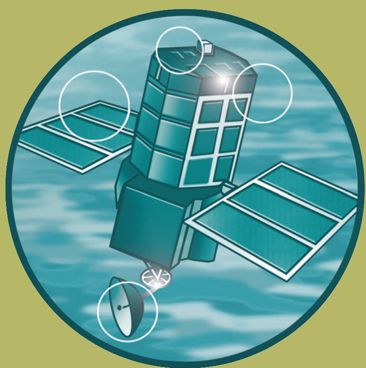
Joint Defra/EA Flood and Coastal Erosion
Risk Management R&D Programme

Understanding the lowering of beaches in front of coastal defence structures, Stage 2

R&D Technical Report FD1927/TR



Environment Agency

defra
Department for Environment
Food and Rural Affairs

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

Understanding the Lowering of Beaches in front of Coastal Defence Structures, Stage 2

R&D Technical Report FD1927/TR

Produced: May 2007

Author(s): J. Sutherland, A.H. Brampton, C. Obhrai, S. Dunn and R.J.S. Whitehouse

Statement of use
This report is intended to be used, in the first instance, by those involved in managing flood and coastal defence assets (particularly coastal defence structures with a sand beach) and assessing the risks associated with coastal defence structures and systems. The report includes new information on the behaviour of beaches in front of coastal defence structures at a range of time and space scales. This will enable better informed decisions to be made about data collection, monitoring and intervention. Moreover, it will assist in the appropriate design, construction and maintenance of coastal defence assets. However, it does not constitute official government policy or guidance.

Dissemination status

Keywords: Beach lowering**,** coastal defence, toe scour, mitigation, monitoring, liquefaction.

Research contractor: HR Wallingford Ltd.

Defra project officer: Stephen Jenkinson

Publishing organisation
Department for Environment, Food and Rural Affairs
Flood Management Division,
Ergon House,
Horseferry Road
London SW1P 2AL

Tel: 020 7238 3000 Fax: 020 7238 6187

www.defra.gov.uk/environ/fcd

© Crown copyright (Defra);(*2008*)

Copyright in the typographical arrangement and design rests with the Crown. This publication (excluding the logo) may be reproduced free of charge in any format or medium provided that it is reproduced accurately and not used in a misleading context. The material must be acknowledged as Crown copyright with the title and source of the publication specified. The views expressed in this document are not necessarily those of Defra or the Environment Agency. Its officers, servants or agents accept no liability whatsoever for any loss or damage arising from the interpretation or use of the information, or reliance on views contained herein.

Published by the Department for Environment, Food and Rural Affairs. Printed in the UK, (August 2008) on recycled material containing 80% post-consumer waste and 20% chlorine-free virgin pulp.

Executive summary

Background needs

A research scoping study (Project FD1916) on beach lowering in front of coastal structures identified the generic elements and processes involved. It also highlighted key shortcomings in the available knowledge for which substantive progress could be made in the short term. This project (FD1927) was the direct follow on to the scoping study and addressed a number of the identified shortcomings. This Technical Report for Users, FD1927/TR, does not update the research scoping study, which reviewed the whole area, but reports on the advances made during project FD1927 (2005-2007).

Main Objectives/Aims

The objectives of this research project were to:

1. Synthesise existing information and approaches to predicting general beach lowering and clearly summarise the implications for beach monitoring. This objective is fulfilled in Section 3 of FD1927/TR.
2. Screen existing scour prediction methods and produce an improved method resulting in less uncertainty. The scour prediction method should be suitable for use both in design and in a risk-based methodology of asset management. This objective is fulfilled in Sections 4 and 6 of FD1927/TR.
3. Provide information on mitigation schemes. Information has been collected on a number of mitigation schemes to assess how well they performed. This objective is fulfilled in Section 5 of FD1927/TR.
4. Assess liquefaction potential of the sediment in front of coastal structures. This objective is fulfilled in Section 7 of FD1927/TR with the development of a new tool to assess the potential for momentary liquefaction.

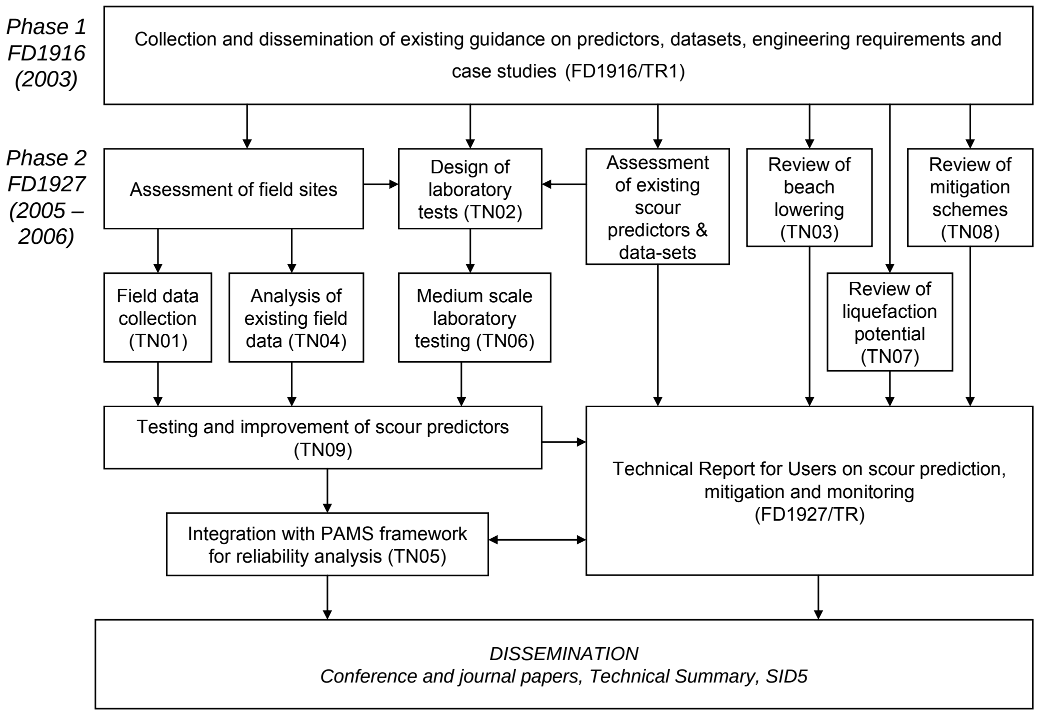
Structure of research project

The project was undertaken in a number of linked stages, which resulted in the production of nine Technical Notes during the course of this project, all of which will be made available as project outputs. These are listed below:

- HR Wallingford, 2006a. Beach lowering and recovery at Southbourne (2005). *HR Wallingford Technical Note CBS0726/01,* with University of Southampton.
- HR Wallingford, 2006b. Design of physical model scour tests. *HR Wallingford Technical Note CBS0726/02*.
- HR Wallingford, 2006c. Assessment of beach lowering and toe scour. *HR Wallingford Technical Note CBS0726/03.*
- HR Wallingford, 2006d. Scour monitor deployment at Blackpool. *HR Wallingford Technical Note CBS0726/04*.
- HR Wallingford, 2006e. Integrating scour research into reliability analysis of coastal structures. *HR Wallingford Technical Note CBS0726/05.*
- HR Wallingford, 2006f. Medium scale 2D physical model tests of scour at seawalls. *HR Wallingford Technical Note CBS0726/06*.

- HR Wallingford, 2006g. Wave-induced liquefaction of sediment in front of coastal structures. *HR Wallingford Technical Note CBS0726/07.*
- HR Wallingford, 2006h. Mitigation methods. *HR Wallingford Technical Note CBS0726/08.*
- HR Wallingford, 2006i. An improved scour predictor for sand beaches. *HR Wallingford Technical Note CBS0726/09.*

The structure of the project and the positions of the Technical Notes within the project are shown in Figure 1, where TN01 refers to Technical Note CBS0726/01, etc. Technical Notes 1, 2, 4 and 6 all provide the inputs to Technical Note 9. Technical Note TN03 also provided input into Technical Note TN05, although this is not shown.



Project map showing where Technical Notes (TNs) and Technical Report for Users (FD1927/TR) fit into project structure

Summary

Development of improved scour predictors

Toe scour can reduce the level of the beach in front of a structure and increase the risk of undermining.

1. An extensive literature review and assessment of existing datasets have been used to identify laboratory test datasets that include measurements of toe scour and maximum wave-induced scour in front of vertical or

sloping seawalls with wave heights sufficiently high to generate suspended sediment transport. A set of new laboratory experiments was then undertaken to extend this dataset.

2. The combined laboratory dataset was then used to derive equations representing low and high values of relative water depth for toe scour depth and for the maximum scour depth.
3. The maximum toe scour depth, $S_{t,max}$ on sandy beaches was predicted not to exceed the deep water value of significant wave height, H_{s0}, i.e. $S_{t,max} < H_{s0}$.
4. Statistical analysis gave a Root-Mean-Square error in the predicted values of relative scour depth of about 0.17. A single equation was derived to calculate toe scour depth as a function of h_t/L_m, although it has systematic and unsystematic errors. An equation was then derived as an alternative with zero systematic error and zero bias, but a slightly higher total root mean square error. The predictor takes beach slope as well as relative toe depth into account and provides significant additional predictive capability for seawall scour in sand beaches.
5. On shingle beaches the predictor of Powell and Lowe (1994) is recommended.
6. The medium-scale flume tests, from which the sand beach toe scour predictors were derived, have used a limited range of wave heights and bed sediments. A set of large scale flume tests is therefore recommended to discriminate the effect of wave height and sediment size on scour depth.

Assessment of seabed liquefaction

Wave induced liquefaction can reduce the bearing capacity of the seabed in front of a structure.

1. An analytical solution for the wave-induced pore pressure response in an isotropic infinite thickness seabed in front of a breakwater was used to study the liquefaction potential of the seabed in front of coastal defence structures subjected to various wave loadings. The liquefaction potential was determined by calculating the minimum total wave height to depth ratio that will cause the momentary liquefaction of the top 0.05m of a sandy seabed in front of a vertical seawall. The liquefaction potential depends on the degree of saturation of the pore water in the sediment which affects its compressibility.
2. Calculations were made for fine, medium fine and coarse sand with the degree of saturation between 0.90 and 1.0, for a range of water depths and a typical storm wave period of 8s. The results can be used to indicate whether liquefaction of the seabed in front of a coastal structure is likely to occur. If so, a more detailed study should be carried out.
3. The likelihood of the occurrence of momentary liquefaction of the seabed increases with a decrease in seabed permeability, which is associated with a decrease in grain size. A seabed of fine sand is therefore more likely to experience momentary liquefaction than a seabed of coarse sand.
4. The likelihood of the occurrence of momentary liquefaction increases with a decrease in the degree of saturation of the seabed.

5. The wave height required to liquefy a fine sand seabed increases significantly when the degree of saturation of the seabed increases higher than 0.995.
6. An S_r-value of 0.95 is recommended for the estimation of the minimum wave height required to liquefy the seabed, in the absence of a site-specific study.
7. Graphs have been developed that can be used to provide a quick check on the potential for momentary liquefaction of the top 0.05m of the seabed. If the potential for momentary liquefaction exists, a more detailed, site-specific study can be carried out by adapting the Mathcad code developed in FD1927, or using another liquefaction model.

Integration into reliability analysis

The results of the research can be implemented in the existing analysis methodologies used to determine the performance of coastal structures.

1. The derivation of the improved scour predictor will inform the future development of fault trees and fragility curves for coastal structures which are known to be sensitive to scour.
2. The identification of the Gaussian distribution of beach levels about a long term trend will reduce the uncertainty in the calculation of the fragility curves of coastal defences at a particular time.
3. The forecasting of changes in the mean beach level in front of a coastal defence, whether through extrapolation of historical data or numerical modelling, will allow the change in the fragility curve with time to be calculated. This contributes to calculations of the deterioration in asset condition with time and could be used to trigger a more detailed form of condition assessment, rather than waiting for scour to be observed or failure to occur.
4. In regions where beaches are regularly monitored the use of a visual condition index for a beach should, in time, be replaced by a quantitative measure of beach performance derived from the measurements. This may require the development of suitable coastal state indicators and of methods to determine suitable threshold levels for them.
5. The development of a method for calculating prediction horizons will inform the duration of the forecasts of future beach behaviour (if sufficient data is available) that can usefully be used in coastal management.
6. The frame of reference approach has been identified as a useful way of linking management objectives to the technical solutions that are used to meet those objectives and evaluating their success.

Prediction tools, shoreline retreat, uncertainty and coastal state indicators

1. Different tools are needed to predict the response of the coastline at different scales. These tools come with different levels of reliability, accuracy, skill and required expertise. These tools may be allocated to one of four basic types: statistical analysis, process-based numerical modelling, geomorphological analysis and parametric equilibrium models. The numerical models attempt to describe fewer and fewer processes in detail as the spatial and temporal scale they are deployed

over increases. The suitable time and space scales have been illustrated for the different model types.

2. In the coastal regions where the Bruun rule can be said to apply, the rate of shoreline retreat is directly proportional to the rate of sea level rise. It follows that the ratio of future shoreline retreat rate to present day shoreline retreat rate (the shoreline retreat rate multiplier) will be the same as the ration of future sea level rise rate to present day sea level rise rat. calculated using present day rates of sea level rise and regional sea level allowances (Defra, 2006). These calculations show that shoreline retreat rates in regions where the Bruun rule applies could increase significantly – in some cases by a factor of 13 - during the 21st century.
3. The shoreline retreat rate multipliers are highest for the Northwest and Northeast of England and Scotland as this region has the lowest present day rate of sea level rise, due to isostatic rebound following the last ice age, which may also imply lower rates of present day shoreline retreat. The systems model SCAPE (Walkden & Hall, 2005) predicted a rather more complex response, with lower overall vulnerability to sea level rise, than the Bruun rule. Therefore the magnitudes of the shoreline retreat rate multipliers should be treated with some caution as they may well be too high. It seems probable that the shoreline recession rate will increase in many places if the rate of sea level rise increases.
4. The uncertainty in shoreline position from OS tidelines is a combination of source uncertainty, interpretation uncertainty and natural variability. Methods for calculating each of these uncertainties have been developed and example values calculated. These errors can be incorporated into analyses of historical shoreline movement, which are often used in Shoreline Management Plans and strategy studies.
5. The best coastal state indicator for assessing the contribution of a beach to the overall risk of flooding or erosion is not yet known. Possible coastal state indicators include the beach level at the toe of a structure, the beach level plus beach slope and the beach cross-sectional area above a set contour.

Beach levels – measurement, results and analysis methods

1. Advances in measurement technology have made it easier than ever before to measure beach levels at a point on a daily basis, or even more frequently.
2. Many of the possible techniques for measuring time series beach levels at a point have not yet been evaluated for the cases of beach level at the toe of a structure.
3. When beach levels have been measured at a point at a rate of at least once a day the time series have generally been short or, in the case of the Blackpool Tell-Tail data, there has been no tie-in to other data gathering programmes. The Blackpool data would have been more useful if it had been collected within an integrated beach monitoring programme.
4. There is a gap in frequency of data collection between the point measurements of beach levels through a tide (sampling about 4 times

per hour) and beach profiles (collected typically 4 times per year). It is therefore impossible to determine from the data if the beach variations at the two frequencies are related, although changes in the beach level at the toe of a structure between tides are the residual of the changes within each tide. An implicit assumption that the processes are unrelated has been made in incorporating these results into the development of fragility curves.

5. A clear seasonal trend was observed in the Linconshire dataset of beach levels at the toe of the local seawalls. The trend was lower than the standard deviation about the trend.
6. Beach levels at a point in front of a structure can generally be de-trended using a simple linear least squares method, providing that neither the coastal defences nor beach management policy changed during the data collection period.
7. Further data analysis should be undertaken to ascertain if a regional approach could be taken to providing guidance on possible changes in beach levels, for use in the design of new structures, in a similar way to the regional net sea level rise allowances (Defra, 2006).
8. There is a need to establish the relationship between the behaviour of a single contour and that of the beach volume at a local level before a contour can be used as a surrogate for beach volume.
9. Beaches around Donna Nook in north Lincolnshire often showed an increase in beach volume (area under a beach profile) combined with a retreat (shoreward movement) in Mean Sea Level. Further south in Lincolnshire (between Mablethorpe and Skegness) beach volumes were found to increase as the Mean High Water advanced (moved seawards).
10. Advanced linear analyses of beach level data (such as the use of wavelets and Empirical Orthogonal Functions) and nonlinear analyses of beach level data (such as Singular Spectrum Analysis and fractal analysis) are becoming more common in academic circles. These sophisticated methods require more data of good quality than the simple linear methods require. They may also impose more constraints on the data, such as the need to be equally spaced in time and position. It will be possible to apply these methods to more areas of the English and Welsh coastlines as coordinated regional data gathering and data management programmes extend their geographical range and temporal duration.

Results obtained from analysis of beach levels

1. Beach surveys that are intended to predict the long-term trends in shoreline position should be made when the standard deviation in the beach level is low. This occurred in August for the Lincolnshire data (but in June/July for Duck, N.C., U.S.A.) and also coincided with relatively high beach levels.
2. Beach surveys that are intended to indicate how low beach levels can fall should be undertaken when the average beach level is low and the standard deviation in beach level is high. In Lincolnshire this occurred around March.

3. When a linear trend is extrapolated to provide a future prediction of beach level is has a site-specific prediction horizon. This is the average length of time over which an extrapolated trend produced a useful level of prediction compared to a baseline prediction (taken to be that future beach levels will be the same as the average of the measured beach levels).
4. Extrapolation of a linear trend fitted to 10 years of data gave prediction horizons between 0 years and 14 years.
5. Extrapolation of a linear trend fitted to 5 years of data always gave a negative skill score (i.e. a worse prediction than the baseline) so should not be used.
6. The use of extrapolated beach levels is more suitable for managed / adaptive beach management policies, rather than the precautionary approach as the latter has a longer timeframe than the longest prediction horizon.
7. The standard deviation in beach level from using 2 surveys per year was 6% different from using 10 surveys per year. The difference was approximately halved by increasing the number of surveys from 2 to 3 times per year.
8. Residual (de-trended) beach levels have a Gaussian distribution about the mean beach level, again providing that neither the coastal defences nor beach management policy changed during the data collection period.

Review of mitigation measures

The likelihood of beach lowering should have been included already in the design of any structure. In many cases the details of how a structure was constructed have been lost so, for example, the level of the structure toe is not known and the allowance made for beach lowering is not known. In some cases it will be possible to accommodate beach lowering by installing storm warning systems, delimiting areas to prevent development, increasing flood resilience, improving drainage, strengthening surfaces behind the structure to withstand higher flows or installing secondary flood defences to limit the extent of flooding.

Various approaches to reinforcing the beach or bed in front of a seawall have been tried over the years. Other techniques and structures have been applied to reduce the rate of shoreline retreat or to maintain a beach in front of a structure:

1. **Fagotting and wave breakers** have been used to reinforce shingle ridges protecting low lying areas from flooding. Most such installations are now redundant, with beach nourishment being widely used instead. The anticipated structural life of faggoting is generally low, possibly as little as 5 to 10 years, while timber wave breakers can be expected to achieve at least twice this lifespan, although they are vulnerable to being damaged by wave impact, and are prone to abrasion by beach sediments.
2. Fagotting and wave breakers may well be of some benefit as emergency works, or as short-term low-cost schemes in instances where more costly methods of protection are difficult to justify. Fagotting, in particular, is a technique that might be suitable in sheltered environments where abrasion is not a serious issue, where low cost is an overriding factor,

where manual labour is available at low rates, and where the possible need for maintenance is not a major concern.

3. **Scour mattresses** are typically deployed to prevent the undermining of structures, as bed levels near them are lowered by scour caused by the presence of the structures themselves. These mattresses, which are normally prefabricated, provide an interface between the normally solid and impermeable structures and the mobile, permeable sediments surrounding them. However, there are few instances of this type of protection being used within the intertidal beach zone anywhere in the UK. Their usage on the open coastline is generally restricted to backshore protection although there are a few examples of these being used as lightweight revetments at or above high water.
4. The availability of suitable **rock,** and greater awareness of the consequences of scour, has led to its increasing use in mitigating problems caused by beach lowering through the construction of rock blankets, toe berms and fillets. A good starting point for the design of rock (or concrete armour unit) structures in coastal engineering is the "Rock Manual" (CIRIA,1991, CIRIA / CUR / CETMEF, 2007).
5. The infilling of scour trenches, and the construction of a scour blanket or a sloping rock toe are designed to prevent the undermining of the structure. They are likely to suffer further beach lowering if the processes of beach erosion continue and this should be considered in their design. They are often installed at or below the beach level so may be covered for much of the time.
6. The construction of a more substantial rock fillet that extends partially up the height of the coastal defence structure may also serve to reduce one or more of wave run-up, overtopping, impact pressures, wave reflections and scour. Care must be taken in the design of rock fillets to ensure it does not increase run-up, overtopping and/or impact pressures by changing the way the wave break onto the structure.
7. The more substantial rock structures commonly have bedding layers or geotextiles to form an interface between the rock and the beach sediment.
8. **Detached breakwaters** are best suited for situations where the existing defences require higher levels of protection at sensitive points, i.e. over short frontages of coastline where beach lowering / scour are causing localised problems of overtopping or undermining. They may be well also suited to frontages where the wider beach formed can be justified for recreation/amenity purposes. In areas where there are large tidal ranges and/ or strong tidal currents, detached breakwaters will have more detrimental impacts than in micro-tidal or sheltered regions.
9. There are no low crested breakwaters or submerged reefs in the UK, but their applicability for mitigating problems of beach lowering or scour in front of coastal structures remains uncertain, based on the observations made elsewhere.
10. **Shore parallel sills** have been used with some degree of success in Mediterranean countries, but only in micro-tidal/moderate wave energy conditions. Their usefulness in high energy/macro-tidal conditions is yet to be proven. However, they have been successful as backshore protection, i.e. in conditions where the tidal range is not a critical factor;

11. The problems of beach lowering will be sufficiently severe and widespread in some situations, as to be worth considering a **major scheme to improve those beaches.** The direct remedy to such problems is to import large quantities of extra beach sediment, i.e. sand or gravel, to replace that gradually lost previously, i.e. a beach recharge scheme. This approach will immediately cover over the toe of coastal structures and decrease water depths in front of them, and in many cases will improve the amenity value and aesthetic appearance of the frontage. Further details on the design and execution of major beach improvement schemes is provided in the Beach Management Manual (CIRIA, 1996a).
12. There will often be a need for ancillary works to accompany an initial "recharge" of a beaches, such as the building of groynes, monitoring and analysis of the changes in beach levels and for periodic addition of extra material in later years. These various elements are now generally identified as components of a beach improvement scheme.
13. **Groynes** are the most widely used method of controlling beach levels in the United Kingdom, where they have been used in a wide variety of situations. On shingle beaches, groynes can be used in any tidal range and under most wave conditions. On sand beaches, however, groynes are most effective in low to medium tidal ranges, because their cost can be prohibitive in areas with large tidal ranges. The spacing of groynes is related to their length. Shorter, higher and closer spaced groynes are used on shingle beaches reflecting the steeper gradients that occur on such beaches, both perpendicular and parallel to the shoreline. In contrast, groynes on sandy beaches are longer, lower and more widely spaced, typically at twice their length or more.
14. Details of the design of groyne systems, with or without beach recharge, can be found in the CIRIA Beach Management Manual (1996a). It is worth noting, however, that the design of a groyne system should not be based purely on such guidelines alone; their design always needs to be matched to local conditions.
15. Installing groynes without recharge will normally lead to problems of erosion further along the coast in the direction to which the sediment is moving, (i.e. "downdrift") potentially extending over many kilometres. The greatest problems of erosion, however, tend to occur just downdrift of the last groyne, and may lead to the "outflanking" of a coastal structure if care is not taken to avoid this possibility. Groynes are often used in combination with beach recharge, and under such conditions model testing becomes almost mandatory.
16. Probably the most common adjustments to a coastal structure are **remedial works** such as underpinning, encasement, or addition of an apron to the wall, although there are few guidelines for design. Such works are rarely a permanent solution to the problem and hence they often have to be repeated later, either extending the protection downwards or further along the coastline. However, they do make maximum use of the existing structure which is often still sound, or can be repaired without great expense. Underpinning typically requires excavation beneath, and often behind, the face of the structure, the construction of a new and deeper "toe" and backfill of the area behind.

Encasement also involves the covering of the front face of the structure, and sometimes building above its existing crest and even over an existing back-slope, i.e. covering some or all of the original structure with a new, normally concrete, layer.

17. The construction of an apron or of steps at the base of an existing structure can prolong the life and improve the performance of that defence, at reasonable cost. However, such an intervention will not remedy the underlying causes of beach lowering. Such additions to a structure will extend it seaward, often occupying an area of the beach that previously provided an amenity area, and affecting the natural sediment transport processes in that area. There is a danger that such seaward extensions of a structure will interfere with longshore sediment transport, and hence reduce sediment supply to downdrift beaches.
18. In some situations the situation may have become so critical, that it is more cost effective to reconstruct/replace an old wall, rather than mitigate scour in front of it. Reconstruction will not remedy the underlying causes of beach lowering and may also interfere with longshore sediment transport.

Recommendations for future research and guidance

The work in FD1927 has identified the following research and guidance needs, some of which may be best suited to funding through a research council (RC).

1) Large scale flume tests of scour in front of coastal structures to confirm the scour relationships performed at a single (medium) scale (RC).
2) Tests on the stability of rock toe protection in front of coastal structures.
3) Detailed processes research and numerical model development to gain a better understanding of cross-shore sediment transport. Without this it will not be possible to produce a morphologically balanced model that can run for months to years without significantly and erroneously eroding or accreting a beach (RC). This could include the following item.
4) Combined field measurements of intertidal beach profiles and beach levels through the tide at a seawall at a timescale of months. This will relate the scour within a tide to the variations in beach level between tides to produce a better understanding of beach level variation in front of coastal defence structures over periods of weeks to months.
5) Continuing work to incorporate the results of this research into reliability analysis. This work is being undertaken within Floodsite and FRMRC and care should be taken to ensure that the outputs from these projects are taken up for general use within reliability analyses using, for example, RASP and/or the PAMS Operational Framework.
6) Work should be undertaken to attempt to quantify which coastal state indicator will best represent the contribution of beaches to overall flood risk.
7) Consideration should be given to the development of a toe scour guidance manual that would place toe scour in the context of coastal management policy, provide methods for predicting toe scour and provide guidance on the development and selection of mitigation options.

Contents

Figures

Tables

1. Introduction

1.1. Background to the project / Overview

A research scoping study (Sutherland et al., 2003, report FD1916/TR1) on beach lowering in front of coastal structures identified the generic elements and processes involved. It also highlighted key shortcomings in the available knowledge for which substantive progress could be made in the short term. This project (FD1927) was the direct follow on to the scoping study and addressed a number of the identified shortcomings. This Technical Report for Users does not update the research scoping study, which reviewed the whole area, but reports on the advances made during project FD1927 and refers readers to the relevant background material for greater detail.

1.2. Objectives of research project

The objectives of this research project were to:

1. Produce evidence for the phenomena of toe scour in front of coastal defence structures. This objective is fulfilled in Section 2.
2. Synthesise existing information and approaches to predicting general beach lowering and clearly summarise the implications for beach monitoring. This objective is fulfilled in Section 3.
3. Screen existing scour prediction methods and produce an improved method for use in design resulting in less uncertainty. This objective is fulfilled in Section 4.
4. Demonstrate how improved knowledge of beach lowering (particularly the development of an improved scour predictor) can be used in a risk-based methodology of asset management (the PAMS framework). This objective is fulfilled in Section 6.
5. Provide information on mitigation schemes. Information has been collected on a number of mitigation schemes to assess how well they performed. This objective is fulfilled in Section 5.
6. Assess liquefaction potential in front of coastal structures. This objective is fulfilled in Section 7 with the development of a new tool to assess the potential for momentary liquefaction.

1.3. Structure of research project

The project was undertaken in a number of linked stages, which resulted in the production of nine Technical Notes during the course of this project, all of which are available as project outputs. These are listed below:

- HR Wallingford, 2006a. Beach lowering and recovery at Southbourne (2005). *HR Wallingford Technical Note CBS0726/01,* with University of Southampton.
- HR Wallingford, 2006b. Design of physical model scour tests. *HR Wallingford Technical Note CBS0726/02*.

