both the design data and the post-construction data. As the design beach azimuth lies at approximately 135°, conditions where waves occur at a greater angle than this will result in sediment transport to the north; this occurs for a greater proportion of the time. There is also a significant proportion of energy in the 105–135° sector, however, which might be expected to drive sediment towards the south-west. Regular drift reversals might reasonably be expected under these conditions.

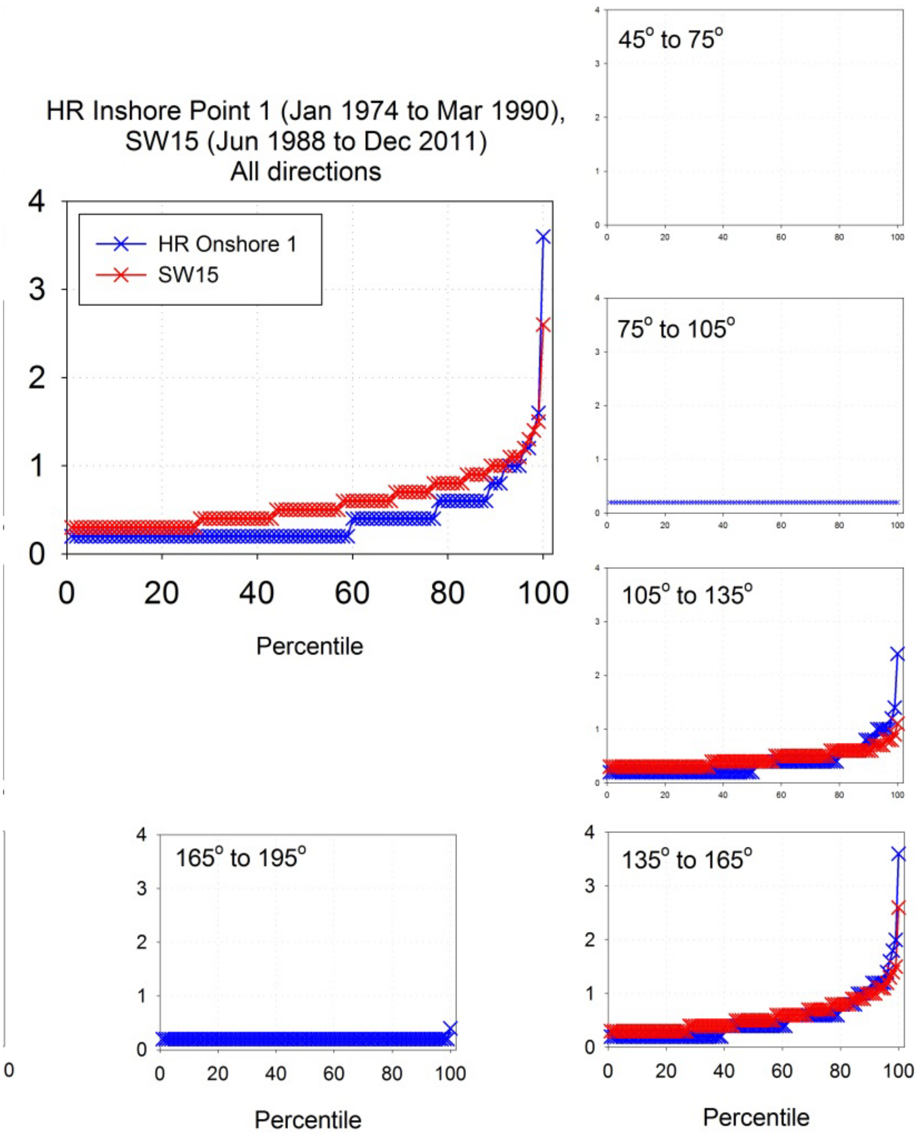

Figure C.9.21 Percentage distribution of pre- and post-construction significant wave heights

C.9.5 Comparative analysis

The original projected scheme life was 50 years, assuming net losses of about 2,000 m^3per year. Losses have been greater than this, particularly during the first eight years of service (Figure C.9.22). However, the rate of loss has reduced significantly since 2003. The slowing rate of change of beach volumes is attributed to gradual adjustments of the beach alignment, with the sediment transport rates reducing as the beach has become more closely swash aligned, approaching an equilibrium plan shape. The implication of this gradual realignment is that the sediment transport rates have generally reduced.

Under current management, the scheme life is now expected to be about 44 years, six years shorter than originally projected. If the sediment losses continue at the recent (past five year) rates, however, there is a realistic prospect of the scheme achieving its target life of 50 years.

These figures present an oversimplification of the scheme performance since beach recycling adds to the losses of material from the at risk sections of the scheme, in addition to the net losses from the system. Recycling logs, which have been completed only since 2, indicate that volumes of recycling are typically between 5,000 and 9,000 ^{3}per year. Original allowances in the design process suggested that recycling might be of the order of 2,000 m^3per year on average. Maintenance commitments are therefore 2.5–4.5 times greater than the design suggested.

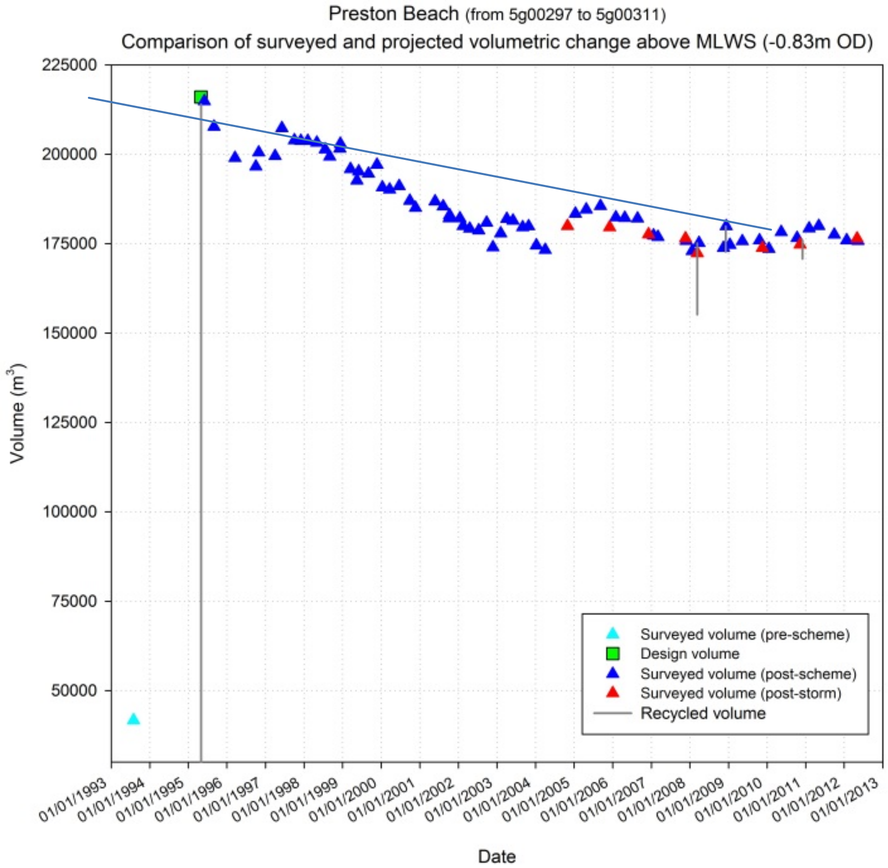

Figure C.9.22 Scheme performance timeline

Numerical modelling of wave climate suggests that wave energy is variable along the length of the beach recharge and that there should consequently be a variable rate of longshore transport and direction along the beach, as suggested by the beach plan model. The general suggestion of longshore variability of wave energy, provided by the wave models, is supported by clear evidence of such variability of longshore transport rates along the length of the beach recharge. Measured wave data since scheme construction similarly suggest that sediment should typically be driven towards the north-east, but with periodic reversals.

Observations have demonstrated some significant differences between monitored performance and predictions at the design phase. Many of the differences in performance are interlinked.

The measured net drift direction is in the opposite direction to that suggested by the design stage beach plan shape modelling. This is demonstrated by gradual accretion to the north-east of the site and loss of material from the zone at the north-east end of the recharge. This is supported further by the requirement to regularly recycle material from the area to the north-east of the recharge site. The orientation of the shoreline and wave approach angle mean that any sediment transport is most often towards the north-east. Analysis of the wave data suggests that this is a reasonable expectation. There is evidence too of small quantities of southerly drift in the zone close to the terminal groyne, which is consistent with design phase expectations.

The actual longshore transport rates (1996-2012) have on average been significantly greater than the initial predictions suggested by the modelling (estimated at ~2,900 m^3 per year net towards the south-west). The observed changes based on monitoring are estimated at about 5,000–9,000 m^3 per year towards the north-east. This might be considered a reasonable result relative to realistic modelling expectations, in a low drift situation, and where drift reversals are predicted by the model; however, these differences have presented significant management challenges requiring much greater and more frequent intervention to recycle sediment along the frontage than was expected to be the case at the design stage. Sensitivity tests conducted in numerical modelling suggested that a mean change of $\pm 2°$ in alignment might result in an annual difference in transport of about $\pm$4,000 m^3 at this site.

Early stage cross-shore performance was considered unsatisfactory following the recharge due to the low permeability arising from the high fine content within the recharge material. The crest width of 20 m tended to be denuded of material in relatively low order storm events and it has proven to be impractical to hold the design beach width. Steep cliffs also formed on the beach, under even quite modest wave conditions. Limited and very slow infiltration of waves was observed into the beach.

The monitoring has had a major impact on management of the beach system. It has demonstrated clear differences by comparison with modelled expectations and has provided the basis for modification of maintenance and long-term planning requirements. The monitoring has been particularly valuable for the purposes of evaluation of threshold damage levels and for long-term planning of interim recharge requirements. Monitoring has identified a need for a general review of the scheme standard of service.

Wave climate

- The design significant wave height has not been exceeded since scheme construction in 1996.
- A small proportion (1%) of storm events is represented by wave conditions with bimodal (period) characteristics.
- The severity of wave conditions since scheme implementation have been generally representative of those modelled at the design stage, although there is clear

evidence that the predominant direction of wave attack lies within the 135–165° sector.

- Wave conditions have been of comparable severity to those tested in design

Plan shape evolution and sediment transport

- The terminal groyne performance has not been tested rigorously as a structure due to the primarily north-easterly drift direction.
- Plan shape evolution has been broadly similar to that suggested by the beach plan shape modelling process, although the north-eastern section has cut back much further than anticipated within the modelling and has yet to stabilise.
- Sediment transport rates have been generally higher than predicted by numerical models. These may reflect the direction of moderate measured wave conditions, which are generally on the opposite side of the beach azimuth to the modelled conditions. Consequently, longshore losses from the system have been higher than design phase predictions suggest. The longshore variability of sediment transport rate has matched that anticipated at the design stage; this is evidenced by a build-up of material in the lower energy zones and evidence of drift reversals.
- Gradual changes to the planform orientation of the beach have occurred since scheme construction, which has resulted in swash alignment in some areas and a consequent reduction in sediment transport rate and sediment losses.

Cross-shore performance

- Cross-shore responses have been broadly similar to those modelled, but cut back of the beach crest has been greater than that modelled in moderate conditions, especially during the first few years following construction. This is attributed to the high fines content in the recharge material, which has enabled the beach matrix to stand up at a very steep slope.
- Cross-shore performance has improved since removal of the fines content from part of the beach. This is evidenced by better energy dissipation across the beach arising from improved permeability, and less cliffing of beach material.
- Beach slopes differ from those modelled, however, and the lower beach slopes are generally flatter than modelling of shingle with no sand fraction might suggest. This appears to be less of a problem for assessment of the upper beach, which comprises the coarser fraction of sediments and which perform more in line with the physical model.

Scheme functional performance

The capital scheme has been entirely successful in ending the regular closure (due to shingle blockage) of the A353 road and it has protected the nearby properties, the nature reserve and the municipal landfill.

The physical model was designed with material with no sand content while the beach was constructed with 40% sand content; this does appear to have had an adverse effect on scheme performance, particularly in the first five years following construction. Anecdotal evidence suggests that removal of 14,000 tonnes of the fine fraction of beach in 2001 does seem to have improved cross-shore performance of the beach, although this is regrettably not supported by monitoring data. Fewer reports of beach cliffing have been reported since this activity took place.

Sediment transport rates that are 2.5–4 times higher than expected and in the opposite direction to that predicted seem likely to be a function of the complex nearshore wave climate, which is difficult to model precisely. Very small differences in the incident wave angle may have significant effects on transport direction and magnitude. This is a particularly challenging site for the modelling of wave climate.

Beach slopes differ from those modelled primarily because the grading of beach material and the consequent permeability is quite different to that tested.

The level of intervention required to maintain the beach crest has been greater than anticipated, but the establishment of a Beach Management Plan in 2009 gives the Environment Agency the certainty that the maintenance work it undertakes is targeted and adaptable to suit changes in Weymouth Bay into the future.

The monitoring programme has provided timely and detailed assessment of performance. It has enabled a more robust assessment of rates of loss from the system and provides better opportunity for planning of maintenance and future model validation. Modifications to maintenance procedures reflect the observations made in the monitoring.

C.9.6 Lessons for future beach modelling/design

Where possible, design wave climates should include, as a minimum, several years of measured wave data to replace or complement numerical hindcasts. Some models do not make allowance for swell conditions and there is clear evidence from measured wave data that these do occur within the bay. The Met Office wave model has also been used in remodelling assessments at the site; this model enables a dataset of more than 20 years to be used in assessments. It is noted, however, that there are some significant biases in this model, which result in more energetic conditions than this that are measured. The Met Office second generation wave model was superseded in 2008 by WAVEWATCH III, which appears to reproduce wave heights more reliably on the south coast, with the bias wave height evident in the Met Office model being removed. In order to provide design conditions appropriately, long-term hindcasts will be needed based on this model. Data will then need to be transformed to suitable nearshore locations and validated against local wave measurements. This approach will improve the ability to model sediment transport more accurately, since this is strongly dependent on wave height and direction data. The plan shape topography presents modelling challenges for transformation models and any outputs should be carefully scrutinised and validated against measured data where possible; this will restrict the possibility of wave directions being incorrectly represented by the modelling.

Drift is generally considered to be at a low rate for an open coast and very small changes in wave approach angles might result in quite different results. Drift of material is generally in the opposite direction to that assumed in the scheme design, although there is some evidence of short-term drift reversals and also some evidence of southerly drift during the initial period following recharge. The drift is clearly very low at the southern end of the site and appears to be predicted correctly in this zone. The monitoring output illustrates the value of comprehensive field observations, as an integral part of the modelling process, when conducting numerical modelling at a complex site such as this. The site is now much better equipped with field data and in a better position to review and tune the output of any modelling exercise. Adequate field data are now available to calibrate any future modelling. Re-running of any models for this frontage would enable the model to be tuned using the measured field data.

Notwithstanding this observation, the calculated and measured sediment transport rates are low and the calculations lie within the expected range of outputs from such a model. It would be beneficial if the anticipated limitations of the modelling approach were highlighted at the modelling stage. Realistic expectations relating to the reliability of the outcomes of modelling need to be highlighted in the reporting. Model reporting is somewhat more matter of fact and does not reflect on the uncertainties or limitations with this approach. Some form of sensitivity assessment would be helpful to identify the range of model outputs.

The outputs from both numerical models and physical models appear entirely reasonable and within the anticipated range of expectations for such models. The model physics appears adequate to replicate the processes, but the input conditions used in the modelling appear insufficiently well defined to permit provide robust results that can be relied upon for predictive purposes. There is no doubt that the hindsight available from the monitoring data would allow the model to be re-run more effectively. Regrettably the level of certainty of the modelling is not highlighted in the reporting.

The detailed approach to scheme monitoring is summarised in a single plot in Figure C.9.18. This approach to scheme management provides a comprehensive review of scheme performance, including scheme maintenance, response of beach and predicted changes to date; it also provides confidence in future projections.

Anecdotal evidence suggests that beach sieving works undertaken in 2001 along a 360 m length of the north-eastern part of Preston Beach have improved the situation. By removing some of the finer fraction of the material placed as recharge in 1995, the beach is better able to absorb wave energy. Loss of beach material by 'cliffing' has been reduced (although it is still quite common along the frontage) and the beach has been less prone to becoming vegetated. Although regrading to remove finer sediment fractions along the remaining beach may improve the performance of the rest of the beach and have amenity benefits, the costs are not considered to be justified at this stage. While the anecdotal evidence is valuable, such an unusual operation merits more rigorous monitoring of changes in performance. This issue is at the crux of the relative performance of shingle and mixed sand and gravel beaches. Any further repetitions of such activities should be supported by a carefully designed monitoring programme. The observations do, however, appear to support the suggestion that different design tools are required to assess the performance of mixed sand and gravel beaches.

Design stage action triggers cannot always be achieved in accordance with the design. The first target should be to achieve the action trigger level with the design slope and a crest width/level, if possible. If this is not possible then it is essential that management options are explored with the aim of restoring the beach to at least the action Level. If this is not possible the beach should be re-profiled to the best possible profile, with the aim of at least protecting the seawall and promenade, by providing more than the emergency trigger level.

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C.10 Seaford

C.10.1 General information

Seaford beach management
1987
Environment Agency Southern Region

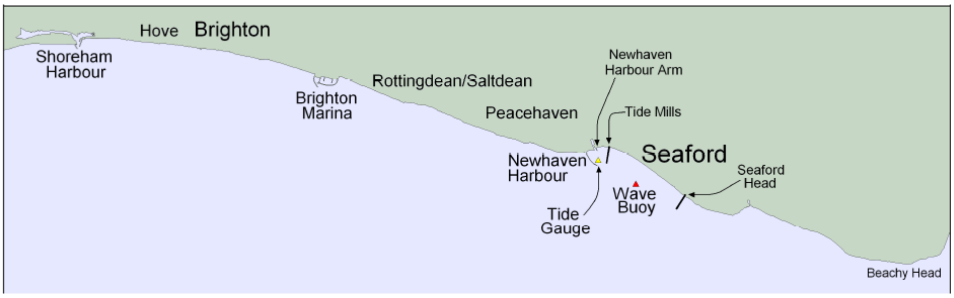

Figure C.10.1 Location of site

Seaford Beach is located between Newhaven and Cuckmere in East Sussex (Figure C.10.1) and is a popular amenity destination for a range of activities ranging from water sports to walking. The frontage comprises a shingle beach overlying a chalk wave-cut platform situated in front of a low-lying hinterland and the town of Seaford. The coastline consists of chalk cliffs, emerging beyond the terminal groynes at both extents of the shingle beach; this forms an open beach, essentially isolated from the regional sediment supply drifting from west to east by the River Ouse training walls (installed in the 17th century), the western breakwater for Newhaven Port (installed in 1880) and the more recent eastern terminal groyne at Seaford head (Figure C.10.2).

In the 60 years before and after the completion of the Newhaven Port breakwater, remedial measures were taken so that by 1912 the whole Seaford frontage was backed by a near-vertical concrete seawall and groynes constructed at 80 m intervals. In April 1981, the responsibility for the frontage passed from the Newhaven and Seaford Defence Commissioners to Southern Water Authority, and then a few years later passed to its current maintainer, the Environment Agency – then known as the National Rivers Authority (NRA).

R. Adur, Shoreham-by-Sea to Beachy Head: Sediment Transport

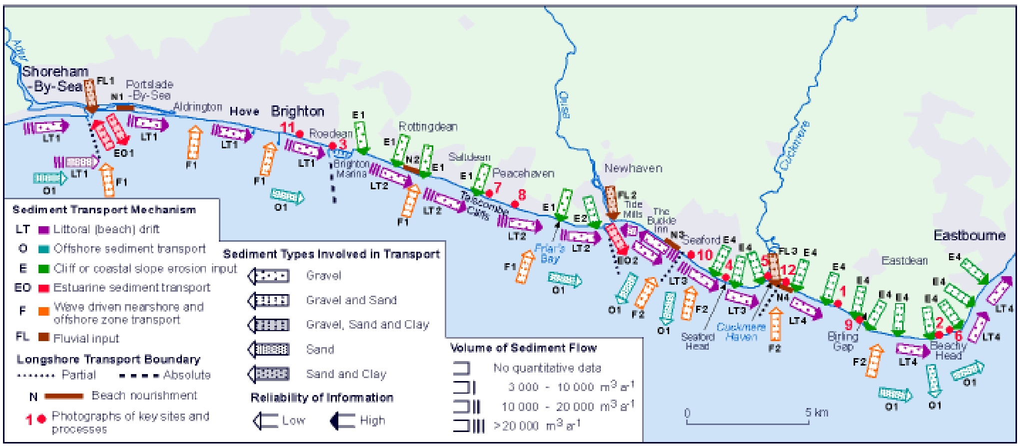

Figure C.10.2 Regional sediment transport patterns (from Bray et al. 2004)

Erosion and depletion were the dominant features, indicating that gravel was difficult to retain and tended to move rapidly alongshore, or offshore. The position of mean low water moved 107 m landwards between 1879 and 1961 (60 m between 1910 and 1945). The first groynes were introduced at Tide Mills in 1836 (Large 1981 and a seawall between the Buckle Inn and Tide Mills was built in the 1880s. Joliffe (1972) states that free gravel bypassing of Newhaven Harbour occurred up to breakwater construction in 1844. After then breakwater and pier construction bypassing appears to have been reduced, especially following the lengthening of the western breakwater, completed in 1890. Significant erosion of the central sector of Seaford Beach occurred within a few years, requiring the insertion of some 80 closely spaced groynes and the upgrading/extension of the seawall (Figure C.10.3). The new breakwater also introduced a drift reversal west of Tide Mills because of localised change in wave climate (Large 1981; Shave 1989).

Figure C.10.3 Seaford beach 1912

Although there was some net periodic accretion at either end of the beach, between the Buckle Inn and Seaford Head, for example, of 160,000 m^3, between 1898 and 1927, and 112,000 m^3, between 1927 and 1961 near East Pier (Joliffe 1972), progressive lowering and steepening was experienced up to the late 1970s. This was despite recharge with gravel, taken from Dungeness, in 1936 and 1958, the construction of a new set of alternating long and short groynes in the 1950s, and attempts at sediment recycling. The latter utilised surplus accretion against the eastern pier of Newhaven Harbour (Shave 1989). The causes of depletion were the subject of considerable debate and controversy (Hydraulics Research Station 1963). The conventional view was that the prime cause was the effect of the western breakwater of Newhaven Harbour on substantially intercepting potential longshore drift. It was becoming apparent that the seawall and groynes promoted both drawdown and scour (May 1966) while erosion of the Chalk platform providing the beach foundation was seen as one other possible cause. The latter was lowered by 3 m between 1900 and 1950 (Joliffe 1972).

Experimental studies carried out by the Hydraulics Research Station in 1961-1962 (Joliffe 1964) used physical models, fluorescent tracers and diving inspection to determine movement of gravel onshore and offshore Seaford Beach. This work demonstrated that gravel clasts had a tendency to move offshore. They did not return if moved into deep water or if they were incorporated into the silt–clay–fine sand layer occupying much of the seabed. This work added a further dimension to the debate on the causes of beach depletion, emphasising that net onshore to offshore transport may be a significant reason why coarse clastic sediment is not inherently stable on Seaford Beach.

Following a sequence of events causing damage to nearby properties, scheme development commenced in 1983. The objective of the scheme was to manage the risk of coastal flooding and erosion to assets on the low-lying land inshore which include assets valued at £28.7 million NPV (1987). In physical terms 414 houses, 13 blocks of flats a school and a sewerage pumping station were considered at risk. Figure C.10.4 shows an example from the 1950s of overtopping – one of the risks of no scheme.

Figure C.10.4 Risks arising from no scheme – overtopping in the 1950s

C.10.2 Approach to/basis of modelling/design

Overview of approach

Scheme design was based primarily on 3D mobile bed physical modelling, which was supported by numerical modelling of sediment transport (Beachplan). The objectives of the model studies were to:

- identify the standard of service of the pre-scheme system
- determine the optimal combined beach geometry and structure configuration to provide protection against overtopping and structure undermining in 1:100 year wave conditions
- determine the anticipated long-term plan shape evolution and longshore transport rates of any management solution
- compare the performance of proposed stabilisation measures with the existing situation
- examine the effects of proposed terminal structures on shingle transport
- estimate the long-term maintenance commitments required to maintain the required standard of service
- identify threshold crest levels and widths to provide alarm levels needed prior to

unacceptable overtopping and failure of the seawall

- identify a planned maintenance programme following beach recharge

A series of hydraulic model studies were carried out to test the proposed designs and to fine tune designs for maximum cost-effectiveness and hydraulic performance. Analysis of beach profile field data indicated that damage occurred most frequently in severe wave conditions associated with storm surges. A range of water levels including extreme storm surges were considered in combination with storm waves and frequently occurring conditions in various sequences. Beach responses to these processes were examined by measuring short-term changes to the beach cross-section profile and plan shape. The beach was modelled in a 3D wave basin at a scale of 1:60. Tests of the cross-section were also carried out in a wave flume to examine rock armour stability.

The test programme was broken down into the following elements:

- mathematical modelling of the offshore and nearshore wave climate, supplemented with nearshore wave measurement and wave radar measurements
- validation of the physical model methodology against pre-scheme layout
- physical modelling alternative cross-section and plan layouts of proposed scheme over a portion of the beach
- numerical modelling of sediment transport and beach plan shape evolution, interactive with the physical model

Design data were derived from long-term deployments of tide gauges at Newhaven. Water levels tested were based on a range of moderate and extreme conditions.

Initial modelling was based on a series of surveys of the beach undertaken in 1983-1984 prior to the modelling. These surveys focused on heavily eroded and healthier sections of beach. The monitoring data were available to calibrate the sediment transport in the plan shape model.

Offshore time series data were derived from a numerical hindcast model driven by Dungeness wind data for the period 1971-1979. This was combined with a deployment of a wave buoy in 10 m CD water depth from 1983-1984.

Note that the model does not include swell waves and that the modelling predates the model generation that includes swell wave modelling. Offshore wave data were transformed using wave refraction modelling to several locations along the study area in 10 m water depth. Subsequent post-scheme construction modelling was based on the Met Office 25 km hindcast model transformed inshore using a ray tracking wave model.

Design rationale

The mean beach plan shape for Seaford Bay is controlled by the two artificial headlands at Newhaven and Seaford. It was assumed at the design stage that there was no significant gain or loss of material into the bay and that the beach would seek to establish a dynamic equilibrium plan shape relative to the headlands and wave conditions. However, the seawall alignment restricted such development of a stable plan shape bay and inadequate beach volume was available in front of the seawall to allow formation of a stable plan shape. The consequence was regular wave attack at the seawall and subsequent undermining.

The varied bathymetry along the length of the site results in a differing wave climate and beach response from west to east. The reorientation of the nearshore bathymetry at the western end arising from the artificial headland structures at Newhaven Harbour results in more severe wave refraction and redirects wave energy towards the west. In contrast, the beach alignment at the eastern portion of the bay and the predominant incident wave conditions directs wave energy towards the east at this location. Wave refraction modelling was undertaken to provide input wave conditions at various locations within the bay.

Physical model

The whole site was too large to model as a single physical model section at a suitable scale, so a portion of the frontage was modelled within a 1:60 scale 3D physical model that was interactive with a numerical plan shape model of the whole bay. An extensive programme of tests was conducted. Design wave conditions, derived from synthetic wave data, provided the basis for physical model testing of the cross-shore and longshore beach response.

Modelling of beach sediment was based on pure shingle sediment with a fine-sediment cut-off at 6 mm. The moderately large model scale allowed the sediment response to waves to be reproduced with a high degree of confidence and rock armour movement to be reproduced and monitored accurately. Changes in alignment of the beach and effects of sediment control structures, such as a terminal groyne, were also examined. Drift calculations derived from a beach plan shape model were used to calibrate the physical model sediment transport rates. A range of beach geometries were considered allowing for a range of levels of investment and allowable levels of overtopping. Testing typically examined the cross-shore response of alternative beach configurations under a range of extreme conditions.

Cross-shore response predicted by physical models

The primary purpose of the model was to determine the appropriate cross-section of the recharge to:

- avoid overtopping in all but the most extreme conditions
- optimise beach volume
- identify critical conditions that could be used as a guide to inform the need for intervention during long-term management

A range of cross-sections were tested, with crests varying in width from 5 to 40 m; these were tested together with a range of alternative sediment control structures with the intention of providing a section that would avoid overtopping during the design conditions. The crest level of the recharged beach must be suitably high and wide to allow the dynamic profile to develop fully within it under design conditions. This beach configuration must be maintained throughout the course of the scheme under conditions where sediment transport rates are high.

Rock armour stability

Flume tests were conducted of a rock armoured toe designed to act as a backstop and to prevent any future undermining of the wall in the event of large-scale loss of beach material. The structure was designed to be stable under design conditions in the absence of a beach.

Beach evolution and longshore transport

The physical modelling was restricted to modelling of selected parts of the frontage length. In addition, the model only permitted a small range of conditions to be tested, which would not be representative of long-term patterns.

A one-line beach plan shape model was used to assess the morphodynamic evolution of the beach plan shape. The model was based on recent beach surveys and some data were available to calibrate the sediment transport rates and sediment size distribution. The eastern portion of the beach is orientated broadly from north–west to south–east with a typical shore normal angle of 218°N. Waves arriving at the frontage within direction sectors >218° should drive sediment towards the south-east, while those arriving from direction sectors <218° should drive material to the east. The key variables in the modelling process relate to wave conditions and the transformation of these conditions to the shoreline, beach orientation and grain size.

Longshore transport was examined in a beach mathematical model. It was tested using time series of conditions based on wave transformations to determine rates of longshore transport and potential longshore losses. The modelling technique is based on a derivative of the CERC formula which is widely used in sediment transport calculations. It is noted that a calibration coefficient for gravel of $K_1 = 0.02$ was used in the design phase modelling. At this stage, only limited plan shape modelling had been conducted of shingle beaches. The modelling is most effective when the beach contours are nearly straight, which is certainly not the case at the western end of the site.

An examination of the original beach plan shape suggested mean easterly drift of material, at the eastern end of the bay near Splash Point; a mean westerly drift direction occurs near Newhaven Harbour. The model was eventually restricted to examination of the frontage between the Buckle and Splash Point due to modelling complexities at the western end. The net longshore drift at the sailing club was westward, though at a low rate; such movements are lost to the zone towards Newhaven. An alternative considered was to introduce a groyne to prevent loss of material to the west, but this was never implemented. Modelling produced a range of outputs to indicate rates and directions of drift for each beach configuration.

Grain sizes for physical modelling and numerical sediment transport modelling were based on grading curves derived from a series of sediment samples captured across the beach profile at the surface and along the beach frontage (Figure C.10.5). The sediment characteristics are summarised below for each stage of the design and construction process.

- Natural sediment: shingle with sand mixed beach, D_{50} ~12–16 mm but varies widely
- Grading envelope used in design
- Modelled sediment type (size): D_{50} = 14 mm (scaled, cut-off at 6 mm in physical model)
- Sediment in as built scheme: shingle with sand mixed beach; grading envelope based on indigenous material (D_{50} = 14 mm)

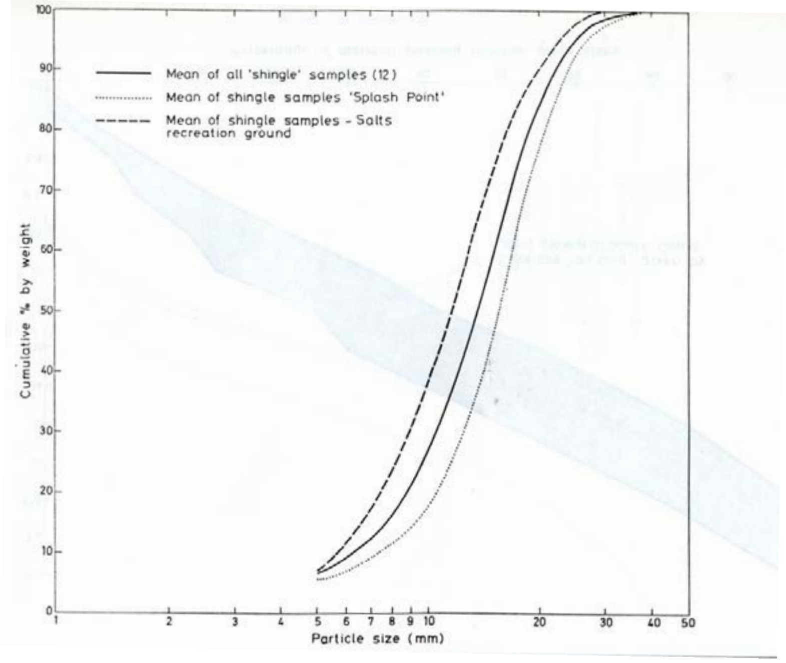

Figure C.10.5 Indigenous sediment gradings

A rock groyne, acting as a terminal structure, was designed at Splash Point to capture material anticipated to be driven from the west. The structure is armoured with 3–6 tonne rock at varying slope angles, with a crest at 2.5 m ODN.

Several alternative configurations were considered including an open beach, groyned beach and shore parallel offshore breakwaters.

The modelling work concluded with a recommendation to renourish with 1,450,000 m^3 of gravel, over a frontage length of 2,500 m, together with a large terminal groyne to contain eastwards littoral drift. This work would involve:

- recharge of the barrier beach with 1.45 million m^3 suitably graded shingle, based on sediment supply from an offshore dredging area
- construction of a rock revetment against 1,000m of the seawall
- construction of a single terminal groyne
- plans to conduct a regular beach recycling programme

Cross-shore response

Hydraulic model tests identified that a beach with a 25 m wide beach crest at a level of 6 m ODN and with an initial seaward slope of 1:7.5 would not be exceeded by green water under any of the combinations of waves and water levels tested, although shingle might be expected on the promenade in the most extreme conditions. The required crest width was 25 m along the length of the frontage. Design profiles (Figure C.10.6) were based on beach recharge with sediment of similar grading to the indigenous beach material. Model tests also identified threshold geometry conditions for each profile beyond which the beach would be vulnerable under the design storm; these suggested that a minimum beach crest width of 15 m would be required to achieve this.

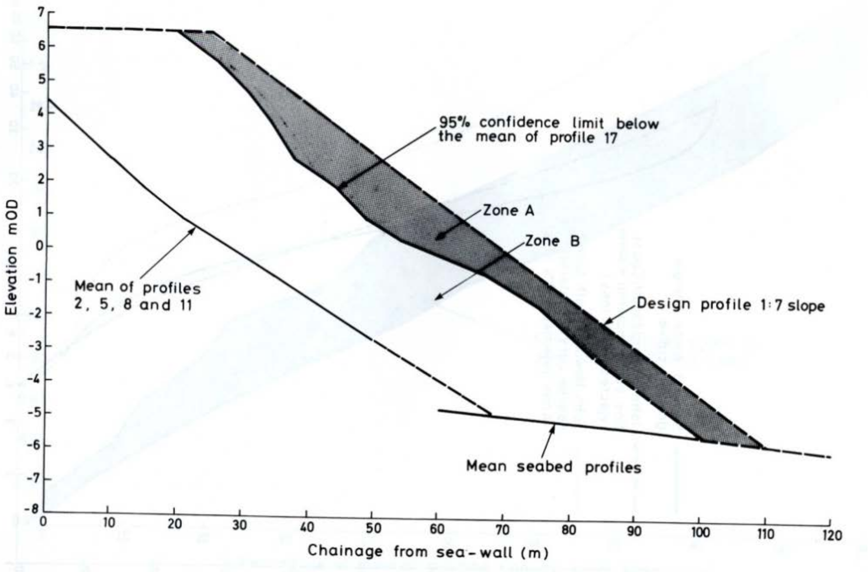

Figure C.10.6 Schematic of the 1987 design constructed at Seaford

Longshore transport

Longshore transport tests suggested that the transport rates would be higher with the recharge solution than the existing beach. Modelling also noted that use of a finer or more widely graded recharge material than that tested might increase the sediment transport rate or affect the cross-shore profile. Net average sediment transport rates of 28,000 m^{3}per year were estimated during a typical year, but these might possibly rise to 46,000 m^3. With these high drift rates it was clear that some parts of the central section of the beach would become denuded fairly quickly and periodic recycling of shingle would be needed from either end of the site.

A responsive recycling policy was suggested, with movement of material being governed by actual rates of drift and erosion. The policy of recycling would enable a satisfactory beach to be maintained, except in very severe storms, when a few hundred metres of wall near the buckle might be temporarily exposed at high water.

Structure design

Should the beach levels fall in front of the promenade, a rock revetment would be exposed. The revetment consisted of a two-layer construction of rock armour designed to provide protection to the toe of the promenade in advance of the beach being recovered by recycling and re-profiling. The rock revetment is designed to withstand direct wave action and, if it is exposed, there is minimal risk of the rock armour being displaced and the promenade being undermined

It was recognised at the design stage that material generally available from offshore dredging sources would contain a significant proportion of fine material. It was anticipated that a good proportion of this fine content would be lost following placement. The expectation was that the fill material would be sorted by wave action until the grading approximates to the indigenous material. A pragmatic assumption was made that a significant proportion of the fines content would be winnowed out with time. Based on available sources, it was suggested that the eventual volume of the recharge might be reduced by about 40% following winnowing of material beneath a size of 4 mm. The design or grading was not modified to reflect the anticipated high fines content. The following comments were offered on the impacts of finer wider graded material to that tested.

- The beach will form a dynamic equilibrium slope at a shallower angle for either finer or more widely graded materials than for the indigenous beach grading. This would require a larger quantity of material to form the capital recharge.

- The longshore sediment transport rate would be faster for finer material than for coarse material. Losses from the system would therefore be greater. This would result in a requirement for more frequent and higher volumes of maintenance to be included in the beach management plan.
- The use of a finer grading or a more widely graded material would reduce the permeability of the beach and reduce the effectiveness of cross-shore performance.
- More widely graded materials would contain a higher proportion of fines, which are likely to be lost from the system at an early stage.

These assumptions were not tested in either physical or numerical models.

C.10.3 Design/modelling outputs – plans for implementation

The scheme comprised 1.45 million m^3 of shingle renourishment between the Buckle and Splash Point, a new, enlarged terminal groyne at Splash Point and the construction of a new rock toe to the 1,000 m stretch of seawall between the Buckle and Salts recreation ground The scheme design life was 50 years (to 2037) and provides a standard of protection with an annual probability of occurrence (APO) of 1% (1 in 100 year return period) against overtopping. This standard of protection is provided by the beach, seawall and promenade and is maintained provided that a suitable beach profile is retained along the frontage.

Recycling of shingle from the Splash Point terminal groyne and the western beaches was anticipated at about 28,000 m^3per year on average. The beach crest level is to be maintained at 6 m ODN at the Buckle, rising to 7 m ODN at Splash Point. The idealised beach section has a crest width of 25 m and a 1:7 seaward slope. Intervention will be required at a trigger level of 15 m crest width.

A preliminary beach management plan was developed at the design stage and is outlined below.

Outline 50-year programme

On completion of the capital recharge scheme, the beach was expected to withstand the design storm conditions without risk of overtopping or structure undermining under design conditions. The recharge was a dynamic solution that would modify rapidly over time due to both cross-shore and longshore transport processes; it will require maintenance throughout its life. The integration of a rock revetment beneath the beach recharge provided an additional safeguard against undermining in the event of large-scale losses. A large-scale commitment to recycling was anticipated from the design stage and allowance made for recycling of shingle on a regular basis.

The beach management plan relies on an understanding of performance derived from a simple monitoring programme in conjunction with predefined alarm conditions to provide a decision support system for the maintenance programme. The scheme has a design life of 50 years, during which there will be a requirement to recycle or top up the recharge. Estimates were made to facilitate development of a preliminary programme of recharge maintenance. The programme was to be revised in conjunction with the results of the planned monitoring programme at strategic (five year) intervals.

No further introduction of additional beach recharge materials was envisaged within the 50 years of the beach management plan. Planned maintenance work was limited to periodic recycling, expected to be at least once per year. The estimated recycling volumes are based on estimates derived from beach plan shape modelling.

Detailed five-year programme maintenance

A programme of planned maintenance was developed based on the results of the physical and mathematical model studies. These suggested that recycling might be required on regular basis (from the south-east end of the beach about once in five years.) An allowance for an average 28,000 m^3 per year was allowed within the scheme for this activity (Figure C.10.6).

The mathematical model examined a number of recycling strategies, which could be refined by monitoring. Projections suggested that the scheme should be adequate for a 50-year lifecycle and the maintenance programme would be also reviewed to reflect monitoring. The maintenance programme was due to be reviewed in epochs of five years.

Threshold levels

Damage threshold and alarm conditions were defined at which beach maintenance is necessary to avoid unacceptable overtopping. The alarm condition is defined as the beach condition required to achieve an acceptable level of overtopping during the design storm (alarm); this has been defined with the aid of physical model studies and relates to the 1 in 100 year (1%) APO.

- The maintenance works should aim to ensure the beach along its length has a crest level of at least +6 m ODN, with a minimum crest width of 15 m, and beach slope no steeper than 1:7.5. This crest elevation should rise to 7 m ODN at Splash Point.
- Beach recycling works are to be carried out as required throughout the year. The need for these works is to be triggered by the monitoring surveys measured against the design profile. The target profile to be maintained was set as the original design profile.

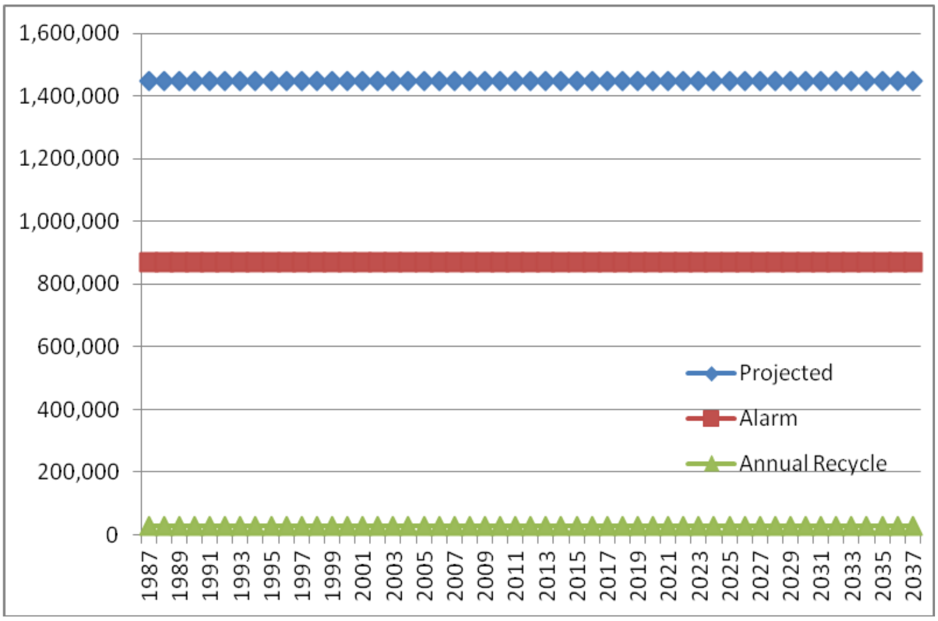

Figure C.10.7 Planned maintenance programme from 1987 to 2037

C.10.4 Beach management and performance

Construction in 1987 involved 1.5 million m^3 of mixed sand and gravel placed between the Buckle and Splash Point, the building of a new, enlarged terminal groyne at Splash Point and the construction of a new rock toe to the 1,000 m stretch of seawall between the Buckle Groyne and Salts recreation ground. Beach recharge was placed adjacent to the seawall and promenade to reduce the volume of overtopping onto the promenade, seawall and properties during storms (Figure C.10.8). The dredged material was won from the Owers Bank aggregate dredging area. The material was pumped onto the beach and then spread to a nominal crest on average 25 m wide with a height of +6.0 m ODN at the Buckle, rising to 7.0m ODN at Splash Point and a seaward slope of 1 in 7. Scheme costs were £11.3 million.

An enlarged terminal groyne was constructed at Splash Point. It was anticipated that an annual recycling programme would be required to maintain the design profiles.

A rock armoured revetment was constructed along a 1,000 m length of the most vulnerable section of the seawall, between the Buckle groyne and Salts recreation ground, and which was subsequently buried within the beach recharge material. If the beach experiences significant cut back and draw-down during a storm event, the 3–6 tonne rock armour is designed to provide protection against undermining and reduce overtopping. The structure has been designed to withstand the design conditions, which could otherwise undermine the promenade, leading to progressive failure of the defences.

Figure C.10.8 Recharged beach in 1998

The as-built scheme reflects all the geometric and volume details developed at the design stage, thereby making comparison of the performance and the design tools fairly straightforward. The geometric characteristics of the final design were based closely on the physical model. Losses of some 15% of volume (approximately 200,000 m^3) were anticipated within six months of scheme completion, mostly due to removal of fines.

The aggregate production area grading envelope (indigenous material) used as the design grading is shown in Figure C.10.9.

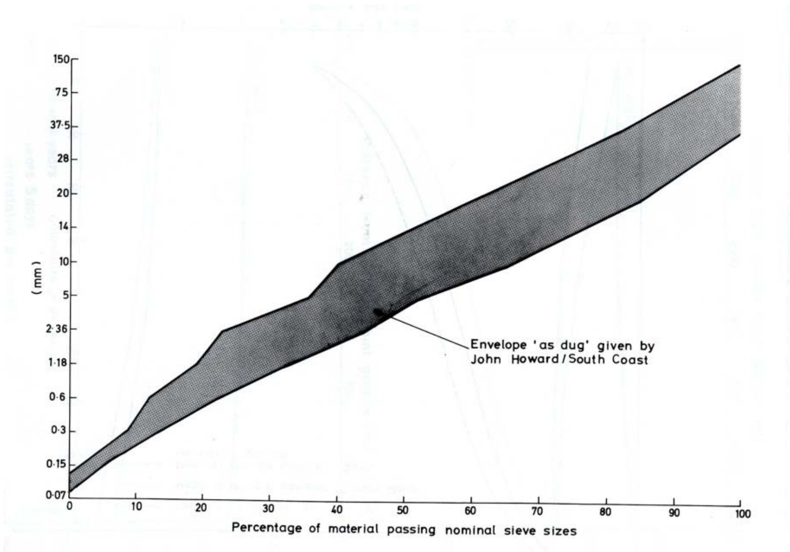

Figure C.10.9 Design and as-built grading envelopes

The main difference between the modelling and the as-built construction relates to the grain-size distributions of the modelled and the prototype recharge material. Design of the physical model sediments was based on the grading of the indigenous beach material, which also formed the basis of the recharge design.