- HR Wallingford, 2006c. Assessment of beach lowering and toe scour. *HR Wallingford Technical Note CBS0726/03.*
- HR Wallingford, 2006d. Scour monitor deployment at Blackpool. *HR Wallingford Technical Note CBS0726/04*.
- HR Wallingford, 2006e. Integrating scour research into reliability analysis of coastal structures. *HR Wallingford Technical Note CBS0726/05.*
- HR Wallingford, 2006f. Medium scale 2D physical model tests of scour at seawalls. *HR Wallingford Technical Note CBS0726/06*.
- HR Wallingford, 2006g. Wave-induced liquefaction of sediment in front of coastal structures. *HR Wallingford Technical Note CBS0726/07.*
- HR Wallingford, 2006h. Mitigation methods. *HR Wallingford Technical Note CBS0726/08.*
- HR Wallingford, 2006i. An improved scour predictor for sand beaches. *HR Wallingford Technical Note CBS0726/09.*

The structure of the project and the positions of the Technical Notes within the project are shown in Figure 1.1, where TN01 refers to Technical Note CBS0726/01, etc. Technical Notes 1, 2, 4 and 6 all provide the inputs to Technical Note 9. The way the Technical Notes have been used to produce this Technical Report for Users is described in Section 1.4.

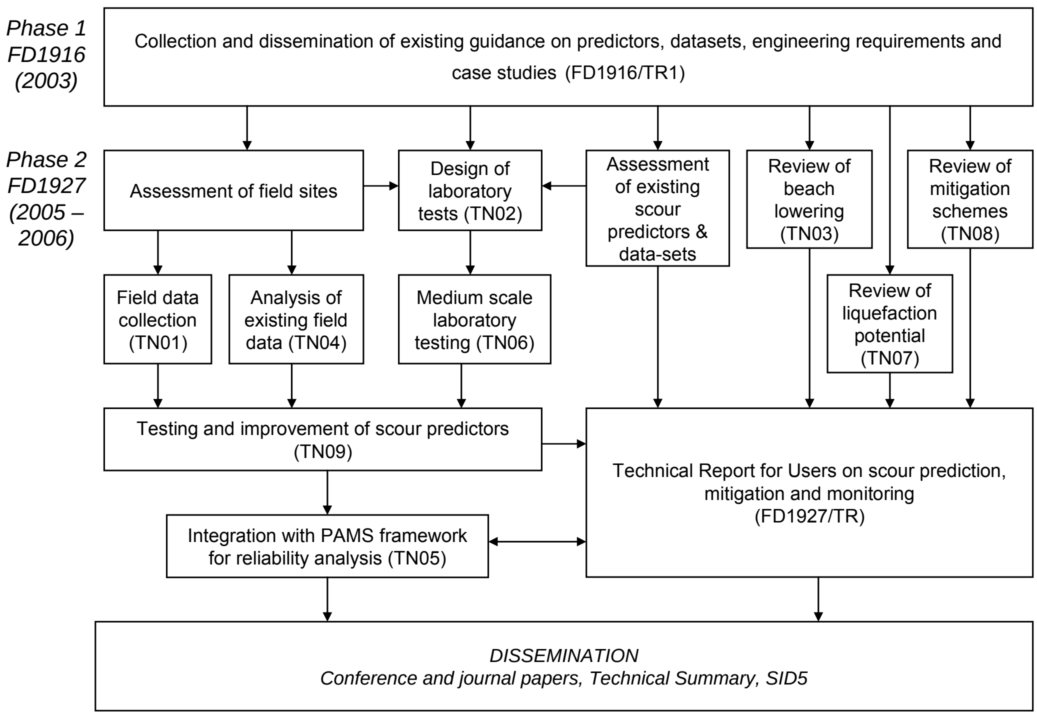


Figure 1.1 Project map showing where Technical Notes (TNs) and Technical Report for Users (FD1927/TR) fit into project structure

1.4. Structure of report

This report contains 8 main sections. Section 1 is this introduction, which gives the background to this research project, outlines its objectives, and the contents of this report. It also includes an expanded statement of use (Section 1.5), which shows where particular results can be obtained.

Section 2 provides evidence for beach lowering, which has been obtained from field measurements of beach levels during tidal inundation (HR Wallingford 2006a, d) and from time series of beach levels at a point in front of a coastal structure (HR Wallingford, 2006c). Both data sets illustrate the variability in beach levels in front of coastal structures.

Section 3 contains a summary of beach lowering and toe scour at different time and space scales (HR Wallingford, 2006c). This includes illustrating the Gaussian distribution of residual beach levels (after the removal of the long-term trend) and describing then demonstrating the use of a skill score to determine the useful length of prediction from extrapolating a 10-year time series of beach levels at a point. It also includes a detailed analysis of the measurement errors obtained from using tidelines on historic OS maps to determine the evolution of the beach.

Section 4 describes the development of an improved toe scour predictor (HR Wallingford, 2006i) which was one of the main outcomes of this project and which used results and knowledge gained from HR Wallingford (2006a, b, d, f) as appropriate. The main data sets are described, the data presented and the improved scour predictor is presented. Further work on this subject is continuing at the University of Southampton using data collected in the project.

Section 5 summarises the extensive review of the use of mitigation measures in the UK (HR Wallingford, 2006h). Four main categories have been identified: monitoring and accommodating scour, ancillary works, adjustment to the structure and major beach improvement methods.

Section 6 describes how the results from this project can be incorporated into ongoing projects on developing reliability analysis methods for coastal structures (HR Wallingford, 2006e).

Section 7 provides a quick and relatively simple method of assessing whether momentary liquefaction could occur in front of a coastal structure subject to wave action (HR Wallingford, 2006g). Liquefaction of the soil reduces the confining pressure at the toe of the structure or reduces the bearing capacity of the foundation. A Mathcad routine has been written to do this analysis which is included in HR Wallingford (2006g).

Section 8 contains conclusions and describes some areas where further work would be advantageous.

1.5. Statement of use

This report is intended to be used, in the first instance, by those involved in managing flood and coastal defence assets (particularly coastal defence structures with a sand beach) and assessing the risks associated with coastal defence structures and systems. The report includes new information on the behaviour of beaches in front of coastal defence structures at a range of time and space scales. This will enable better informed decisions to be made about data collection, monitoring and intervention. Moreover, it will assist in the appropriate design, construction and maintenance of coastal defence assets.

Fore example, fragility curves can be improved using information from this report on the prediction of toe scour, the long-term trend in beach levels and the Gaussian distribution of beach levels about the long-term trend. The development of a method for determining a prediction horizon for the long-term trend may also be of interest as it indicates the timeframe over which predictions have a useful (positive) level of skill. The timing of condition indexing surveys may also be affected by the research into seasonal trends and the variation about those trends.

The management of assets requires input from different organisations. Some potential uses of this report by different groups are listed in Table 1.1.

Table 1.1 Potential users of this report, subjects that may be of interest and locations

User Type	Subject of interest	Section
Academic	Use of laboratory and field data	4
Data gatherers and analysts	Review of instrumentation Error analysis of OS map data Limits to predictability of beach level Effect of varying the number of surveys per year Use of seasonal trend in planning surveys	3.1 3.2 3.3.3 3.3.6 3.3.8
Local Engineers	Methods of mitigation Frame of reference approach	5 6.4.4
Asset Managers	Implication of prediction horizon for asset management Prediction of bed levels at different scales Possible regional approach to beach lowering Improving fragility curves using Gaussian distribution of, and long-term trend in beach levels Timing of condition indexing surveys	3.3.4 3.7 3.3.9 6.4.1 6.3.1
Consultants	Gaussian distribution of residual beach levels Improved predictor for toe scour Trigger level for undermining	3.3 4 3.3.2

2. Evidence for beach lowering

2.1. Length scales and time scales of morphological changes

Research in this project (FD1927) concentrated on small length scales (a few metres cross-shore) and small time scales (typically a few hours). The overall performance of a coastal structure depends on morphological changes over a much broader range of length scales, detailed below and illustrated in Figure 2.1.

- Toe scour - often occurring and recovering completely during the course of a single tide (if in the intertidal zone, at least). Occurs over a cross-shore lengthscale of a few metres but may extend considerably further in the longshore direction;
- Storm response - lasting for a few tides and causing toe scour, beach lowering and recovery over cross-shore scales of up to a few hundred metres and rather longer distances in the longshore direction;
- Recovery between storms - the beach will respond to the changing forcing conditions and variations in beach level can be observed. Recovery from a storm can take 10s of tides and will affect a similar area to the storm;
- Seasonal variation – it is commonly observed that beach levels draw-down more in winter (due to storm-induced erosion) and build up during summer, leading to a seasonal variation in beach levels at the toe of a structure;
- Inter-annual variability in climate – this will have a net effect on the coastline by generating erosion or accretion and there are considerable variations between years. The annual wave climate affects the whole coastline so its effects are felt over the scale of the sediment cell, say 10s of km alongshore by of the order of 1 or 2 km cross-shore; and
- Coastal evolution and sea level rise – changes are driven by sea level rise and dominated by longshore transport. Occurs over longer timescales and even larger spatial scales than beach changes due to variations in annual conditions.

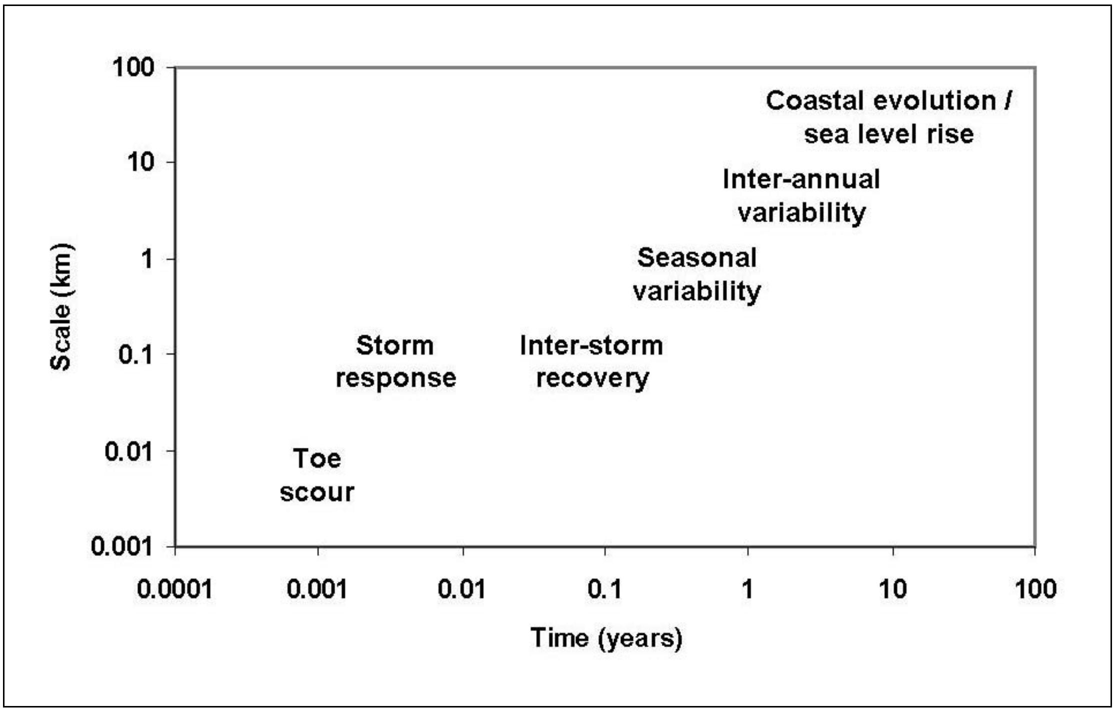


Figure 2.1 Beach responses to natural forcing, indicating associated length-scales and time-scales

In general, the spatial scale increases with the timescale and longshore sediment transport processes increase in importance compared to cross-shore transport processes as the timescale increases. Data on beach levels are therefore needed at a range of spatial scales and time scales in order to assess the changes to a beach in front of a coastal defence structure.

Two examples of data collected at different time scales but which both show variations in the beach level at the toe of coastal structures are given below. Section 2.2 shows beach elevation data collected at a point and at a time interval of 15 minutes through a few tidal cycles. Section 2.3 shows beach elevations collected at low tide in front of a coastal structure at time intervals between about 1 month and 1 year over a period of 32 years. Both sections provide evidence of beach lowering and recovery. The data on beach lowering and recovery during a tide in Section 2.2 was used in the development of an improved scour predictor (Section 4). The 32-year long time series of beach level at a point collected at low tide was used to illustrate the distribution of residual beach levels (see Section 3.3.3).

2.2. Beach lowering and recovery during a storm

HR Wallingford deployed two of their Tell-Tail scour monitors at Fisherman's Walk at Southbourne (Bournemouth) between 9th May and 7th June 2005 (HR Wallingford, 2005a) see figure 3.1. The monitors were installed along Channel Coastal Observatory (CCO) profile line 5f00409, which has been surveyed three times per year since July 2002. Water levels were obtained from the Bournemouth Pier tide gauge, provided by the National Tidal and Sea Level

Facility <http://www.pol.ac.uk/ntslf/>. Wave data was obtained from the Directional WaveRider buoy in 10.4mCD depth in Boscombe Bay, via the CCO website at <http://www.channelcoast.org/>.

The Tell-Tail scour monitor system consists of 8 omni-directional motion sensors, mounted on flexible "tails" and connected to a solid state data recorder. Under normal conditions, the sensors remain buried and do not move. When a scour hole begins to develop, the sensors are progressively exposed and each begins to oscillate in the flow. Each oscillation is logged. Use of an eight level array of sensors provides a more accurate measurement of the depth of scour and also indicates when the scour hole fills in again.

Figure 2.2 shows that as the wave height and water level rose during the morning of the 24th, the beach level dropped by at least 0.60m. The bottom monitor became exposed, so there is no record as to exactly how far the beach level dropped below this level. However, as water levels fell during the afternoon, the beach recovered to its previous low-tide level. The beach level fell again as water levels rose during the afternoon of the 24th, even though wave heights were lower. The bottom scour monitor again became exposed and again the beach recovered fully by low tide. There was only a small change in bed level during the next high tide as water levels were lower and wave heights were smaller.

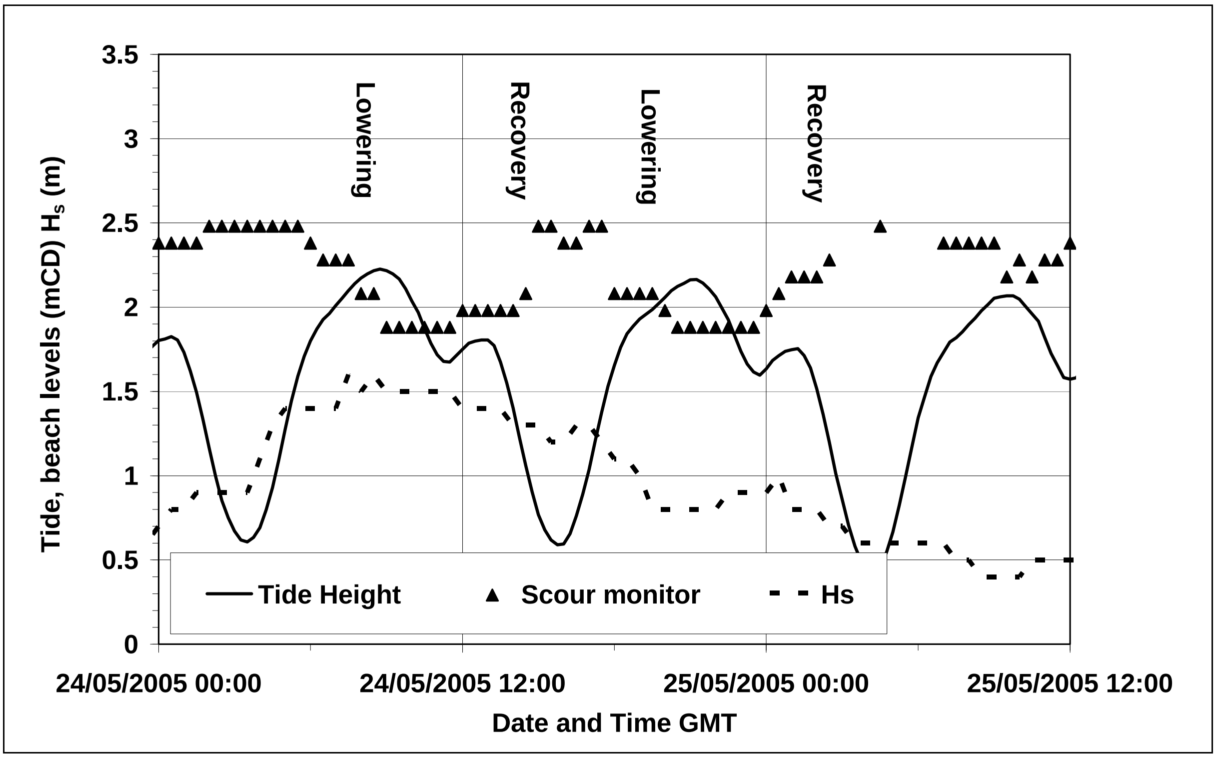


Figure 2.2. Scour monitor data showing beach lowering and recovery during a tide measured at Southbourne

The results from Southbourne and extensive analysis of scour monitor data collected at Blackpool between 1995 and 1998 (HR Wallingford, 2005b) has shown that beach levels frequently drop and recover to, or close to, their original level within a single tide, providing the water levels and wave heights are high enough. This beach lowering and recovery *could not have been detected* from beach profiles conducted at low tide, even if the profiles had been collected at successive low tides before and after the tide in question, as the beach levels recovered partially or completely during the falling tide.

2.3. Beach levels in front of a seawall on a monthly to decadal scale

Cross-shore beach profiles were collected approximately monthly at 18 locations in Lincolnshire from Mablethorpe to Skegness between 1959 and 1991 (HR Wallingford, 1991). Typically about 310 profiles were measured at each location during this period and most of the profiles started from a seawall. Each beach profile has been interpolated using HR Wallingford's Beach Data Analysis System (BDAS) to produce the beach level at the same chainage in front of the seawall. The time series of levels at 10m chainage at Mablethorpe Convalescent Home (NRA profile 12 at 551278mE, 384400mN) is shown in Figure 2.3. The straight line is the best-fit straight line through the points and is used to define the mean beach level in any year and the trend in beach level. The profile was falling at an average rate of 23mm per year during this period.

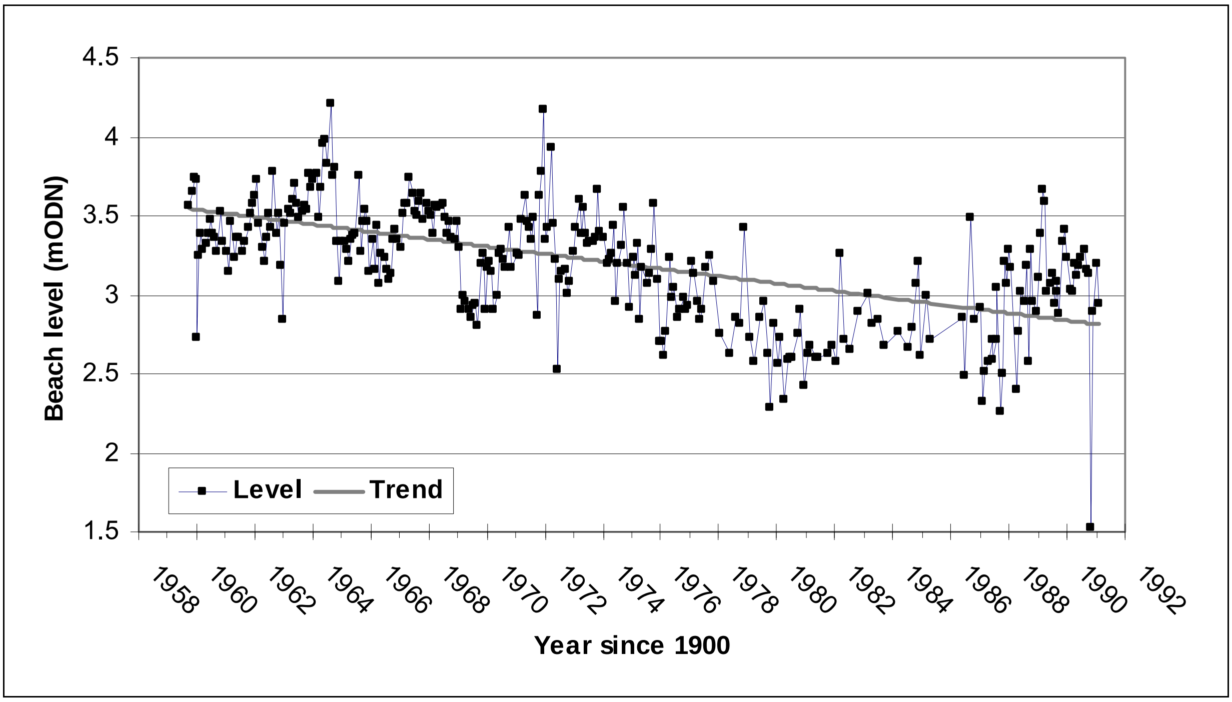


Figure 2.3. Time series of beach levels in front of a seawall at Mablethorpe in Lincolnshire

Figure 2.3 shows that there is a significant amount of variability about the best fit straight line, which is, nevertheless, a reasonable representation of the trend in the beach levels over 30 years.

3. Assessment of beach lowering and toe scour

Section 2 has illustrated that beach lowering and recovery occurs over a range of spatial scales and time scales. It is therefore necessary to have data and models that will allow the range of beach levels in front of coastal structures to be assessed in order for coastal management decisions to be made. This was the subject of HR Wallingford Technical Note CBS0726/03 (HR Wallingford, 2006c), summarised in this chapter.

Section 3.1 summarises the available technology for monitoring beaches in front of structures. This ranges from small-scale devices that record time series of beach levels at a point through the tidal cycle to the use of airborne Light Detection and Ranging (LIDAR) to survey 10's of kilometres of beach in a single flight. The review includes a number of instruments that have been developed recently and have never or only rarely been used to monitor beach levels in front of structures, although all have the potential to do so. In particular the increasing range of systems that can operate for weeks or even years with little or no human intervention is opening up possibilities for the collection of relatively long datasets with low recurring costs. These possibilities have not been significantly exploited for research or coastal management.

Section 3.2 contains an error analysis of tideline data on OS maps, which are commonly used to determine long-term rates of coastal erosion, with time series extending back over 100 years. Examples of the use of tidelines can be found in many Shoreline Management Plans and strategy studies as well as in Futurecoast (Halcrow, 2002, Defra, 2003b) and resulting publications (Taylor et al., 2004). The tideline positions have generally been presented without any consideration of the errors in their positions. An honourable exception is the analysis of the (lack of) intertidal shortening along the chalk platforms of southeast England by Dornbusch et al. (2007). The error analysis performed in Section 3.2 allows error bounds to be placed on the position of OS tidelines. This error analysis allows the reliability of any best-fit trend fitted to the tideline positions on OS maps to be estimated.

Different tools are needed to predict the response of the coastline at different scales. These tools come with different levels of reliability, accuracy, skill and required expertise. These tools may be allocated to one of four basic types:
1. Statistical analysis;
2. Process-based numerical modelling;
3. Geomorphological analysis; and
4. Parametric equilibrium models.

The statistical analysis of beach levels is discussed in Section 3.3, with particular regard to the relatively simple forms of statistical analysis that can be easily applied to coastal management problems. The extrapolation of a linear trend in beach level is one of the simplest predictors that can be used (Section 3.3.2). A method for determining the prediction horizon (the useful duration over which a prediction is useful) is derived in Section 3.3.3. The implications of having known prediction horizons for coastal management are demonstrated in

Sections 3.3.4. Other issues considered are the Gaussian distribution of de-trended beach levels (Section 3.3.5) the effect of varying the number of surveys in a year (Section 3.3.6) the seasonal variations in beach level (Sections 3.3.7 & 3.3.8) a possible new regional precautionary allowance for beach levels (Section 3.3.9) advanced linear and nonlinear analyses of beach levels (Sections 3.3.10 and 3.3.11) the appropriateness of considering shoreline position or beach volume (Section 3.3.12) and the implications for beach monitoring (Section 3.3.13).

The other tools for predicting coastal evolution (i.e. process-based models, parametric models and geomorphological analysis) are summarised in Section 3.4. A sub-section describes each of these types of tools. It is not the purpose of this report to describe these in detail, rather an overview of each is provided, except where new methods have been developed and demonstrated. This is intended to be complementary to the descriptions and analysis provided in Defra's revised Shoreline Management Plan (SMP) Procedural Guidance, Appendix D (Defra 2003a), which is intended to apply to relatively long timescales (10 to 100 years). Defra (2003a, Appendix D) summarises the available techniques for predicting long-term shoreline evolution and provides a more detailed framework for the prediction of shoreline interactions and response in second round SMPs. This framework encourages the use of the results from the Futurecoast project (Halcrow, 2002, Burgess et al., 2002, Defra, 2003b) in determining future coastal behaviour as it gives a 100-year view of coastal behaviour and evolutionary tendency at a high level.

A variety of sources and techniques must be used to develop an understanding of shorter-term and smaller-scale behaviour up to a period of years. This report concentrates more on the techniques that can be used for shorter-term determination of beach levels. This may have been expressed in terms of a change in the plan shape of beaches, rather than changes in beach level at a structure, but changes in plan shape can be converted into changes in elevation using some knowledge of the beach slope. The options available for predicting bed levels at different timescales are outlined in Section 3.5.

3.1. Monitoring methods

Monitoring of beaches provides important information about the state of the coastal system. The data from monitoring provides the input into the statistical descriptors and numerical models of beach behaviour. It also provides the information with which to judge the bias, accuracy or skill of any predictor (Sutherland et al., 2004). This section of the report describes much of the equipment available for monitoring beach levels and the other data sources, such as Ordnance Survey maps, that provide useful information on beach widths. It pays particular attention to the equipment's ability to measure levels at the toe of coastal structures, but also acts as a general review of beach monitoring techniques. Some of the equipment has not been used to monitor beach levels in front of coastal structures, as far as the authors are aware, but has been included if this is a potential future use. It is convenient to categorise monitoring methods by the length scale that they cover as each method can be used for different purposes at different timescales. Further details can be found in HR Wallingford, (2006c).

3.1.1. Small scale bed level measuring devices

Small scale devices have been separated into the following classes: point measurements of beach level through the tidal cycle, underwater acoustic measurements of the seabed, measurements of emerged toe level and measurements of mixing depth. These classes are summarised below. Further details can be found in HR Wallingford 2006c, Section 3.1).

Point measurements of beach level through the tidal cycle can be obtained using the following devices:

- HR Wallingford's "Tell Tail" scour monitoring system is based on a linear array of omni-directional motion sensors, buried in the sea bed adjacent to the structure. Identifying the elevation of the lowest active sensor places an upper limit on beach level. The system records the onset of scour, the depth of scour reached, and the in-filling of scour holes following storm events. Tell-Tail scour monitors have been deployed in front of seawalls at Teignmouth (Whitehouse et al., 2000), at Southbourne (HR Wallingford, 2006a, Pearce et al., 2006) as shown in Figure 3.1 and at Blackpool (HR Wallingford, 2006d);
- Linear array of electrical conductivity meters (Ridd, 1992, Cassen et al. 2005), which rely on the fact that sea water has a high electrical conductivity while dry sediment has a low conductivity and saturated sediment has an intermediate conductivity. Cassen et al. (2005) measured erosion in the inter-tidal zone of a beach at Bicarrosse (France);
- Photo-Electric Erosion Pin of Lawler (1991) which detects daylight at an array of optical sensors and has been used in the swash zone by Robinson et al. (2005);
- Sedimeter of Erlingsson (1991) which used an array of infra-red transmitters and backscatter detectors.

Figure 3.1 Deployment of Tell-Tail scour monitors at Southbourne

An acoustic bed level measuring device can be used to detect the level of the seabed and acoustic backscatter devices can give information about sediment in suspension in situations where the seabed and instrument are fully submerged. As far as the authors are aware no such system has been used to measure scour in front of a seawall. However, these systems have been used in the surf zone (Gallagher et al., 1996, Hoekstra et al., 2004) and could be deployed at a seawall.