Physical modelling of the beach was undertaken using lightweight materials (crushed anthracite) designed to simulate the hydraulic performance of shingle. The model sediment was scaled to be representative of a shingle grading with a D_{50} of 14 mm, but without the sand content and an effective cut off of material below a grain size of about 6 mm. This is a standard modelling practice, since mixed sediments cannot be modelled effectively at the selected scale. There is a reasonable expectation therefore that the profile response of the prototype and model recharges might be expected to differ, since the model effectively represents a clean shingle while the prototype represents a mixture of sand and shingle, with lower permeability. Academic studies of mixed beach performance suggest that mixed beaches perform similarly to sand beaches when the sand content reaches about 40%, although there is limited published guidance to quantify this difference. There is an expectation therefore that the prototype beach might theoretically develop a flatter slope and with a lower crest than that achieved in the model. Similarly there is a reasonable expectation that sediment transport rates might be higher than those modelled. The potential implications of the differences in grain size distribution are well documented in the modelling guidance.

Regular beach surveys have tracked progress of the project performance, since construction, initially via the Environment Agency ABMS, which has provided a broad brush assessment of changes. The monitoring programme has been developed further since 2002 with the introduction of the regional coastal monitoring programme (Figure C.10.10). Surveys are now conducted three times per year and also following storm events and maintenance. The seaward extent of surveys has been increased since 2002 in conjunction with the Southeast Regional Coastal Monitoring Programme (SRCMP), so data are not strictly comparable prior to this date.

Monitoring studies carried out after completion of the beach recharge in March 1988, using 58 transects (Hydraulics Research 1988, 1989, 1991; Brampton and Millard 1996) revealed close correspondence between theoretical expectations and actual performance. Losses were less than anticipated at first, but storms in 1989-1990 and 1992 generated strong mid/backshore scarping following the consolidation of the matrix between gravel clasts. This assisted wave reflection, created significant losses from foreshore scour.

Longshore transport

Between 1987 and 1994, monitoring revealed a spatial pattern of net losses and gains over different sectors of the beach, with gains recorded at both eastern and western ends (Brampton and Millard 1996). These were equalised through periodic recycling, with a small net gain of 5,000 m^3 for the beach as a whole over this period. Since 1995, the practice of recycling has been maintained, with quantities determined by analysis of ABMS data for preceding years. ABMS data for 1991-2000 indicate an average annual gain of 13,800 m^3 (maximum accretion,1992, of 65,000 m^3, maximum depletion, 2000, of 75,000 m^3). Management has thus sustained the equilibrium of this recharged and modelled beach. Seaford Beach has been surveyed three times a year since the summer of 2003 as part of the SRCMP using aerial surveys. This consists of biannual profile surveys (Figure C.10.10) and an annual survey of the whole beach. The summary of volumetric changes for the period 2003-2011 (Figure C.10.10) demonstrates that the beach volume has remained fairly constant over this period.

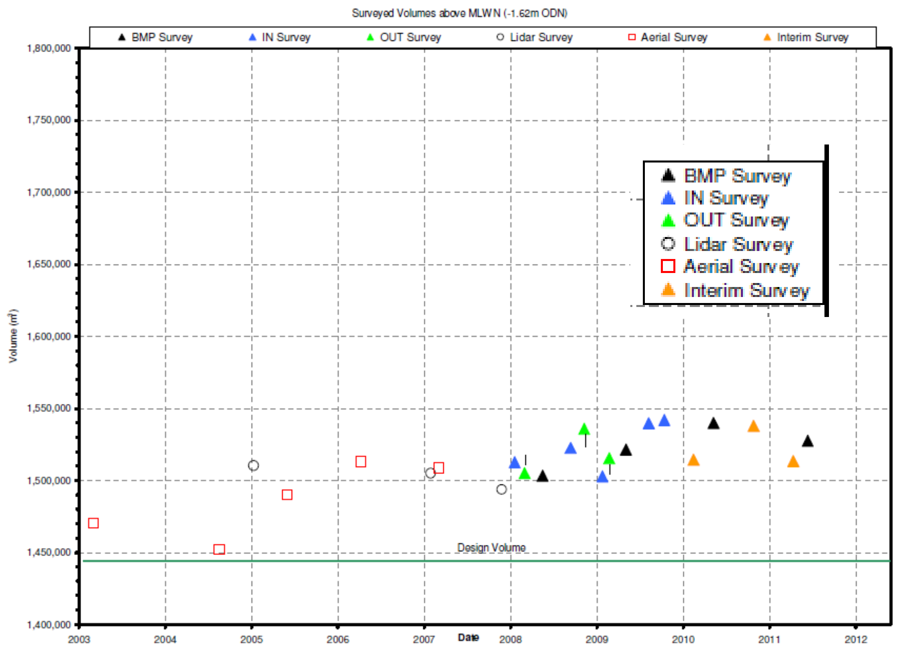

Figure C.10.10 Total beach volume fluctuations, 2003-2011

Beach performance varies along the frontage. Regular accumulations occur at the north-western and south-eastern extents of the frontage, while erosion predominates in the central area. The beach volume has been balanced since 1987 by recycling, to maintain similar volumes close to the design profiles in each zone (Figure C.10.11). Note that the beach is typically very close to the design volume in the central zone, which is controlled closely by monitoring of recycling operations.

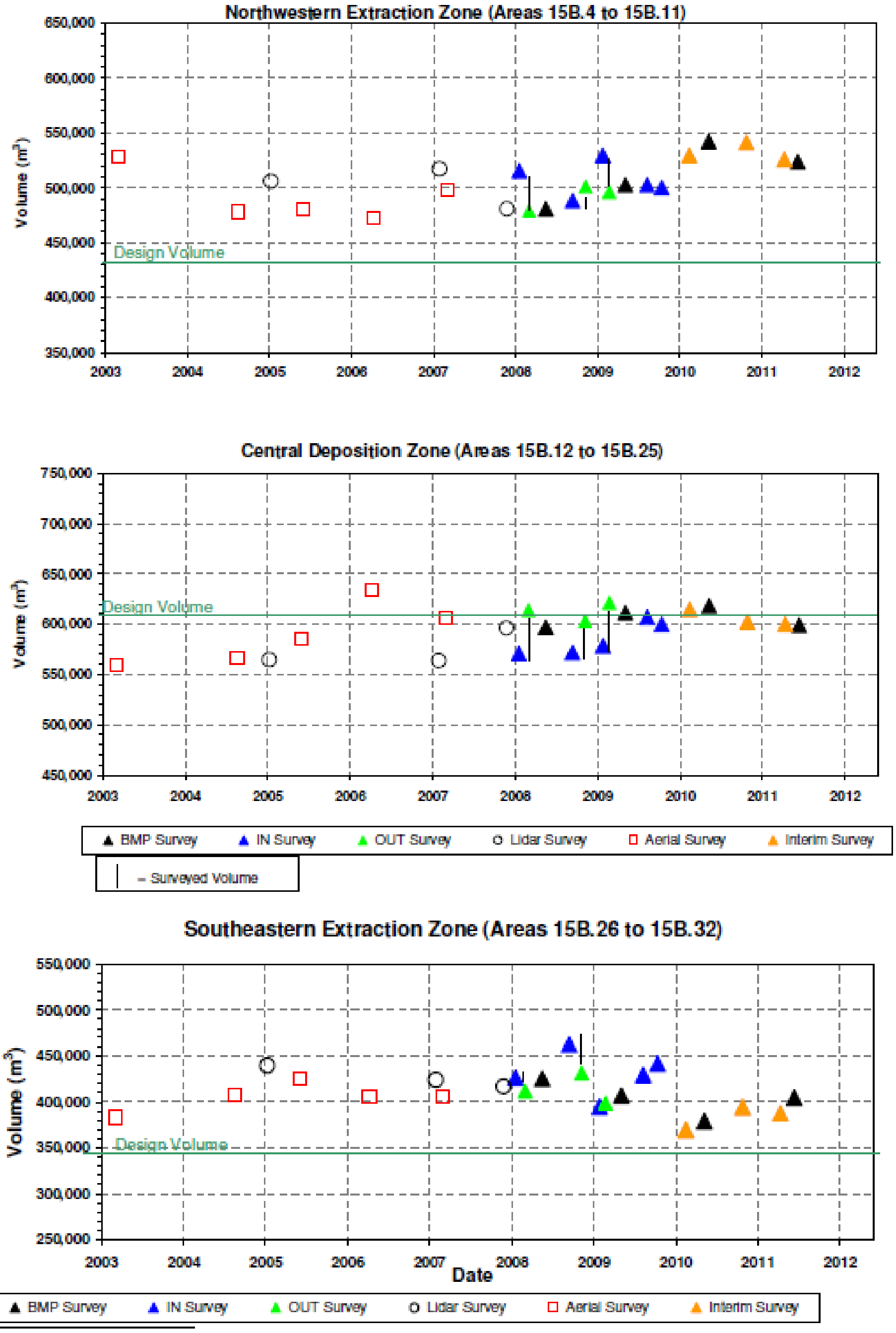

Figure C.10.11 Comparison of beach volumes within recharged zone and adjacent recycling source zones

Comparison of the recharge and downdrift zones indicates an approximate symmetry, which balances the total beach volume. It is relatively straightforward to determine a coarse approximation of longshore rates by assessing the losses from the beach recharge zone, which is provided in the recycling logs (Table C.10.1). The overall recharged beach volume has remained roughly constant since 1987. Assuming that this is a closed sediment cell, the net drift can be calculated simply by assessing the annual recycled volumes, which result in no net change in total beach volume. This indicates a net drift towards the east of approximately 50,000–120,000 m^3 per year (based on recycling logs).

Planform development

The planform developed following construction indicates that there has been regular accumulation of material to the southeast of the recharge site and also at the north-east end. Erosion is predominant in the central section of the recharge site. There is clear evidence that there has been significant net transport direction towards to both the east and the west for the whole of the period following recharge. A net build-up of sediment has been evident to the east of the beach recharge scheme on each survey following the recharge.

The beach has been unable to establish an equilibrium plan shape, since this is adjusted each year by recycling of material. The plan shape of the western zone has remained fairly constant from 2003 to 2011 (Figure C.10.12). Note the flatter foreshore seawards of the MHW contour adjacent to the breakwater; this reflects a build-up of fine material lying at a flatter slope in this less energetic area.

Figure C.10.12 Plan form development of beach adjacent to harbour training wall 25 years after scheme construction

The central 1 km of the recharge area has undergone rapid and regular erosion. Evidence suggests that material is moving primarily to the east (Figure C.10.13). The recent planform of the beach adjacent to the terminal groyne is shown in Figure C.10.14 and this has remained relatively constant over a period of several years, with occasional fluctuations, that largely reflect maintenance activities.

Figure C.10.13 Plan form development of central (eroding) beach section 25 years after scheme construction

Figure C.10.14 Plan form development of beach adjacent Splash Point terminal groyne 25 years after scheme construction

In parallel with the monitoring, further assessments were made of the plan shape modelling (Brampton and Millard 1996). This modelling, based on data from 1988-1991, indicated that to achieve the measured drift rates of about 70,000 m^3per year, the model K_1 factor for sediment size needed to be adjusted from 0.02 to 0.04. These studies also confirmed that wave conditions had subsequently been more severe than at the design phase.

Profile response

Recent storm events (since 2002) have been used to assess the profile performance following construction. Earlier surveys were not conducted at suitable times to assess the storm response. The field data indicate rapid changes to the cross-shore profiles and formation of steep scarps at the upper beach. There is evidence of steep cliffs at the beach crest along much of the frontage. There is clear evidence of regular onshore offshore exchanges of beach material, demonstrated by a series of detailed surveys that have been turned into terrain models and which show transfer of material from the upper to lower beach and vice versa. Several notable storm events have occurred during which the crest has been cut back and new crest berms formed reaching a maximum level of 7.2 m OD. A notable event occurred in October 2004 (Figure C.10.15). The beach response, which has been monitored by topographic surveys, in parallel with wave measurement since 2008 has been remarkably close to that predicted by modelling for the storm events with the characteristics.

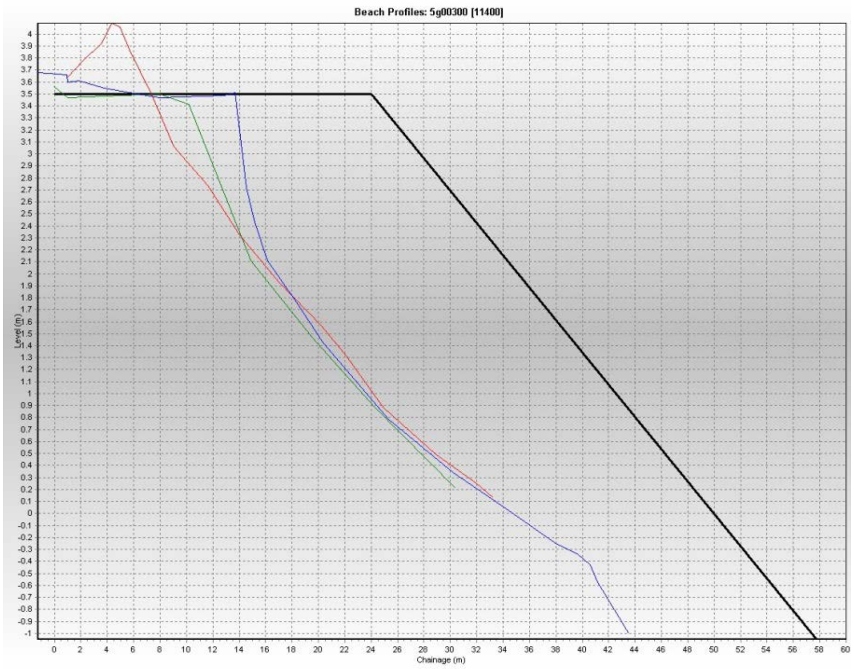

Figure C.10.15 Profile response of beach under storm conditions

Allowance was made within the design for a maintenance programme of recycling every year, following recharge (Figure C.10.16). Allowance was made at the design stage for an average total 28,000 m^3 of recycling every year from Tide Mills (at the western end of the bay) and Splash Point (to the east) where material was expected to build up; this was based on a 'typical' year. Sensitivity calculations suggested that it might be possible for drift rates to reach about 46,000 m^3 per year.

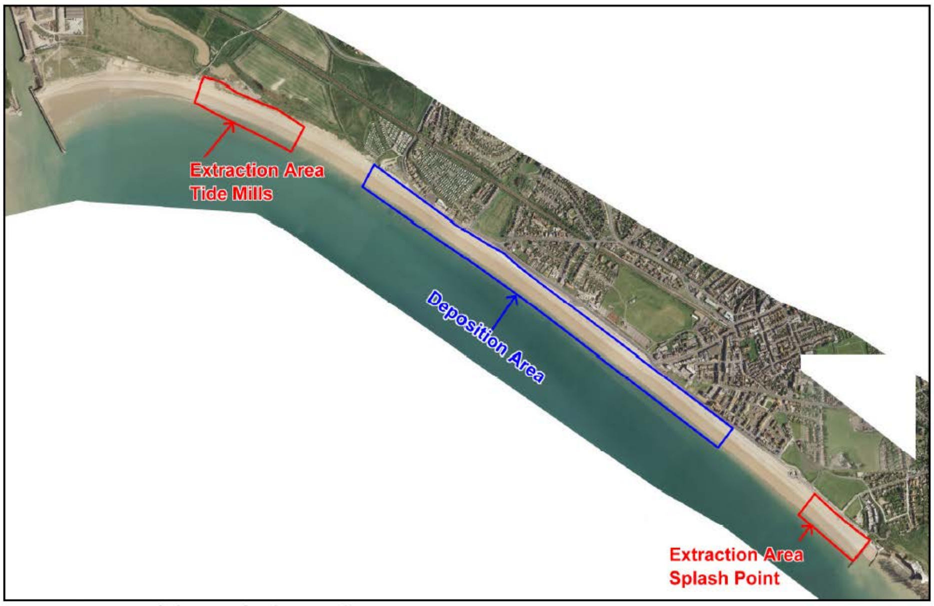

Figure C.10.16 Location of maintenance activities

The beach crest elevation has been maintained at a level of between 6.0 m OD at the Buckle, rising to 7.0 m ODN at Splash Point. This is in accordance with the original design. The idealised beach section crest width of 25 m has been maintained with a 1:7 seaward slope. The total beach volume remains above the design volume for the frontage.

Records of actual maintenance are not available for much of the period since construction. It has been reported that recycling has been undertaken each year since construction and that the volume of material recycled has been 'similar' each year. Recycling logs have been introduced together with in and out surveys and these data have provided some very valuable information since about 2006. Procedures for the recycling process are outlined in Figure C.10.17. This methodology of IN, OUT and BMP surveying allows a useful distinction to be made between the changes in beach material volumes that are due to the recycling works, and those due to natural sediment transport processes, providing an overview of the effectiveness of the management of Seaford frontage.

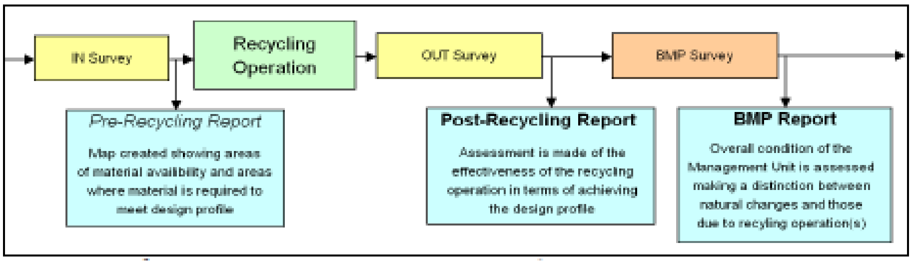

Figure C.10.17 Workflow sequence for maintenance and monitoring

Considerably more recycling has been required than original projections suggested throughout the scheme life; this has generally been from areas of accumulation at Splash Point and Tide Mills to the central area near the recreation ground (Figure C.10.16), as originally anticipated. It is estimated that the average recycled volume has typically been 3–4 times greater than the average originally suggested by the numerical modelling as shown in the partial records of recycling (Table C.10.1.)

Table C.10.1 Partial record of annual beach recycling at Seaford

Year	Deposition volume (m^3): recreation ground	Source: Splash Point (m^3)	Source: Tide Mills (m^3)
1989-1990	107,000	80,000	27,000
1990-1991	112,000	72,000	40,000
1991-1992	97,000	58,000	39,000
1992-1993	101,000	81,000	20,000
1993-1994	99,000	66,000	33,000
1994-1995	125,000	82,000	42,000
2004-2005	60,000	40,000	20,000
2006-2007	60,320	22,872	37,293
2007-2008	50,176	37,441	12,734
2008-2009	89,943	3,000	86,943
2009-2010	119.367	48,701	70.665
2010-2011	102,061	41,640	60,420

Modifications to the maintenance process have been made to reflect the monitoring results and profile response. It has generally been possible to restore the 1987 design crest width 25 m. The ‘design’ slope of 1 in 7 is often altered by storms, with mini cliffs being created in the shingle. These cliffs undermine the beaches ability to offset the impacts of storm events. Sheeting over run is often noted across the surface of the upper beach which has become impermeable, primarily as a result of the recycling activity, which causes compaction of the crest and binds recycled fine materials into a cohesive matrix.

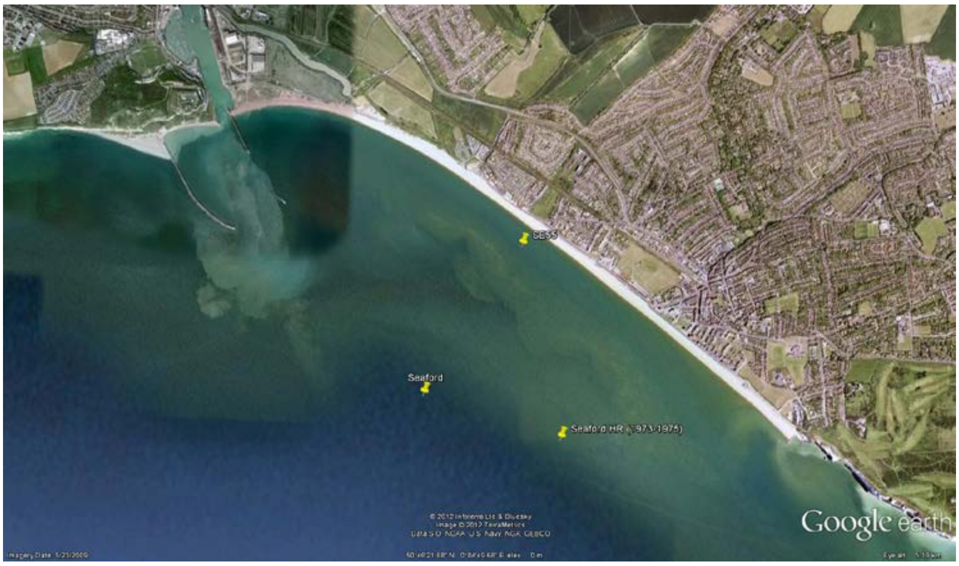

Figure C.10.18 Location of wave prediction and measurement points

Design phase extreme wave conditions were determined for events with a range of return periods in deep water (20 m). The wave climate was transformed to suitable nearshore locations in about 10 m water depth at MHWS (points A, B, C, D, E) as shown in Figure C.10.18. A wave rider buoy was located at point C from 1983-1985. Note that the wave buoy is in similar water depth to the wave transformation points. Extreme conditions were calculated for each location (Table C.10.2).

Extremes were calculated from the two datasets for design and post-construction. The highest individual value in the hindcast design data spanning eight years was 4.2 m; it is the top few values in the probability distribution that impact most on the extrapolated extremes (Table C.10.2). Data at the wave buoy indicate six events between 2008 and 2012 with H_s > 4.2 m; this site is located in shallow water and waves here should typically be no more than 90% of those at the offshore boundary, using the HR Wallingford look-up table as a basis. An offshore H_s >4.2 m has been regularly exceeded since scheme construction (444 three- hourly records exceed 4.2 m) and this is reflected by the suggested offshore extremes determined from the Met Office model. The 1:100 year event calculated at the design phase has been equalled or exceeded twice in the last 25 years.