There are a number of techniques that can be used to measure emerged coastal defence structure toe levels at a point every low tide. These include:

- Acoustic distance measurements in air. Such products are sometimes used to measure wave heights but could also be used to measure beach levels at a point;
- Photography/video of the seawall (possibly with marked elevations) from a camera mounted overhead of a sloping seawall or offshore from a vertical wall, perhaps on a pier; (see also Section 3.1.2) and
- Counting the number of steps above the beach level at access points, or the number of planks visible on either side of a groyne.

The first two could be operated remotely, so could collect a large amount of data with little running cost, once the system is set up.

The seabed mixing depth is the maximum depth below the seabed where sediment motion occurs. Immediately below this level the sediment is immobile (Ferriera et al., 2000). The mixing depth therefore determines the vertical limit of sediment transport. The available methods for determining mixing depth include:

- Plug holes filled with marked materials, such as a stack of numbered aluminium disks, up to surface level. The disks are left in the beach for a period, then the uppermost undisturbed ring identifies the mixing depth.
- Graduated sticks or rods with or without washers. Beach levels at rods without washers must be observed, so this technique is generally confined to the swash zone.
- Analysis of the distribution of tracers, such as dyed sand, with depth. Native sand should be used wherever possible. In these tests dyed sand is injected into the beach face.

3.1.2. Medium Scale

A large amount of beach survey data has been collected in the last ten years. Large-scale data collection programmes have been set up such as the Environment Agency Anglian Region bi-annual measurement of beach profiles and the Channel Coast Observatory mixture of beach profiles, topographic surveys and aerial photography. Beach profiles and topographic surveys are typically collected using standard surveying techniques including the use of total stations, and kinematic Global Positioning System (GPS) systems mounted on a pedestrian or on a quad bike. Recently data has also been collected using laser scanning systems; repeated digital photography (such as the Argus system, Wijnberg et al., 2004) and, in one experimental case, X-band radar (Bell, 1999). More details of the use of these techniques can be found in HR Wallingford, (2006c, Section 2.2).

3.1.3. Large scale

One of the major sources of information on large scale coastal changes is the mapping of tidelines (and clifflines, where appropriate) on Ordnance Survey (OS) maps. These cover large spatial scales and extend back over 100 years, as described in HR Wallingford (2006c, Section 2.3.1). OS map data is commonly used in SMPs and strategy studies, as shown in Figure 3.2, but there has been little critical analysis of its shortcomings (Carr, 1962, 1980, Oliver, 1996, 2005, Ryan, 1999, Dornbusch et al., 2005). An error analysis of OS tideline data has therefore been carried out (HR Wallingford, 2006c, Section 3.5). It is summarised in Section 3.2.

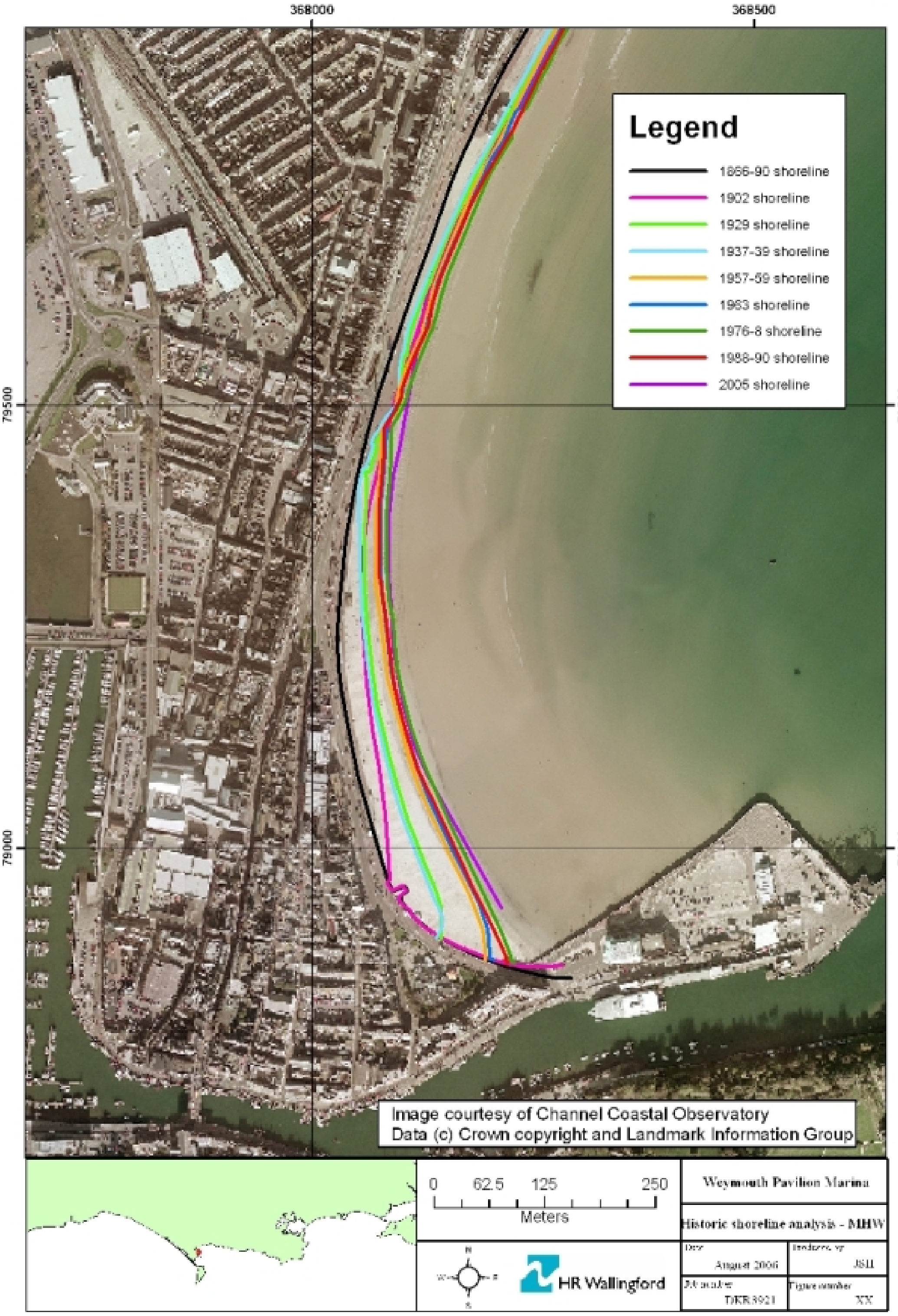


Figure 3.2 Historic shorelines from OS maps

Since the 1970's much of the tideline information on OS maps has been obtained from photogrammetry. Not all photogrammetry has been for the OS, however so additional sources of tidelines and beach levels may be obtainable.

The development of LIDAR in recent years in both topographic and hydrographic forms has provided a large increase in the point density of beach level data. For example, Environment Agency LIDAR surveys involve flying at a height of about 800 metres above ground level, which allows a swathe width of about 600 metres to be surveyed. Individual measurements are made on the ground at 2 metre intervals (with a vertical accuracy of ±0.10m to ±0.25m depending on system). A large and extensive archive of EA LIDAR data files is available and is searchable using a downloadable database.

The use of low level, low speed LIDAR systems has recently been investigated for use in asset management (see Investigation of “Fli-map” System for Flood Defence Asset Monitoring, by Tim Burgess, R&D Technical Report W5A-059/TR/1 or ATLAS – High resolution Laser Terrain Mapping). These systems are similar to LIDAR surveys by aircraft, only operated at lower speed and altitude thereby offering a greater density of points and a better vertical resolution. Typically resolution is 12 –28 points per metre squared for ATLAS at 150m elevation and 60kph, with a typical swath width of 60m. There was a quoted standard deviation of 80mm on vertical height for Fli-map compared to 170mm for LIDAR from a comparison at one site. ATLAS promises an absolute 3D accuracy of 50mm from 150m altitude. During ATLAS and Fli-map flights, detailed videos are recorded which allow for asset identification and condition monitoring.

3.1.4. Summary of monitoring methods

The longshore length of coverage and time between sampling of the different monitoring methods are summarised in this section and represented in Figure 3.3

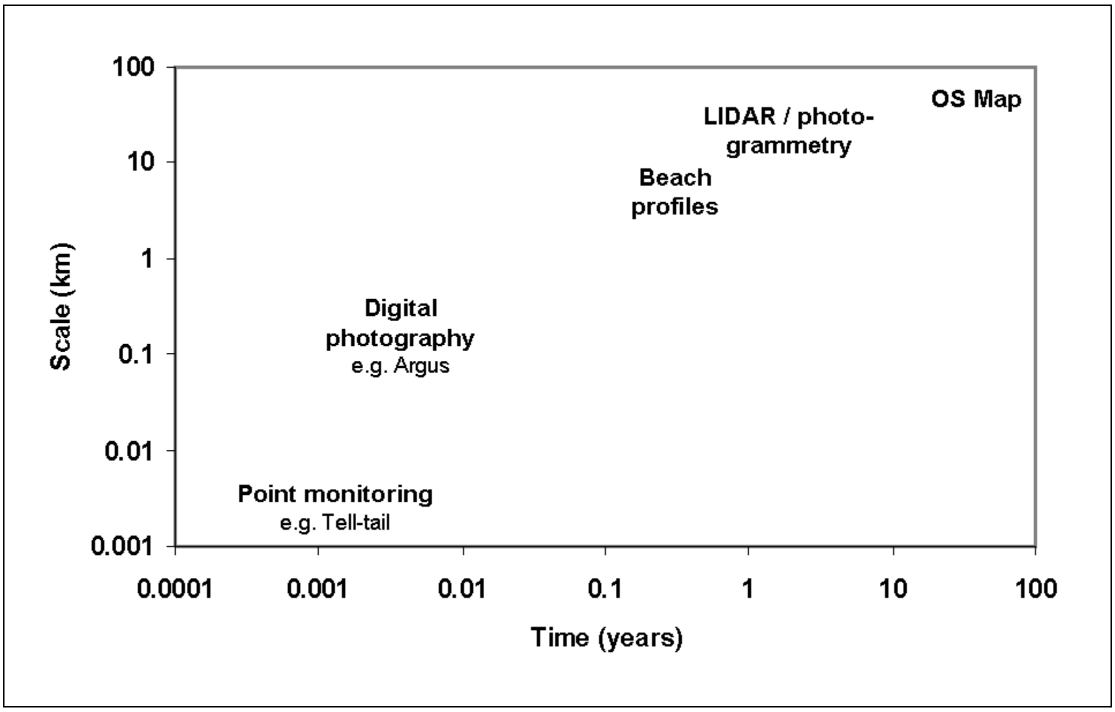


Figure 3.3 Beach monitoring: longshore length of coverage and time between sampling

Any of the point monitoring, vertical arrays of measuring devices should give a reasonable time series of beach levels over several tides before the data has to be downloaded and batteries replaced. These devices also measure time series through a tide during storms, which manual methods cannot.

The underwater acoustic measurements have the potential to measure the changes in bed level through a tide and measure the sediment transport at the same time, thereby providing more information on the processes involved. They cannot measure through the air-water interface so require a certain depth of water to work in. They cannot therefore capture the full tidal evolution in intertidal zones. Also they have not been used to measure time series of beach levels in front of seawalls, where the interaction of incident and reflected waves can lead to severe pressures and forces, so their robustness for use there has not been proven.

Of the three methods, the analysis of the distribution of tracers gives the clearest picture of the lateral and horizontal extent of mixing, but requires by far the greatest amount of work. Either the filled plug holes or the graduated rods with washers should be able to measure the greatest depth of sediment transport in front of a seawall reasonably well and can be left for days or even weeks at a time. Linear arrays of measuring devices therefore provide the best way of obtaining time series of beach lowering and recovery through a series of events. However none of the mixing depth methods provides a time history through a tide.

The longest time series of shoreline positions can be obtained from OS maps. The best accuracy can be obtained from 1:1250 or 1:2500 OS maps from 1879 onwards (providing that Mean High/Low Water of Ordinary Tides were mapped rather than Mean High/Low Water of Spring Tides). Records of aerial photographs sometimes extend back over 50 years and have been the most common source of OS tidelines since the 1970s.

Airborne SAR and LIDAR can survey a large area faster than ground surveys. The use of DGPS on a backpack or quadbike is a faster method of ground survey than conventional triangulation. LIDAR systems can therefore survey large lengths of defence in a day and are particularly useful for remote defences or those with difficult access (because of, say, saltmarshes).

LIDAR systems record the first returned signal, which can be from the top of vegetation, so routines have been written to remove such surface features. The lower-level higher-resolution systems, such as Fli-Map, collect a much larger number of points per metre squared (have a higher point-cloud density) so they are more likely than conventional LIDAR to see through gaps in vegetation and record the ground level underneath tree cover. A ground survey can obtain more than just top surface level and position, so can contribute more to a condition survey than even high-resolution LIDAR.

A ground survey is still the most accurate form of survey. Conventional (higher level, faster speed) LIDAR is suitable for large area surveys (>10km^2) where detail is not too important, while lower level, higher resolution LIDAR is suitable for long lengths of structure (>2km) with video images being used to assist in condition surveys. Ground surveys are suitable for detailed descriptions of small areas or vegetated areas, particularly where further information is required.

One of the most important data needs is for the beach level at the toe of coastal defence structures. In order to be able to identify the beach levels with reasonable confidence, a high resolution is required. Conventional LIDAR can now provide elevations within ±0.15m, which is good enough for this purpose, but if the data is at 2m intervals, the LIDAR system may miss a seawall. High-resolution LIDAR can provide greater accuracy and reduced distance between surveyed points, so the combined use of this system and ground-survey would achieve the required resolution. All remote sensing systems need a good network of control points to be at their most effective.

3.2. Error Analysis of OS tidelines

Estimates of the total uncertainty in shoreline position are made up from a combination of source uncertainty, interpretation uncertainty and natural variability. Source uncertainty reflects the errors involved in the measurement of any point and includes errors in triangulation, the resolution of and type of corrections applied to aerial photos and GPS errors. Interpretation uncertainty represents the error in turning the data into a shoreline. This includes the difficulty of determining the shoreline from an aerial photo and the error in determining the mean high water position from a single visit. Natural variability reflects the dynamic changes in the shape of the beach that occur in response to changes in waves and water levels. An analysis of these errors is given by HR Wallingford (2006c, Section 3.5) and is summarised below.

Consideration of the work of Ryan (1999) Dornbusch (2005) and the Ordnance Survey (OS, 1997a:5 and OS, 1997b:D10, cited from Ryan, 1999) indicates that a suitable Root-Mean-Square Source Error (RMSSE) for the tideline is 3.3m for OS County Series (1:10,560) maps and 2.8m for the National Grid maps, including Mastermap. The Root-Mean-Square Source Error is no lower in Mastermap than in older National grid maps, despite the implementation of a Positional Accuracy Improvement (PAI) programme as areas with sea were not covered by PAI.

The interpretation uncertainty includes errors in the elevation of high or low water relative to the target value and uncertainty in deciding the position of a moving shoreline. Four components were identified for an example site: (HR Wallingford, 2006c):

- Truncation of levels in Admiralty Tide Tables to nearest 0.1m;
- Surveys can be taken when predicted high or low water is within ±0.3m of the target value. A Root-Mean-Square Error (RMSE) from the target of 0.21m was obtained from analysing the results from 4 tide gauges in 1996;

- The variation in water level within ±0.5hours of high tide (the allowable time frame for surveys) was estimated at 9 English and Welsh ports from Admiralty Tide Tables. These values increased with tidal range. Avonmouth was included in the calculation but then left out of the overall averaging due to its unusually high tidal range. An estimate of root-mean-square error of 0.05m for high water and 0.10m for low water was made.
- The Root-Mean-Square (RMS) vertical error in determining the instantaneous position of the tideline was estimated at 0.05m as measurements should have been taken in calm conditions.

The four errors are assumed to be independent, so are combined by calculating the square root of the sum of the squares of the standard deviations. This gives typical values of RMSE in level of 0.23m for high tide and 0.29m for low tide. This combination of errors ignores any measured mean errors which will bias the results. The method is therefore only valid if the mean errors are essentially invariant with time. The validity of this assumption has not been investigated.

The horizontal error then depends upon the beach gradient, which will be greater for High Water Levels (HWL) than for Low Water Levels (LWL). This should be determined for each site from local beach profiles. As an indication of the difference, a number of cross-shore RMS Interpretation Errors (RMSIE) are given in Table 3.1 for a range of potential beach slopes. These range between 4.6m and almost 57.5m.

Table 3.1 Potential Root-Mean-Square Interpretation Errors (RMSIE) in cross-shore position for a range of beach slopes

Slope	High water RMSIE (m)	Low water RMSIE (m)
1:20	4.6	5.8
1:30	6.9	8.6
1:50	11.5	14.4
1:75	17.2	21.6
1:100	22.9	28.8
1:200	45.9	57.5

The RMS Variability Error (RMSVE) is a measure of the horizontal variability in the cross-shore position of a given contour, due to natural changes in the waves, currents and water levels. Values for this figure should be obtained for each site by analysing beach profiles. An example of this is provided by beach profile data collected in Lincolnshire between 1959 and 1992. The HWL was estimated to be the average of MHWS and MHWN at Skegness, while LWL was estimated as the average of MLWS and MLWN at Skegness (from the 1991 Admiralty Tide Tables). The resulting RMSVEs are given in Table 3.2 for a number of cross-shore profiles, which also includes values for MHWN, MWL and MLWN. In some cases there were insufficient surveys at MLWN and/or MLW to provide a reasonable estimate. These cells have been left blank.

Table 3.2 RMS horizontal Variability Errors (RMSVE) for Lincolnshire profiles

Location	MHW (m)	MHWN (m)	MWL (m)	MLWN (m)	MLW (m)
Convalescent Home	7.45	9.14	10	11.08	
Convalescent Home	6.07	8.62	10.36	11.08	
Trusthorpe Outfall	0	3.85	9.33	7.79	10.82
Sutton Pullover	2.01	9.26	14.34	10.73	22.25
Boygrift Outfall	6.31	4.84	13.65	14.88	15.57
Jacksons Corner	14.5	15.69	14.52		

Table 3.2 shows that the RMSVE increases on going down the beach profile, as the beach profile flattens. The RMSVE is zero at MHW at Trusthorpe Outfall as the beach level was so low, only the position of the seawall was recorded. The range of RMSVE at MHW was from 0 m to 14.5 m. The RMSVE at MLW was not obtained for half the profiles where there were insufficient results at that level. The range of measured errors at MLW was from 10.8 m to 22.2 m.

The sources of error are summarised below:

1. Root Mean Square Source Error (RMSS) for 1:2500 scale mapping decreases from 3.3m for County Series maps to 2.8m for National grid maps. Mastermap mapping is taken to have the same error as National Grid mapping.
2. The RMS Interpretation Error (RMSIE) is given approximately by 0.23/tan(α) m for MHW and 0.29/tan(α) m for MLW where α is the beach slope at MHW/MLW. Similar values apply for County Series, National Grid and Mastermap. Regional differences are probably larger than differences between map series.
3. RMS Variability Error (RMSVE) can be determined from beach profiles. As an example, in Lincolnshire between 1959 and 1991, the RMSV at MHW varied between 0m and 8m, while that at MLW varied between 10m and 23m. Beach profiles were relatively steep, being around 1:30 at MLW. Larger errors may be anticipated on flatter beaches or on flatter beaches with topographic features such as a ridge or runnel.

These values are not necessarily applicable outside the areas they were derived for and local values should be estimated in all cases. If the different errors are independent and have normal distributions, as we have assumed, then the total RMS error, RMSTE, is given by Equation 1.

$$RMSTE = \sqrt{RMSSE^2 + RMSIE^2 + RMSVE^2} \qquad (1)$$

The range of expected values will then be about 4 times the RMS total error (at 95% confidence level). A number of examples from Lincolnshire are set out below:

- MHW on a National Grid map with a 1:25 slope would have a RMS total error of 6 m to 10 m.
- MLW on a National Grid map with a 1:30 slope would have a RMS total error of 14m to 24 m.
- MLW on a National Grid map with a 1:100 slope would have a RMS total error of 31 m to 37 m.

So, for example, two surveys of MLW (if on a 1:100 slope) could be up to 150m apart, with the differences being caused by the survey methods used and the natural variations in the beach morphology. No net erosion or accretion need have taken place. The above examples are not the worst-case scenarios as there are obvious problems in determining LWL in cases where there are sandbanks (if the inshore channel level is about MLW) and ridge and runnel beaches. In the former case the channel bed may be above MLW and MLW will run at the seaward side of the sandbank or it may be below MLW and the MLW will run along the beach side of the channel. In the latter case the position of low water will depend on the configuration of ridges and runnels. Estimates of the error in MLW assume that MLW was surveyed, whereas in practice this was not always the case. Trends from MLW are therefore less reliable than trends from MHW.

3.3. Statistical Analysis of beach data

Statistical models rely on the extrapolation of historic data to predict future coastal evolution. A statistical model can only predict behaviour under conditions that are similar to those in the historic record and cannot cope with changes in forcing conditions, beach management or geological controls. Statistical methods can use long-term data sets, such as OS maps, which are available for the entire coastline at a number of times. The use of long-term datasets may allow extrapolation further into the future than from using shorter datasets. Shorter-term, often more detailed datasets, can be used to try and confirm the long-term behaviour and can be used for analysis at shorter timeframes.

Statistical models are derived from data using an analysis method. The available monitoring methods for collecting data have been summarised in Section 3.1, while the available analysis methods are summarised here. The large majority of statistical modelling performed for SMPs has been carried out using the simpler linear analysis methods detailed in Sections 3.3.2 to 3.3.5. The more complicated linear analysis techniques (Section 3.3.10) and the non-linear analyses (Section 3.3.11) have only recently been applied to beaches. Their use generally requires larger quantities of high-quality data than have historically been collected.

Larson et al. (2003) noted that the choice of method for data analysis depends crucially on the quality and the quantity of data. The more sophisticated methods require more data of good quality and may pose additional constraints on the data (Möller, 1997), such as the need for data to be equally spaced in time and position. This will restrict their use to the limited regions where long term high quality datasets of coastal morphology exist. The shortage of locations with high quality data on morphology extending over years to decades is one major obstacle in the quest to understand and predict beach response over these scales. The shortage is being addressed through the development of regional monitoring programmes, such as the Environment Agency's Anglian Region beach monitoring programme that has been performing beach profile surveys twice a year since 1991 and the Channel Coast Observatory that has been operating a regional beach monitoring programme since 2002. The use of the more advanced linear and non-linear techniques is likely to become more widespread in time as the quantity and quality of data collected increases, provided that they can be shown to be useful for coastal management as well as interesting to research. Most are not yet at this stage.

Any analysis is likely to start with a review of bulk properties such as the mean and standard deviation of the beach level at each point or the cross-shore position of a contour line. The more advanced methods allow the morphological response at different scales to be identified, with the analysis and modelling at that scale being independent of processes at other scales. In some areas statistically-based models may show as much skill as physically-based models, but the application of a statistical model to a different beach to the one it was developed at is likely to require more data to recalibrate the model than a physics based model would require. 'Skill' is defined as a non-dimensional measure of the accuracy of a prediction compared to the accuracy of a baseline prediction (Sutherland et al., 2004).

3.3.1. Example beach level data

The linear analysis of beach level data is demonstrated here using a set of beach profile measurements carried out at eight locations along the Lincolnshire coast between 1959 and 1991, as described in HR Wallingford (2006c, Section 3.1). Locations backed by a seawall were chosen and time series of beach levels were output at points near the seawall toe. An example of a time series has been given in Figure 2.3. Details of the 8 chosen stations are given in Table 3.3

Table 3.3 Details of chosen stations for more detailed analysis.

Station number		Place Name	National grid coordinates (mE mN)	Bearing grid N (°)	Chainage at wall (m)
HR	NRA				
A4	12	Mablethorpe Convalescent Home	551278 384400	54 17 15	10
B1	13	Trusthorpe Outfall	551504 384115	83 31 55	15
B2	14	Bohemia Point	551815 383288	90 31 59	20
B3	15	Sutton Pullover	552251 382143	85 06 51	10
C1	17	Boygrift Outfall	553401 379919	86 18 40	40
D1	21	Chapel Point	556240 373285	71 54 55	15
D3	23	Trunch Lane	556620 371068	86 16 01	15
E2	26	Jacksons Corner	557320 366403	108 57 00	15

3.3.2. Extrapolation of best-fit linear trend as predictor

Straight lines fitted to beach level time series give an indication of the rate of change of elevation and hence of erosion or accretion. The measured rates of change are often used to predict future beach levels by assuming that the best-fit rate from one period will be continued into the future. Alternatively, long-term shoreline change rates can be determined using linear regression on cross-shore position versus time data.

Douglas and Crowell (2000) have shown that simple regression is superior to end-point rate and complex statistical methods for calculating shoreline erosion rates. Genz et al. (2007) reviewed methods of fitting trend lines, including using end point rates, the average of rates, ordinary least squares (including variations such as jackknifing, re-weighted least squares, weighted least squares and weighted re-weighted least squares) and least absolute deviation (with and without weighting functions). Genz et al. recommended that weighted methods should be used if uncertainties are understood, but not otherwise. The ordinary least squares, re-weighted least squares, jackknifing and least absolute deviation methods were preferred (with weighting, if appropriate). If the uncertainties are unknown or not quantified then the least absolute deviation method should be preferred.

Confidence limits can be calculated to provide a measure of the reliability of the erosion or accretion rate. They provide a range for the calculated erosion or accretion rate and depend on the variance of the data, the number of samples and the desired level of confidence. They strictly apply only to the time period the data was collected in. The extrapolation of trends and confidence limits into predictions assumes that the future hydrodynamic climate will be statistically similar to the climate during the period the measurements are made. That assumption is tested below.

The following question then arises: how useful is a best-fit linear trend as a predictor of future beach levels? In order to examine this, the thirty years of Lincolnshire data have been divided into sections: from 1960 to 1970, from 1970 to 1980, from 1980 to 1990 and from 1960 to 1990, for most of the stations. In each case a least-squares best-fit straight line has been fitted to the data and the rates of change in elevation are shown in Table 3.4.

Table 3.4 Rates of change in elevation in front of seawalls for different periods

Period	Convalescent Home Rate of change (m/year)	Bohemia Point Rate of change (m/year)	Boygrift Outfall Rate of change (m/year)	Chapel Point Rate of change (m/year)
1959-1991	-0.023	-0.023	-0.030	-0.031
1960-1990	-0.025	-0.021	-0.030	-0.028
1960-1970	-0.017	-0.001	0.010	0.069
1970-1980	-0.063	0.010	-0.035	-0.028
1980-1990	0.047	-0.061	-0.051	-0.186

The data above indicates that 10-year averages provide little predictive capability for estimating the change in elevation for the next 10-years, let alone for the planning horizon that might need to be considered for a coastal engineering scheme. Few of the 10-year averages are close to the 30-year average (1959-1991). At all locations except Chapel Point only one of the four 10 year averages lie within ±10% of the 30 year average, and at Chapel Point two lie within ±10%.

Also, the above analysis does not reveal how far ahead a best-fit straight line can be extrapolated to give a useful prediction of future beach levels, compared to, for example, using the average measured beach level. This can be assessed as follows; what is the average prediction horizon from extrapolating a best-fit linear trend fitted to a *M* year long beach profile record? The prediction horizon is defined as the average length of time over which a trend produces a better level of prediction of future beach levels than a simple baseline prediction. A procedure has been developed to determine the average prediction horizon from a time series of beach levels at the toe of a structure. The procedure is outlined below and demonstrated using the Lincolnshire dataset.

3.3.3. Procedure to establish an average prediction horizon

A procedure to establish the average prediction horizon, defined as the average length of time over which a trend produces a useful level of prediction of future beach levels, was developed in HR Wallingford (2006c, Section 3.1.2) and is summarised here. The procedure uses the Brier Skill Score (Murphy and Epstein, 1989, Brady and Sutherland, 2001, Sutherland et al., 2004), which is a non-dimensional measure of the accuracy of the linear trend (fitted to *M* years of data) relative to the accuracy of a baseline prediction of future beach levels. In this case the baseline prediction of future elevations is that they will all remain at the average level of the *M* years of measured data. Murphy and Epstein found that their meteorological model had a skill score that decreased smoothly with time, on average. The prediction horizon was the maximum length of prediction that gave a useful level of predictive skill, determined from when the average skill score dropped below a threshold value. Here the useful threshold value of the Brier Skill Score is zero, as it is at this level that the baseline prediction is as good as the prediction obtained from extrapolating the trend.