Table C.10.2 Extreme wave conditions

	Return period H_s 1:1 year	Return period H_s 1:10 year	Return period H_s1:100 year
Offshore deep water (HR 1971-1979)	N/A	5.90	6.65
Offshore Met Office (1988-2011)	5.71	6.81	7.85
SE55 (5 m CD) (1988-2011)	3.18	3.70	4.17
WaveRider (10 m CD) (2008-2012)	4.42	5.24*	Not determined

*Depth limited at MLWS

Modelled data distributions of design stage and post construction offshore significant wave heights are compared for 1971-1979 and 1988-2011 in Figure C.10.19. The sites are not precisely co-located and the datasets were generated using different wave models, but are sufficiently close to enable a reasonable comparison to be made. Both locations are in deep water Comparisons show the percentage of wave heights within each height band and for each direction sector. The plot for all data (Figure C.10.19) shows that hindcast post construction conditions (1988-2011) were generally more energetic than those used for design of the beach plan modelling; this might reasonably explain the higher than anticipated sediment transport rates that have occurred since construction. The HR offshore data suggest no energy in the sectors to the north, but this reflects the cut-off of direction sectors used within the modelling. The Met Office data for the northern sectors seem very energetic, but the data have been generated at some distance from the shoreline. Data availability for the nearshore datasets did not allow such comparisons for pre- and post-construction conditions.

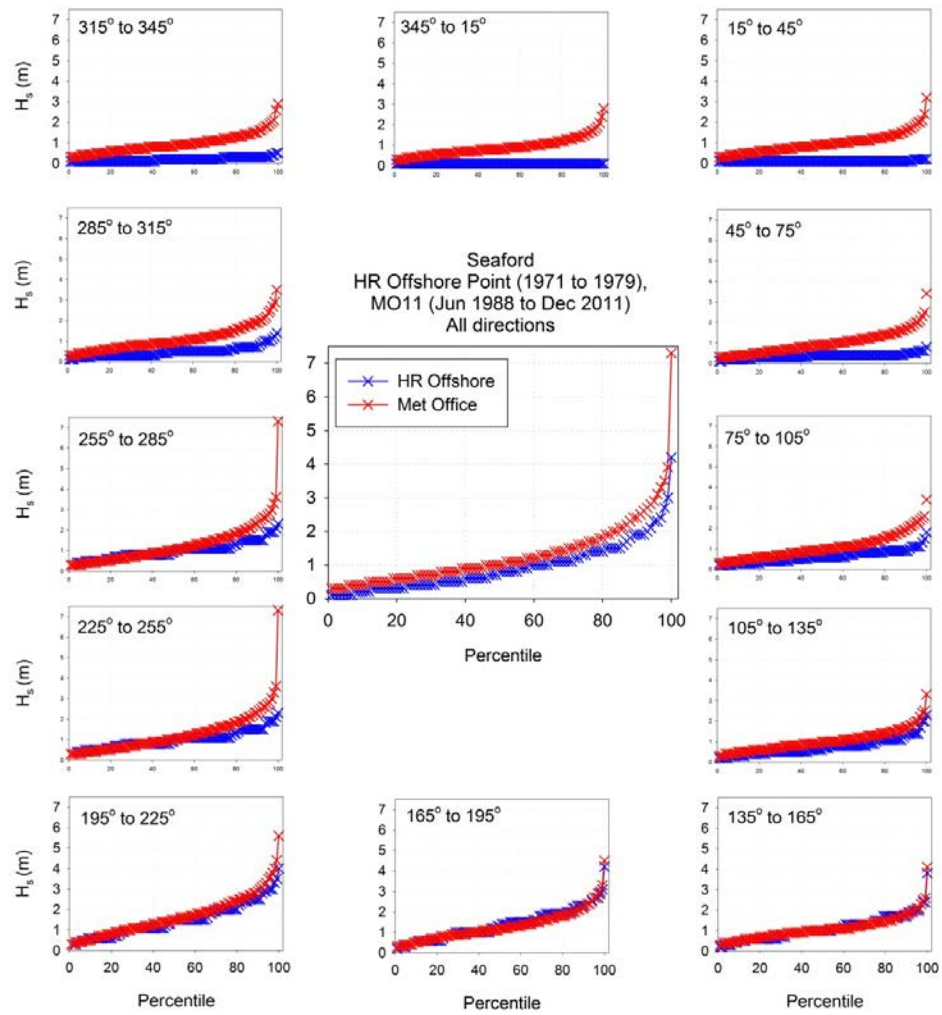

Figure C.10.19 Percentage directional distributions of pre- and post-construction offshore significant wave heights

The data are too far offshore to make specific assessments of the conditions at the shoreline. However, a significantly greater proportion of the more energetic design offshore conditions (those where H_s >0.5 m) within the 195–285° sectors occurred during the post-construction period; these are the conditions that might reasonably be expected to drive the sediment transport to the east at the shoreline. It is clear, however, that data for the south–south-east sectors are very comparable for both the design stage and following construction. The south–south-easterly sectors also indicate a significant proportion of energy, which might be expected to drive sediment towards the west, as indicated also in the beach plan modelling. The most energetic sectors are from the south-west to western sectors and each of these suggests significantly more energetic conditions have occurred on average since the scheme construction. The design conditions were generated therefore from a comparatively less energetic epoch than those following construction.

A comparison of measured data between the design stage wave rider deployment (1983-1985) and the deployment of the regional monitoring wave buoy (2008-2011) suggests that this short-term deployment at the design stage is more energetic than has been the case for the past three years (Figure C.10.20). The locations of the two wave rider buoys were very close and might reasonably be considered to be co-located. Long-term records of transformed synthetic wave data are also compared, but these are in significantly shallower water where significant energy losses might be expected.

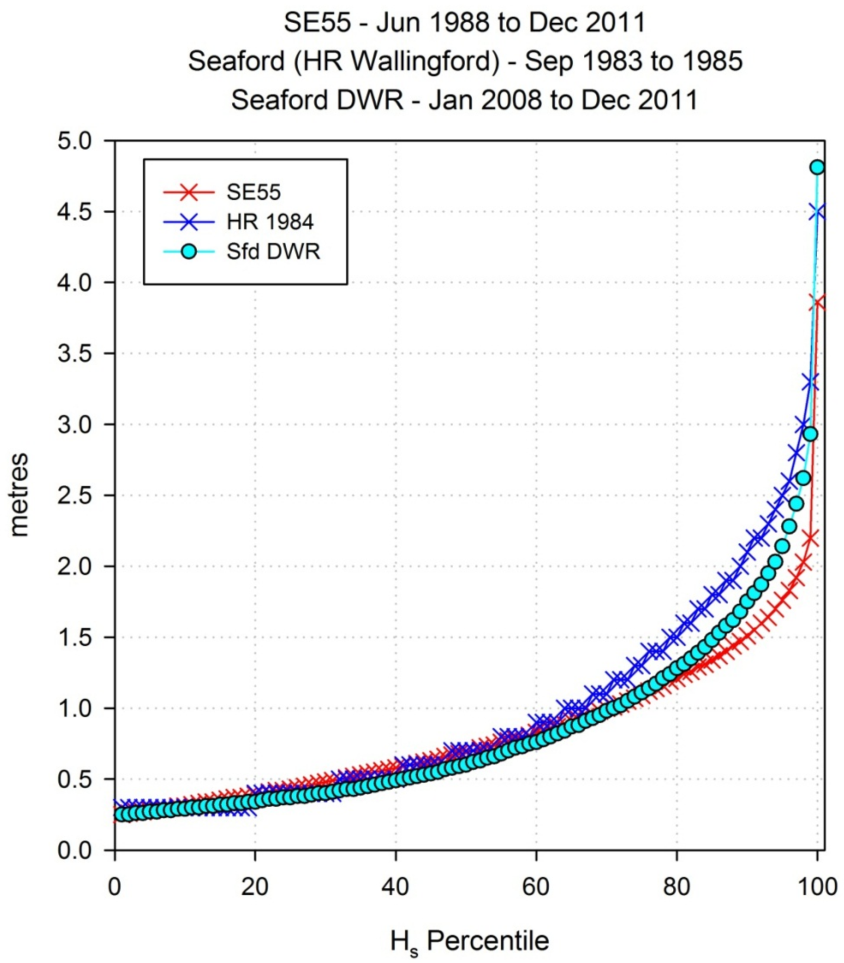

Figure C.10.20 Comparison of design stage and post-construction wave buoy deployments

Post-construction probability distributions based on transformed Met Office model data show considerable inter-annual variability of measured wave conditions at point SE55 from 1988-2008 (Figure C.10.21). Note that these conditions were derived following design. These data suggest periods of fairly severe conditions during the 1990s; these wave conditions were generally more energetic than were typical over the 1971-1979 hindcast used in design. Regrettably the design data are not currently available in a form that permits direct comparison.

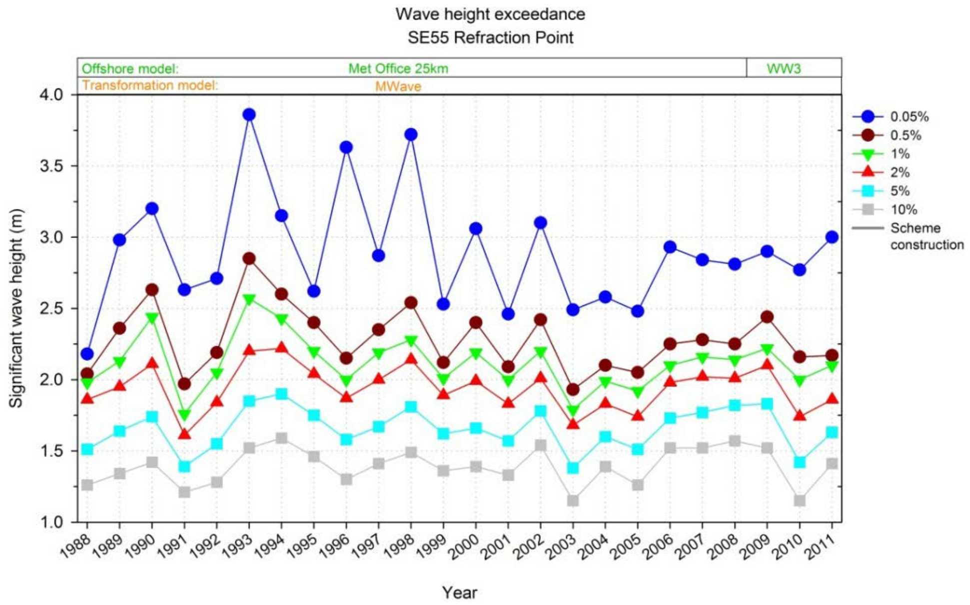

Figure C.10.21 Modelled wave climate timeline for post construction 1988-2011

Modelled post-construction data suggest the intensity of post-construction storm conditions have been more severe than those assessed during the design phase (Figure C.10.22). One storm event in 1993 stands out as being more severe than any other. Note that the comparison is made at a location in shallower water depths to the wave prediction sites used in the design. Measured data at the deeper water buoy site are also added for comparison, where conditions are significantly more energetic.

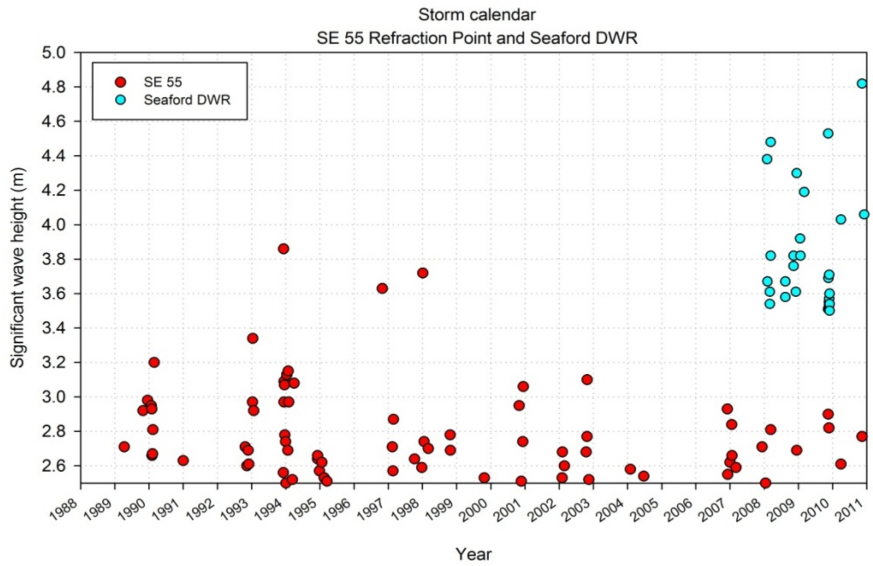

Figure C.10.22 Hindcast post-construction storms (5 m CD) above 2.5 m threshold and measured post-construction storms above 3.4 m threshold (10 m CD)

More recent investigations have identified wave condiions with bimodal wave periods as an additional factor that should be considered in the design process at some sites. Evidence from the WaveRider buoy indicates that bimodal conditions have occurred for just 1% of the time from 2008 to 2011.

Percentage scatter distributions of measured post-scheme significant wave height and direction (Figure C.10.23) indicate that a very high proportion of the wave energy approaches the shoreline at an angle greater than the beach azimuth of 218° at this location (near the recreation ground), suggesting that sediment transport should be predominantly towards the east at this location, and which clearly is the case.

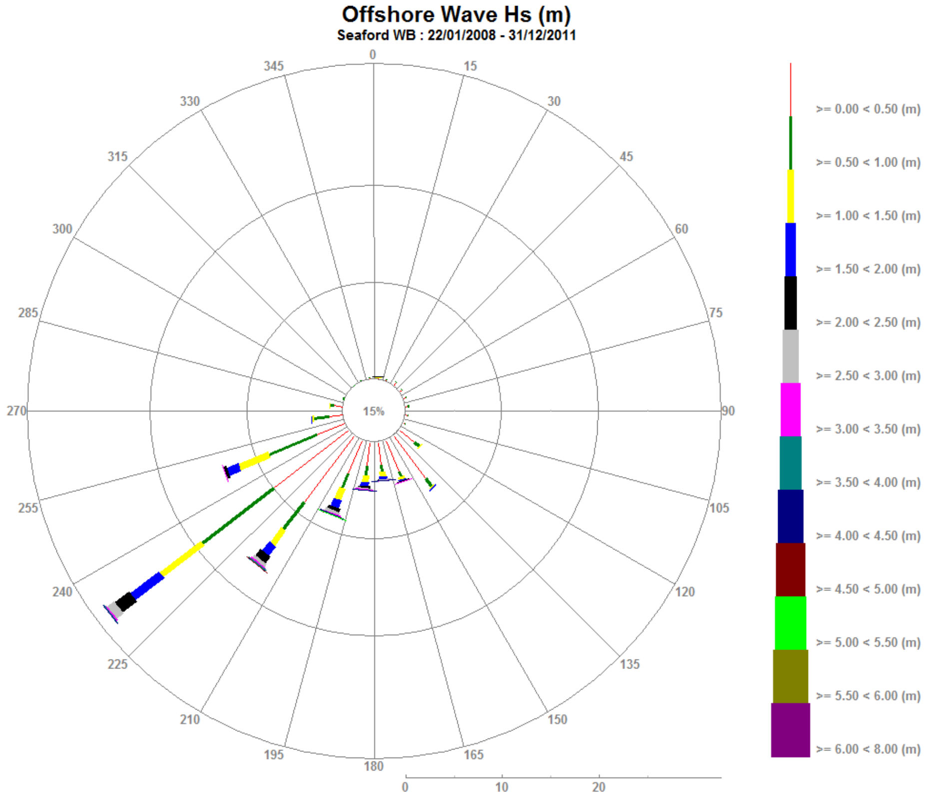

Figure C.10.23 Distributions of modelled significant wave height and direction post-scheme

The distribution of measured significant wave height and wave direction is shown in scatter form in Figure C.10.24, which resolves direction more finely (5° bins). This also demonstrates that some moderately energetic conditions occur also for directions <218°, but that a great proportion of the energy lies within the >218° sector. The location of the buoy is in deeper water than the shallow water sites modelled data sites used in the design and this might impact on actual directions at the shoreline arising from further refraction. Regrettably the data do not extend back earlier than 2008.

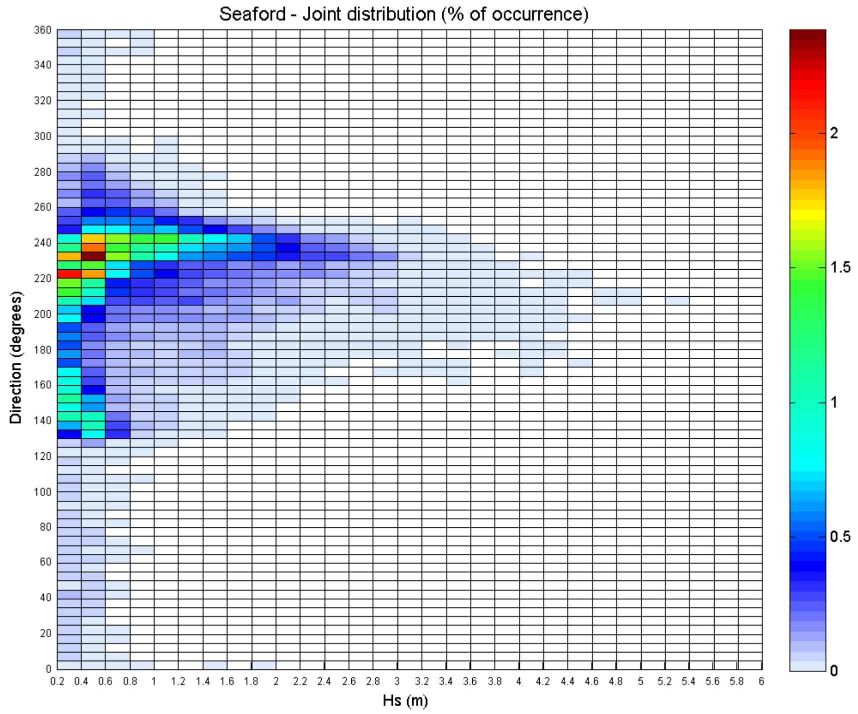

Figure C.10.24 Directional distribution of measured wave data, 2008-2011

Measured exceedance probabilities indicate a constant year-to-year distribution of significant wave heights since 2008 (Figure C.10.25).

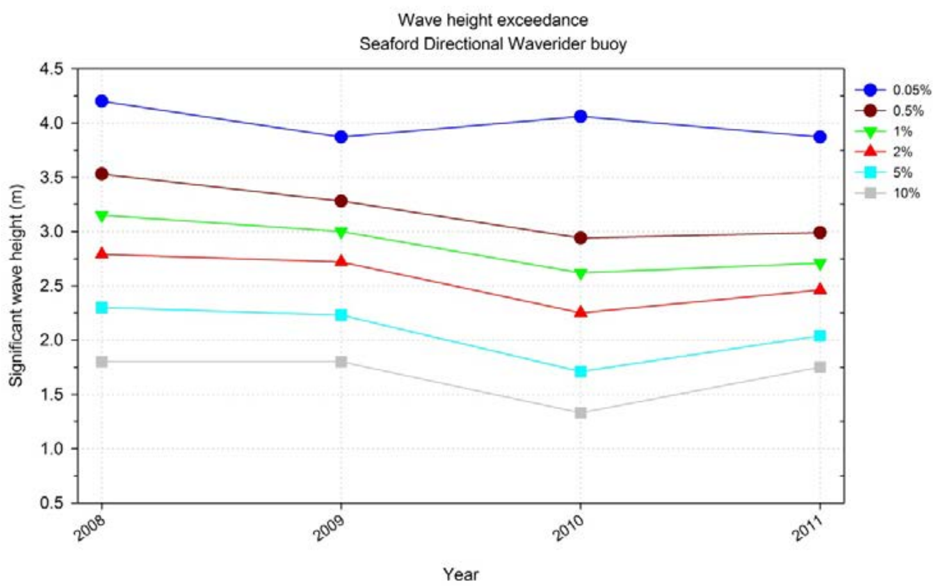

Figure C.10.25 Measured wave climate timeline since 2007

C.10.5 Comparative analysis

The original projected scheme life was 50 years, assuming recycling at a rate of 28,000 m^3per year on average. Drift rates have been significantly greater than this. Under current management, the scheme life is expected to easily achieve the originally projected design expectations of 50 years. The fact that there have been no net losses from the system indicates that the scheme is likely to be sustainable for considerably longer, subject to continued beach recycling. The measured net drift direction is in the same directions as that suggested by the design stage beach plan shape modelling. This is demonstrated by gradual accretion at both Splash Point and Tide Mills, at either end of the site. This is supported further by the requirement to regularly recycle material from both areas.

The monitoring figures alone present an over-simplification of the scheme performance, since beach recycling adds to the losses of material from the at risk sections of the scheme. Recycling logs which have been completed sporadically since 1989 and more rigorously since 2006 indicate that volumes of recycling are typically between 50,000 and 125,000 m^3per year.

Original allowances in the design process suggested that require recycling rates might be of the order of 20,000–28,000 m^3per year on average. Maintenance commitments are therefore 2.5–6 times greater than the design suggested; this difference can be attributed partially to the wave conditions used in design, which have not been representative of the more energetic post scheme wave conditions. Additionally the high fines content within the recharge volume is likely to have resulted in faster transport rates than originally modelled. It should be noted that this was a design phase expectation. This might be considered a reasonable result relative to realistic modelling expectations in a high drift situation. The potential for such differences are also highlighted in the design reports, which explain the uncertainty associated with a variety of variables in a clear manner. Notwithstanding this, the design calculations suggest that the calculated drift rates are underestimates of the actual volumes. A recent assessment of longshore variability of performance indicates that the beach cross section is generally above the original design levels (Figure C.10.26).

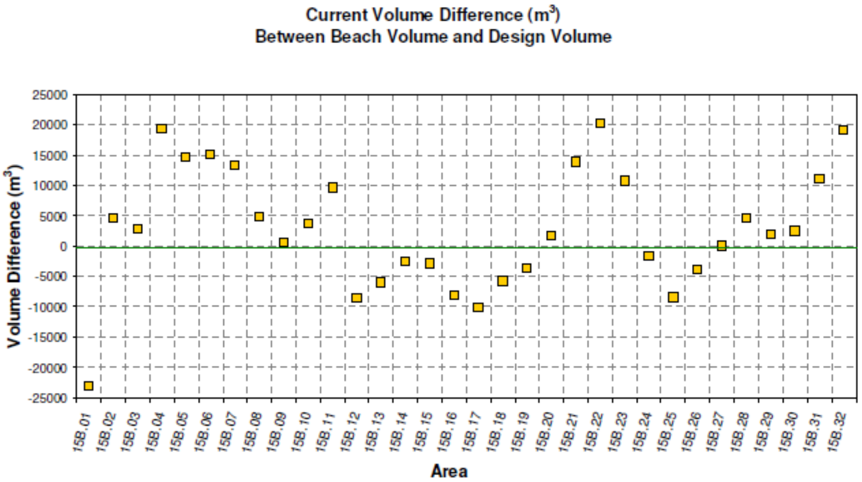

Figure C.10.26 Scheme performance timeline since 2003

Numerical modelling of wave climate suggests that wave energy is variable along the length of the beach recharge and that the beach azimuth varies significantly from west to east. There should consequently be a variable rate of longshore transport and direction along the beach, as suggested by the beach plan model. The general suggestion of longshore variability of wave energy, provided by the wave models, is supported by clear evidence of variability of longshore transport rates along the length of the beach recharge. Measured wave data since scheme construction similarly suggest that sediment should typically be driven towards the east, but with periodic reversals.

It is therefore possible to draw the following conclusions.