The procedure for determining the average prediction horizon given by a trend line fitted to *M* years of a time series of beach levels at a point is given in detail in HR Wallingford (2006c, Section 3.1.2) and is summarised below. Fit a straight line to the first M years' data, starting from the first point. For each data point beyond the data used in the fitting, extrapolate the fitted line to that point and record the following three values together:

1) the duration of the extrapolation (time between last point used in fitting and data point),
2) the difference. x-y. between the measured elevation, x, and the extrapolated, y.

3) the difference between the measured elevation, x, and the baseline prediction of the elevation, B., which is the average elevation of the data used in the fitting.

Repeat the above procedure, only starting from the next point each time until the fitted time series extends to the end of the time series. Sort the results by duration of extrapolation into bins of, say, 1 year (i.e. all results with duration between 0 and 1 year, 1 and 2 years, etc). Calculate the Brier Skill Score for each bin, *i*, using equation 2.

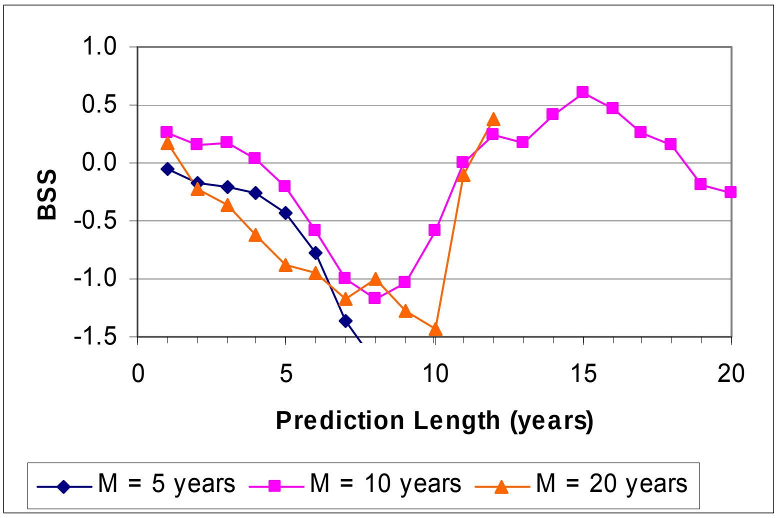
$$\mathrm{BSS}(\mathrm{i}) = 1 - \frac{\left\langle (\mathrm{x}-\mathrm{y})^2 \right\rangle}{\left\langle (\mathrm{x}-\mathrm{B})^2 \right\rangle} \tag{2}$$

Plotting the Brier Skill Score (Equation 2) as a function of duration of a prediction shows how much more accurate it is to use a trend line rather than the average of the measured points to predict future beach levels. Perfect agreement gives a Brier skill score of 1 whereas modelling the baseline condition gives a score of 0. If the model prediction is further away from the final measured condition than the baseline prediction, on average, the skill score is negative. This skill score is reduced by errors in the prediction of amplitude, phase and mean. It provides an objective measure of a model's performance (Sutherland et al., 2004).

The extrapolation of the best-fit trend in historic beach profile time series will act as a better predictor of future beach levels than the average beach level for time differences where the average skill score remains above zero.

The above method has been used to calculate the prediction horizon for some of the long-term Lincolnshire datasets. The results from the Mablethorpe Convalescent Home and Bohemia Point are shown in Figure 3.4 for the least-squares best-fit straight lines fitted to *M* = 5, 10 and 20 years. The results show that the best-fit straight line has a *negative* level of predictive skill for all durations of forecast for *M* = 5 years. In other words the use of a straight-line trend from 5 years' data is on average worse than the use of the average beach level as a predictor of future beach levels. This occurs as the relatively short timespan leads to a wide range of gradients from the fitted lines. These can lead to relatively large errors when extrapolated beyond the period of the measurements used.

Only for the cases of *M* = 10 years and *M* = 20 years does the best-fit straight line provide a better prediction of future beach levels than the average beach level for the first few years of prediction. The prediction horizons at Mablethorpe Convalescent Home and Bohemia Point are 4 years and 7 years from fitting to a 10-year record length (i.e. defined as when BSS drops below zero). The prediction horizons for 20-year record length are actually shorter, which may be because there are fewer samples to average over.



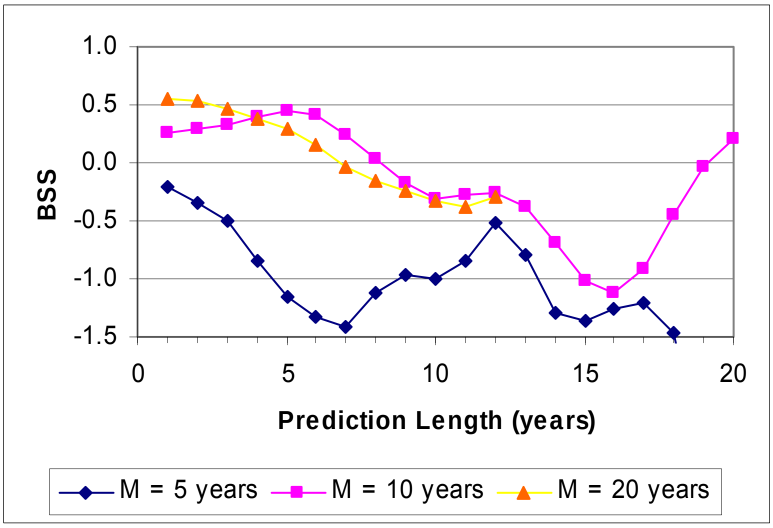


Figure 3.4 **Brier Skill Score(BSS) versus duration of prediction for linear trends fitted to 5, 10 and 20 years' data for Mablethorpe Convalescent Home (top) and Bohemia Point (below).**

Figure 3.5 shows the Brier Skill Score versus time for all the calculated profiles in Lincolnshire, based on fitting to 10 years of data. The linear trend is a better predictor than the average beach level for prediction durations between zero years (Jackson's Corner) and 15 years (Boygrift Outfall). The average Brier Skill Score from the 8 profiles is also shown. This has a positive value for both 1 and 2 year lengths of prediction, but a negative value thereafter.

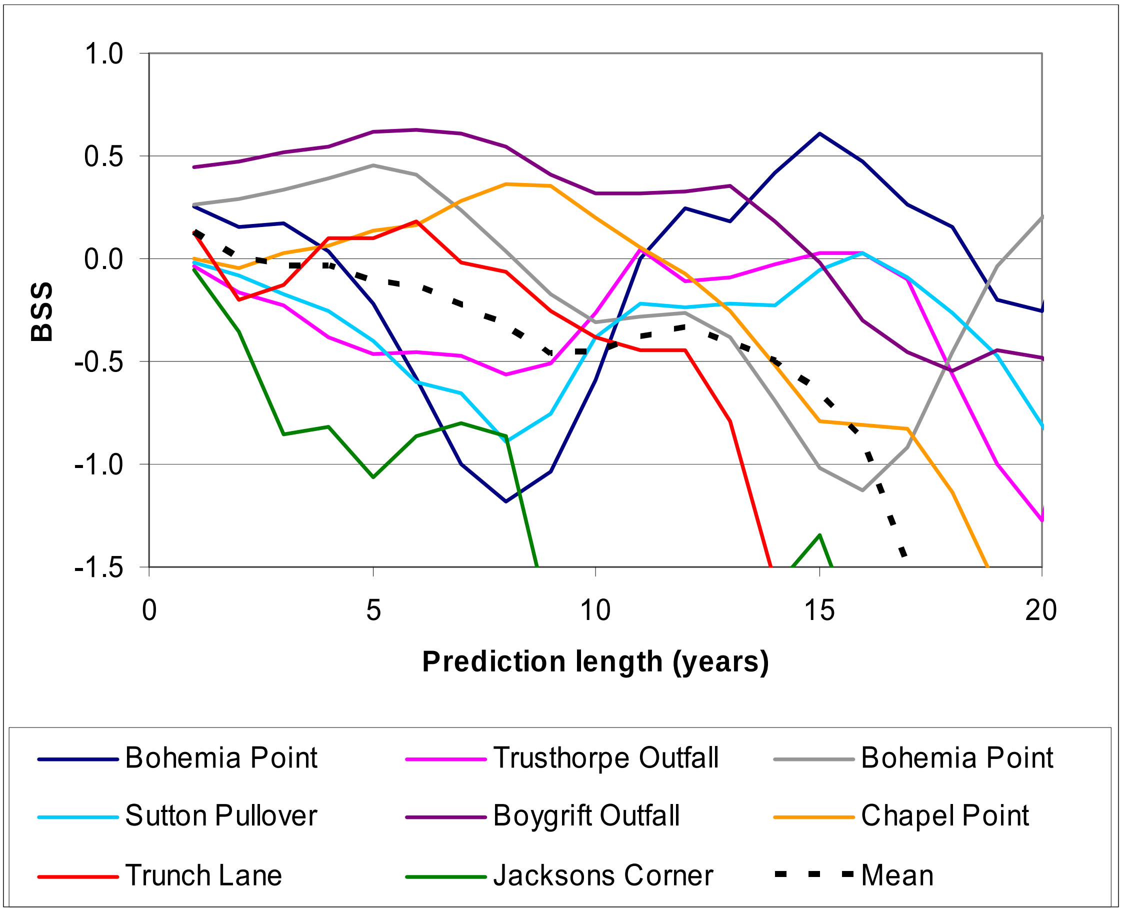


Figure 3.5 Brier Skill Scores versus time for Lincolnshire profiles based on linear trends fitted to 10 years' data

The above procedure has been applied to sandy beaches only in the present study. Zuzek et al. (2003) considered shoreline change rates in cohesive shores. They recommended that the length of the data record be at least as long as the planning horizon if past erosion rates are to be used to predict future shoreline positions in cohesive shores. Zuzek et al. (2003) also noted that when the erosion rate is averaged over a number of cross-shore transects, "the population mean provides a poor indication of the future erosion hazards because approximately half the transects will erode at a rate greater than the mean." This comment will also apply to sandy beaches.

3.3.4. Implications of prediction horizon for coastal management

Section 3.3.3 outlines a procedure that can be used to determine the length of time (or prediction horizon) over which the extrapolated best-fit straight line will provide a better predictor of future beach levels than the average of the measured beach levels. The results from an analysis of 8 profiles in Lincolnshire indicated that it is better to use an average beach level than a linear trend if only 5 years' data is available. If 10 years of data is available then a linear trend can provide a better predictor of future beach levels than the average value for prediction lengths between 0 and 15 years, with an average value of 2 years.

These results apply only to Lincolnshire and a similar regional analysis should ideally be carried out for other coastal regions when sufficient data is available. The results illustrate that the predictive ability of a straight line fit to data is limited to a few years beyond the end of the dataset. This duration is shorter than the timeframes normally considered for the precautionary approach to coastal management. However this duration is likely to be suitable for a managed / adaptive policy of tracking risk and performing multiple interventions (Defra, 2006).

Prediction horizons should be calculated for other predictive methods so that their suitability for informing coastal management decisions can be assessed in light of the timescales involved.

3.3.5. Gaussian distribution of residual beach elevations

Residual beach levels are obtained when the long-term trend is removed from a time series of beach levels. The eight Lincolnshire datasets were de-trended by subtracting the best-fit straight line from the time series. The probability distribution of residual beach levels was then calculated and plotted with a Gaussian distribution, which had measured average and standard deviation (HR Wallingford, 2006c, Section 3.1.3). The results from Mablethorpe Convalescent Home (from the time series in Figure 2.3) are shown in Figure 3.7

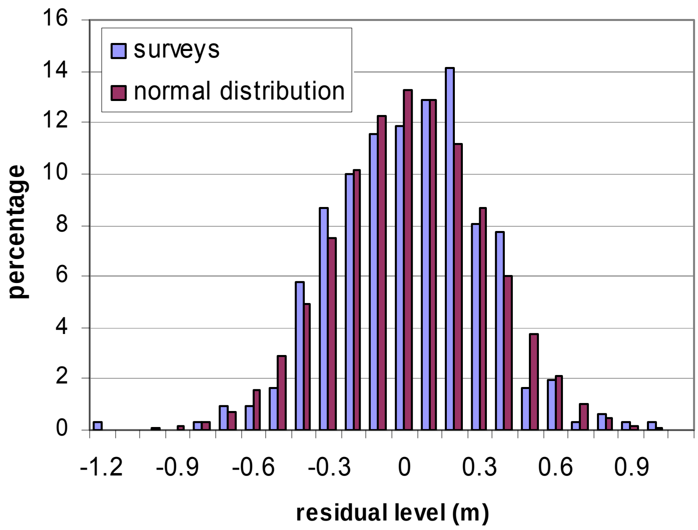


Figure 3.7 Measured and Gaussian distribution of residual beach levels at Mablethorpe Convalescent Home.

This work has confirmed the correctness of the assumption that the residual beach levels (after removal of the long-term trend) have a Gaussian (or normal) distribution, provided that the long-term trend in beach level has been satisfactorily removed. It is clear, however, that the long-term trend is not always adequately represented by a straight line profile, particularly where there have been changes to the methods of beach management during the duration of the measurements.

3.3.6. Effect of varying the number of surveys per year

The effect of collecting different numbers of surveys per year has been illustrated using the results from Mablethorpe Convalescent Home by first analysing the results from all surveys then by re-sampling 2, 3 or 5 times per year. The results indicated that the differences from using all the profiles increased as the number of surveys per year decreased. The standard deviation of beach level was on average 6% different from 2 surveys per year compared to all the surveys, while the percentage change in trend was on average 11% different. The differences in mean level and trend could be approximately halved by increasing the number of surveys from 2 to 3 per year. More details are contained in HR Wallingford (2006c Section 3.1.4). Information on when to perform surveys is provided in the following section.

3.3.7. Seasonal variations in beach levels at the toe of a structure

It is common to find beach levels lower in winter than in summer, due to the increased occurrence and severity of storms during winter. It also follows that beach levels may show a greater variation about their seasonal mean during winter. This will affect the optimum number timing of beach surveys, although these will vary depending on the purpose of the surveys.

In order to investigate this, the best-fit line of the form given in Equation 3 was fitted to seven out of the eight Lincolnshire stations analysed(HR Wallingford, 2006c, Section 3.1.5). Chapel Point was not included as the linear trend did not represent the long-term behaviour of beach levels well.

$$Z(T) = a - bT + c\sin(2\pi/T) + d\cos(2\pi/T) \qquad (3)$$

where Z(T) is the best-fit beach level at the toe of the structure, T = time (in years) since 1900 and a, b, c and d are the fitted variables. The latter two terms can be combined to give the amplitude and phase of the best-fit seasonal trend, represented as a sine function. Figure 3.8 shows the best-fit seasonal trend for the seven stations calculated. Six out of seven stations had seasonal trends between 0.1 m and 0.2 m in amplitude which had their highest values in August or September. The other profile, from the Convalescent Home has a much lower amplitude (22mm) and peaked in October. The average profile had an amplitude of 110 mm and peaked in September, with its lowest value coming in March.

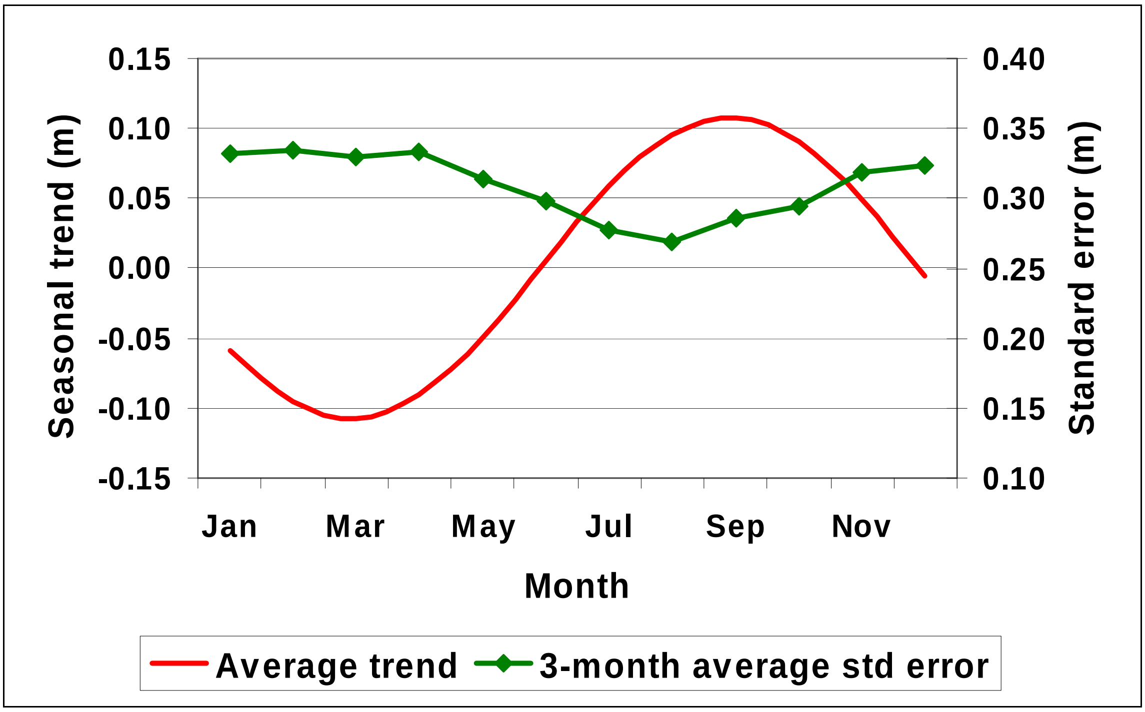


Figure 3.8 Best-fit seasonal trend from Lincolnshire stations

The residual level was calculated by subtracting Z(T) from the measured values. For four of the stations the mean residual level was calculated for each month (noting that the annual average residual level is zero). The standard error (standard deviation of the residual) was calculated for all stations.

3.3.8. Use of seasonal trend in planning surveys

The presence of a seasonal trend in both the mean and standard deviation of the beach level in front of a coastal structure will affect the beach monitoring programme. If the intention is to determine the best long-term trend in beach levels, the measurements should be taken when the standard deviation in the residual beach levels is at its lowest as this is when the signal-to-noise ratio will be at its highest. In other words the beach should be monitored in or around August (at least in Lincolnshire) when the variability in the beach level is at it's lowest. At that time, however, the beach level will be close to its highest, so it is unlikely that any particularly low beach levels will be recorded.

Therefore, if the intention is to detect extreme low beach levels then surveys should be undertaken when mean beach levels are low and the standard deviation in the residual beach levels is at its highest. In other words beach levels should be monitored in or around March (at least in Lincolnshire).

Note, however, that the relative timing of the seasonal trend will vary from place to place. For example, Zhang et al. (2002) analysed a detailed dataset of 588 high water line positions collected at Duck, North Carolina, USA between January 1996 and December 1999. They showed that the standard deviation of the high water line position was a minimum in June and July. They concluded that beach surveys that are to be used to predict long-term trends in shoreline position should be performed in June and July between spring and neap tides and should not be performed immediately after a storm.

3.3.9. Possible regional approach to beach lowering

A regional approach can be taken to provide guidance on possible changes in beach level at a structure toe over 10 to 20 years. Table 3.4 showed the change in beach level per year at the toe of the structure for four profile locations. An indicative allowance of, say, 30mm per annum for Lincolnshire would provide a guide to potential beach lowering rates that could be used for the design and maintenance of coastal defences. The indicative allowances for beach lowering would be applied in the same way as indicative allowances for sea level rise.

3.3.10. Advanced linear analysis of beach level data

There are a number of linear data analysis and modelling techniques that are useful for the prediction of the long-term evolution of beaches (Larson et al., 2003, HR Wallingford, 2006c Section 3.2). Correlation may be used to assess the effect of, say, wave height on bar movement. Fourier analysis and Random Sine Function analyses are useful in identifying features of different durations. Fourier theory assumes that the signal has a constant average and is periodic in nature.

Wavelet analysis uses an oscillating signal called a mother kernel that is localised in time or space, so wavelets are well suited to looking at phenomena that vary in time or space. Li et al. (2005) used the adapted maximal overlap discrete wavelet transform (AMODWT) to analyse beach profiles. Their first analysis used spatial wavelets to look at the relative importance of variations at different lengthscales across the beach. The second analysis used wavelets of different timescales, which showed that there were no dominant timescales of variation throughout the record.

Empirical Orthogonal Functions (EOFs) are shape functions extracted from morphological data. They correspond to a statistically optimal description of the data (Larson et al., 2003) with respect to how variance is concentrated in modes. The variance decreases as mode number increases so a finite (often small) number of modes explain most of the observed variance in the data (Winant et al., 1975, Aubrey, 1979, Wijnberg and Terwindt, 1995, Möller, 1997, Larson et al., 1999b). There is no reason for the EOFs to have a physical meaning, although EOFs often can be matched to physical processes.

Canonical Correlation Analysis (CCA) can identify correlations between patterns that tend to occur simultaneously in two different data sets (Larson et al., 2003). Larson et al. (1999a) used CCA to determine the covariability between waves and profile response at Duck, North Carolina, USA. The profile response was reasonably well correlated to the nearshore wave conditions. CCA could therefore be used to provide a predictive tool for beach levels in front of coastal structures.

Beach profile data can also be analysed using Principal Oscillation Patterns (POPs) - patterns based on approximate forms of dynamical equations. They may be used to identify changing patterns, such as standing waves and migrating waves (Larson et al, 2003, Jansen, 1997, Różyński and Jansen, 2002). POP is a linearised form of the more general Principal Interaction Pattern (PIP) analysis. An EOF analysis should be carried out first.

3.3.11. Non-linear analysis of beach level data

There are a number of non-linear data analysis and modelling techniques that are useful for the prediction of the long-term evolution of beaches (HR Wallingford, 2006c Section 3.3). Southgate et al. (2003) noted that the available time series of morphological data from in-situ measurements are usually too small for a full non-linear statistical analysis of the system dynamics. In these cases there may still be enough data to test a hypothesis.

Singular Spectrum Analysis seeks to identify the type of attractor state and the number of independent variables needed to describe the system (Southgate et al., 2003). SSA is an application of EOF analysis that uses time-lagged variables. SSA could be employed for predictive purposes in the coastal zone if it was combined with an autoregressive model to form a linear forecasting algorithm. Różyński (2005) studied the long-term shoreline response at Lubiatow (Poland) using multi-channel SSA (MSSA). The typical period of the North Atlantic Oscillation corresponds to that of the most frequently encountered 2nd standing wave component (7–8 years) in the shoreline response indicating that the NAO may drive a component of the morphological evolution.

A fractal shape is self-similar so it looks similar if seen at different scales. Every fractal process has a Hurst exponent, H, that represents the amount of persistence in the system. Fractal analysis requires less data than SSA and has been applied to beach profile data from Lincolnshire by Southgate and Beltran (1996) and Duck, NC, USA by Möller (1997) and Southgate and Möller (2000). The fractal analysis showed which timescales were dominated by self organised behaviour and which by forced behaviour, so could be useful for determining the length of dataset needed to reach a meaningful conclusion about a beach’s behaviour.

3.3.12. Is it more appropriate to use shoreline or beach volume?

The Environment Agency (2003) has analysed beach profile data collected in north Lincolnshire, between Grimsby and Mablethorpe, from 1991 to 2000. Summer beach profile data from 1991 through to 1999 was analysed for mean annual shoreline retreat/advance (m/yr) at mean sea level (MSL) and mean annual volumetric rate of change (m^3/yr) using the volume of the compartment 500m to either side of each beach profile. The mean annual volumetric rate of change is the mean annual rate of change of the area under the profile multiplied by a typical distance between profiles.

Seven out of the 26 profiles between Grimsby and Saltfleet showed advancing MSL and 19 retreating, including 7 around Donna Nook. On the analysis of volumes 18 out of the 26 beach profiles showed increases in volume including 6 of the profiles at Donna Nook that exhibited MSL retreat. This indicates that the retreat/advance of a contour line on the beach may not be a good indicator of changes in volume of the beach.

An alternative method of exploring the link between beach volume and position of high water was applied to the beach profiles further south, between Mablethorpe and Skegness by HR Wallingford (2006c, Section 3.4), summarised below. Here, the chainage of Mean High Water Neaps (MHWN) was plotted against the area under the profile for four of the Lincolnshire datasets: Convalescent Home, Sutton Pullover, Chapel Point and Jacksons Corner. MHWN was chosen as there were more data points on the beach for the neap tide level than for the spring tide level (where the seawall was encountered more often). Moreover, MHWN was chosen over MWL as the longest records of shoreline position are from maps, which show MHW and MLW but not MWL. The area under the measured profile was calculated between set chainages with the inshore chainage set at the wall and above a stratum level, which is arbitrary but sufficiently low that the beach never drops to that level.

Figures 3.9 and 3.10 show the cross-sectional area plotted against the chainage of MHWN at Mablethorpe Convalescent Home and Jacksons Corner respectively. In each case the relationship between cross-sectional area and the chainage of MHWN is approximately linear so it appears that the position of MHWN can be used as a surrogate for beach volume.

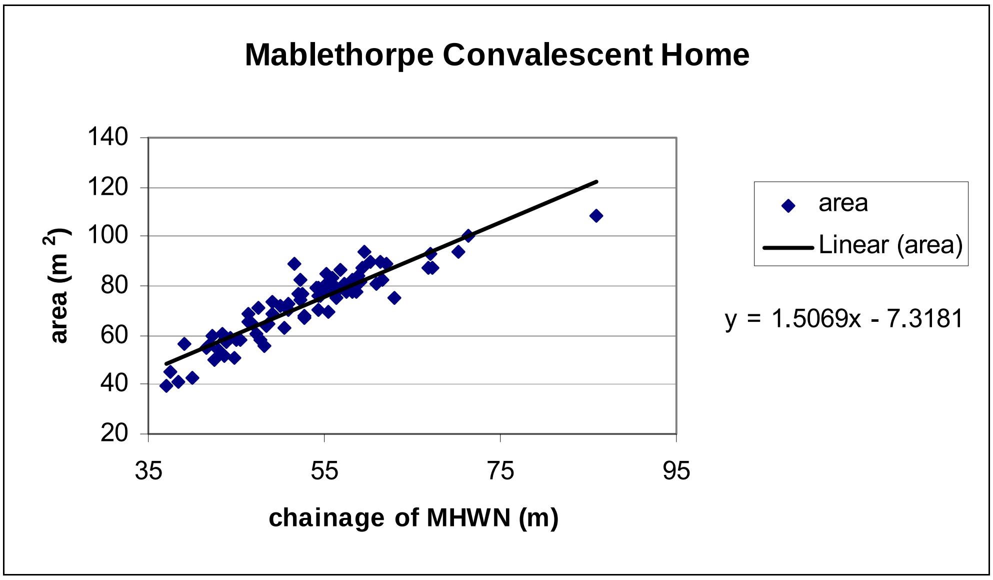


Figure 3.9 Cross-sectional area (above arbitrary datum) against chainage of MHWN for Mablethorpe Convalescent Home

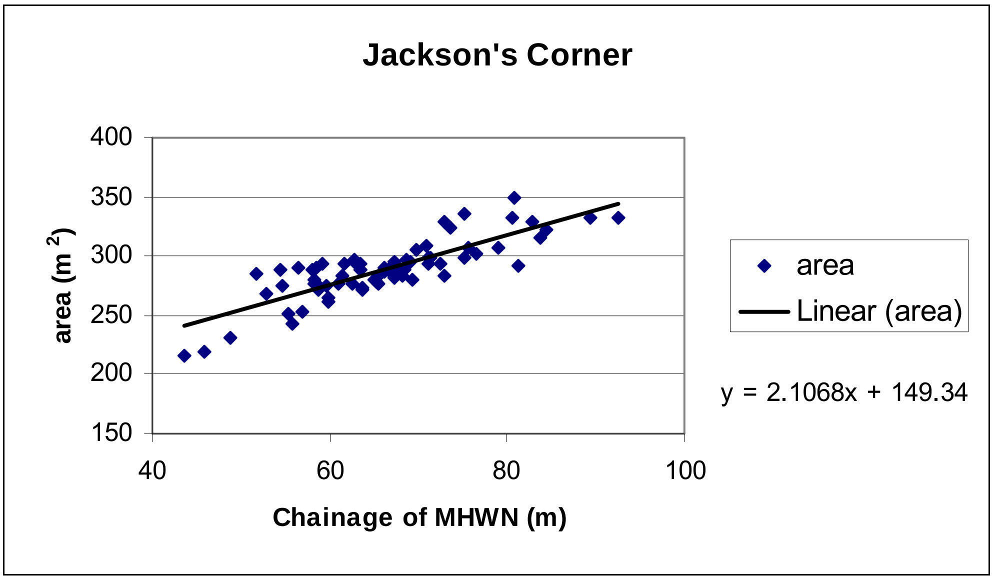


Figure 3.10 Cross-sectional area (above arbitrary datum) against chainage of MHWN for Jackson's Corner

However, the variation between the two sets of analyses indicates that there is a need to establish the relationship between the behaviour of a single contour and that of the beach volume at a local level before a contour can be used as a surrogate for beach volume, or vice versa.

3.3.13. Implications for beach monitoring

The beach management tools that have been derived from beach level measurements have been described in Section 3.3. The tools derived from simple, linear analyses are shown in Figure 3.11 with an indication of the spatial and temporal scales they can be applied over.

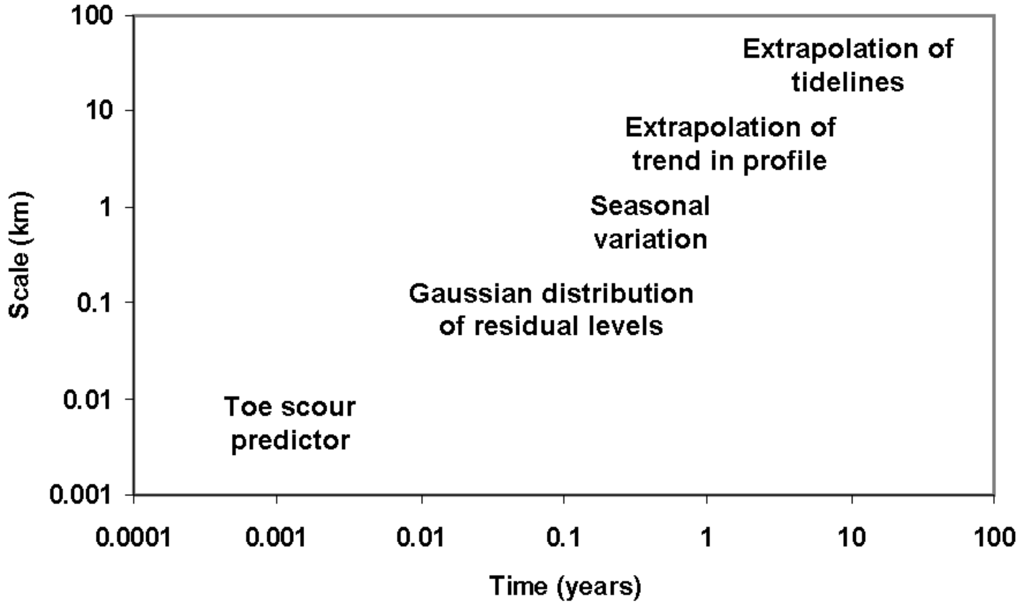


Figure 3.11 Beach management tools derived from simple linear analyses of beach level data

The implications for beach monitoring are:

- Advances in measurement technology have made it easier than ever before to measure beach levels at a point on a daily basis, or even more frequently;
- Many of the possible techniques for measuring time series beach levels at a point have not yet been evaluated for the cases of beach level at the toe of a structure;
- When beach levels have been measured at a point at a rate of at least once a day the time series have generally been short or (in the case of the Blackpool Tell-Tail data, section 3.1.1) there has been no tie-in to other data gathering programmes. The Blackpool data would have been more useful if it had been collected within an integrated programme for beach monitoring data;
- Statistical predictors of beach scour during a tide have been derived from field and laboratory data (see Section 4);
- There is a gap in frequency of data collection between the point measurements of beach levels through a tide (sampling about 4 times per hour) and beach profiles (collected typically 4 times per year). It is therefore impossible to determine from the data how the beach variations at the two frequencies are related;
- Beach levels at a point in front of a structure can generally be de-trended using a simple linear least squares method, providing that neither the coastal defences nor beach management policy changed during the data collection period;
- When a linear trend is extrapolated to provide a future prediction of beach level is has a site-specific prediction horizon. This is the average length of time over which an extrapolated trend produces a useful level of prediction compared to a baseline prediction (taken to be that future beach levels will be the same as the average of the measured beach levels);
- Based on an analysis of data from Lincolnshire, extrapolation of a linear trend fitted to 5 years of data always gave a negative skill score (i.e. a worse prediction than the baseline) so should not be used; and
- Extrapolation of a linear trend fitted to 10 years of data gave prediction horizons between 0 years and 14 years;
- The use of extrapolated beach levels is more suitable for managed / adaptive beach management policies, rather than the precautionary approach as the latter has a longer timeframe than the longest prediction horizon. It follows that predictions of leach levels made with an extrapolated trend are likely to have a negative level of skill when made for timescales of decades common for a precautionary approach;
- Residual (de-trended) beach levels have a Gaussian distribution about the mean beach level, again providing that neither the coastal defences nor beach management policy changed during the data collection period;
- The standard deviation in beach level from using 2 surveys per year was only 6% different from using 10 surveys per year. However, the difference was approximately halved by increasing the number of surveys from 2 to 3 times per year.

- A clear seasonal trend was observed in the Linconshire dataset of beach levels at the toe of the local seawalls. The trend was lower than the standard error about the trend.
- Beach surveys that are intended to predict the long-term trends in shoreline position should be made when the standard deviation in the beach level is low. This occurred in August for the Lincolnshire data (but in June/July for Duck, N.C., U.S.A.) and also coincided with relatively high beach levels.
- Beach surveys that are intended to indicate how low beach levels can fall should be undertaken when the average beach level is low and the standard deviation in beach level is high. In Lincolnshire this occurred around March.
- Advanced linear analyses of beach level data (such as the use of wavelets and Empirical Orthogonal Functions) and nonlinear analyses of beach level data (such as Singular Spectrum Analysis and fractal analysis) are becoming more common in academic circles. These sophisticated methods require more data of good quality than the simple linear methods require. They may also impose more constraints on the data, such as the need to be equally spaced in time and position. It will be possible to apply these methods to more areas of the English and Welsh coastlines as coordinated regional data gathering and data management programmes extend their geographical range and temporal duration.
- Beaches around Donna Nook in north Lincolnshire often showed an increase in beach volume (area under a beach profile) combined with a retreat (shoreward movement) in Mean Sea Level. Further south in Lincolnshire (between Mablethorpe and Skegness) beach volumes were found to increase as the Mean High Water advanced (moved seawards).
- There is a need to establish the relationship between the behaviour of a single contour and that of the beach volume at a local level before a contour can be used as a surrogate for beach volume.

Further data analysis should be undertaken to ascertain if a regional approach could be taken to providing guidance on possible changes in beach levels, for use in the design of new structures, in a similar way to the regional net sea level rise allowances (Defra, 2006).

3.4. Process-based, parametric and geomorphologic tools for predicting coastal evolution

This section covers the use of process-based models, parametric models and geomorphological analysis for predicting coastal evolution. A sub-section describes each of these types of tools. It is not the purpose of this report to describe these methods, except where new methods have been developed and demonstrated. This is intended to be complementary to the descriptions and analysis provided in Defra's revised Shoreline Management Plan (SMP) Procedural Guidance, Appendix D (Defra 2003a), which is intended to apply to relatively long timescales for coastal management (10 to 100 years).

3.4.1. Process-based numerical modelling

A considerable amount of research has been carried out over the least 20 years to develop predictive numerical models of coastal evolution covering periods of up to 20 years or more. These models are based on representations of physical processes and typically include forcing by waves and/or currents, a response in terms of sediment transport and a morphology-updating module. However, there are still major gaps in our understanding of long-term morphological behaviour (de Vriend et al., 1993, Southgate and Brampton, 2001, de Vriend, 2003, Hanson et al., 2003) which mean that modelling results are subject to a considerable degree of uncertainty. Their use requires a high level of specialised knowledge of science, engineering and management.

Southgate and Brampton (2001) provide a guide to model usage, which considers the engineering and management options and the strategies that can be adopted, while working within the limitations of a shortfall in our scientific knowledge and data. An introduction to the following model types can be found in HR Wallingford (2006c):

- One-line models
- Coastal Profile Models
- Coastal Area Models
- Systems model SCAPE

These reviews are not included here, apart from the systems model SCAPE, which is less well known and could form the template for a systems models covering an increased range of geomorphological units.

Recent advances in the understanding of skewness and asymmetry in the surf zone, incorporation of swash processes, the development of phase resolving nearshore numerical wave models and the improvement of coastal sediment transport models all hold out the possibility of improving coastal sediment models to be able to model beach recovery. If this can be done then, for example, profile models may be able to model periods between a tide and a few weeks where there is presently a shortage of understanding of beach behaviour due to a shortage of data and model skill.

A three-dimensional model system built around coastal profile modelling for long stretches of coastline has been developed and applied by Stripling and Panzeri (2007). This offers opportunities for a regional modelling capability including the assessment of management options.

3.4.1.1. Systems model: SCAPE

Walkden and Hall (2005) have developed a long-term model of the effect of waves, tides and sea level rise on littoral transport and the erosion and profile development of soft cliffs and shore platforms, called Soft Cliff And Platform Erosion (SCAPE). This models the development of a system of geomorphological units: the shore platform, beach, talus and cliff, at a series of representative cross-shore profiles, each of which is represented by a column of elements. The quasi-3D representation is achieved by allowing the profiles to interact, by exchanging beach material alongshore between profiles using a simple one-line model of the beach.

Each cross-shore profile can also be run independently (provided the beach volume is set by the user). SCAPE is effectively a set of relatively simple cross-shore profile models that are linked by a one-line model. The development of the system is controlled by feedback mechanisms. For example, cliff erosion produces tallus, which erodes partly into beach sand, which increases the beach volume, which protects the shore platform and cliff from erosion.

SCAPE models the interactions between different elements of the system and the emergence of system properties, particularly profile shape. The model is process-based (although with relatively large-scale behavioural elements) so allows the effects of climate change and the construction of local defences to be included. The model may be run over the timescales of decades (Walkden & Hall, 2005) and centuries (Dickson et al, in press) and over tens of kilometres. SCAPE has been used to model the soft cliff and platform erosion at the Naze, Essex (Walkden and Hall, 2005) and the between Weybourne and Happisburgh, Norfolk (Dickson et al., 2005).

3.4.1.2. Limitations of applicability

The approximate limitations of applicability of the types of numerical models are illustrated in Figure 3.12. Coastal tract models are based on sediment budgets (see section 3.4.2). Figure 3.12 shows that the numerical models attempt to describe fewer and fewer processes in detail as the spatial and temporal scale they are deployed over increases.

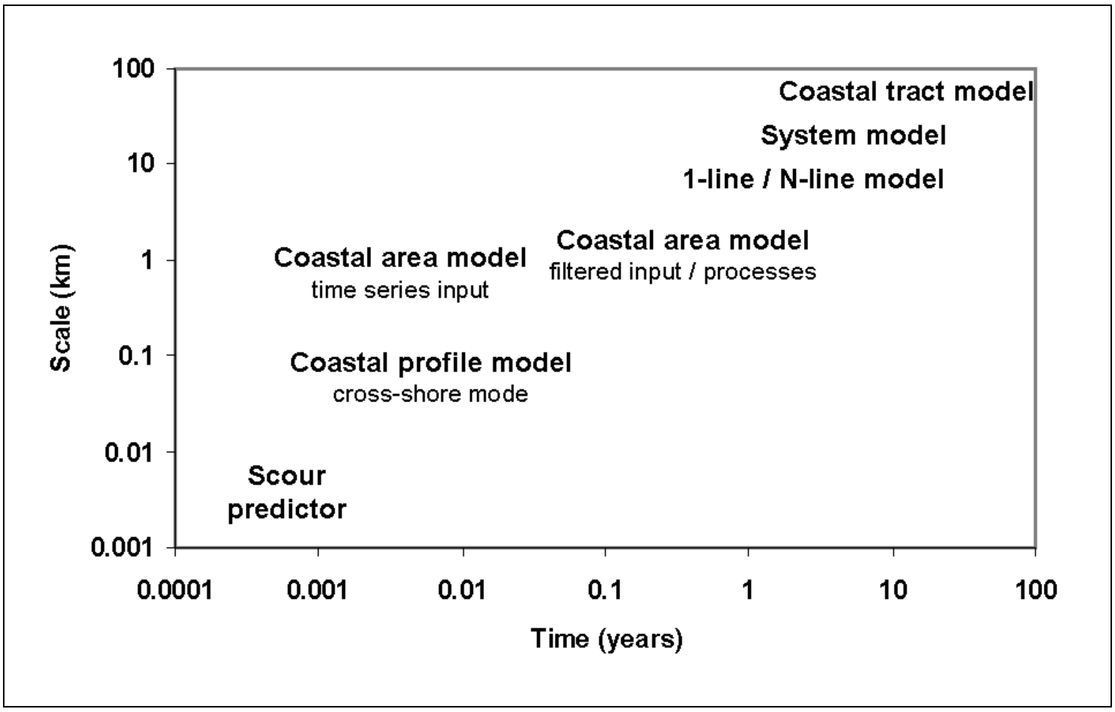


Figure 3.12 Indication of spatial scale and length of prediction for different numerical model types

3.4.2. Geomorphological analysis

Geomorphology is the study of the features that make up the earth's surface and their relationship to the underlying geology. A geomorphological study will provide a conceptual picture of coastal processes and the potential behaviour of the coastal system. This includes taking into account changes in the bedrock composition that could affect the potential rate of future coastal evolution. The results tend to be qualitative, rather than quantitative. Reviews of the following approaches and projects are given in HR Wallingford (2006c):

- Sediment budget
- Futurecoast
- Eurosion
- Coastal tract modelling

Many geomorphology studies use a range of tools, including predictive numerical models. As such many geomorphology studies are effectively a composite of the different modelling techniques, as advocated by, for example, Cooper and Pilkey (2004). A similar approach has been taken for estuaries in project FD2116 (Whitehouse et al., 2006).

A good sediment budget will provide a useful indication of whether a beach in front of a coastal structure is likely to be subjected to beach lowering due to loss of sediment from the entire beach. Even if this is not the case and beach volumes have been constant or increasing, a coastal structure may be subject to beach lowering due to local effects.

Eurosion (European Commission, 2004) concluded that a more strategic and proactive approach to coastal erosion is needed for the sustained development of vulnerable coastal zones. It developed the concept of coastal resilience: the inherent ability of the coast to accommodate changes induced by sea level rise, extreme events and occasional human impacts, whilst maintaining the functions fulfilled by the coastal system in the longer term. To promote coastal resilience, Eurosion introduced the concept of favourable sediment status: the situation where the availability of coastal sediments supports the objective of promoting coastal resilience in general and of preserving dynamic coastlines in particular. This should be achieved for each coastal sediment cell by designating strategic sediment reservoirs: supplies of sediment of appropriate characteristics that are available for replenishment of the coastal zone, either temporarily (to compensate for losses due to extreme storms) or in the long term (at least 100 years). They can be identified offshore, in the coastal zone (both above and below low water) and in the hinterland.

The European Commission has subsequently funded a Specific Targeted Research Project called Concepts and Science for Coastal Erosion, or CONSCIENCE (2007-2010) to provide guidelines and tools for coastal managers to implement the concepts of coastal resilience developed in Eurosion.

3.4.3. Parametric equilibrium models

Parametric equilibrium models represent the shape of the coastline or its response to forcing through simple equations that have been derived through a mixture of curve-fitting and theoretical considerations. They are necessarily simplistic, but quick to apply. Three main equilibrium beach concepts are commonly used to predict coastal morphology:

- Equilibrium beach profile (Bruun, 1954 as reviewed by Dean, et al., 2002);
- Bruun rule for coastal retreat (Bruun, 1962);
- Log-spiral coastlines (Sivester and Hsu, 1997).

The use of all three is reviewed in HR Wallingford (2006c). An application of the Bruun rule to study the potential affects of accelerated sea level rise has been developed and is given in Section 3.4.3.1, below.

3.4.3.1. Bruun rule for coastal retreat

Bruun (1962) proposed Equation 5 for the equilibrium shoreline retreat, R, of sandy coasts that will occur as a result of sea level rise, S.

$$R = S\frac{L}{h + B} \tag{5}$$

Here L is the cross-shore width of the active profile (i.e. cross-shore distance from closure depth to furthest landward point of sediment transport), h is the closure depth (maximum depth of sediment transport) and B is the elevation of the beach or dune crest (maximum height of sediment transport). The equation balances sediment yield R(h+B) from the horizontal retreat of the profile with sediment demand, SL, from a vertical rise in the profile (Dean et al., 2002). The magnitudes of h and B are difficult to determine, however and the actual seabed will need time to respond to a change in sea level.

The Bruun rule does not depend on a particular coastal profile, but does assume that no sediment is lost from the coastal system (which is likely to happen if there are fines in the area eroded). It assumes a coast of unconsolidated sediment, mainly sand, with (originally) a coastal dune and makes no allowances for gradients in the longshore or cross-shore transport of sand. However, the Bruun rule has been extensively modified, developed and used (see Dean et al., 2002 for a summary).

In the coastal regions where the Bruun rule can be said to apply, the rate of shoreline retreat (dR/dt) is directly proportional to the rate of sea level rise (dS/dt). It follows that the ratio of future shoreline retreat rate to present day shoreline retreat rate (the shoreline retreat rate multiplier) will be the same as the ratio of future sea level rise rate to present day sea level rise rate.

The shoreline retreat rate multiplier can be calculated using present day rates of sea level rise (IPCC, 2007) and regional sea level allowances from Defra (2006). IPCC (2007) states that the average rate of sea level rise (eustatic change) from 1961 to 2003 was 1.8 mm/yr. This was combined with the regional rates of vertical land movement (the isostatic change from Defra, 2006, derived from Shennan and Horton, 2002) to give regional rates of net relative sea level rise from 1961 to 2003, as shown in Table 3.5.

Table 3.5 Net rate of sea level rise from Defra (2006) and IPCC (2007).

Region	Land movement (mm/yr)	Net rate of sea level rise (mm/yr) From 1961 to 2003	From 1990 to 2025	From 2025 to 2055	From 2055 to 2085	From 2095 to 2115
E & SE England (South of Flamborough Head)	-0.8	2.6	4.0	8.5	12.0	15.0
SW England & Wales	-0.5	2.3	3.5	8.0	11.5	14.5
NW & NE England, Scotland	0.8	1.0	2.5	7.0	10.0	13.0

The shoreline retreat rate multiplier was then taken as the net rate of sea level rise from future periods divided by the net rate of sea level rise from 1961 to 2003. The results are shown in Table 3.6 and show that shoreline retreat rates in regions where the Bruun rule applies could increase significantly – in some cases by a factor of 13 - during the 21st century. The shoreline retreat rate multipliers are highest for the Northwest and Northeast of England and Scotland as this region has the lowest present day rate of sea level rise, due to isostatic rebound following the last ice age, which may also imply lower rates of present day shoreline retreat.

Table 3.6 Shoreline retreat rate multipliers for different time spans

Region	Shoreline retreat rate multiplier 1961 to 2003	1990 to 2025	2025 to 2055	2055 to 2085	2095 to 2115
E & SE England (South of Flamborough Head)	1.0	1.5	3.3	4.6	5.8
SW England & Wales	1.0	1.5	3.5	5.0	6.3
NW & NE England, Scotland	1.0	2.5	7.0	10.0	13.0

These results should be treated with some caution, however, as the Bruun rule is a very simplistic analysis tool and difficult to validate. Bray and Hooke (1997) adapted it to look at the erosion of soft cliffs by adding sediment exchange and considered it particularly suitable for assessing the sensitivity of eroding soft cliffs to future climate change. On the other hand both Cooper and Pilkey (2004) and Stive (2004) cautioned against its use due to its simplicity and restrictions.

Dickson et al. (2007) compared the predictions of recession from the modified Bruun rule and the systems model SCAPE (Walkden and Hall, 2005) for 50km of the soft rock shoreline of northeast Norfolk. They found that the systems model SCAPE predicted a rather more complex response, with lower overall vulnerability to sea level rise, than the Bruun rule. Where beaches overlie shore platforms both SCAPE and the Bruun rule gave accelerated recession rates in response to sea level rise. However, in some areas with large beaches and gradients in the longshore transport rates the Bruun rule predicted recession where SCAPE predicted accretion due to sediment transport from eroding up-drift stretches of the coastline. This indicated the inadequacy of the Bruun rule in regions where there is a significant variability in the longshore transport rates.

Therefore the magnitudes of the shoreline retreat rate multipliers in Table 3.6 should be treated with some caution as they may well be too high. However, it appears probable that the shoreline recession rate will increase in many places if the rate of sea level rise increases.

3.5. Prediction of bed levels at different scales

3.5.1. Predicting bed levels at a scale of tides and storms

3.5.1.1. Monitoring

Field work (HR Wallingford, 2006a, Sutherland et al., 2006a and HR Wallingford, 2006d) has shown how bed levels can fall and recover during a tide. Measuring the variations during a number of tides can allow the distribution of local scour depths to be determined. This data can be used to determine the likely range of scour depths that can be achieved at a location. Alternatively the results can be analysed to give an empirical predictor of the short-term scour depth (as has been done in Section 4). The data itself could be used in real time to trigger a warning system, should beach levels drop too low.

3.5.1.2. Process-based modelling

Process-based numerical models of cross-shore beach evolution have been used for a number of years to predict the (generally) short-term cross-shore response of beaches to storms (van Rijn et al., 2003, Southgate and Nairn, 1993, Nairn and Southgate, 1993). Cross-shore profile models assume longshore uniformity and model the cross-shore hydrodynamics, sediment transport and bed level changes. These models have often been used to model the short-term cross-shore beach profile response to storms, but are generally less capable of modelling the recovery of beaches after a storm. They could be combined with a wave forecasting system (driven by a weather forecast) to predict changes in bed level over periods of up to about 5 days.

3.5.1.3. Empirical Scour Predictors

A variety of empirical scour predictors have been developed for predicting the short-term response of beaches to waves and water levels. These are dealt with in Section 4.

3.5.2. Predicting bed levels at the scale of weeks and seasons

3.5.2.1. Coastal profile modelling

Coastal profile models could be used over a timescale of weeks and in some calibrated cases potentially months to predict bed levels (van Rijn et al., 2003).

3.5.2.2. Extrapolation of measured beach levels

Measured beach levels can be used to identify a linear trend, a seasonal trend and a Gaussian distribution of beach levels about that trend (see Section 3.3).

3.5.3. Predicting bed levels at the scale of years

Bed levels at the toe of structures are not generally calculated at a timescale of years and decades. It is more common to try and predict the behaviour of the shoreline and methods for doing this are discussed in Defra (2003a, Appendix D) and in Section 3.4 here. Changes in shoreline position can be related to beach level at the toe of a structure through knowledge of the beach slope. Defra 2003a Appendix D includes a comparison of the following methods for analyse shoreline interactions and responses:

- Extrapolation of historical data (covered in Section 3.3 here);
- Numerical modelling (covered here in Section 3.4.1);
- Geomorphic extrapolation (covered here in Section 3.4.2);
- Parametric equilibrium models (covered here in Section 3.4.3).

These methods could also be used over shorter timescales of a few years. In particular bed levels at the toe of a coastal structure can be predicted in some locations using the extrapolation of a linear trend obtained from at least 10 years' data, as discussed in Section 3.3.2. Different locations along the Lincolnshire coastline were found to have different prediction horizons (i.e. different lengths of prediction before the predicted beach levels from the extrapolation of the linear trend became on average worse than the use of the average beach level). Further work should be undertaken to establishing typical prediction horizons from other stretches of coastline where sufficient data is available.

Intrinsic limits to knowledge mean that predictions of future shoreline position over a timescale of years to decades will never be definitive, particularly when considering the effects of climate change. Therefore it is useful to take an approach based on a range of available methods and data to improve confidence in shoreline position and to arrive at the most likely position.

4. Improved scour predictor

This chapter sets out the process by which the development of an improved scour predictor for sand beaches in front of seawalls has been undertaken. The scour predictors have been developed using an extensive database of new and previously published laboratory data. The new data was obtained to deal with shortcomings in the previous studies.

4.1. Shortcomings in previous studies

The Stage 1 scoping study report (Sutherland et al., 2003) identified two main shortcomings in the available laboratory data for scour prediction in front of seawalls and hence in the suitability of the empirical predictors of toe scour. The first is that the majority of laboratory scour tests have been performed in relatively small wave flumes where the sediment transport was dominated by bedload transport. These experimenters were aware of the mode of sediment transport being generated and explicitly formulated empirical formulae for scour by bedload transport. However, in many cases in the field, especially with sand beaches, sediment transport is dominated by suspended sediment transport. There has been a shortage of controlled laboratory experiments where suspended sediment transport has been the dominant mode of transport. This is important as the response of the beach to bedload and suspended load transport is different (Whitehouse, 1998, HR Wallingford 2006b, Sutherland et al., 2003).

The second major shortcoming is that many scour tests have been performed with regular period waves, rather than irregular waves with a natural spectral shape. It is not clear how to apply a scour predictor developed from regular wave laboratory experiments to a natural situation with irregular waves where wave reflections can be expected to become increasingly out of phase with distance from the wall (Hughes and Fowler, 1991).

4.2. Development of the database

In order to address these shortcomings a database was established of laboratory tests (Fowler, 1992, Kraus and Smith, 1994) that used irregular waves and were large enough to generate suspended sediment transport (HR Wallingford, 2006b). A set of laboratory flume experiments was devised to extend this database (HR Wallingford, 2006b) and conducted (HR Wallingford 2006f, Sutherland et al., 2006b). The combined dataset was then analysed (HR Wallingford, 2006i, Pearce et al., 2006, Pearce et al., 2007).

The tests performed at HR Wallingford were within the following ranges:

- $0.000 \le h_t/L_p \le 0.073$;
- $0.059 \le Ir \le 0.430$;
- $0.00 \le h_t/H_s \le 2.08$;

- $0.006 \le H_s/L_p \le 0.052$;

Where h_t is the still water depth at the toe of the structure, L_p is the deep water linear theory wavelength based on the spectral peak period, Ir is the Iribarren number or surf similarity parameter and H_s is the offshore spectral significant wave height. Tests were performed using a smooth impermeable vertical seawall with beach slopes of 1:30 then 1:75 and with a smooth impermeable seawall, sloping at 1:2 (V:H) with a beach slope of 1:75. Some tests were performed with varying water levels, which included negative toe depths (i.e. emerged beach at the seawall toe) for part of the time. However, only the tests with constant water levels were used in the development of the scour predictor, hence the negative toe depths are not included in the ranges above.

The other datasets used were those of Fowler (1992) Kraus and Smith (1994) and Xie (1981). In all cases only the subset of all the tests that used irregular waves and generated suspended sediment transport were used. Fowler (1992) performed 18 tests using a smooth impermeable vertical seawall with beach slopes of 1:15. Kraus and Smith (1994) ran one Supertank test with a smooth impermeable vertical seawall and an initial bed slope of approximately 1:23, while Xie completed two tests with a smooth impermeable vertical seawall and a flat seabed in relatively deep water.

Two scour depths were identified for each test after three thousand spectral peak periods: the scour depth at the structure toe, S_t, and the maximum scour depth, S_{max} and equations were derived for these parameters by fitting curves to the data. In some cases the maximum scour depth occurred at the seawall. In these cases $S_t = S_{max}$.

The non-dimensional scour depths were found to vary as a function of relative water depth, h_t/L_m where L_m is the deep water linear theory water wavelength based on the spectral mean period, as shown in Figure 4.1 for toe scour and Figure 4.2 for the maximum scour depth. Negative values for the toe scour denote accretion at the structure, while negative values of the relative water depth at the structure toe denote the presence of a beach that extended above the still water level at the start of the test.