- The current regime of response recycling has been effective in maintaining the standards of service set by the 1987 scheme.
- Beach volumes have increased slightly since the 1987 scheme; this may reflect more onshore movement of sediment.
- The original assumptions that accretion would occur at either end of the site and that erosion would predominate in the central section, has been proved.

The average annual volume of recycling is 84,000 m^3per year but there is no clear inter-annual trend, with recycling volumes varying between 50,000 and 125,000 m^3. The natural variation in wave conditions produces regular variations in sediment transport. An economic review in 1996 suggested that, despite the increased costs of maintenance, the recycling strategy still provided an economic approach to management. An annual allowance for 100,000 m^3 recycling is now allowed for.

The monitoring has had a major impact on management of the beach system. It has enabled effective modification of the maintenance strategy, which requires greater recycling than originally envisaged. The monitoring has been particularly valuable for the purposes of evaluation of threshold damage levels and for long-term planning of recycling requirements.

Early stage cross-shore performance was considered problematic following the recharge due to the low permeability arising from the high fine content within the recharge material. Steep cliffs formed on the beach under even quite moderate wave conditions at an early stage following construction. Limited and very slow infiltration of waves was observed into the beach. More recently, concerns have been expressed at the apparent lack of permeability, which accentuates wave run-up and results in waves skating across the compacted crest surface. This characteristic seems to be getting worse and is attributed to the regular running of heavy plat across the surface, which is compacting and binding the fine material within the sediment matrix.

Scheme functional performance

- The capital scheme constructed in 1987 has been entirely successful in ending the regular damage to properties and closure (due to shingle blockage) of the coast road and it has protected the nearby properties.
- The scheme as constructed in 1987 has consistently provided the required level of protection to the town of Seaford.
- The beach volumes within the coastal cell have remained fairly constant over a period of 25 years.
- The original maintenance strategy of regular beach recycling has been maintained and modified to reflect increased drift rates over the entire period.
- The design beach geometry conditions have remarkably been maintained at the site for the whole of this period.

Wave climate

- The design offshore (1:100 year) significant wave height, as calculated at the design stage, has been exceeded twice since scheme construction in 1987.
- There has been a greater frequency of severe storm conditions than anticipated at the design phase.
- The wave conditions since scheme implementation have been generally more energetic than those modelled at the design stage. There is clear evidence that the time series used in the design phase has not been representative of the period following construction (Figure C.10.19).
- A small proportion (1%) of storm events is represented by wave conditions with bimodal (period) characteristics.

Plan shape evolution and sediment transport

- Plan shape evolution has been similar to that suggested by the beach plan shape modelling process, although the central section has cut back much further than anticipated within the modelling.
- An equilibrium plan shape has not formed, but this cannot occur with the recycling policy.
- Sediment transport rates have been generally higher than predicted by the beach plan numerical model.
- Consequently recycling rates have been 2–5 times greater than originally envisaged at the design phase.
- The longshore variability of sediment transport rate has matched that anticipated at the design stage; this is evidenced by a build-up of material at both ends of the bay.
- In parallel with monitoring, further assessments were made of the plan shape modelling (Brampton and Millard 1996). This modelling, based on data from 1988 to 1991, indicated that to achieve the measured drift rates of about 70,000 m^3per year, the model K_1 factor for sediment size needed to be adjusted from 0.02 to 0.04.
- Wave conditions have subsequently been more severe than at the design phase.

Cross-shore performance

- Cross-shore responses have been broadly similar to those modelled, but cut back of the beach crest has been greater than that modelled in moderate conditions.
- The upper beach has tended to form extremely steep cliffs where the fine material has been bound into a matrix with the coarser fraction of sediments. The consequent permeability is quite different to that tested in the physical model. The physical model was designed with material with no sand content while the beach was constructed with 40% sand content. Run-up is higher than predicted and this possibly reflects the low permeability of the beach and the ability of waves to skate across the surface.
- The level of intervention to maintain the beach crest has been greater than anticipated, but the establishment of a Beach Management Plan in 1996 gives the Environment Agency the certainty that the maintenance work it undertakes is targeted and adaptable to suit changes.
- The monitoring programme has provided timely and detailed assessment of performance and has enabled a more efficient assessment of changes to the system and improved efficiency of operational activities is possible.

C.10.6 Lessons for future beach modelling/design

While the time series of wind data and measured wave data used was the best available at the time of scheme design, the dataset was somewhat shorter than is desirable for a project of this type. Unfortunately the time series was not representative of the more severe conditions that occurred during the following 10 years. Ideally a duration of 20 or more years' data should be used, enabling inter-decadal variations to be considered, although even this may be inadequate in times of rapid climate change. This is generally possible now for open coast sites when using one of the long-term Met Office offshore datasets, which date back to 1988. These datasets also include swell waves which may be significant at some sites.

Where possible, design wave climates should include, as a minimum, several years of measured wave data to replace or complement numerical hindcasts. There is clear evidence that systematic biases occur within the modelling process and measured data can be used to assess this. Much longer (>20 years) wave datasets are now available from the Met Office second generation wave model. The Met Office wave model itself was superseded by WAVEWATCH III in 2008; this model appears to reproduce wave heights more reliably on the south coast, with the bias evident in the Met Office model being removed. In order to provide design conditions appropriately, long-term hindcasts will be needed based on this model. Data will then need to be transformed to suitable nearshore locations and validated against local wave measurements. This approach will improve the ability to model sediment transport more accurately, since this is strongly dependent on wave height and direction data. The bathymetry presents modelling challenges for transformation models and any outputs should be carefully scrutinised and validated against measured data where possible; this will restrict the possibility of wave directions being incorrectly represented by the modelling.

Some hindcasting models do not make allowance for swell conditions and there is clear evidence from measured wave data that these do occur within the bay.

Drift is generally considered to be at a very high rate for shingle on an open coast. Drift of material is in the direction suggested by the numerical model. The monitoring output illustrates the value of comprehensive field observations as an integral part of the modelling process when conducting numerical modelling at a complex site such as this. The site is now much better equipped with field data and in a better position to review and tune the output of any modelling exercise. Adequate field data are now available to calibrate any future modelling.

Taking account of the above observations relating to wave climate, the calculated sediment transport rates are significantly higher than expected. The difference in measured and modelled rates may reflect a combination of wave climate, sediment size and perhaps model calibration for grain size. However, these limitations were highlighted extremely thoroughly at the modelling stage. Model reporting reflects on the uncertainties or limitations with these variables. Some form of sensitivity assessment would be helpful to identify the range of uncertainty relating to (for example) different wave conditions. The outputs from both numerical models and physical models appear entirely reasonable and within the anticipated range of expectations for such models.

This approach to scheme management provides a comprehensive review of scheme performance, including scheme maintenance, response of beach and predicted changes to date; it also provides confidence in future projections. Subsequent remodelling of the site some years after construction has taken account of the design stage weaknesses, integrated additional monitoring responses and also included more representative wave climate data. This has resulted in a revised and more robust assessment of conditions; this approach might be useful elsewhere, under conditions that result in significant differences to the initial modelling.

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C.11 Southend-on-Sea

C.11.1 General information

Southend-on-Sea: Pier to Thorpe Hall Avenue Flood Defence Improvements
2001-2002
Southend-on-Sea Borough Council

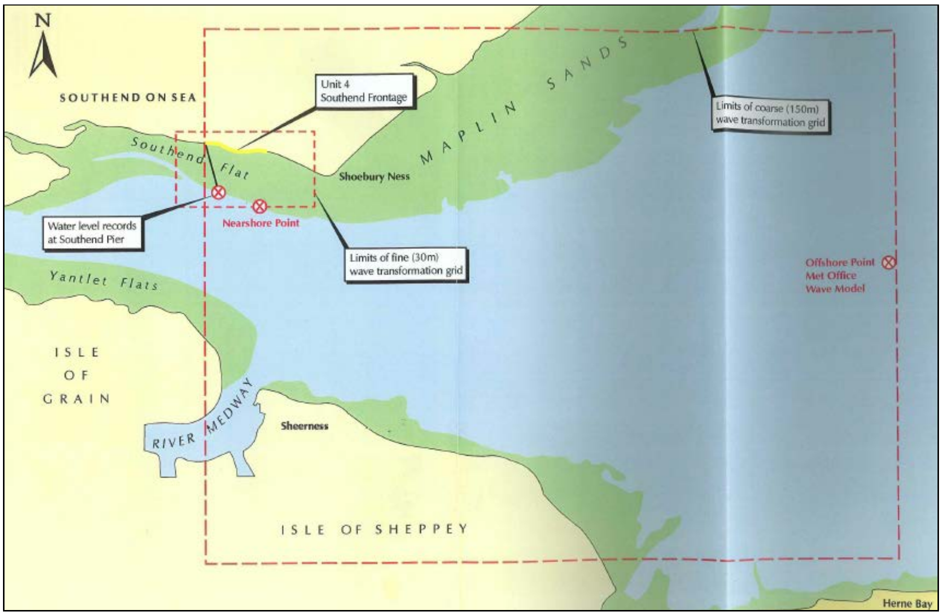

Figure C.11.1 Location of site, also showing area covered by wave model

Background

A coastal defence strategy for the whole of the Southend-on-Sea frontage (Figure C.11.1) was developed in 1998 (Mouchel 1998). The strategy recommended a beach recharge scheme for the frontage eastwards from the Pier to Thorpe Hall Avenue where there were over 700 properties in the low-lying area behind the defences that would be subject to annual coastal flooding in the event of a breach (see Figure C.11.2).

The site is located in the Thames Estuary and is exposed to both locally generated waves and waves that propagate into the area from the outer Thames and southern North Sea. There is a large tidal range and wide environmentally important intertidal sand/mud flats backed by a coarse beach and seawall.

Prior to the scheme the existing defences comprised a beach backed by a sloping (1:3) blockwork revetment capped with a 1 m high concrete wall. The existing beach was about 25 m wide with mixed sand and shingle grading. There were timber groynes, spaced at approximately 40 m intervals.

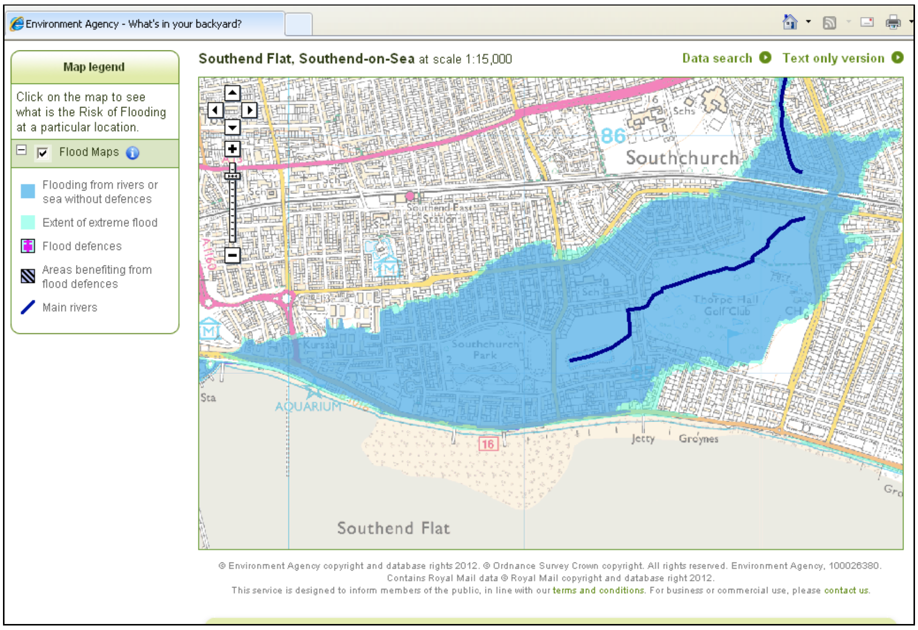

Figure C.11.2 Flood risk area protected by defences

Numerical modelling was undertaken in 2001 as part of development of an Engineer's Report to support a funding application for grant aid and subsequently to develop the detailed design. The works included repairs and refurbishment works to the seawall as well as the beach nourishment scheme.

The length of frontage improved was 2.2 km. The scheme development included assessment of the design beach profile in relation to standard of protection and options for the use of extended timber groynes or an open beach. The preferred standard of protection was selected to reduce the annual chance of flooding to 1 in 100. The open beach solution was preferred due to the estimated whole life cost being lower and the environmental benefits of avoiding the use of timber groynes, particularly the need for maintenance vehicles to pass over the foreshore seaward of the groyne ends which forms part of the internationally designated site for wading birds.

The Engineer's Report proposed an initial placement of 190,000 m^3 of beach material (Halcrow 2001). This allowed for placing the nourished beach with a slope of 1:10, a crest level of 4.65 m OD and a minimum crest width of 5 m.

Environmental issues were given high priority in the scheme development due to the internationally important wide intertidal sand and mud flats off Southend that support significant numbers of wild birds. There was concern that the beach recharge material could spread onto and smother areas of the mud flats. The specification of the grading of the recharge material was therefore optimised to maintain a stable steep profile and avoid significant loss of fines onto the sand flats. The environmental report identified a preferred maximum and thresholds for the grading curves that were agreed with English Nature and subsequently translated into the contract. The D_{50} range was 1.5–10 mm.

The capital beach recharge scheme was completed in June 2002.

C.11.2 Approach to modelling and basis of design

Rationale

As a coarse sediment beach was planned, it was expected from outset that wave driven sediment transport on the beach would be the dominant process. However, there were no measured nearshore wave data available for use in the modelling or to help calibrate the models. Wave modelling using wind data and offshore Met Office model data was therefore undertaken; the approach is summarised in Figure C.11.3.

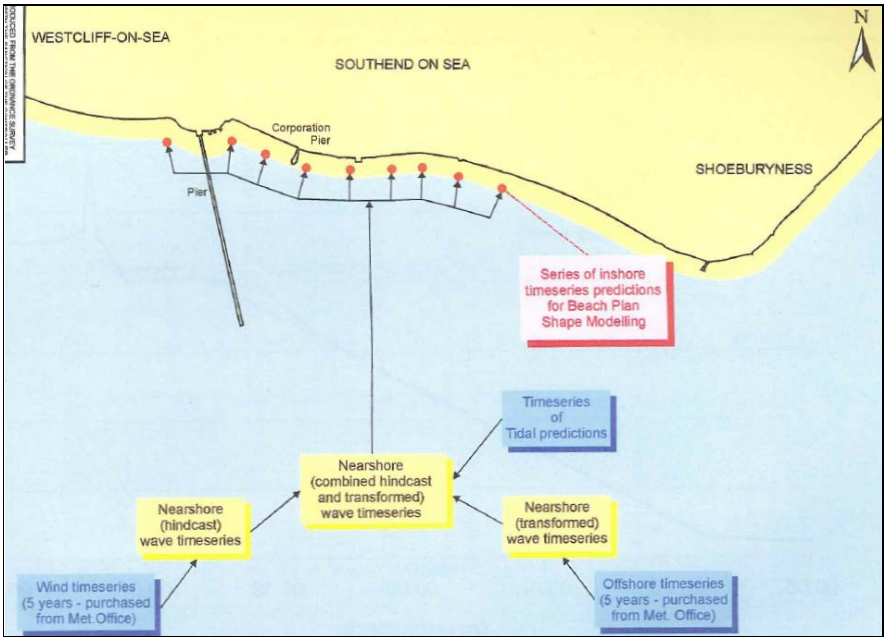

Figure C.11.3 Approach to wave modelling

Models used

- Wave modelling: Due to the relatively sheltered location in the outer Thames, swell and locally generated waves needed to be considered. This involved wind wave hindcast modelling, transformation of offshore wind and swell waves with a 2D numerical wave model (see grids on location map) and derivation of a combined nearshore wave climate.
- Beach plan shape and alongshore drift modelling: one-line beach evolution model.
- Beach profile cross-shore storm beach response modelling**:** both numerical cross-shore and parametric (SHINGLE) used.

- Met Office offshore wind and wave hindcast data, five years of data were used (1 January 1994 to 31 December 1998) from Outer Thames open sea point at 51.5°N, 1.1°E
- One year of locally measured wind data from Shoeburyness April 1991 to March 1992
- No measured data or nearshore data were available for calibration.

- Offshore time series data from Met Office was transformed inshore using a 2D wave model and combined with a local wind-wave hindcast, which used the Met Office wind data from offshore

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- Annual maxima from Southend Pier for 1911 to 1993 were used to derive extreme water levels.
- Tide predictions for the wave transformation used the NP159 method and were based on published harmonics and shallow water corrections from the Admiralty

Tide Tables.
• Topographic survey of beach from 1999 • Two beach profile locations from the Anglian Regional Monitoring, E4A4 and E4A5, with data from surveys in 1993, 1995 and 1997 • Modelled sediment: for both short-term storm response modelling with the numerical cross-shore model and one-line beach evolution modelling, a D50 of 2 mm was used. The profile storm response was also analysed with the SHINGLE parametric model using D50 between 2 and 10 mm. • As only one set of beach profile surveys was available, there was insufficient data to calibrate the beach plan shape model. All that could be done was to compare predicted drift directions and shoreline change with evidence of transport directions from observations during a site inspection.
• Tidal flood risk to 1000+ properties • Impacts of recharge and beach management on the environmentally designated habitats on the foreshore: important to avoid losses of recharge material onto the sand/mud flats to avoid impacts on bird populations • Need to ensure existing amenity value of the beach not compromised • Wave attenuation due to the wide inter-tidal flats • Natural sediment: a limited amount of coarse sediment on upper beach, sand/mudflat lower intertidal foreshore
• Prior to the scheme: there were wooden groynes on the frontage with typical length of 30 m. These were ineffective as beach was denuded. • Options considered: the modelling of the recharge scheme considered three options: open beach, 30 m groynes and 50 m groynes. • Several outfalls, slipways and the Corporation Landing Pier (removed in 2007) act as partial beach control structures. • Groynes: Modelling of options initially proposed to include 50m groynes in preference to the options of 30m groynes or an open beach. However, groynes were subsequently dropped and the open beach design adopted due to environmental concerns (such as the need for beach management plat to track across the foreshore seaward of the groynes) and lower costs. • Beach material: sediment in final design; sandy gravel with grading envelope based on analysis of the existing beach at eight locations (see Figure C.11.6) Figure C.11.4 summarised understanding of wave conditions and littoral drift.

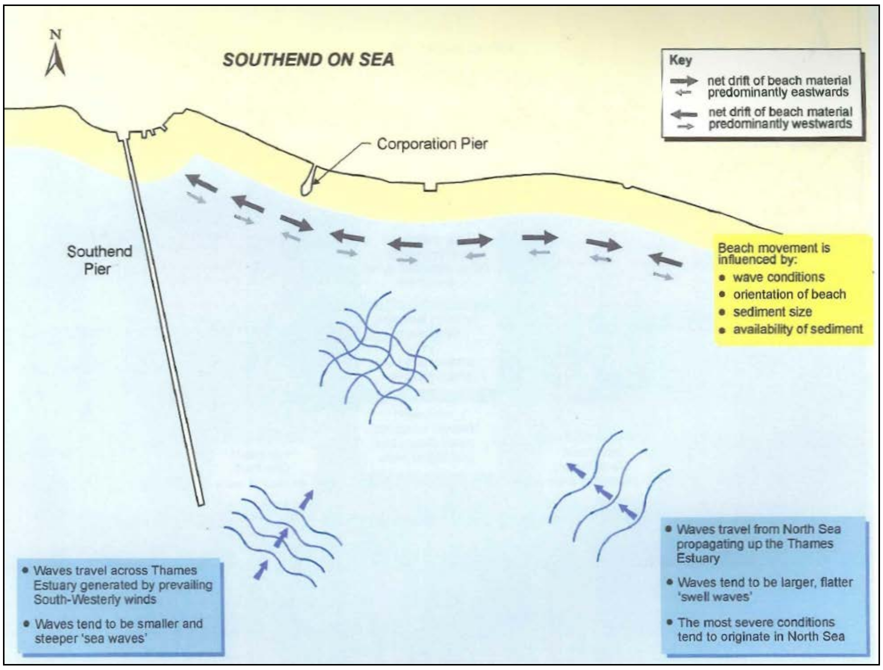

Figure C.11.4 Understanding of wave conditions and littoral drift

The beach plan shape model showed that longshore drift rates were low due to the dissipation of wave energy and refraction of wave directions to be close to shore normal by the extensive mudflats. Predicted maximum drift rates were 2,000–3,000 m^3per year.

C.11.3 Design/modelling outputs – plans for implementation

Extreme waves were derived from a nearshore combined time series of transformed swell waves and locally generated waves at the toe of the beach generated using a fine grid wave propagation model. The estimated extreme wave heights were subject to significant uncertainty due to the short five-year record length used. However, they were checked against results from a nearby study in the outer Thames that had used 14 years of data. The larger waves are essentially limited by the depth of water across the mudflats and so the estimated wave heights given in Table C.11.1 were adjusted for sea level rise scenarios separately.

Table C.11.1 Estimated wave heights

Return period (1:x year)	Wave height, H_s (m)
1	1.3
5	1.6
20	1.8
50	1.9
100	2.0
200	2.1

The beach profile (Figure C.11.5) was optimised through modelling tests and knowledge of the existing and nearby beaches. The modelling used the numerical cross-shore model to estimate response to storms for sand sized sediment and use of the SHINGLE parametric model to estimate the storm response for a gravel-sized beach. The estimated total volume of material required was 190,000 m^3.

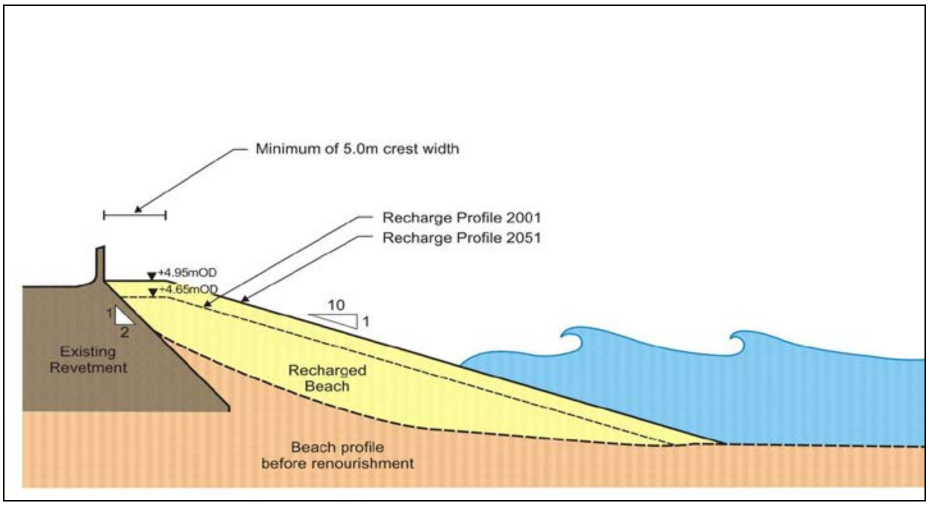

Figure C.11.5 Design profile

Sediment was to be placed in accordance with strict requirements of grading curves (Figure C.11.6) and environmental conditions with regard to the placement.