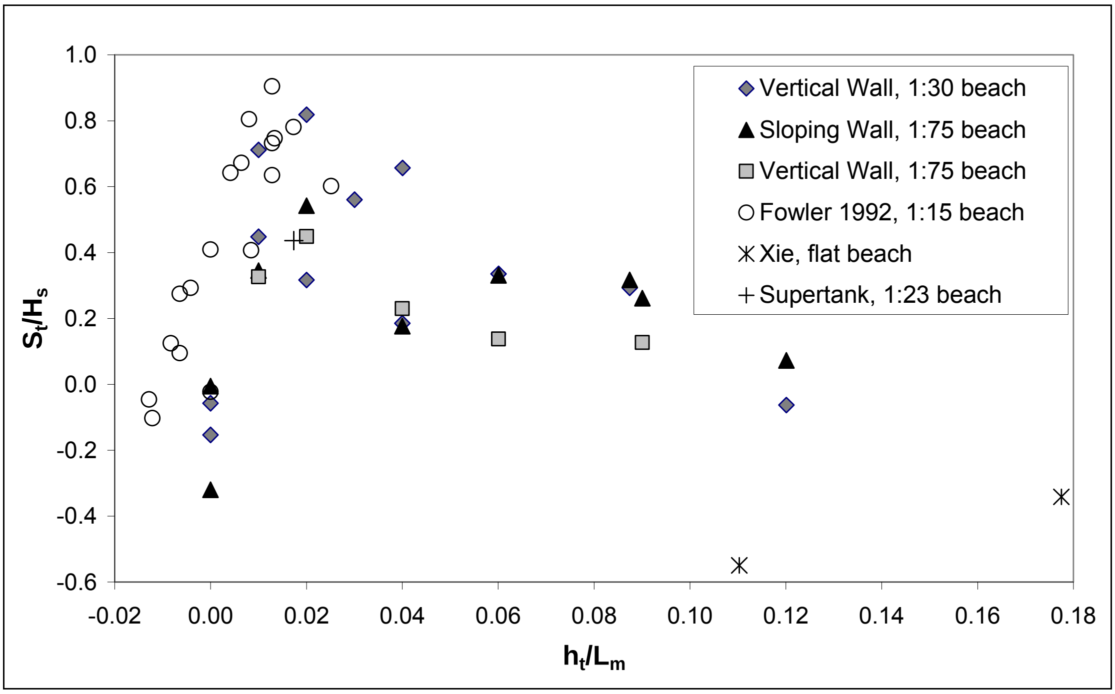


Figure 4.1 Relative toe scour depth plotted against relative water depth at the structure toe

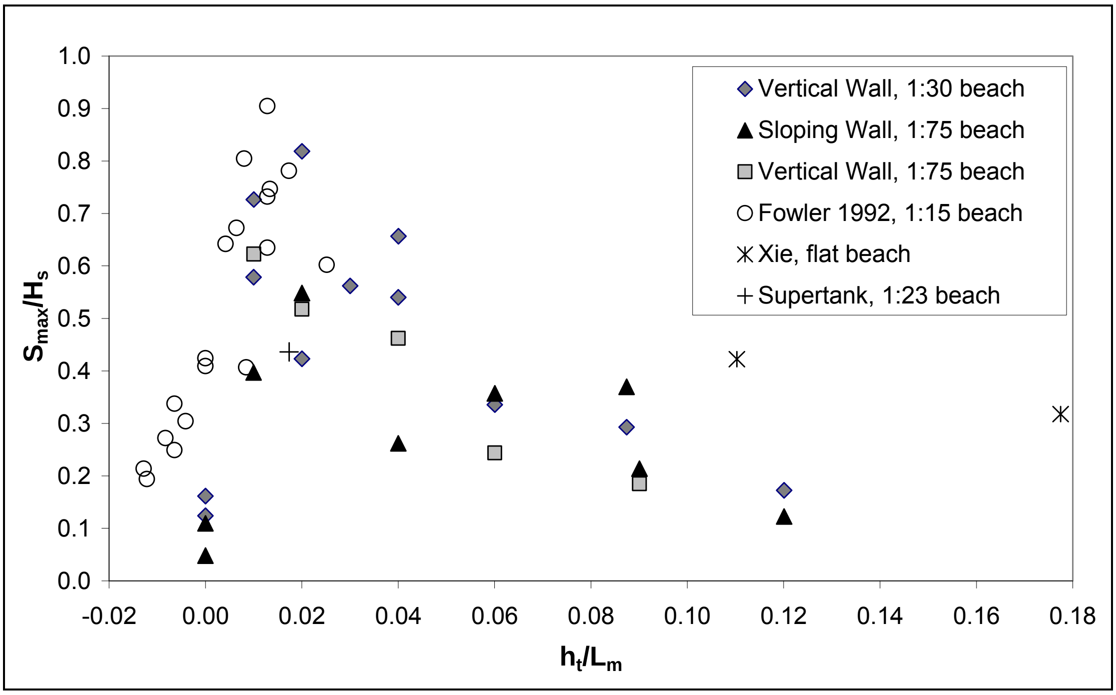


Figure 4.2 Relative maximum scour depth plotted against relative water depth at the structure toe

The values of relative scour depth on both graphs peak at about h_t/L_m = 0.016 and show that the scour predictors of Fowler (1992) and Xie (1981) both had the correct form of the relationship between relative scour depth and relative water depths for their respective ranges of relative water depths. Fowler (1992) predicted increasing relative toe scour for increasing relative depths at low relative depths while Xie (1981) and subsequently Sumer and Fredsøe (2000) and others had predicted decreasing relative toe scour for increasing relative depths at higher relative depths. Both are correct in their relative ranges.

4.3. Development of improved scour predictors

4.3.1. Best-fit pairs of equations

Best fit equations were derived for toe scour and maximum scour, by splitting the data into low and high ranges of relative depth, denoted LHS and RHS and fitting separate curves to each, which intersect at the break point around h_t/L_m = 0.016 identified above. The scour predictors need to be able to work for the case when the initial beach level is above the still water level. In this case the beach may become submerged by wave set-up at the shoreline. This was achieved by incorporating Holman and Sallenger's (1985) expression for the maximum set-up into the calculation of the toe depth.

Holman and Sallenger's (1985) expression for the maximum set-up, $\overline{\eta}_{max}$, that would occur on a natural beach is given in Equation 6, where both the wave height and wavelength (in the Iribarren number, Ir) are calculated in deep water but the beach slope, α, is calculated at breaking.

$$\overline{\eta}_{max} = 0.45 H_s Ir = 0.45 \tan(\alpha)\sqrt{H_s L_p} \qquad (6)$$

A new relative toe depth was then calculated given by Equation 7:

$$\frac{h_t^*}{L_m} = \frac{h_t}{L_m} + \overline{\eta}_{max} \qquad \text{for } h_t/L_m < 0 \qquad (7)$$

Equation 7 was only applied for cases where $h_t/L_m \leq 0$ as the set-up is a maximum at the shoreline and decreases to the breaker line, where set-down will occur. This relatively simple approach was adopted to see if this change made a difference to the results. In practice there will be an interaction between the incident and reflected waves so parameterisations of setup derived for the open coast may not be particularly accurate. This allowed the Root-Mean-Square Error (RMSE) to be reduced while simplifying the equations for toe scour and maximum scour at a vertical seawall on a sand beach.

The best-fit equations for toe scour depth are given in Equations 8 (LHS) and 9 (RHS), which have the break point at $h_t^*/L_m = 0.018$, where h_t^* is the toe depth including setup for $h_t/L_m \leq 0$.

$$\frac{S_t}{H_S} = \left(50\frac{h_t^*}{L_m}\right) \qquad h_t^*/L_m \leq 0.018 \qquad (8)$$

$$\frac{S_t}{H_s} = \frac{0.014}{\sinh\left(h_t^*/L_m\right)^{1.05}} \qquad h_t^*/L_m > 0.018 \qquad (9)$$

The best-fit lines are shown in Figure 4.3, with the error statistics given in Table 4.1, where:

- Equ^n is the number of the Equations represented in that row;
- BP is the breakpoint – the value of h_t/L_m which represents the split between LHS and RHS;
- RMSE is the Root-Mean-Square Error, which has been split into systematic and unsystematic parts $RMSE_S$ and $RMSE_U$, respectively (as described in HR Wallingford, 2006c).
- Bias is the difference between the predicted and observed mean value.

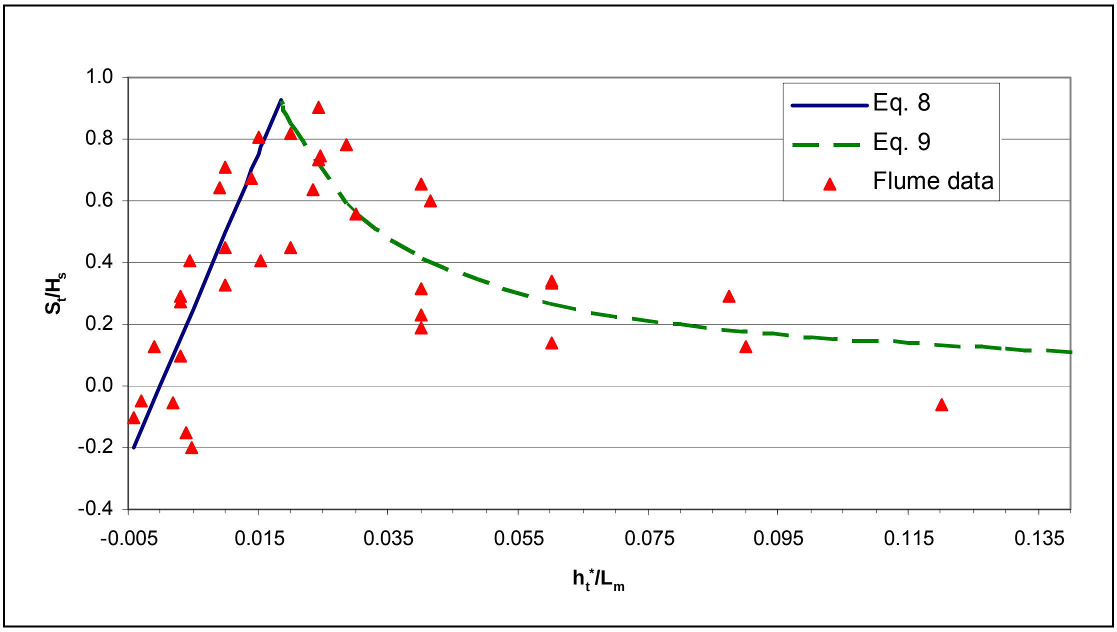


Figure 4.3 Simplified best fit lines plotted on graph of S_t/H_s versus h_t/L_m

Table 4.1 Error statistics from simplified equations

		$RMSE_S$		$RMSE_U$		Bias	Bias	RMSE	RMSE
Equ^n	BP	LHS	RHS	LHS	RHS	LHS	RHS	LHS	RHS
8, 9	0.018	0.088	0.095	0.184	0.138	0.021	0.009	0.204	0.168
10, 11	0.016	0.012	0.054	0.155	0.101	-0.011	-0.011	0.155	0.115

The best-fit equations for maximum scour depth are given in Equations 10 and 11, which have the break point at $h_t^*/L_m = 0.016$, where h_t^* is the toe depth including setup given by Equation 6 for $h_t/L_m \leq 0$.

$$\frac{S_{max}}{H_S} = \left(35\frac{h_t^*}{L_m} + 0.40\right)^2 \qquad h_t^*/L_m \leq 0.016 \qquad (10)$$

$$\frac{S_{max}}{H_s} = \frac{0.036}{\sinh\left(h_t^*/L_m\right)^{0.80}} \qquad h_t^*/L_m > 0.016 \qquad (11)$$

The best-fit lines are shown in Figure 4.4, with the error statistics given in Table 4.1.

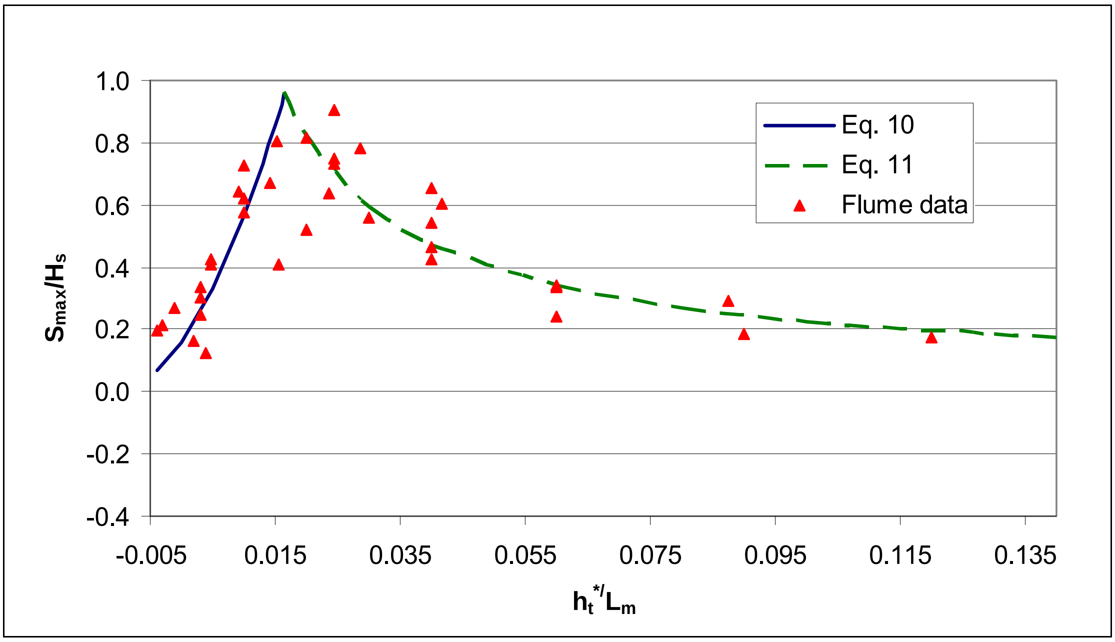


Figure 4.4 Simplified best fit lines plotted on graph of S_{max}/H_s versus h_t^*/L_m

Equations 8 and 9, for calculating toe scour, and Equations 10 and 11, for calculating the maximum scour depth give relative scour depths of $S/H_s \leq 1$, a well known rule of thumb for scour prediction. In each case the RMS error can be used to estimate the uncertainty in the prediction.

4.3.2. Conservative equation for toe scour

The single equation has also been fitted to the maximum value of relative toe scour depth, S_{tmax}, for each range of relative water depth. This is intended to provide a conservative estimate of the toe scour depth and was fitted by eye to the data. It is given in Equation 12, which is considered a reasonable predictor of the maximum toe scour depth in sand likely to be encountered for a given water depth at the structure toe, h_t, and offshore linear theory mean wavelength, L_m. Setup is not included in this method.

$$\frac{S_{t\,max}}{H_s} = 4.5e^{-8\pi(h_t/L_m+0.01)}\left(1 - e^{-6\pi(h_t/L_m+0.01)}\right) \qquad (12)$$

The tests were within the following ranges: $-0.013 \leq h_t/L_m \leq 0.18$ and $0 \leq Ir \leq 0.43$. Equation 12 should only be applied within those ranges and users should note that some parts of those ranges were covered more thoroughly than others. The maximum scour depth appears to decrease as beach slope decreases for the same offshore wave conditions. Moreover, the maximum scour depth seems to occur at larger relative depths for lower beach slopes. However, neither phenomenon has been well validated so they are not included in this method. Equation 12 is plotted with the laboratory and field test data in Figure 4.5

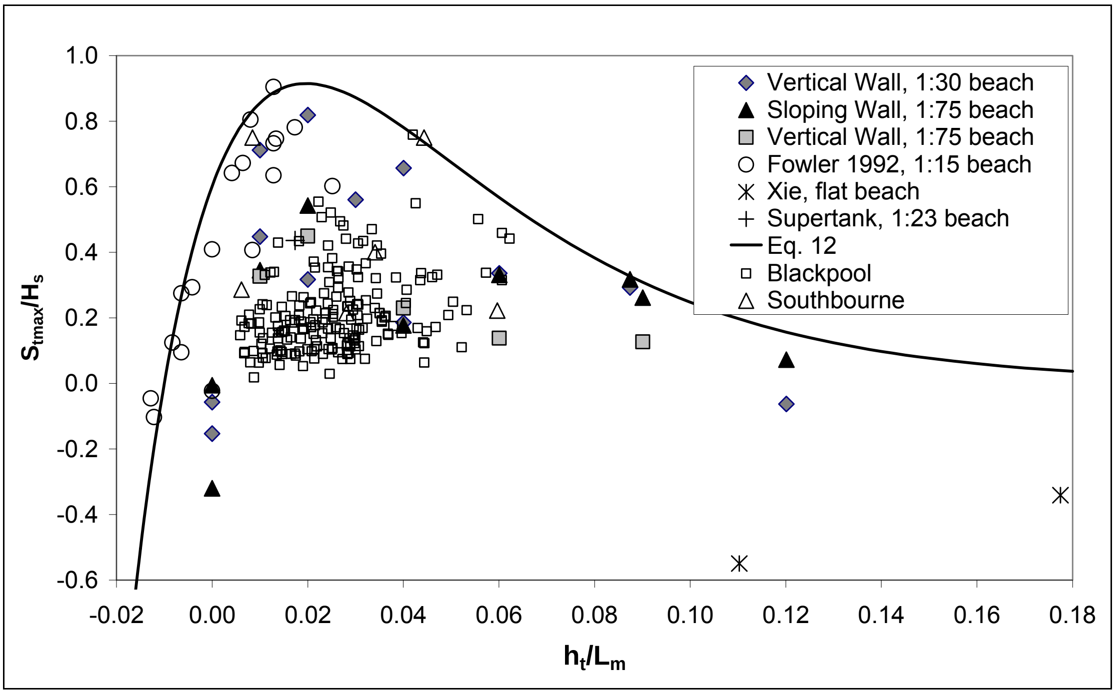


Figure 4.5 Conservative predictor of toe scour depth

4.3.3. Single equations for toe scour depth

Equations of the form of Equation 12 were fitted to the data (this time including the Holman and Sallenger setup for $h_t/L_m \leq 0$) to provide best estimate of the scour depth as a function of relative depth. The fit to the data that minimised the Root-Mean-Square Error is given in Equation 13, the error statistics for which are given in Table 4.1.

$$\frac{S_t}{H_s} = 10e^{-7.25k_m h_t^*}\left(1 - e^{-1.35k_m h_t^*}\right) \qquad (13)$$

Table 4.2 Error statistics for Equations 13 and 14

Equation	$RMSE_S$	$RMSE_U$	Bias	RMSE	Best fit line slope	Best fit line intercept
13	0.107	0.169	-0.014	0.200	0.652	0.116
14	0.000	0.226	0.000	0.226	0.999	0.000

Equation 13 is plotted with the measured data in Figure 4.6 Equation 13 produces a relatively low RMSE = 0.20 but this consists of both systematic and unsystematic components. When the predicted relative scour depths are plotted against the measured, the resulting best-fit straight line has a slope of 0.65 and an intercept of 0.12. Equation 13 also under predicts the highest relative scour depths and has a small bias.

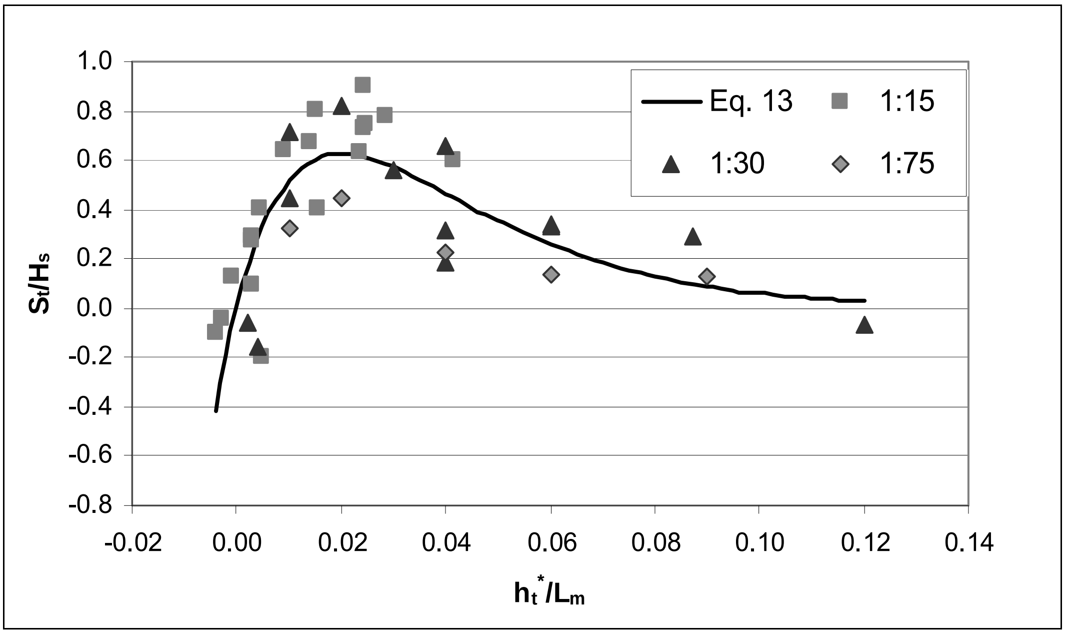


Figure 4.6 Measured and predicted (Equation 13) relative toe scour depth as a function of relative toe depth

Therefore an equation of the form of Equation 13 was fitted to reduce the systematic Root-Mean-Square Error, $RMSE_S$. The predicted scour depths were subtracted from the measured to give the residual scour depths. These were shown to increase with beach slope and reduce as h_t/H_s increases. Equation 14 was developed to take into account the variation of scour depth with beach slope (α).

$$\frac{S_t}{H_s} = 6.8(0.207\ln(\alpha) + 1.51)e^{-5.85k_m h_t}\left(1 - e^{-3k_m h_t}\right) - 0.137 \qquad [-0.04 \le h_t/L_m \le 0.12] \qquad (14)$$

Equation 14 is plotted with the measured data in Figure 4.7, where 'O 1:N' and 'P 1:N' are the observed and predicted scour depths with a beach slope of 1:N (with N = 15, 30 or 75) respectively. Its error statistics are given in Table 4.2. Equation 14 has zero bias and systematic error and predicts the highest toe scour depths relatively well. In Figure 4.7 Equation 14 is plotted for the range of h_t^*/L_m for which data was obtained at that particular beach slope.

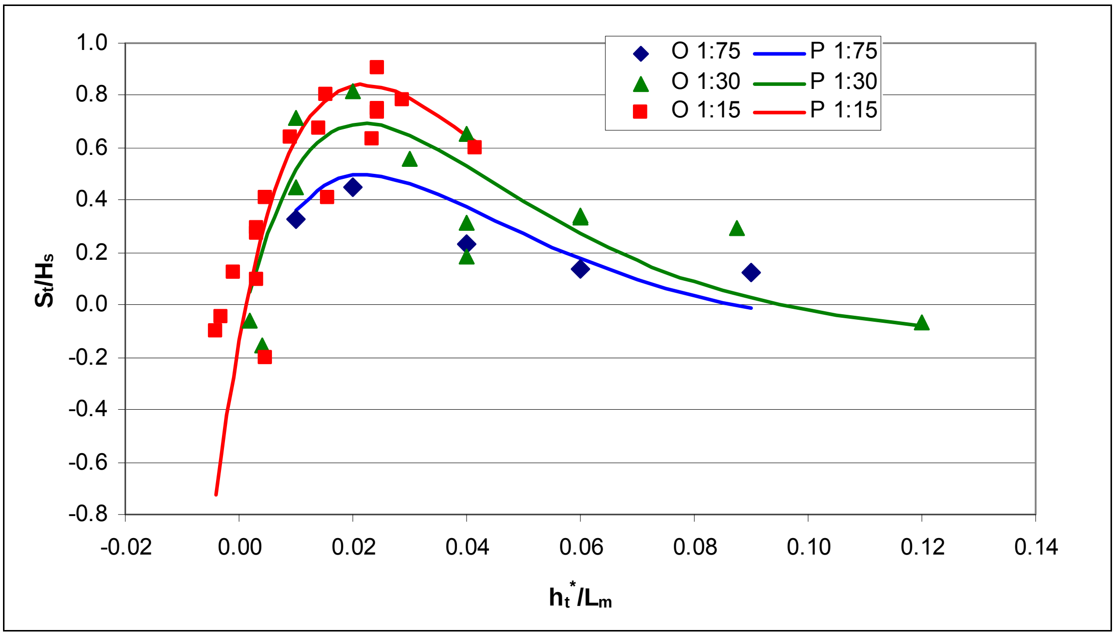


Figure 4.7 Measured and predicted (Equation 14) relative toe scour depths as a function of relative toe depth

The predicted relative scour depths are plotted against the measured (or observed) in Figure 4.8. This shows that the best-fit straight line has a slope of 0.999 and an intercept of zero. This indicates that the relationship between relative toe scour and relative toe depth has been represented accurately. Moreover, there is relatively low scatter about the best-fit line for the high relative scour depths, which are likely to be the most important. For observed relative scour depths greater than $S_t/Hs = 0.5$, the predictions all lie within 50% of the observed values. For all observed scour depths greater than St/Hs = 0 65% lie within 50% of the observed values.

The worst results by far occur for negative observed scour depths (i.e. accretion at the toe of the structure). For observed scour depths less than $S_t/Hs = 0$ (i.e. accretion) only one of the six predictions lie within 50% of the observed values (i.e. 17%). These cases are relatively unimportant, at least as far as the stability of a structure is concerned. The laboratory tests may also be affected by scale effects at such shallow depths. The differences between measured and predicted relative scour depths are large at low relative toe depths in Figure 4.8 partly as the gradient of the curve is high, so small changes in relative toe depth lead to large changes in relative scour depth. The errors may be as much a reflection of errors in the toe depth – such as in the calculation of setup – as they are in the relative scour depth.

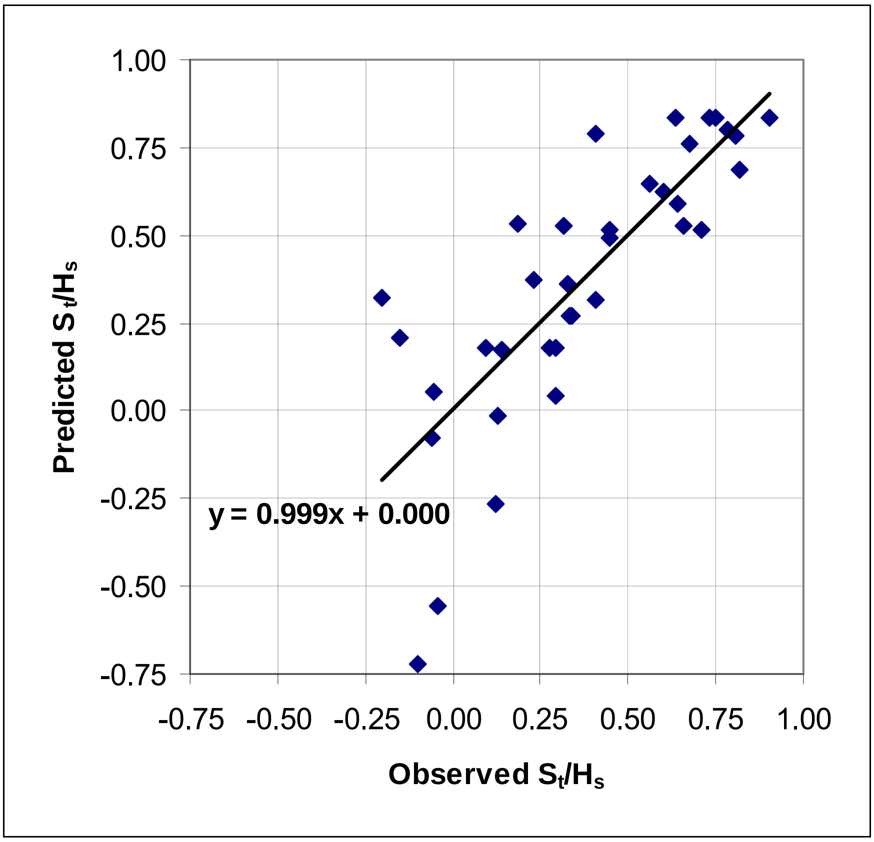


Figure 4.8 Predicted versus observed relative scour depths, using Equation 14 for the predictions

4.4. Summary of improved scour predictors

An extensive literature review (Sutherland et al., 2003) and assessment of existing datasets (HR Wallingford, 2006b) have been used to identify laboratory test datasets that include measurements of toe scour and maximum wave-induced scour in front of vertical or sloping seawalls with wave heights sufficiently high to generate suspended sediment transport. A set of new laboratory experiments was then planned (HR Wallingford, 2006b) to address some of the shortcomings identified. The new test programme was then executed and documented (HR Wallingford, 2006e) and interpreted (Sutherland et al., 2006b, Pearce et al., 2007).

The combined laboratory dataset was then used to derive Equations 8 and 9 (representing low and high values of relative water depth) for toe scour depth and Equations 10 and 11 for the maximum scour depth. Statistical analysis gave a Root-Mean-Square error in the predicted values of relative scour depth of about 0.17. Equation 13 was derived as a single equation to calculate toe scour depth as a function of h_t/L_m, although it has systematic and unsystematic errors. Equation 14 was then derived as an alternative. Equation 14 has zero systematic error and zero bias. It takes beach slope as well as relative toe depth into account. This predictor provides significant additional predictive capability for seawall scour in sand beaches.

Scour depths in shingle beaches can be predicted using the parametric plot of Powell and Lowe (1994) reproduced in Whitehouse (1998), which was based on an extensive set of laboratory tests.

5. Review of mitigation measures

5.1. Introduction

Beach lowering and / or toe scour can lead to unacceptably high risks of coastal erosion or flooding in two ways. First it can affect the condition of a coastal defence structure, by increasing the probability of it being undermined or breached. Secondly, the greater water depths just in front of a structure at high tide will allow greater wave overtopping, thus reducing the standard of defence that coastal defence provides or increasing the likelihood of erosion of dunes, cliffs or coastal slopes behind the defence. Further consideration of these issues within the context of the PAMS Operational Framework is presented in the following chapter.

In these circumstances, and where the coastal defence management strategy for the frontage has been defined in the relevant Shoreline Management Plan as either ‘Hold the Line’ or ‘Advance the Line’, consideration will need to be given to reducing the risks of erosion or flooding by mitigating the beach lowering and / or toe scour.

At present, there is no established “Good Practice” guidance manual for such mitigation works. Where these have been undertaken around the UK coastline, they have often been developed on an *ad hoc* basis. In this scoping study we have reviewed a number of mitigation techniques, ranging from very low-cost and unsophisticated schemes through to major beach recharge projects. In many cases, however, we found no information available on the reasons for the choice of mitigation measures or on their design, construction or effectiveness.

A summary of these methods is presented in HR Wallingford (2006h). For any particular location, however, the choice of a suitable mitigation method will depend not only on its technical feasibility, but also on a number of other factors, such as impacts on amenity, access, ecology, aesthetics, the length of frontage affected, the required lifetime of the works, the initial and maintenance costs etc. It is beyond the scope of this study to discuss these issues in detail.

5.2. An approach to choosing mitigation methods

There are many factors that need to be taken into account when considering intervention to reduce existing or potential future flooding or erosion risks caused by beach lowering or toe-scour, for example:

- Level of expenditure warranted;
- Coastline length over which intervention is needed;
- The structural condition of existing defence structure(s);
- Depth of beach, its sediments and the character of solid rock below it;
- Environmental sensitivities, particularly amenity and aesthetic concerns;
- Strengths of longshore currents and drift rates; and
- Required lifetime of scheme.

It is clear from this that the choice of an appropriate mitigation method will depend considerably on local conditions. Consequently, different methods might be appropriate for two locations where the waves, tides and beaches are very similar.

HR Wallingford (2006h) contains reviews of the most common schemes used to cope with beach lowering, starting with those that are likely to be least expensive. HR Wallingford (2006h) contains reviews of:

- Section 3 - Monitoring and accommodating the effects of beach lowering;
- Section 4 - Ancillary works to minimise/control scour;
- Section 5 - Adjustments to the existing defence structure(s); and
- Section 6 - Major beach improvement methods.

In each section, a brief summary of the likely applicability, strengths and weakness of each of these is provided and the reader is guided to the relevant section for details. Section 5.3 of this report draws some preliminary conclusions and then sets out some recommendations for improving and disseminating good practice on mitigation methods.

5.3. Mitigation measures: conclusions and recommendations

A variety of mitigation measures have been used to reduce the problems caused to coastal defences by beach lowering and toe scour (HR Wallingford, 2006h). This section first briefly summarises the main conclusions of this review, and then sets out some recommendations for future research into and guidance for such techniques.

5.3.1. Conclusions

The following conclusions seek to summarise the main advantages and disadvantages of each mitigation technique, and comment on their applicability.

5.3.1.1. Monitoring and accommodating beach lowering

In many cases, there is no need for immediate action to mitigate beach lowering or toe scour. Rather it may be sufficient to monitor the situation and reduce the consequences of the erosion and/ or flooding problems that are being experienced.

Fundamental to this, and indeed to other mitigation options, is the need to assess both the condition and performance of coastal defences, both at present and taking into account future beach lowering. This assessment requires monitoring and analysis of survey results, to understand how beach levels may lower in future and whether the coastal defence structures will be undermined. In addition, it will be necessary to carry out calculations to assess how the defences will be affected by severe conditions, i.e. high tides and large waves, again for present-day and future beach levels. This two-stage assessment process should be set within the wider context of a Performance-based Asset Management System (PAMS) for coastal defences (HR Wallingford, 2004).

Provided that there is no immediate threat of structural failure of defence structures, e.g. caused by undermining, then the problems caused by wave overtopping can be tackled in a variety of ways, ranging from “immediate” measures involving storm warnings and evacuations to longer-term initiatives such as improving the flood resilience of, or relocating, important assets that are liable to be flooded.

Such measures do not tackle or remedy the causes of beach lowering, and it is likely that the frequency and intensity of wave overtopping, for example, will increase over time. There is also a need for careful and repeated monitoring and analysis to assess the changing condition and hence performance of the coastal defences. If this is not carried out, there is a risk of a sudden and potentially catastrophic failure could occur, for example the undermining and breaching of a seawall. The advantages of this approach lie mainly in the modest costs involved while delaying, and possibly eventually avoiding, more substantial expenditure.

5.3.1.2. Ancillary works

There are a number of techniques to mitigate beach lowering and toe scour that involve installing additional structures to the seaward of the main coastal defences. These range from low-cost and low-technology measures, such as installing faggotting just in front of a seawall, to much more substantial and expensive works such as installing detached breakwaters or groynes.

Some of the older methods such as faggoting and installing timber “wave breakers” are now unlikely to be acceptable from amenity, recreation and aesthetics viewpoints, as well as being supplanted in many cases by alternative approaches. There is however a potential role for faggotting in tidal inlets and estuaries, especially if labour costs are not a concern and there is a desire to avoid more substantial engineering works.

There has been little use, as far as we are aware, of “scour blankets” that have been placed directly in front of coastal defences to counter toe scour. The few examples found suggest that gabion baskets, or possibly geotextile containers filled with sand, might be suitable as a short-term measure to prevent undermining. However, such lightweight construction is likely to have a very limited life-span, being vulnerable to abrasion and corrosion, and also unlikely to be popular on beaches with high amenity or recreation usage.

The increased availability of armour rock, at reasonable cost, has led to its increased use in schemes to extend the life and improve the performance of seawalls around the UK. A modest amount of rock placed at the toe of a defence structure may serve to protect its toe from undermining, reduce the abrasion of its front face and even reduce overtopping problems (Figure 5.1). There are sometimes concerns about the impacts on aesthetics, access and public safety especially where such schemes are installed on beaches of high amenity and recreational usage. There is also a danger that such works can increase wave overtopping if not designed carefully.

Figure 5.1 Rock infill of scour trough, Le Dicq, Jersey, 2005

Detached breakwaters can efficiently reduce wave energy arriving at the shoreline, and cause higher beach levels to protect coastal structures such as seawalls or any developments to landward. However, care must always be taken to address the potentially severe risks to the downdrift coastline, where erosion may become a continuing problem. As with the construction of groynes (HR Wallingford, 2006h, Section 4.5) it is best to anticipate the widening of the beach in the lee of a detached breakwater and charge the beach with sufficient extra sediment to avoid beaches becoming narrower elsewhere. Because of their effects on longshore drift, they may be less suitable for long, straight sections of coastline where it is likely that the frontage downdrift of them will suffer from greater problems of beach lowering. In areas where there are large tidal ranges and/ or strong tidal currents, detached breakwaters are likely to have more disadvantages than in micro-tidal or sheltered regions.

Detached breakwaters built close to the shoreline are smaller and thus less expensive to build, and have less severe effects on the downdrift coastline. However, there may be greater impacts on the amenity and aesthetic attributes of the coastline in this case. Where such breakwaters are built further offshore, there is a greater risk of reducing supply to downdrift beaches, and potentially of losing sediment offshore.

In general, detached breakwaters are perhaps best suited for situations where beach lowering / scour are causing localised problems of overtopping or undermining (e.g. Rhos-on-Sea, Figure 5.2). They may be particularly well suited to frontages where the wider beach formed can be justified for recreation/amenity purposes, but need to be carefully designed to reduce their visual impact. Potentially hazardous rip currents can develop near detached breakwaters, particularly when waves are large or tidal currents are strong. As well as the obvious risks to swimmers, these rip currents may add to the scour problems along the seaward face or around the ends of detached breakwaters.

Figure 5.2 Detached breakwater, Rhos-on-Sea, Clwyd, 1986

As an alternative to detached breakwaters, there is a possibility of using similar structures that are submerged at most or all tidal levels, although there is little or no experience of these in the UK so far. They can have similar disadvantages to detached breakwaters in terms of affecting adjacent stretches of the coastline, although such effects will be less intense. They could potentially provide benefits for recreation (e.g. for surfing) and a niche habitat for marine life, but may also pose a hazard to navigation. Further research into such structures as an aid to reducing localised problems of beach lowering is needed.

As a more direct approach, there are a number of locations where rock sills have been built on beaches, aimed at promoting higher beach levels at the beach crest and hence reducing the wave energy reaching existing defences or cliffs (Figure 5.3). Again these structures may hamper access and pose dangers on beaches of high recreational / amenity usage, and the underlying problems of beach lowering may simply transfer to the seaward face of the sill. The effects of such structures on longshore drift rates, and hence on the tendency for erosion along the downdrift coastline, is less than for detached breakwaters or groynes but there may still be problems of scour at the ends of such sills.

Figure 5.3 Rock sill, California, Norfolk

In the UK, groynes have long been the most common method used to improve and retain high beach levels, and have generally been regarded as successful in this regard. Details of the design of groyne systems, with or without beach recharge, can be found in the CIRIA Beach Management Manual (1995) which is being updated in 2008. Groynes can be effective in reducing the problems caused by toe scour in front of coastal defences by diverting longshore currents further seawards. In addition, they can improve and retain higher beach levels along stretches of coastline where this is necessary, although unless supplemented by beach recharge this is often accompanied by increased rates of beach lowering and retreat further along the coast. Traditional vertical-sided timber groynes are now being replaced in some areas by rock structures, which allow the possibility of adding T- or Y-heads to reduce scour along the main stem of the groyne (Figure 5.4). In addition, some rock groynes have been built with walkways along their crest, and these have proved popular with holiday-makers as well as local residents.

Figure 5.4 Variations in beach width caused by rock groynes at Jaywick, 2006

5.3.1.3. Underpinning and encasement

The underpinning, and if necessary encasement of seawalls reduces the threat of undermining of an existing coastal defence structure, with little effect, adverse or beneficial, on beach lowering in front of that structure. This technique is commonly used, but is always likely to provide only a short-term or medium-term benefit, since it does nothing to alter the causes of beach lowering, or to reduce the propensity for toe scour just in front of the structure.

5.3.1.4. Adding steps or aprons

The construction of an apron or of steps at the base of an existing structure can prolong the life and improve the performance of that defence, at reasonable cost compared to rebuilding that defence entirely. However, such an intervention will not remedy the underlying causes of beach lowering. Such additions to a structure will extend it seaward, often occupying an area of the beach that previously provided an amenity area, and affecting the natural sediment transport processes in that area. There is a danger, for example, that such seaward extensions of a structure will interfere with longshore sediment transport, and hence reduce sediment supply to downdrift beaches.

Despite these disadvantages, such measures have been used frequently around the UK, and new techniques have been developed, for example using a sloping asphalt apron to both protect the original structure against undermining and abrasion, and reduce wave overtopping (Figure 5.5). The new steps and apron may also have an amenity value, e.g. for sunbathing or sitting above the beach.

Figure 5.5 Asphaltic revetment under construction, Porthcawl, Glamorgan, 1984

5.3.1.5. Rebuilding defences to reduce toe scour

The reconstruction of a coastal defence structure such as a seawall, while expensive, can undoubtedly improve its performance, and reduce localised problems of scour at its toe. However, this approach will not address the underlying problems of beach lowering, which can generally be much more widespread.

Where such reconstruction results in a defence structure protruding further into the inter-tidal zone, it may add to problems along the adjacent, particularly the downdrift, coastline and as well as reducing the amenity and aesthetic attributes of the beach. As with the addition of an apron or steps, however, it is possible that the new structure will offer some amenity benefits as well.

5.3.1.6. Major beach improvement schemes

Virtually all of the mitigation measures described in the earlier parts of this section have been restricted to rather modest lengths of coastline, although some groyne schemes may have been extended, over time, along several, exceptionally more than 10km of beach. While early beach recharge schemes in the UK were restricted to modest lengths of the shoreline, typically only a few

kilometres, there has been a growing trend to consider and implement such measures over much longer frontages.
Such schemes directly redress the losses of sediment that predominantly cause beach lowering, and immediately reduce the risk of undermining of defence structures. In all but a few exceptional cases, recharge schemes also reduce the flooding risks caused by wave overtopping. Further advantages usually include an improvement in the amenity, recreational and conservation value of the beaches, and an improvement in beach levels along adjacent stretches of the coast.

The main disadvantage of such schemes is their cost, not only initially but also for subsequent “top up” or maintenance operations. There are some doubts as to the long-term affordability of recharge schemes for shingle beaches, because of the limited supplies of suitable sediments on the offshore seabed (or from inland sources for which greater transport costs will usually result). The sustainability of recharging sand beaches seems less contentious. The other potential disadvantages of such schemes include problems with sand blown inland, or blocking of outfalls, intakes and perhaps even the siltation of harbours or marinas.

5.3.2. Recommendations

It has been found in this research project that there is very little, if anything, in the way of established “good practice” guidance for dealing with beach lowering or toe scour in front of coastal defences. Where measures have been undertaken to reduce the resultant risks of coastal erosion or flooding, then these have rarely, if ever, been well publicised. The rationale for the design adopted, and perhaps more crucially the effectiveness of any chosen scheme, are very difficult to establish, despite the large number of such schemes and the substantial investment that has been made in them. The exception is large-scale beach recharge schemes, which have been reasonably well described, monitored and analysed.

There is therefore a pressing need for a guidance manual to assist in the planning of mitigation schemes.

This guidance should begin with advice on establishing the condition and performance of coastal defences fronted by lowering beaches. While there has been a welcome increase in the volume and accuracy of coastal monitoring in recent years, this has been understandably aimed at a general understanding of how the coastline of England and Wales is changing. The prediction of how a coastal defence may perform in severe conditions will require more specific information than provided by the present programmes of beach monitoring and wave recording around our shorelines. For example, it may be beneficial to monitor beach levels along the toe of the whole length of a seawall, as well as surveying cross-sectional beach profiles at intervals along it. More frequent monitoring of beach levels along the toe of a seawall, for example before and after storm events, would help identify short-term fluctuations caused by toe-scour. These data should be analysed together with knowledge of the structure itself, with a view to reducing the risks of undermining of its toe. A possible way of achieving this is to measure beach levels downwards from a fixed mark or marks at known levels (relative to ODN) running along the defence structure. These measurements could then be taken manually or perhaps remotely by the use of time-lapse photography techniques.

Such measurements will only record the changing condition of the defences, and need to be analysed further to allow the prediction of the performance of the defence (beach plus structure) in a severe event. There are a growing number of methods for carrying out calculations, for seawalls and the like, of overtopping, impact forces and overturning moments and structural responses. Such methods range from empirical through numerical/ computational to, for very complex situations, the use of laboratory modelling, all aimed at providing quantitative estimates of present-day defence performance. Such methods should be repeated for a range of assumptions about future conditions, in which tidal levels, wave conditions and, particularly in the context of this study, changes in beach levels are altered (e.g. Sutherland and Gouldby, 2003, Burgess and Townend, 2004) . These sensitivity tests, together with information about the defence structure itself, will indicate the present standard of protection offered by the coastal defences, and define a minimum “threshold” beach level beyond which action will be needed to reduce flood or erosion risks to an acceptable level.

There is a clear need to gather and disseminate information about coping with problems of wave overtopping caused by low beach levels, for example the use and experience of storm warning and evacuation procedures, and on any short-term intervention measures used to deal with problems of undermining of seawalls and the like. This is an area where there are social and economic issues that need consideration as well as the operational practicalities of predicting, responding and acting to reduce flooding risks in particular.

Information on relatively low-cost measures that have been undertaken to remedy beach lowering is also difficult to find. There is a need to review the efficacy, advantages and disadvantages of various “ancillary” works designed to reduce problems caused by beach lowering. In particular, it would be helpful to gather and disseminate experience gained from lower-cost measures such as placing scour mattresses or modest amounts of rock at the base of seawalls or cliffs.

The limited number of more substantial schemes aimed at improving beach levels, such as the installation of offshore breakwaters and large scale beach recharge schemes, have generally been better documented, at least in terms of their design, implementation and initial performance. There is a need for a longer-term assessment of the performance, sustainability and of the overall advantages and disadvantages of these schemes, but this is likely to be reasonably well covered by other research and development projects such as the forthcoming update to the first edition (1996) of the CIRIA Beach Management Manual.

6. Integration into reliability analysis for coastal structures

6.1. Introduction

The results from research project FD1927 are being integrated with the programme of research already underway into the reliability analysis of coastal structures, particularly Establishing a Performance-based Asset Management System Phase 2 (PAMS2) which was funded by the Joint Defra/EA Flood and Coastal Erosion Risk Management R&D Programme. PAMS Phase 1 (HR Wallingford, 2004) was a scoping study that provided the Environment Agency with a new vision for managing its flood defence assets. The overall aim is to manage flood risk as efficiently and effectively as possible by inspecting, maintaining, repairing and if necessary replacing flood defences in order to achieve the required performance and to reduce risk. Central to PAMS are two concepts that can be helped by receiving improved information from FD1927:

1. Asset Condition Assessment; and
2. Fragility.

Condition assessment refers to the process by which data about assets is collected and the methodologies by which asset condition is determined. Data can be collected visually or by coastal monitoring (see Section 3.1). Condition indexing uses visual indicators that relate directly to Performance Features (PFs) that may be specific to one function of a defence element (or one failure mode). Condition Indexing has been developed in the TE2100 (Thames Estuary 2100) project, led by the Environment Agency (HR Wallingford, 2005b) and in the development of the Condition Assessment Manual (Environment Agency, 2006). If condition indexing suggests that an asset is in poor or very poor condition and if it makes a significant contribution to flood (or coastal erosion) risk management then the asset is likely to be monitored to produce a quantitative condition assessment.

Fragility has been defined as the probability of failure of a particular defence or system given a load condition (HR Wallingford, 2005a). Fragility can be expressed in the form of a univariate distribution when one loading variable is considered, in the form of a fragility surface when two loading variables are considered or multidimensional fragility space when three or more loading variables are considered. Combined with descriptors of decay/deterioration, fragility functions enable future performance to be described. The concept of fragility curves has been expanded beyond what was in PAMS Phase 1 in "Performance and Reliability of Flood and Coastal Defences – Phase 1" which was Project FD2318 in the Risk Theme of the Joint Defra/EA Flood and Coastal Erosion Risk Management R&D Programme. FD2318 explored ways to assess the performance and reliability of flood and coastal defences in order to make better assessments of risk. Further detailed work on the failure modes of coastal defences has been undertaken in Task 4 of the EC Floodsite project (www.floodsite.net).

This section of the report describes the PAMS operational framework (in Section 6.2) followed by descriptions of condition indexing (Section 6.3) fragility curves (Section 6.4) and the potential for using coastal monitoring data in condition assessment (Section 6.5). Sections 6.3 to 6.5 describe how the outputs from the present project can be used in the development of the PAMS operational framework.

6.2. PAMS Operational Framework

The PAMS operational framework developed in PAMS Phase 1 is shown in Figure 6.1. The main elements cover the following issues (taken from HR Wallingford, 2004):

- "Inspection and condition assessment methodologies – To improve asset management decisions it will be important that PAMS includes an improved approach to condition assessment. This module of PAMS refers to the process by which data is collected and asset condition is assessed. It will also include recommendations on minimum information requirements, for example the features of an asset that should be collected as a matter of routine (crest level for example) and which should only be gathered if the collection costs can be justified in risk reduction terms.
- System analysis (Performance assessment) – To understand flood risk and the effectiveness of any intervention the decision maker must first have an understanding of how risk is generated and how it can be influenced (reduced). The general concepts of system analysis are currently being addressed outside of PAMS through projects such as RASP (Environment Agency, 2003) and the review of risk methods within flood and coastal defence (Environment Agency, 2002) [and now Floodsite]. However, PAMS will need to develop these methods to cover the issues relevant to asset managers. The systems analysis module of PAMS will involve the integration of source, pathway and receptor terms together with information on how these drivers of risk are modified through management intervention and/or asset deterioration as well as climate or social change. Therefore this module will include the analysis undertaken to provide an understanding of the performance of an asset (in its present, deteriorated or improved state) and the defence system in the context of risk and risk reduction.
- Decision approaches and option selection techniques – As with the system analysis a number of generic issues are currently being addressed – or are planned - outside of PAMS. However, significant effort will be required to develop the specific decision approaches within PAMS to reflect the interface with higher level plans and the broad spectrum of criteria to be considered in selecting the preferred maintenance or operational intervention. This module of PAMS will therefore cover the process of the decision-making and option selection.

- Common databases and data and information management – Allowing data to be stored and accessed for re-use will be a key feature of PAMS. Maximising the use and re-use of data will inform any of the modules outlined above. In particular, PAMS will specify the asset data to be recorded; including format, mandatory and optional parameters, histories, uncertainties etc and appropriate fields developed within NFCDD and the use of related databases on flood plain assets."

The present research into scour can assist in four main ways by:

1. Improving condition indexing (Section 6.3);
2. Updating the fragility curve for seawalls to include the improved knowledge of scour depths and variations in beach levels developed within FD1927 (Section 6.4);
3. Demonstrating how routine coastal monitoring could be used in condition assessment and introducing the *frame of reference* approach (Section 6.5);
4. Identifying technically feasible options for maintenance or improvement of critical assets (Section 5)

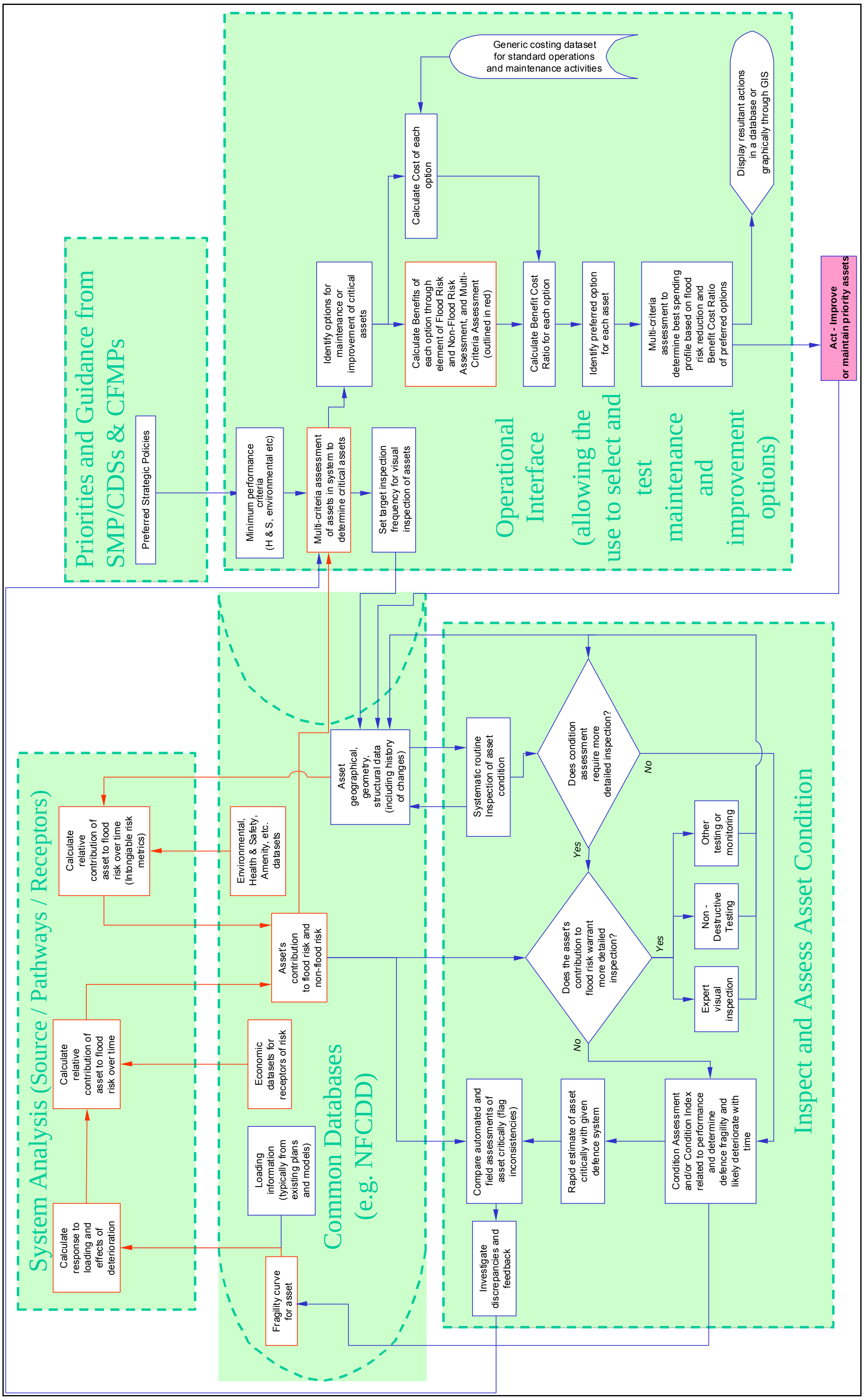


Figure 6.1 PAMS Operational Framework (from HR Wallingford, 2004)

6.3. Condition Indexing

Condition Indexing is the process of assessing the likely performance of a defence asset through observation of Performance Features (PFs) and subsequent calculations based on the findings (HR Wallingford, 2005b). No detailed measurement of asset geometry is required and the inspection process should not require a qualified engineer. In this respect the process fits into Option C "Non-expert inspection and condition assessment with periodic expert inspection and condition assessment" identified during the PAMS Phase 1 scoping study (HR Wallingford, 2004).

The proposed process of Condition Indexing and its use within an overall asset management system is shown in Figure 6.2 (from HR Wallingford, 2005b). Steps 1 to 3 form the revised method of visual inspection described in HR Wallingford (2005b), whilst Steps 4 to 6 show the asset management actions that would make use of the visual inspection results. Failure modes and the Performance Features that apply to them are predefined by asset type. Further failure modes and PFs can be added to the method and will require alteration to the contribution weightings that link PFs to failure modes or failure modes to the asset.