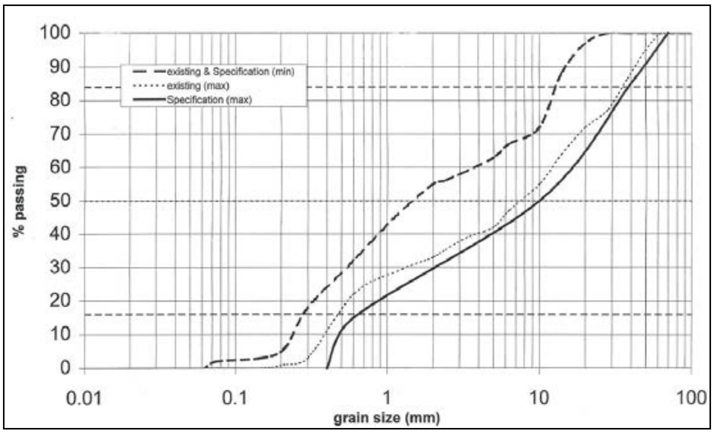

Figure C.11.6 Design grading envelope

A Beach Management Plan was prepared in 2001 as part of the scheme design. The estimates of long-term beach management requirements in the scheme appraisal were uncertain due to the lack of data to calibrate the numerical models. According to the Engineer's Report (Halcrow 2001), a contingency was added to the modelled estimates. As a result it was considered that recycling of beach material may be required on an annual basis, with an expected total of 40,000 m^3 recycling over each five-year period. Additional recharge of 20,000 m^3 was also allowed for at 10-year intervals over the scheme design life of 50 years.

Action and emergency trigger levels for flood defence were defined in the beach management plan as shown in Table C.11.2.

Table C.11.2 Trigger levels for flood defence

Action level	Emergency level
Beach crest <3 m wide at +4.65 m OD along a total length of 200 m over the 2.2 km frontage	Beach crest <2 m wide at +4.65 m OD along a total length of 200 m over the 2.2 km frontage.

Beach monitoring was originally recommended at six-month intervals but has taken place annually in late spring. The monitoring includes a topographic survey and visual inspection with photographs. The monitoring was developed to answer the following questions:

- Does the beach still provide flood protection to Southend to the design standard?
- Are works required immediately and what form should these works take?
- Is the forward programme of recycling and recharge still appropriate to maintain flood protection?

C.11.4 Beach management and performance

Estimates of long-term beach management requirements for the scheme were documented in the Engineer's Report (Halcrow 2001). Expected rates of sediment transport and thus recycling/recharge top-up requirements were uncertain due to the lack of data to calibrate the numerical models. According to the Engineer's Report, a contingency was added to the modelled estimates and it was considered that recycling of beach material may be required on an annual basis, with an expected total of 40,000 m^3 of recycling over each five-year period. Additional recharge of 20,000 m^3 was also allowed for at 10-year intervals over the scheme design life of 50 years. Beach monitoring was originally recommended at six-month intervals but has taken place annually in late spring.

The subsequent beach management plan prepared in 2001 as part of the scheme design indicated that at the time of construction beach management was expected to involve 40,000 m^3 of recycling every five years and 15,000 m^3 of recharge every 10 years. A more detailed programme for implementing this recycling was to be developed from monitoring actual performance to determine if recycling would be required annually or in a single block. Specific mitigation measures to reduce the potential effect of maintenance operations on the Benfleet and Southend Marshes SPA that extends along the frontage were also specified regarding timing/seasons and so on

The beach grading was designed such that, under normal conditions, the coverage of the foreshore would not vary substantially after the initial placement. However, the extent of actual coverage was to be monitored as part of the beach surveys and the results considered in relation to the bird survey data.

Figure C.11.7 shows an example of beach profile monitoring data.

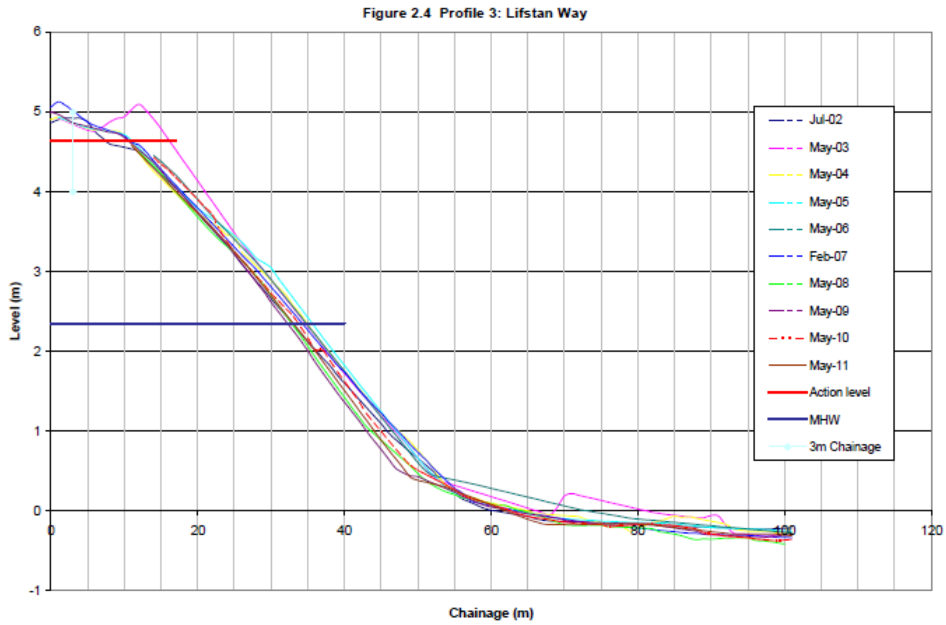

Figure C.11.7 Example of beach profile monitoring data

The capital recharge works took place between April and July 2002. The contract drawing indicates that the contractor was required to achieve a beach crest level of 5.9 m OD, that is, 0.25 m above the design profile and the levels specified in the beach management plan to trigger beach management action.

The specification of the grading envelope was amended slightly following agreement of consultees to the Food and Environment Protection Act (FEPA) licence to better take into account the available sediment from the offshore dredging site. However, the changes from the envelope shown in Figure C.11.6 were minor.

- **2002 to 2007.** No works were required. Over the first few months and years there was some natural re-profiling on the beach to a slightly steeper slope, typically with some very slight accretion of finer material, at the toe of the beach. There was some small movement of material eastwards to the non-renourished section; there were also slight accumulations at barriers to alongshore drift such as the outfalls and paddling pool. However, the action thresholds were not met and so no recycling took place.
- **2007.** Following unusual storm conditions, the 2007 monitoring survey was undertaken early (February whereas others have all been in May). The report indicated that a length of the frontage west of the Corporation Loading Pier had suffered erosion to reduce the crest below the 3 m action level, but the length affected was not yet at the 200 m threshold.
- In the autumn of **2007** the Corporation Pier located at the eastern end of the main promenade was removed. The pier was thought to be a retaining/control structure for the beach material. Consideration was given to the effects of removal of this structure before it took place. It was expected that, due to the reduction in wave sheltering, there would be some erosion and redistribution of the material that had built up on the west side.
- **2008.** A survey indicated 375 m of frontage below action level and 185 m at emergency width at the western end of the frontage. In September it was recommended that approximately 6,000 m^3 of beach material should be recycled from further east where there had been accretion in the region of Camper Road slipway.
- **2009.** A survey indicated the impacts of the September 2008 recycling and that the

beach profile had not been restored to the design profile at the extraction or deposition locations as it appeared that the contractor had incorrectly extracted material from the beach crest and placed it on the active beach, resulting in addition lengths not meeting the trigger levels for the crest. Overall beach volumes were healthy, but recommendations were made for re-profiling in order to meet crest level targets. No beach management work was undertaken in 2009, possibly due to lack of funding.

- **2010.** In the spring survey, the length of frontage below trigger levels had reduced slightly. Recommendations for re-profiling repeated. Some re-profiling undertaken in autumn 2010.
- **2011.** Spring survey showed improvements from autumn 2010 beach management works. The beach profiles were generally healthy, but crest levels were below trigger levels and further re-profiling was recommended. It was also noted that the original plan recommended recharge in 2012 and that this should be considered at the same time. However, none or minimal re-profiling or recycling was undertaken.
- **2012.** The June 2012 survey found that the length of frontage that falls below the emergency threshold level had reduced further in the period to the survey, but it was considered that there remains the need to undertake further works to restore the standard of protection (SoP) against flooding. It was noted that the volume on the beach appeared to be sufficient for re-profiling and recycling to be used to restore the beach to meet the crest level targets. Alternatively recharge could be undertaken in 2013.

There has been no wave monitoring at the site. In order to assess estimates of the wave conditions experienced since construction and compare these to those expected at the design stage, Met Office synthetic offshore wave data were obtained specifically for this case study analysis. The full available dataset from 1988 through to June 2012 was used. The wind-wave hindcasting model was used with the wind data from the Met Office model data to hindcast locally generated waves for a point offshore from the mudflats at Southend. The offshore waves from the Met Office dataset were also transformed to a nearshore site using the spectral refraction model within SANDS. This new modelling of waves for this case study is not directly comparable with the original modelling as the approach, models used and nearshore locations and depths differ. The results show wave heights in deep water rather than at the toe of the beach.

Figure C.11.8 shows an annual wave height exceedance plot for the new hindcast. It also shows the period of data that was used for the original modelling study in order to compare the relative storminess. Figure C.11.9 shows a wave height exceedance plot for the resultant offshore waves transformed to an inshore point near to Southend. Figures C.11.10 and C.11.11 show storm calendars respectively for hindcast waves offshore from Southend and for transformed swell waves before and after the scheme.

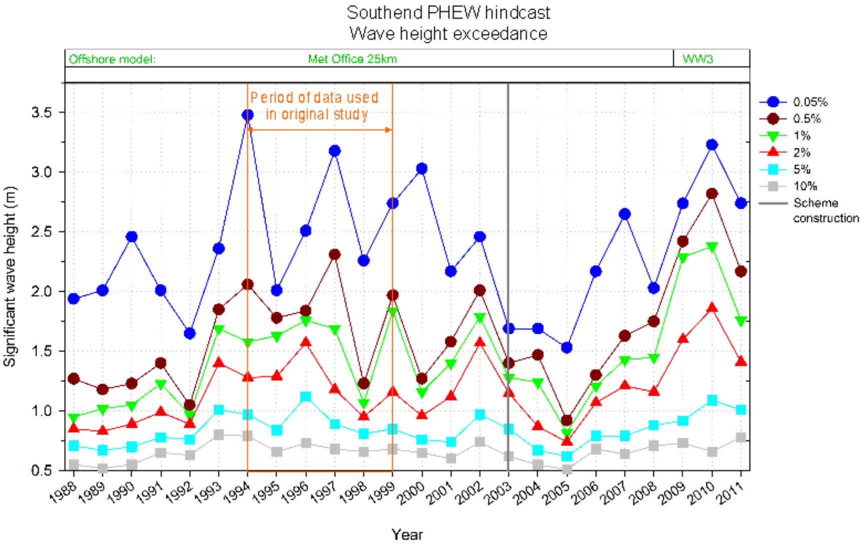

Figure C.11.8 Hindcast waves offshore Southend before and after scheme

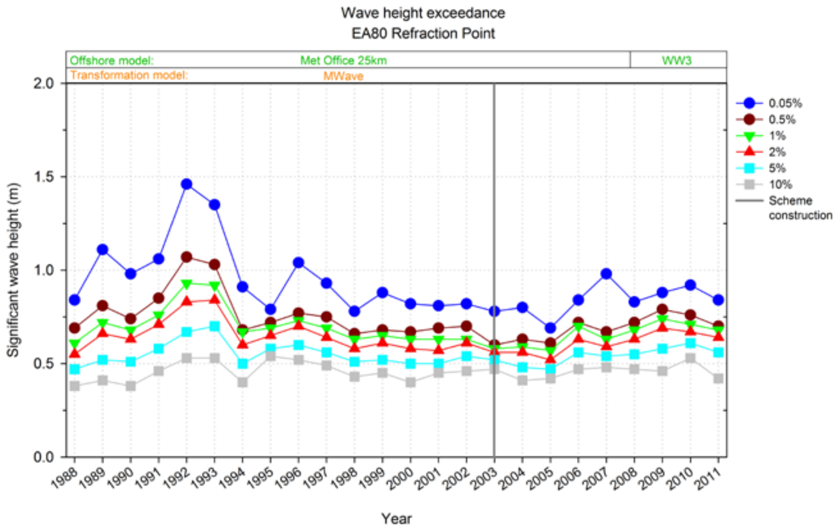

Figure C.11.9 Transformed swell waves before and after scheme

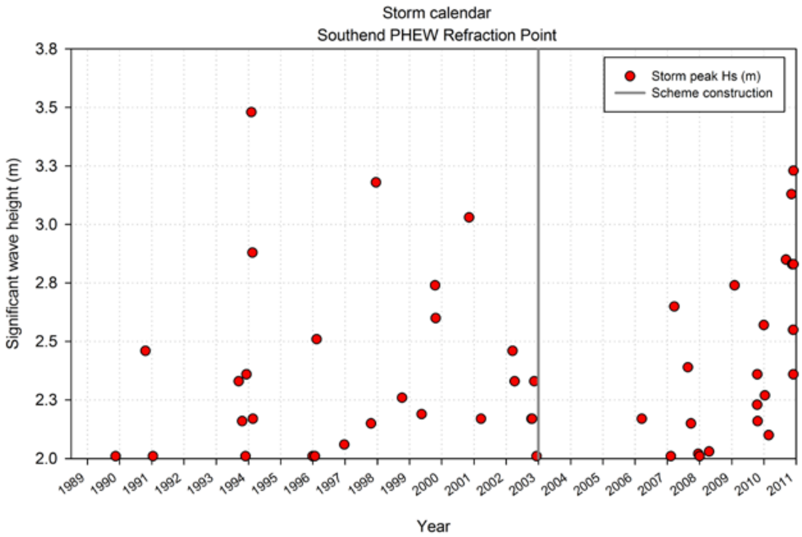

Figure C.11.10 Storm calendar for hindcast waves offshore from Southend

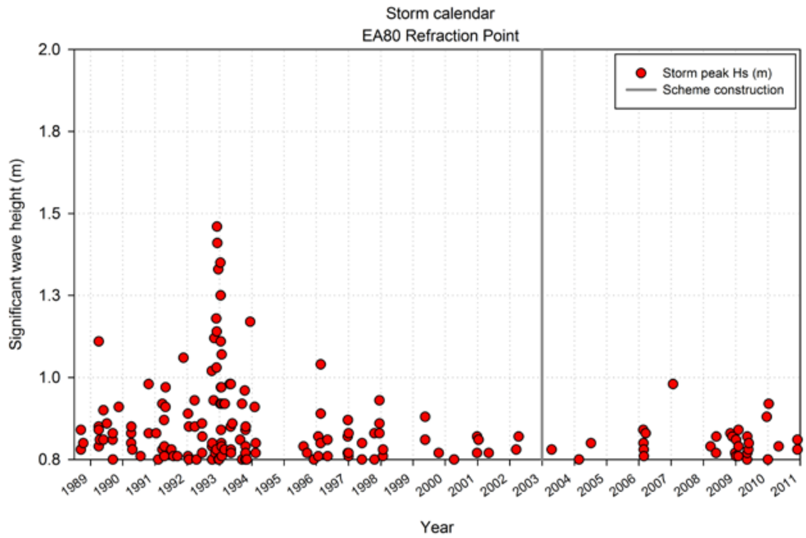

Figure C.11.11 Storm calendar for transformed swell waves before and after scheme

C.11.5 Comparative analysis

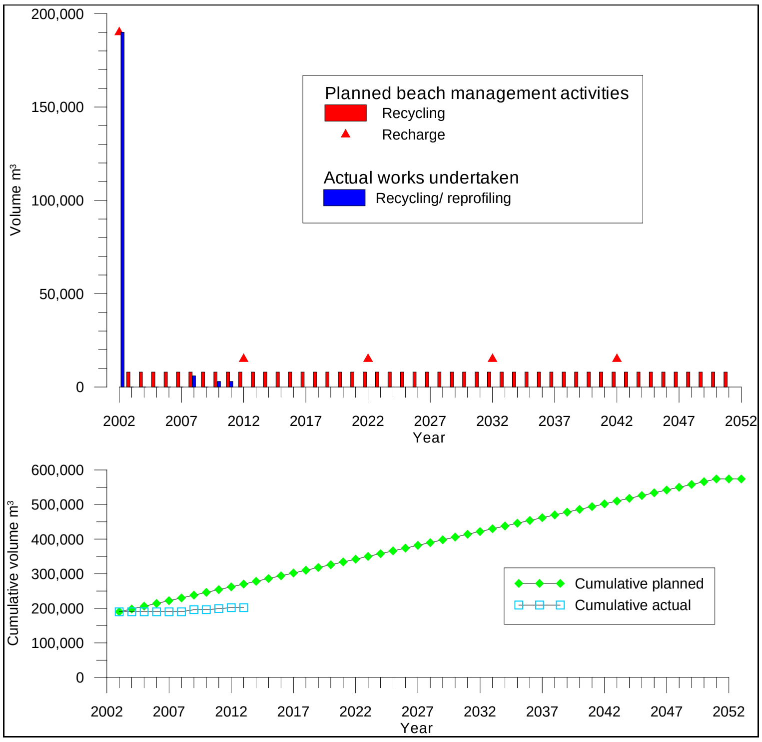

Figure C.11.12 Timeline of beach management actions

The scheme was completed in June 2002, and it is interesting to note from Figures C.11.8 and C.11.9 that the following three years, 2003-2005, had a comparatively benign wave climate compared with the rest of the wind-wave record. Consequently no beach management works were required until 2007 and the cumulative volume of beach management works is now significantly less than expected. A timeline of beach management actions is shown in Figure C.11.12.

Both hindcast and transformed waves show a significant peak in extreme waves in

2007; this is presumably the stormy conditions reported in the 2007 monitoring report and which resulted in the spring survey being brought forward and identifying erosion at the west end of the frontage. The storm conditions during the five-year period used for the analysis in the design studies appear to have been reasonably representative of the more severe conditions occurring in the overall 23-year data period analysed for this study. This was by chance rather than design – the data from 1989 to 1994 could have been used at the time but were not, probably due to budget limitations. If the 1989 to 1994 data had been used it appears likely that estimates of future recharge/recycling may have reduced slightly.
The conservative nature of the design was due to the lack of calibration data and short period of wind/wave data used. There was an unusually low level of storms during the three years after construction.
The sheltered nature of the site, which is protected by the extensive sand and mud flats of the Thames outer estuary at low tide, combined with the coarse recharge material resulted in limited expected movement of the beach recharge material and this has proved to be the case. The mud flats dissipate wave energy and refract the larger waves to be nearshore normal.

C.11.6 Lessons for future beach modelling/design

The wave and sediment transport models were not calibrated as there were no measured data in the vicinity and so a conservative view was taken on beach management activity. Although the scheme has required less beach management than allowed for and actual costs are less than expected, a conservative approach can lead to the overestimation of costs which could make a scheme appear less well economically justified than is actually the case. Only five years of wind and wave data were used in the modelling study that informed the design. This is not generally considered to be long enough to derive the mean or the range of the expected annual wave climate reliably. Although by chance the period of data used was reasonably representative, if relatively energetic, this may have led to high estimates of sediment transport and added to the conservative nature of the design. It is recommended to always use a dataset as long as possible and a minimum of 10 years. Beach behaviour in an estuary environment can potentially be affected by locally generated and open sea wave activity. These need to be effectively combined to fully represent the environmental characteristics at the site in any modelling; otherwise unexpected beach behaviour may occur and need to be managed. Wide intertidal flats will have a significant effect on wave energy and direction. Consequently, gravel beaches sitting behind these may be less affected by regular conditions and only susceptible to cross-shore or alongshore movement from infrequent storm events. This same threshold to mobility will then also apply to the beach recovery, which means that the potential for natural recovery may be limited as the necessary energy to enable this does not occur at the site. Modelling should therefore consider this possibility too for informing the beach management planning.

C.11.7 Bibliography

- Halcrow, 2000. Study to Inform Appropriate Assessment, Halcrow Maritime, August 2000.
- Halcrow, 2001. Southend on Sea Flood Defence Improvements – Engineers Report, Halcrow Maritime, March 2001.
- Halcrow, 2001. Southend on Sea Flood Defence Improvements – Beach Management Plan, Halcrow Maritime, April 2001.
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Appendix D Generic tests

D.1 Introduction

D.1.1 Background

One component of the project to develop guidance for beach modelling was reviewing the application of 'one-line beach plan shape models', one of the engineering tools commonly used to predict beach evolution through time.

Although Phase 1 of this research project identified that it was not appropriate to recreate a numerical model specifically for any of the candidate sites, it was determined that a non-location specific, one-line beach plan shape model might be established to illustrate the sensitivity of a beach system to differences in key variables within a 'controlled environment' including:

- differences in beach material
- changes in wave climate
- impact of changes in beach nourishment (volume and timing)
- differences in scheme type (recycling, recharging, with and without groynes)

It is important to fully comprehend what is going into and coming out of such models. The purpose of this exercise was not to try and quantify the impact that changes in variables have to inform future choices; that would be a much more extensive exercise. It was simply to inform those commissioning such modelling of the issues and sorts of questions to be asking.

D.1.2 Software

It was also not the intention of this exercise to evaluate the merits or shortcomings of any specific software. However, a choice needed to be made from the large number of packages, programmes and code that are available.

Several of the more commonly used packages were appraised. The modelling package chosen for the purposes of this review was one that is a commercially available product that is widely used within the coastal engineering industry, and which has been subjected to various quality assurance and verification processes in its development.

D.2 Model establishment

D.2.1 Model construction

The modelled beach was defined as a simple straight line 5 km long (500 nodes at 10 m spacing). The model was established with straight bed contours to eliminate localised effects resulting from a change in alignment or discontinuities along the shoreline.

The updrift boundary was closed to eliminate any input of sediment into the model other than that introduced by modelled nourishment. In this way the direct impact of changes in any of the variables could be determined and compared without question of the model ‘compensating’ for any shortfall or excess of sediment.

The primary area of focus for comparison of results was the central portion of the model where any issues relating to boundary conditions would have least if any impact upon outputs.

A 10-year wave dataset (1 January 1991 through to 31 December 2000) taken from an arbitrary inshore location on the coast of the UK was used to drive the model (see later section).

D.2.2 Testing regime

Three different scheme types were tested to ascertain whether changes in certain parameters would have different impacts depending upon the scheme type. These were:

A. Recharge; adding material to the beach from an external source

B. Recycling, taking material moved by littoral transport from the downdrift boundary and redistributing along the frontage

C. Groyned beach, with recharge from an external source

Three different nourishment regimes were examined to determine the potential impact of nourishment work being carried out differently from that originally modelled, as follows:

i. Annual nourishment (base cases)

ii. Nourishment with double the volume every second year

iii. Nourishment with five times the volume every fifth year

In all cases the model was run for the first year before the initial nourishment taking place at the end of that first year and the subsequent nourishment frequencies listed above applied thereafter.

Variations in sediment sizes (and associated beach slope) were modelled to establish the potential impact of recharge material being of a finer sand size from the base case, as follows:

1. 1.0 mm grain size (base case)
2. 0.2 mm grain size, that is, smaller than base case
3. 10.0 mm grain size

The third size of sediment (shingle sized) was modelled so as to compare with the results from (1) and (2), noting that one-line beach plan shape models are often used to model this size material although it generally falls outside of the limits of applicability of the sediment transport equations available:

Finally, changes to the wave climate were used to model beach response and to examine the sensitivity of outputs to wave data used for modelling purposes. Partial datasets were taken from the base case 10-year record to construct two variations in wave climate as follows:

a. Full 10-year dataset (base case)

b. Selected years with a different directional energy spread

c. Selected years with a yet different directional energy spread

All model runs were carried out for a 10-year period, although outputs were taken at different points within that period to draw comparisons between modelled beaches performance.

D.2.3 Initial set-up

Initial set-up involved running the model several times to develop the 'base case' (= annual nourishment with 1.0 mm sand size) for each of the scheme types, varying the nourishment volumes, placement locations, groyne lengths and spacings until an 'optimum' scheme was established.

This process determined the average annual nourishment volume required and whereabouts along the beach this needed to be placed to maintain the 'best' (most seaward) beach position (see Figure D.1). The resultant line was then taken as the baseline against which all other variations were compared.

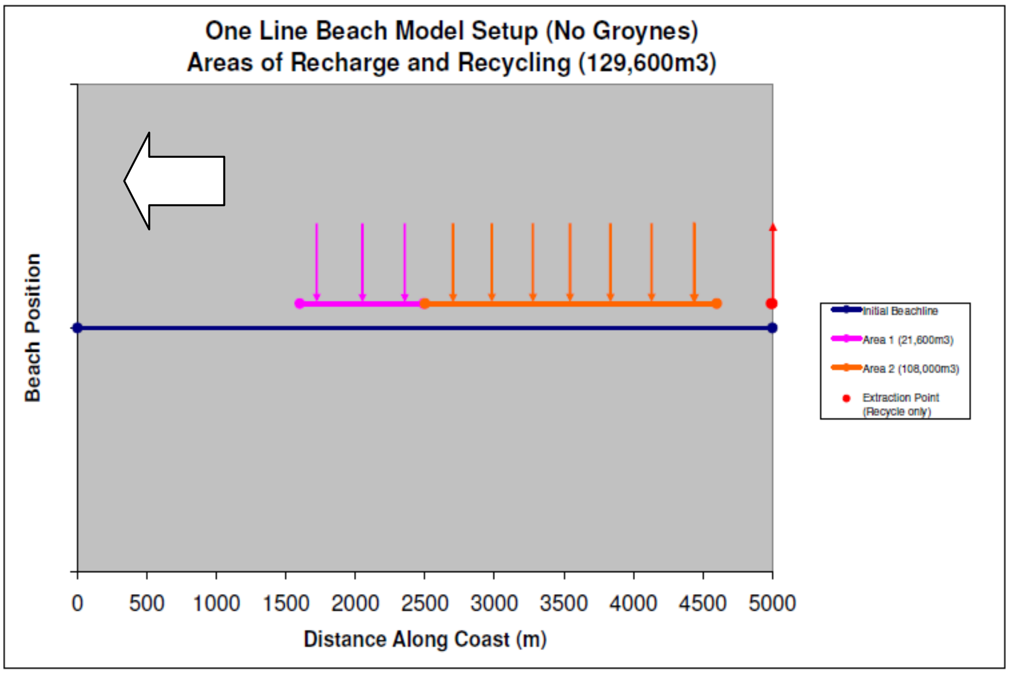


Figure D.1 Model set-up (scheme types A and B)

It was concluded from this exercise that there was limited value in further testing sensitivity of groyne length and spacing, or time of removal/installation. This was because:

- the limitless number of permutations that existed made comparisons of relative performance difficult and potentially meaningless
- all other things being equal, in the medium term, the groyne fields fill and stabilise – just the timing of this alters (unless the wave climate continues to be varied through time, that is, not repeated once the end of the record is reached)

The 'optimised' groyne scheme therefore became the one in Figure D.2.

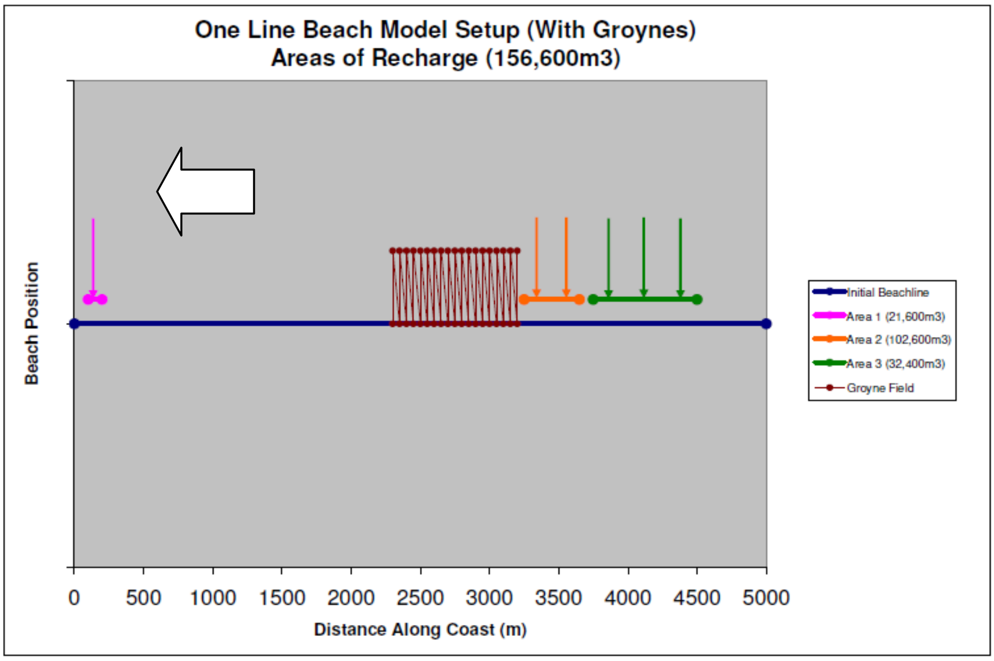


Figure D.2 Model set-up (scheme type C)

D.3 Results

D.3.1 Impact of changes to nourishment regime

These sets of runs compared the impacts of beach nourishment with a similar sized material (1.0 mm) and in the same placement locations, but altering the timing and quantity of nourishment.

The purpose of this was to determine the sensitivity of schemes to this change in management practice, for example, an operator choosing to alter the regime to better match funding requirements or to capitalise on opportunities for material sourcing.

Comparisons of the resultant modelled beach contour position were made for the month just prior to renourishment of the beach, that is, representing the most likely ‘worst beach’ position. Example results are provided in Figures D.3 to D.5b, which show the beach contour position versus chainage along the frontage.

Comparing ‘annual’ and ‘two-yearly’ scenario results four years into scheme (1995) and eight years into scheme (1999) indicated negligible difference in beach position for open beach schemes (that is, types A and B). Although there were notable differences between the performances of the different scheme types (that is, whether it was recharged or recycled), the beach contours between the two nourishment scenarios were consistent. A similar situation was observed for the ‘annual’ and ‘five-yearly’ scenario results five years into the scheme (1996).

Obviously the periods between the above output times do show differences between nourishment scenarios, as the quantities being placed are quite different at different times. But these results all suggest that, for an open beach scheme, the ‘maximum retreat’ position is not dramatically affected by the frequency of recharging as long as the total volume placed is matched.

In practice however it is not as simple as that. The way in which the material is placed will affect how it is dispersed and indeed whether losses (for example, offshore) might be greater or lesser. For example, with a greater quantity placed at one time (for example, five times the annual volume), it would be expected that the toe of the beach will be in deeper water, waves will be larger and potentially hitting the beach at a slightly different angle, and the outer section has the potential to erode more quickly. Depending on the subtleties of the model and the skill of the modeller to redefine the initial conditions in the model itself, these factors may well not get accounted for and a false impression of an alternative management approach may result.

With the introduction of beach control structures (scheme type C), the frequency of nourishment does seem to become more significant in the model itself. Within the groyne bays themselves, the beaches in the model tend to stabilise once the bays are filled in all cases, but sizeable differences in performance are seen downdrift of the groyne field.

Through comparison of the results, as a general rule the longer the interval between nourishment campaigns the greater the landward cut back of the beach directly downdrift of the groyne field, that is, 'five-yearly' is worse than 'two-yearly' which in turn is worse than 'annual' nourishment. That would suggest any changes in management regime could have profound implications and from these results it may be concluded that beach plan shape modelling becomes increasingly important with schemes where structures are incorporated. But those conclusions need to be caveated with the same points made above for open beach schemes: is the model or modeller actually reproducing the geometrical differences in beach line and slope as a result of placement and the differences in waves on the beach face?

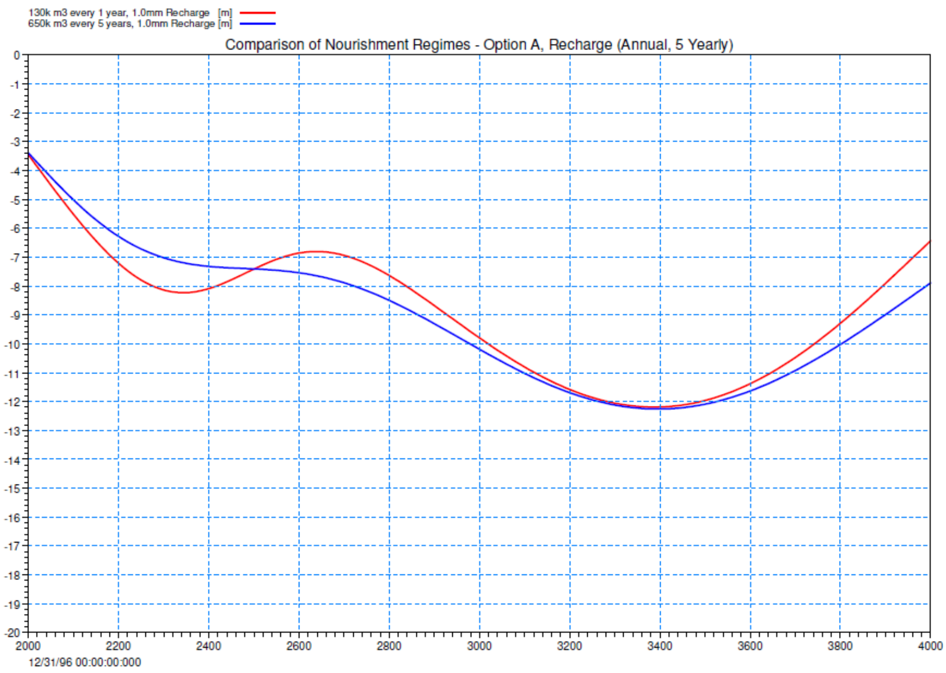


Figure D.3 Scheme type A – annual vs. five-yearly (after five years)

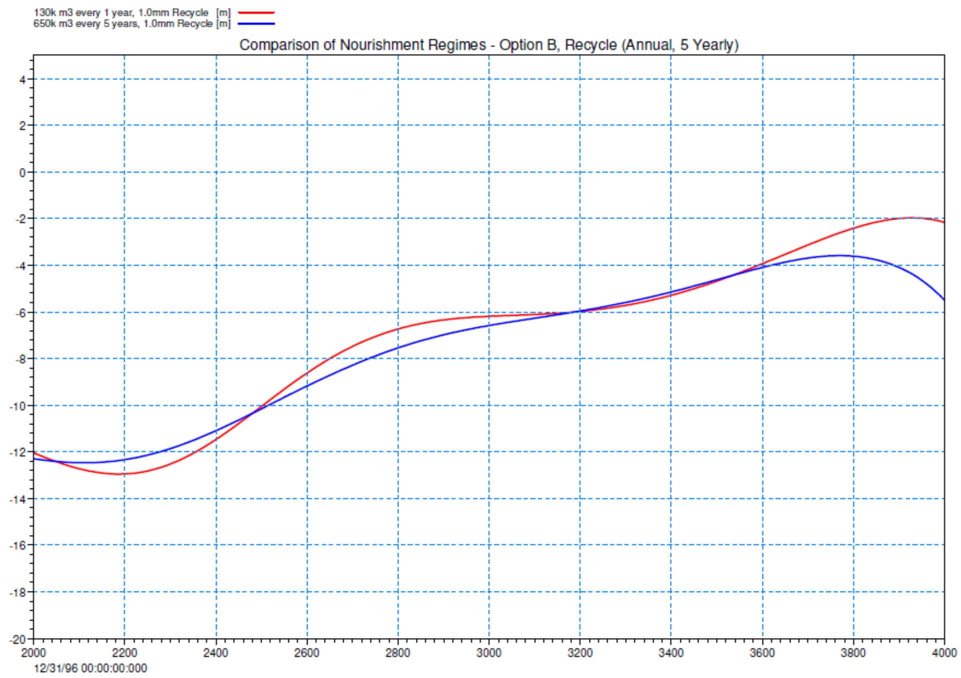
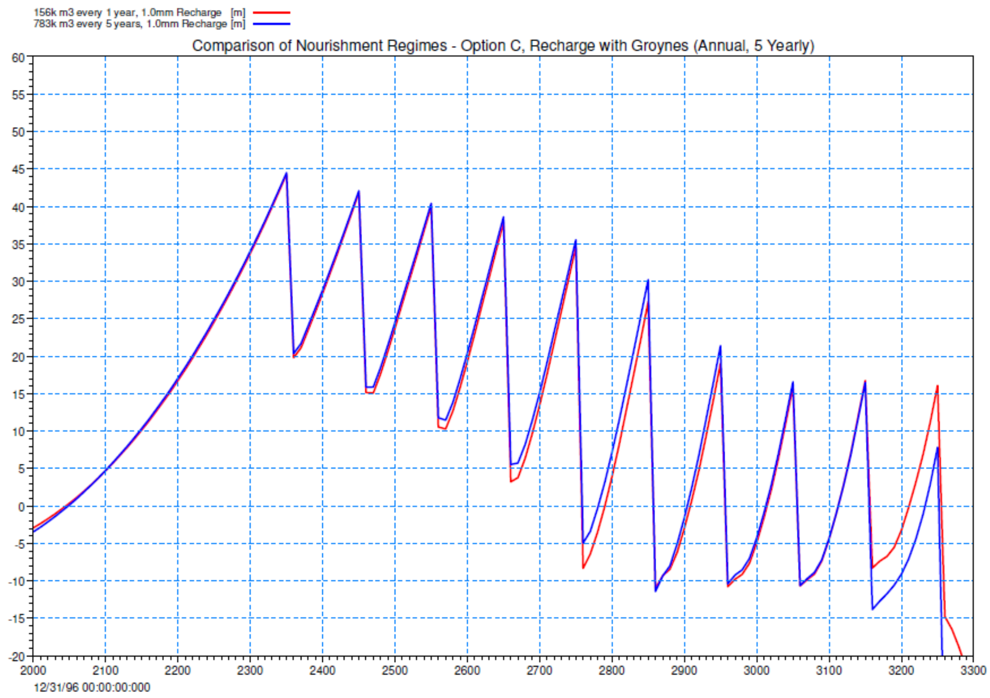
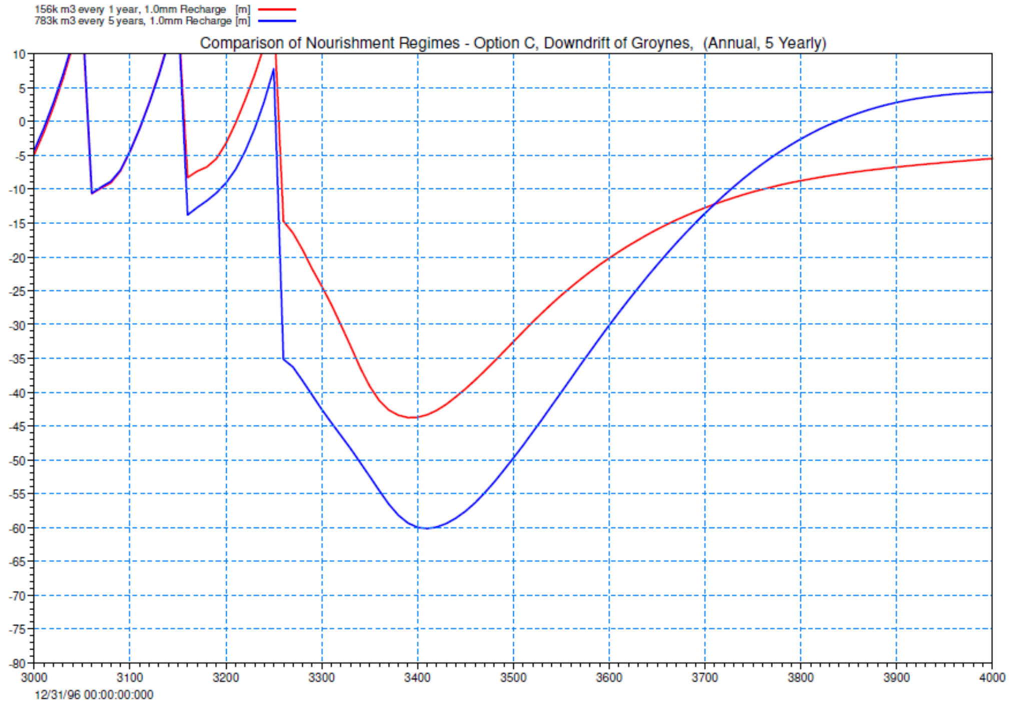


Figure D.4 Scheme type B – annual vs. five-yearly (after five years)



(a)



(b)

Figure D.5 Scheme type C – annual vs. five-yearly (after five years): (a) recharge with groynes and (b) downdrift of groynes

D.3.2 Impacts of variations in sediment size

These sets of runs compared the impacts of annual beach nourishment with similar volumes in the same locations but using different size material.

The purpose of this was to determine the sensitivity of schemes to this change in scheme implementation, for example, availability of actual material not matching that expected to be placed by the modelling. This could result from lack of available information at the time of design, or an alternative source being used either due to the original source no longer being available or for commercial reasons.

For this exercise, comparisons were made of the annual drift rates produced by the model at different locations along the frontage, the plot for one of which is shown in Figure D.6.

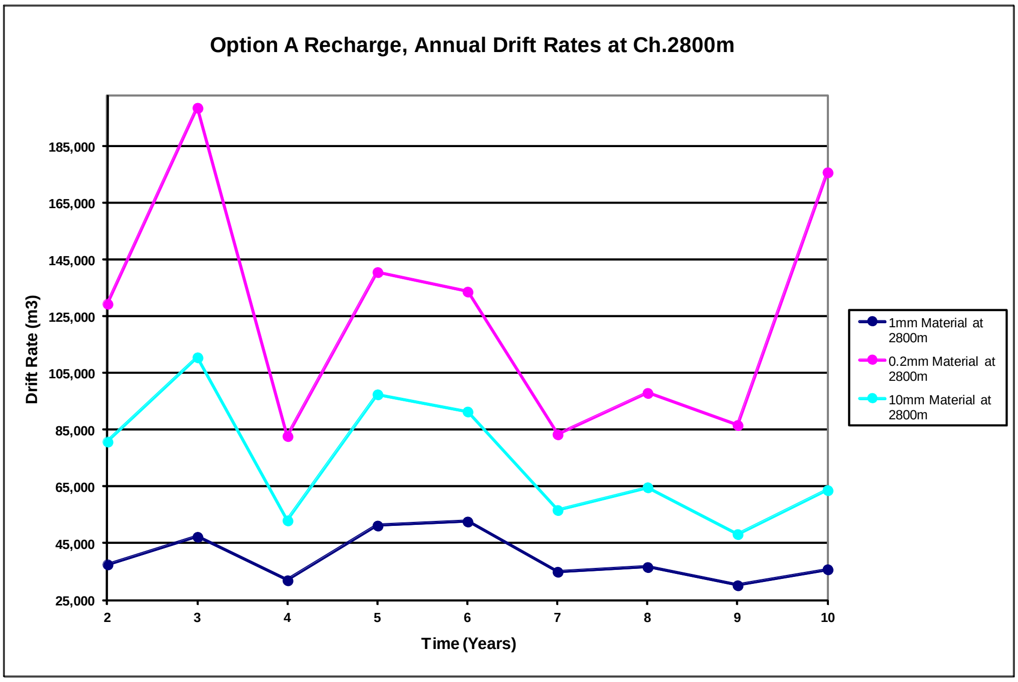


Figure D.6 Scheme type A – net sediment drift rates

Comparing all sand size runs for the open beach scheme type A (scheme type B, recycling, not being applicable here), outputs for smaller (0.2 mm) sediment sizes show substantially higher rates of transport than in the base case (1.0 mm). This is as would be expected and observed in nature.

The first notable point here is the magnitude of difference with rates for the finer material is typically up to 3–5 times that of the base material. This could have significant implications for beach management with a much higher maintenance commitment than expected if all, or even part, of the actual sourced renourishment was finer than intended. The second point of note is that the difference in rate between the two material sizes is not a constant factor year-on-year. This is because the finer material has much higher gross drift in both directions and, while under some wave conditions the increase in rate is only a factor of 1.5 times greater, under other wave conditions it is 5–10 times greater. This again is significant when thinking about how to design and manage a beach. Where material is likely to be or is found to be of a different size to that modelled, it will behave differently to that expected and not necessarily in a linear fashion. If the design is reliant upon or sensitive to this, then the scheme ought to be remodelled to reflect this change in a key variable.

In addition, further model runs were carried out for a 10 mm shingle size beach material. This did not appear to produce different beach behaviours, simply different drift rates but following the same overall trends as observed for the sand sized materials. However, what is significant is that the rates of transport output by the model are higher than those observed for the 1.0 mm material, which is not what would be expected to occur in nature. On further inspection of the (daily output) data, it is clear that in fact the increase is not consistent either; under some wave conditions the drift of the coarser material is higher and under other conditions it is lower than the finer sand material. This leads to two conclusions. The first is similar to that above; it should not be assumed that there will be a consistent and linear relationship between beach performance for two different material sizes. The second, and possibly more important, is that this result most probably illustrates the reliability of outputs where a variable used in a model is outside of its theoretical limits of applicability, or perhaps where other factors within the model are not also adjusted to accommodate that fact (for example, the sediment transport calculation may not be simply a function of grain size). In short, calibration of models under these circumstances is critical.

For a groyned beach (scheme type C), considerable differences were also observed between the results for different sediment sizes, but trends altered with time (see Figure D.7). This is because the finer sand (0.2 mm) was in this example found to fill and stabilise within the groyne bays very quickly, after which higher rates of transport occurred as material could move alongshore beyond the ends of the groynes. By comparison the 1.0 mm sand never filled the groyne bays fully and so very low rates were experienced.

A similar albeit slightly later occurrence is observed for the 10 mm shingle, but as with the open beach scheme type above, in practice this would not be expected to have responded in a manner closer to the finer material and would be in fact be more likely to replicate that seen for the 1.0 mm sand size in this instance. The same conclusions made above for the open beach will apply here too.

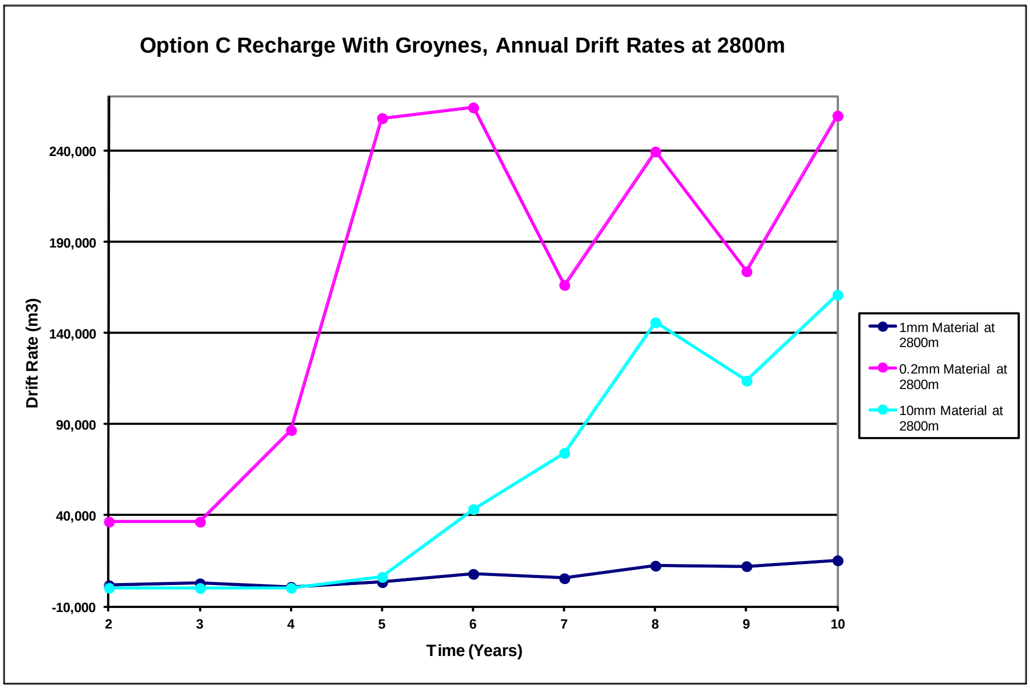


Figure D.7 Scheme type C – net sediment drift rates

From all of the above, it is concluded that:

- the choice of sediment size used in the model does have a dramatic impact upon results and thus scheme design for all types of beach scheme
- altering the size of material at scheme implementation from that which was modelled, potentially renders the original design obsolete unless remodelled to reflect those changes and the design is revised accordingly

It is also important to understand whether the model has been set up and calibrated correctly for the specific size of material being analysed.

D.3.3 Impacts of variations in wave conditions

The initial 'base cases' used a 10-year wave dataset (1 January 1991 through to 31 December 2000) – see wave rose (WAVES004) in Figure D.8 (note plots re-orientated to correspond with the beach plots).

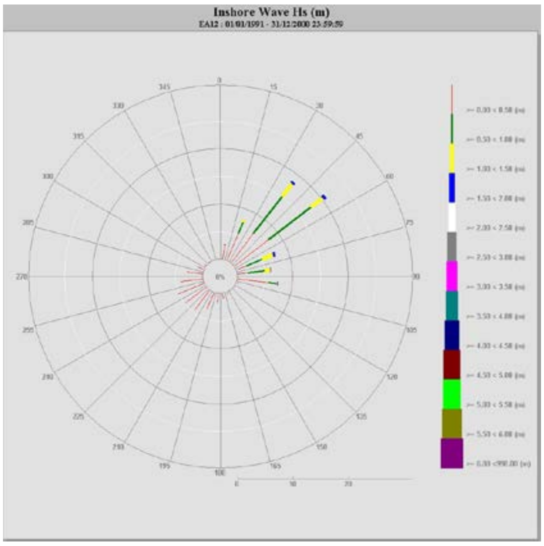


Figure D.8 Baseline wave data rose

Further sets of runs were undertaken to examine the impacts of variations in the wave climate. Two ‘synthetic’ time series wave inputs were generated from the same data to investigate and illustrate the risks and limitations of the repeated use of a ‘short’ dataset in situations where a sufficiently long dataset may not exist, or a situation where the actual wave climate simply differed from the data available at the time of modelling.

To produce these additional datasets, years with notable variation in distribution of wave energy from the 10-year dataset were identified, these being in years 1993 and 1995 (see wave roses in Figure D.9). For example, year 1993 shows considerably more wave activity from the 45–60° sector than the 10-year average, while year 1995 shows higher energy than average arriving from the 30–45° sector and less from 45–60°.

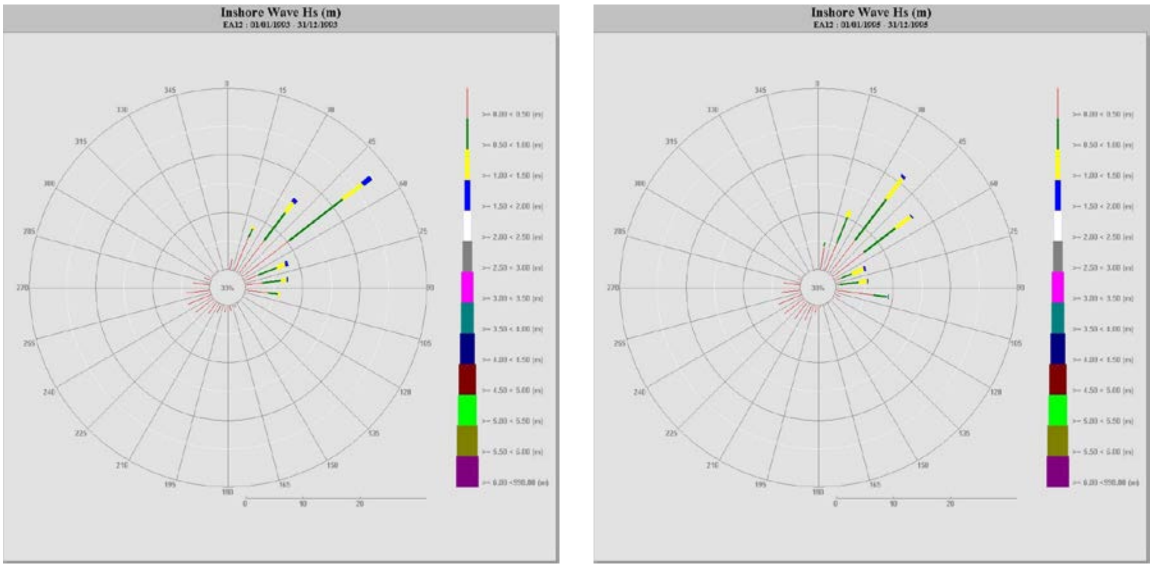


Figure D.9 Modified wave data roses: (a) 1993 and (b) 1995

In constructing the two synthetic datasets, to ensure that the first year of beach evolution prior to nourishment was comparable with the base case, the same initial year of wave data were used, followed by a new dataset repeating one of the above annual climates. So, the first synthetic dataset (WAVES008) uses the data for 1991 once followed by the data for 1993 duplicated nine times to create a 10-year dataset.

The second synthetic dataset (WAVES 009) uses the data for 1991 once followed by the data for 1995 duplicated nine times to create a further 10-year dataset.

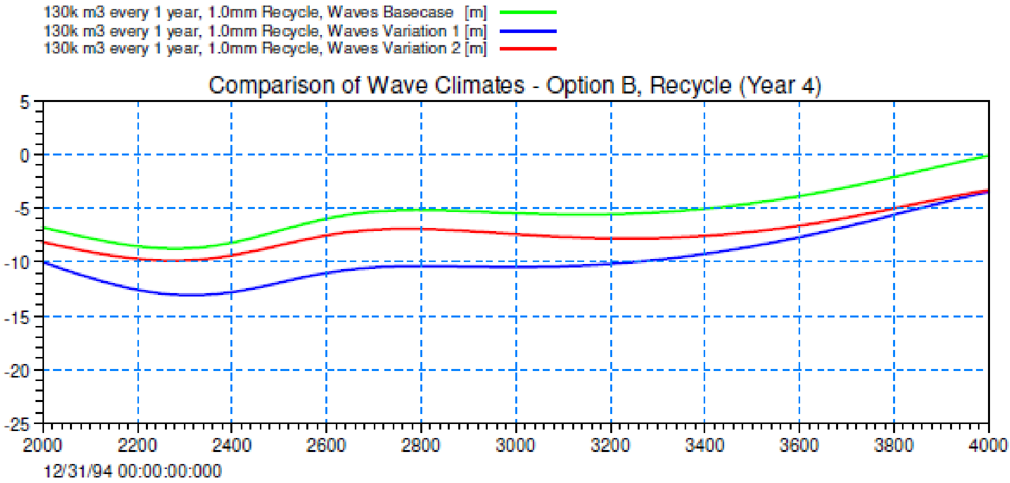
Using these modified wave datasets, the nourishment base cases were remodelled (annual nourishment with 1.0 mm sediment size) and compared. Both beach position and drift rates were output and examined; some example outputs are provided in Figures D.10 to D.13.

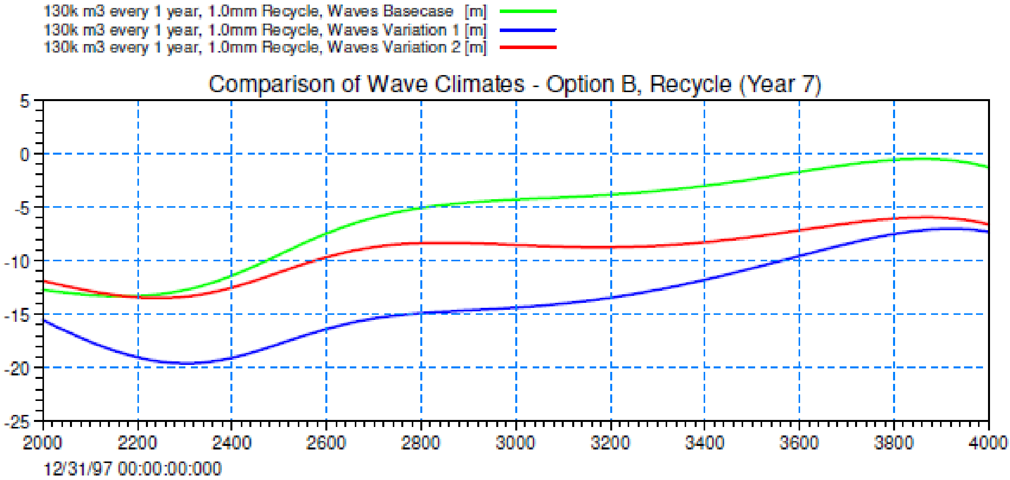
Unsurprisingly, it in all cases tested the resultant beach position does differ, and within the test period, differences of 10–20 m occurred for both of the open coast scheme types (types A and B) despite what is a relatively modest shift in the wave energy. In practice, this level of cut back could be quite significant for management of a beach and providing a particular standard of protection if not accounted for. This illustrates the importance of having best possible definition of the wave climate for modelling, and where there is uncertainty, modelling sensitivity to slight changes in the wave climate used.

For the groyned beach (scheme type C), differences in the beach positions within the groyne bays is also noted (up to 5–10 m) as the alignments of the beach face are not the same (due to the subtle differences in direction of the prevalent wave fronts), and cut back downdrift of the groyne field was between 30 and 50 m greater than in the base case.

Further runs were then conducted for the other sediment sizes (0.2 and 10 mm) to assess whether similar trends were observed or not. For the open beach schemes (types A and B), considerable differences were observed but not consistently between the two sediment sizes. For the more constrained groyned beach (scheme type C), the development of the beach up until the groyne bay filled showed different rates and patterns depending on the wave climate. There was no discernible pattern or consistency between the development of these beaches across the three different sediment sizes. The conclusion to be drawn from these latter runs is that the influence of different wave climates will also produce a different response depending on the size of the sediment and that response may not follow the same pattern to that of a beach of different material.

These results illustrate that even modest changes in wave climate can have a significant impact upon the outputs from beach models. This can be the result of only having a small sample dataset with which to run the model and draw long-term conclusions from, or even where a longer term dataset exists, the actual conditions still differing from those used and having a relatively instantaneous effect. From this it is concluded that the sensitivity of any scheme to potential variability in wave conditions needs to be examined and accounted for when using beach plan modelling. Furthermore, where beach management is an ongoing process, it would be prudent to update the wave record and re-run the model to confirm continued validity of current regime and determine any appropriate modifications to it.





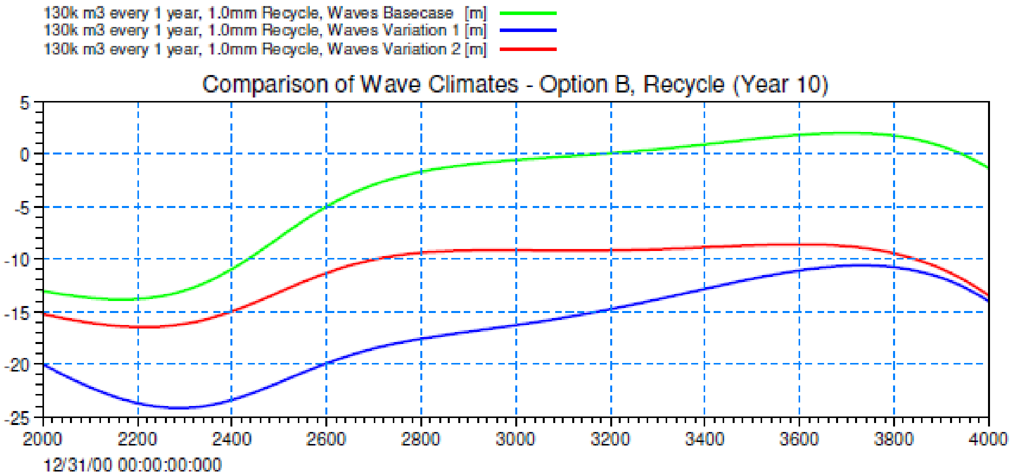


Figure D.10 Scheme type B – comparison of impact of wave climates on beach position

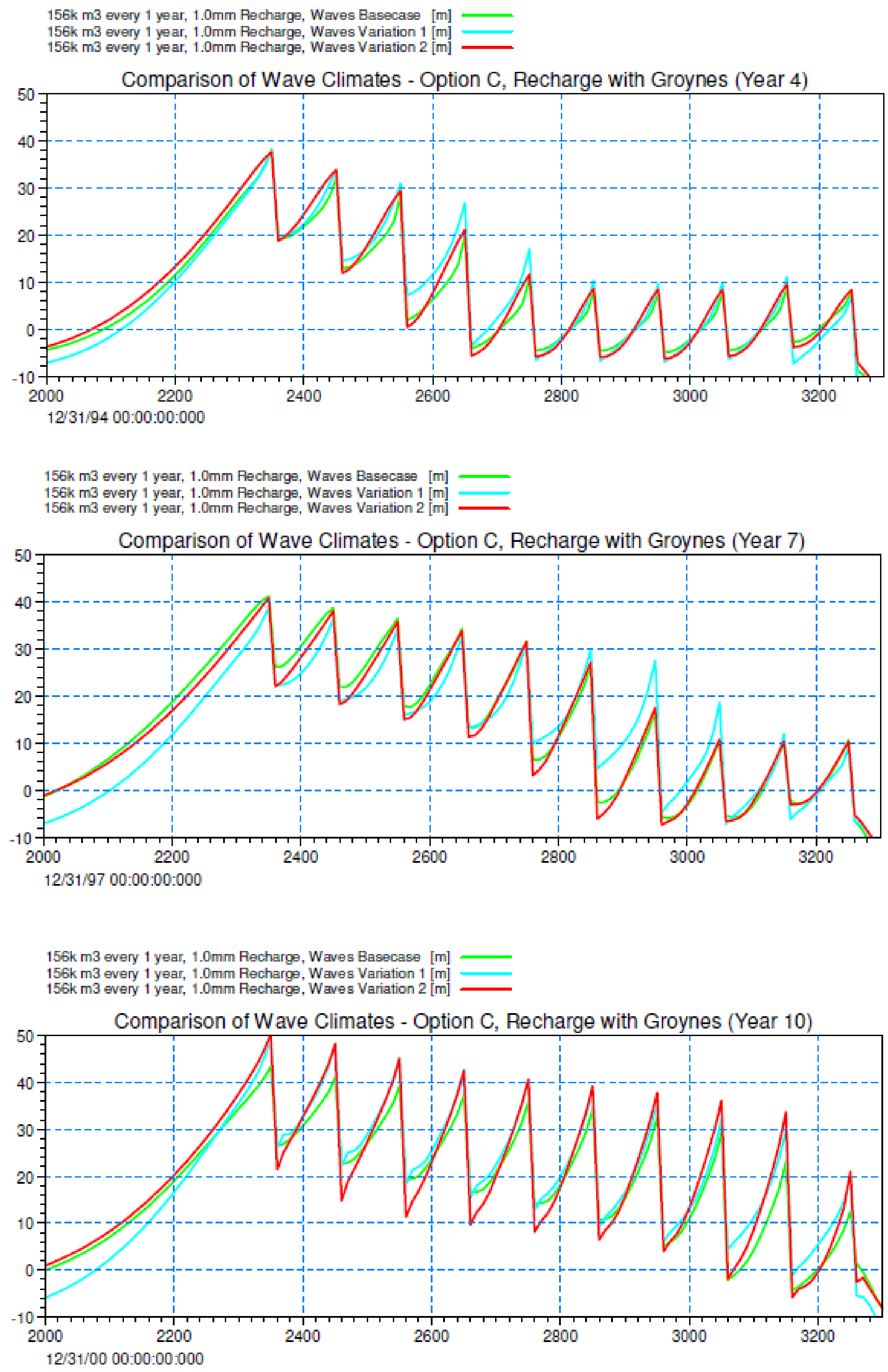


Figure D.11 Scheme type C – comparison of impact of wave climates on beach position

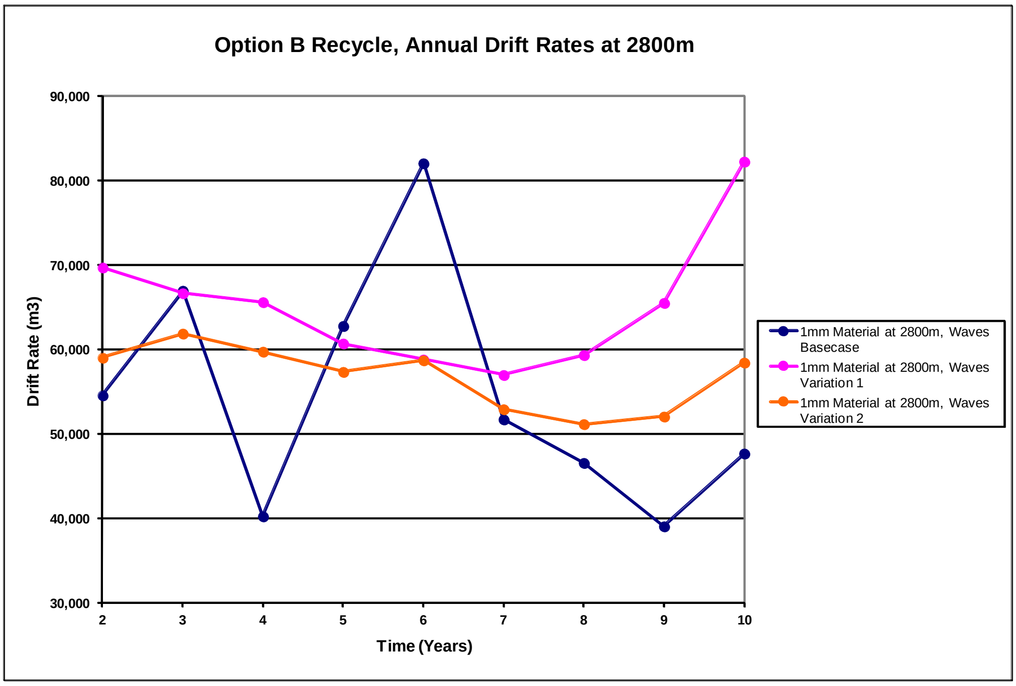


Figure D.12 Scheme type B – comparison of impacts of wave climates on drift rates

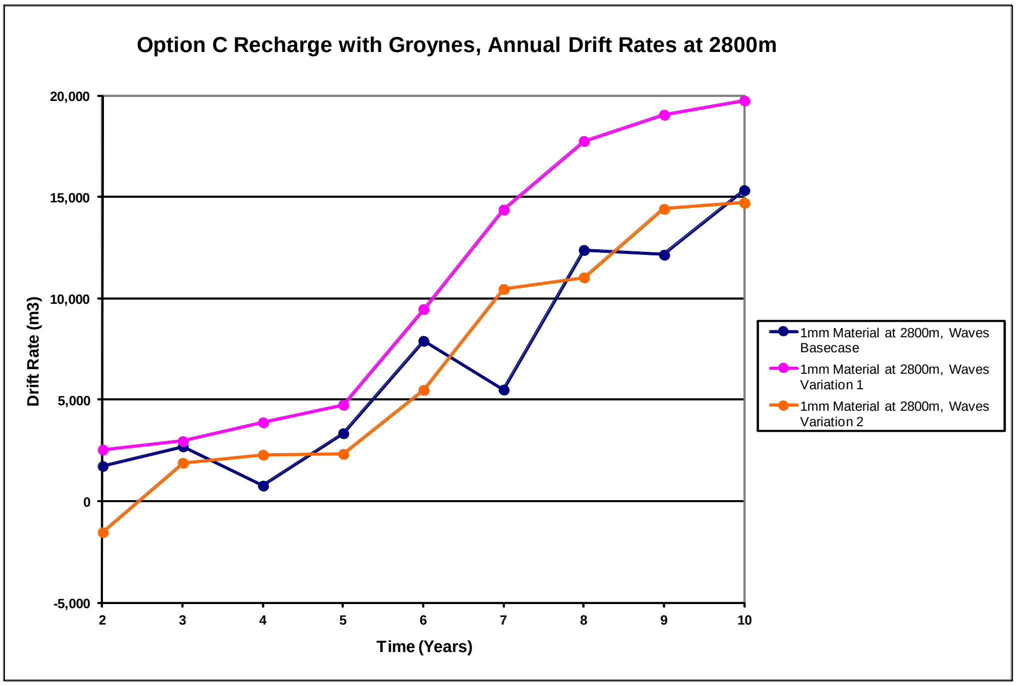


Figure D-13 Scheme type C – comparison of impacts of wave climates on drift rates

D.4 Conclusions

From the non-location specific one-line beach plan shape modelling discussed above, the following conclusions are derived for consideration by those commissioning or undertaking such modelling in the future.

- The choice of sediment size used in the model does have a dramatic impact upon results and thus scheme design for all types of beach scheme.
- Altering the size of material at scheme implementation from that which was modelled, potentially renders the original design obsolete unless remodelled to reflect those changes and the design is revised accordingly.
- It is important to understand whether the model has been set up and calibrated correctly for the specific size of material being analysed.
- The sensitivity of any scheme to potential variability in wave conditions needs to be examined and accounted for when using beach plan modelling.

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