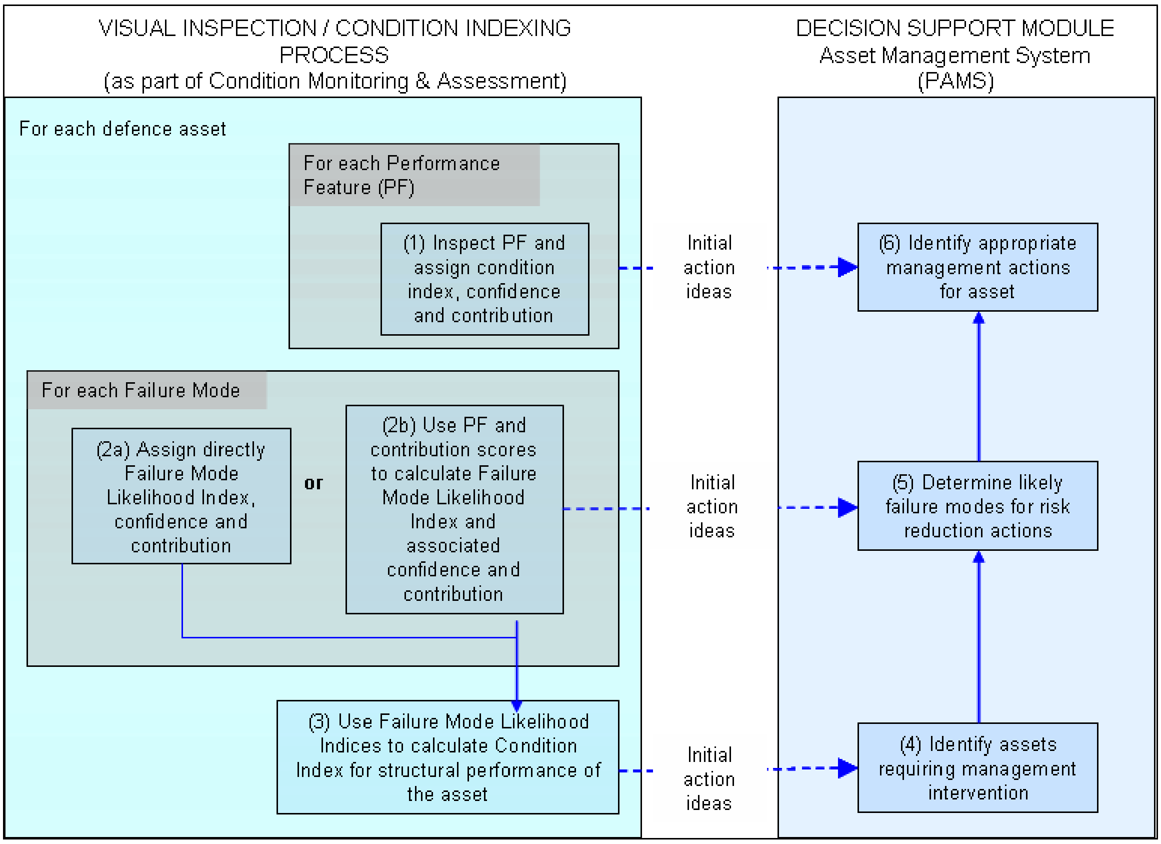


Figure 6.2 Outline of condition indexing process and links to asset management system

Step (1) is the on-site visual assessment of condition, confidence and contribution for the Performance Features. Prior to undertaking this process an inspector should review historical data on the assets to be inspected. This will determine the asset type and therefore the set of Failure Modes and PFs to be inspected in relation to a particular asset. There are two important outcomes from this step, the PF Condition Index and Confidence. The method of obtaining a condition index is outlined below.

Each PF will be inspected and assigned an index in accordance with guidance provided in the form of tabular descriptions and/or simple flowcharts. The definition of the PFs that relate to specific Failure Modes is critical to the Condition Indexing process. There are a number of factors to take into account in their definition:

- From an operational viewpoint there is a limit to the number of PFs that can be inspected without increasing the inspection duration beyond practical constraints.
- Each PF should be related, directly or indirectly, to at least one failure mode either through performance models, or in their absence by expert judgment.
- There must be enough PFs to evaluate individual Failure Mode Likelihood Indices.
- PFs must be unambiguous in terms of their definitions and supporting information.
- All PFs must be visually inspectable without the need for detailed measurement.
- All PFs must be able to be given a Condition Index in discrete steps ranging from 'Insignificant – no negative impact on desired performance' to 'significant – major negative impact on performance'.
- A PF will relate to an individual defence element, to a group of defence elements or to a defence asset as a whole.

Table 6.1 shows examples of PFs for the four main structure types encountered in the TE2100 area (from HR Wallingford, 2005b). It includes toe scouring / undermining for gravity walls, sheet piles walls and revetments. An example of the guidance provided for assessing the condition index for the toe scour/erosion performance feature of a gravity wall is given in Figure 6.3 (HR Wallingford, 2005b). Note that these examples do not include an assessment of the beach, except as a means of producing toe scour or undermining.

The Environment Agency's Condition Assessment Manual (EA, 2006) has a section on beaches as defence assets in their own right. This uses subjective terms, such as narrow, wide, steep and shallow to describe the condition of a beach. It also relies on the position of the strand line (a surrogate for the high water level) which will vary between spring and neap tides, with the time of year and the weather conditions. This gives a qualitative assessment of the beach condition and it should be possible to assess the deterioration in the condition of the beach (as a defence asset) by tracking its condition index through time as an eroding beach will change condition as it looses volume.

Table 6.1 Performance Features for the TE 2100 area

Embankments	Gravity Walls	Sheet Piled Walls	Revetments
Visible deformation of cross-section (e.g. slumping, heave, local translation)	Obvious deformations of structure &/or surroundings relevant to failure mode	Obvious deformations of structure &/or surroundings relevant to failure mode	Obvious deformations of structure &/or surroundings relevant to failure mode
Animal burrowing / infestation	Toe scouring / undermining	Toe scouring / undermining	Toe scouring / undermining
Foreign objects in the crest or rear slope	Condition of the wall material	Condition of the wall material	Condition of the revetment material
Cracking &/or fissuring	State of the joints	State of the joints	State of the joints
Third party damage (cattle, vehicles etc)	Third party interference with load or resistance	Third party interference with load or resistance	Third party interference with load or resistance
Direct evidence of seepage or piping	Animal infestation in ground around structure	Animal infestation in ground around structure	Animal infestation in ground around structure
Revetment condition	Seepage through, behind or in front of structure	Seepage through, behind or in front of structure	Seepage through, behind or in front of structure
Vegetation condition	Presence of foreign objects	Presence of foreign objects	Presence of foreign objects
Erosion of cross section			

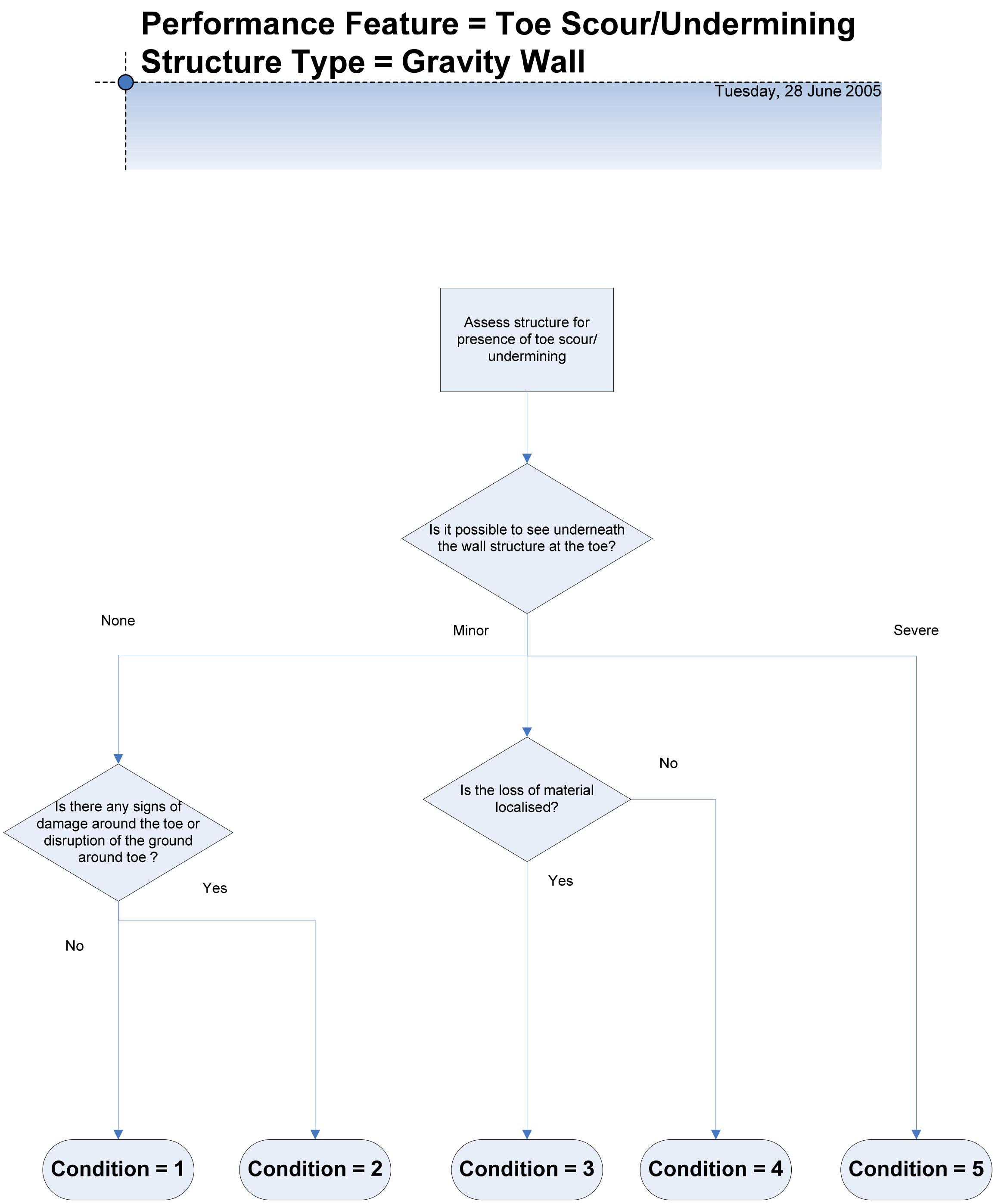


Figure 6.3 Flow chart for toe scour/undermining performance feature for a gravity wall

6.3.1. Input from FD1927

The condition indexing system developed for TE2100 includes toe scour/undermining as a performance feature for gravity walls, sheet pile walls and revetments. HR Wallingford (2006c) looked at the changes in the mean and standard deviation of the measured beach levels at the toe of coastal defences. In particular it noted that beach levels were on average lowest and the standard deviation in the beach levels were highest in spring (February to April) for the Lincolnshire beach profiles analysed (see Section 3.3.7 of the present report). This type of analysis could be repeated for any stretch of coast in order to inform the timing of condition indexing surveys. Clearly a survey between February and April is more likely to provide observations under the wall structure at the toe than a survey in September or October, when beach levels are relatively high and the standard deviation in levels is relatively low. Moreover, the differences in the average and the standard deviation of the beach level caused by altering the number of surveys per year was demonstrated (see Section 3.3.6).

A local analysis of beach levels could be used to influence both the timing and the frequency of Condition Indexing surveys. There may be a need to add an additional performance feature: the scouring or undermining of mitigation measures, as well as of walls themselves, when dealing with the coastal zone.

One prominent failure mechanism for vertical seawalls is structural failure due to wash out of fill following joint failure. This could be expanded to include structural failure due to wash out of fill following scour. Cases have been noted where there was no obvious sign of damage before collapse occurred due to the loss of fill, caused (it is believed) by scour. This is difficult to detect as there may be no visible outward signs of damage until failure occurs. In other cases the visible signs of loss of fill material include movement, such as the development of cracks, in the top surface of the defence. No comments on the condition of the top surface of coastal defences have been included in the Condition Assessment Manual (EA, 2006). This document could be expanded to include the condition of the top of coastal defences as a preliminary way of assessing the risk that fill material has settled or been lost.

The use of non-destructive testing to assess the presence of significant voids within a structure is one relatively costly option to identify those that are at risk of failure. An alternative method for identifying structures that may have lost fill material when beach levels dropped below the structure toe is outlined below, in locations where routine beach surveys are undertaken.

As noted above, Condition Indexing is highly subjective, particularly for beaches. In regions where beaches are regularly monitored the use of a visual condition index for a beach should, in time, be replaced by a quantitative measure of beach performance derived from the measurements. This may require the development of suitable coastal state indicators and of methods to determine suitable threshold levels for them, which may be derived using fragility curves.

6.4. Fragility Curves

The concept of fragility is described in detail in HR Wallingford (2005a) from which the following text is derived. The concept of fragility in flood and coastal erosion risk management represents the link between the likelihood of defence response (pathway) and different loading conditions (source). A shortage of knowledge about how defences fail and variations in the characteristics of defences result in a range of defence responses and associated likelihood. The concept of fragility tries to capture the probability of a range of defence responses to a given load. A typical fragility curve is shown in Figure 6.4. Where the probability of failure is a function of two loads, a fragility surface may be formed.

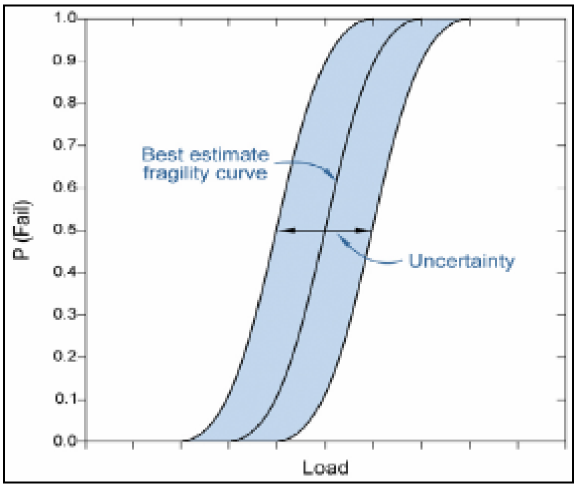


Figure 6.4 Generic fragility curve

A failure curve or surface can be defined for each failure mode in a fault tree, provided sufficient information is available. Altering a fragility curve, or surface, will alter the relative importance of the failure model it represents in the fault tree.

The generic sources of uncertainty are of two types: natural variability and knowledge uncertainty. The latter includes the following categories:

- Statistical model uncertainty. A distribution function represents the best fit to a dataset, and therefore does not capture all of the data within the statistical model. The quality of the fragility curve depends on the quality of the underlying statistical models.
- Process model uncertainty. Process models that quantify failure processes are limited in their representation of reality. Model uncertainties can be quantified and incorporated in fragility. An increasing quality in process based model reduces these uncertainties.
- Decision uncertainty. This is the strength of belief in the decision made and of its robustness. This type of uncertainty is part of the overall decision process informed by fragility and other performance measures and targets.

6.4.1. Input from FD1927

The PAMS fragility curves for vertical coastal defences, such as anchored sheet piles, cantilever walls and masonry, concrete or gabion walls, contains a toe scour term (HR Wallingford, 2005c, Table 8). Two options are offered (taken from McConnell, 1998), which are:

1. Scour depth is the maximum unbroken wave height.
2. Scour depth is obtained from the parametric scour plot of Powell and Lowe (1994).

The improved scour predictor developed in this project can be used in the calculation of fragility curves for coastal defences, using the procedure below:

1. For sand beaches the depth of scour can be predicted using Equation 14 (Section 4.3.3). This requires the beach slope, the offshore significant wave height, the mean wave period and the depth of water at the toe of the structure.
2. For shingle beaches the parametric scour plot of Powell and Lowe (1994) can be used in form of a lookup table to predict the depth of scour. This requires the offshore significant wave height, the mean wave period and the depth of water at the toe of the structure.

This will reduce the process model uncertainty. The details of the scour predictor have been supplied to Task 4 of the Floodsite project to assist in their description of the failure modes for seawalls.

However, this is not the only contribution of the present project, FD1927, to the development of fragility curves. Section 3.3.5 demonstrated that the beach level at the toe of a coastal defence does not just vary with the short term toe scour depth, but also has a medium term Gaussian variation about a long-term mean. These variations in level systematically alter the ratio between the water depth at the structure (from mean water level to un-scoured seabed level) to the buried depth of seawall (from the un-scoured seabed level to the structure toe). Variations in the water depth and buried depth alter the forces on the seawall and hence the elements of the limit state function (HR Wallingford, 2005c, Sections 3.2 to 3.4).

Inputting the Gaussian variation in beach levels into the calculations will produce a corresponding variation of the limit state function. This will lead to a spread in the fragility curve about its central position. Inputting knowledge on the Gaussian distribution of residual beach levels will therefore quantify this element of the uncertainty in the fragility curve for each structure.

Moreover, the long-term trend in beach level can be obtained from, for example, the extrapolation of the historical data (possibly with seasonal variation – see Sections 3.3.2 and 3.3.8). This will allow the variation with time of the best estimate of the fragility curve to be predicted for a few years into the future, depending on the prediction horizon (Sections 3.2.3 and 3.3.4). Using the predicted trend in mean beach level will allow the change in the fragility curve with time to be calculated, by again altering the water depth and buried depths in the calculation of the limit state function. This procedure will assist in calculating the deterioration of performance with time.

The level of information about the distribution of beach levels will vary from case to case. Four classes of information have been identified:

1. Where repeated surveys have taken place at the structure over a few years it will be possible to input a site-specific distribution with an estimate of its uncertainty;
2. Where repeated surveys have taken place at a similar structure close to the one being studied and with a similar level of exposure to waves,

the distribution from the similar structure may be applied to the one being studied;
3. Where no local survey data is available the distribution from another site with similar characteristics may be used;
4. Where no local survey data is available the distribution in beach levels may be estimated using a numerical model. The main options are the use of a coastal profile model to simulate the response of the beach to storms (van Rijn et al., 2004) and the use of a one-line model to simulate the variability in shoreline position combined with estimates of beach slope, which may be different in summer and winter (Stripling and Panzeri, 2007).

Class 1 information should be preferred to class 2, which is better than classes 3 or 4. The choice between class 3 and class 4 is a matter of engineering judgement. A regional assessment of beach level variability could be performed where coastal monitoring data has been routinely collected in recent years. This would identify the variations in beach level distributions around the country. If the variations within regions are relatively low then a regional set of figures could be used. Examples of regional beach survey programmes include the Anglian Region biannual beach surveys or the Channel Coastal Observatory. Increasing lengths of coastline are covered by routine beach surveys and the resulting quantitative data can be used within PAMS.

Fragility curves are also used within the RASP model. In this case the increased risk of structural failure with time, due to coastal erosion, may be incorporated using the methods developed in the Defra/EA Joint R&D project FD2324, Risk Assessment of Coastal Erosion (Burgess et al., 2006) the results from which will become available from the Defra Science Search web site http://sciencesearch.defra.gov.uk/.

6.4.2. Example of the extrapolation of beach survey data

The use of an extrapolated trend to hindcast beach levels is shown in Figure 6.5 using data collected at Boygrift Outfall between 1970 and 1990. A linear trend in beach level was fitted to the data from 1970 to 1980 and the 95% confidence limits were calculated on the assumption of a Gaussian distribution of residual beach levels (Section 3.3.5). Figure 6.5 shows that only 3 out of the 92 measured beach levels between 1970 and 1980 fell outside the 95% confidence limits. The linear trend between 1970 and 1980 was then extrapolated between 1980 and 1990, as were the confidence limits. Over a quarter of the measured beach levels from 1980 to 1990 were outside the extrapolated 95% confidence limits (despite the Boygrift outfall 10-year linear trend having a prediction horizon greater than 10 years – see Figure 3.5).

The use of extrapolated beach level data to predict future beach levels should therefore be limited to periods of a few years only. As already noted in Section 3.3.4, this duration is shorter than the timeframes normally considered for the precautionary approach to coastal management. However this duration is likely to be suitable for a managed / adaptive policy of tracking risk and performing multiple interventions (Defra, 2006).

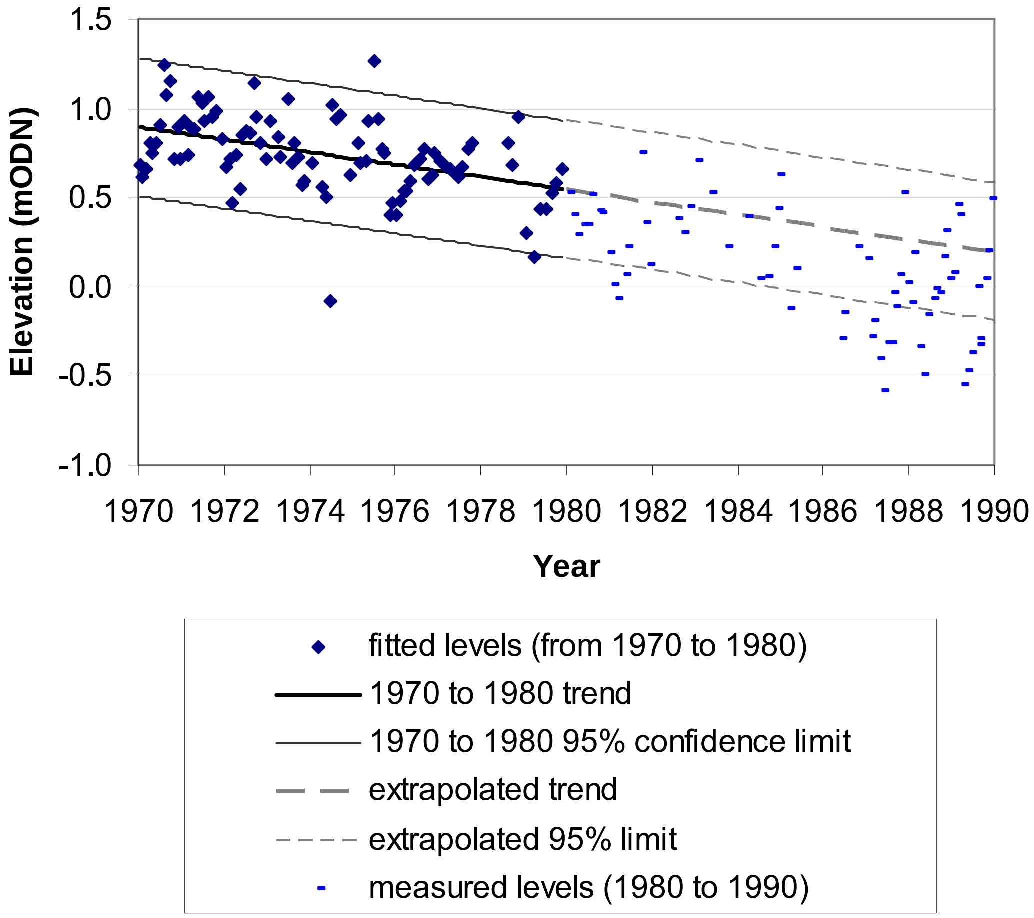


Figure 6.5 Linear trend in beach levels at Boygrift outfall from 1970 to 1980 and extrapolated trend from 1980 to 1990 plotted with measured beach levels and 95% confidence limits.

6.4.3. Identification of potential loss of fill

A graph plotted in the form of Figure 6.5 can be used to identify whether the beach level is likely to drop below the level where washout under the structure can begin (assuming that level is known). It may also be possible to combine this with predictions from the toe scour equation (Sections 4.3 and 4.4) although some, as yet unproven, assumption about the joint probability of low beach levels and deep scour events would need to be made.

If a structure has voids caused by a loss of fill due to scour, it will have an increased risk of failure and could be worth investigating in more detail – possibly using non-destructive testing. Therefore the trigger for a more detailed asset inspection (shown in the 'Inspect and Assess Asset Condition' box in Figure 6.1) could be the extrapolation of beach level data, rather than the result of a standard inspection.

In Figure 6.5 the lowest line represents the 95% probability that the beach will not drop below that level (at low tide). The probability of exceedence that should trigger a more detailed inspection would have to be set by engineering judgement, possibly using historic records of structural failures. One this level has been set the calculations of present probability of undermining and the extrapolation to future probabilities could be automated in the regions covered by routine coastal monitoring surveys. The success of this approach in identifying locations with loss of fill could be determined using the frame of reference approach (Section 6.4.4).

Information on longer term beach level movement does not have to be obtained from extrapolation of historic data (see Section 3.4). For example, a one-line numerical model has been used to help develop a risk-based method of predicting coastal erosion (Stripling and Panzeri, 2007) funded by the Flood Risk Management Research Consortium (FRMRC). In this model the long term development of the shoreline is modelled using the one-line model and the beach level at the coastal defence is set using typical beach slopes for summer and winter. The beach level is used as an input to the coastal defence’s fragility curve.

If sufficient information is known about a structure and the forcing on it, it may be possible to determine a beach level, known as a trigger level or benchmark level, at which the probability of failure becomes unacceptably high. In some cases it may be possible to have a set of trigger levels, each with its own probability of failure. Trigger levels may be determined using a fragility curve or by a more simple analysis. The lower confidence limit of the extrapolated best fit straight line (fitted to the measured beach levels, as shown in Figure 6.5) may then be used to predict when the trigger level will be attained. This time is referred to as the intervention date.

It will always take a period of time, referred to here as the mobilisation duration, to decide what form of intervention should take place, then to have it designed, permitted and built, all of which should take place before the intervention date. Clearly, if the mobilisation duration is greater than the prediction horizon an extra allowance should be made for safety or a confirmatory study performed.

6.4.4. The frame of reference approach

The use of a trend and a benchmark to determine an intervention date within a managed / adaptive policy framework has been formalised in the Dutch frame of reference approach (van Koningsveld and Mulder, 2004, van Koningsveld and Lescinski, 2007), which is illustrated in Figure 6.6. The frame of reference approach explicitly links the management objectives and the technical and administrative solutions that are used to meet those objectives.

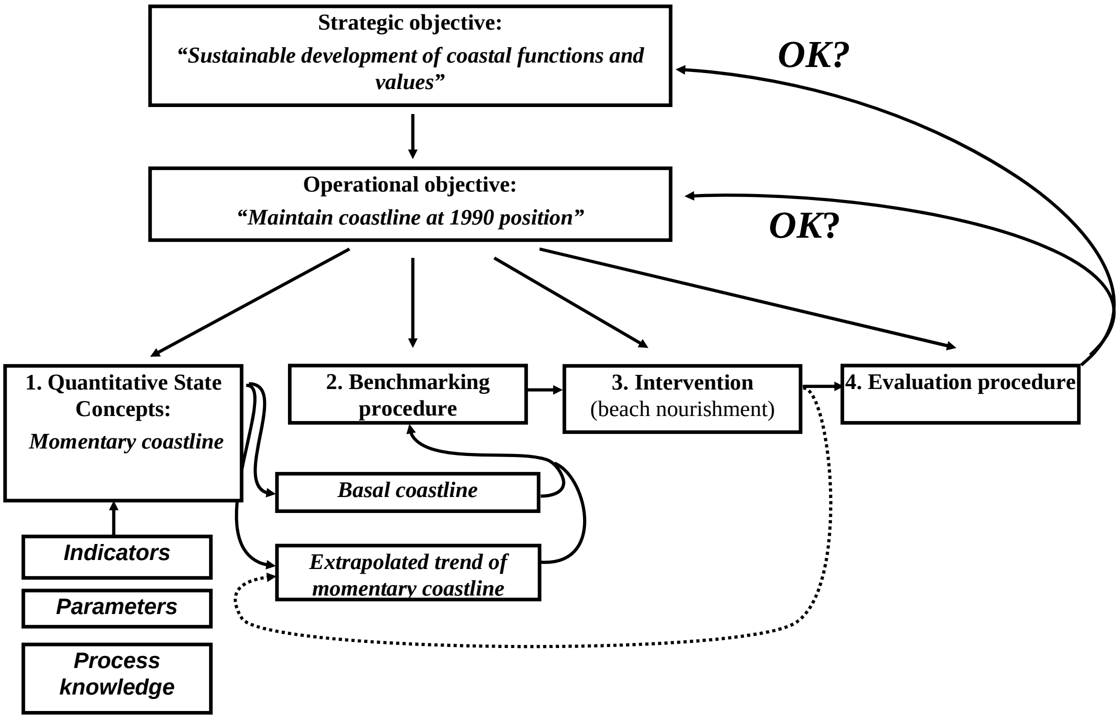


Figure 6.6 Application of frame of reference approach to Dutch coastline (after van Koningsveld and Mulder, 2004)

In the Netherlands the strategic objective set in 1990 was to 'guarantee sustainable development of coastal functions and values' (i.e. to guarantee a safety level). The operational objective followed from this and was set as *the coastline will be maintained at its position in the year 1990* (van Koningsveld and Mulder, 2004).

The following steps were taken to implement the policy. A quantitative state concept called the momentary coastline was derived from a cross-shore beach profile and was used to assess the state of the coastal system. A benchmarking system was derived, with the position of the momentary coastline estimated at 1 January 1990 set as the basal coastline (the equivalent of the trigger level above). The 10-year trend in the momentary coastline position was then extrapolated and beach nourishment performed in any year when the trend was predicted to cross the basal coastline position. This procedure has been repeated each year, with newly measured momentary coastlines. The policy has been since been extended by the addition of a second operational objective (van Koningsveld and Mulder, 2004) and has been successful in preventing flooding.

The Dutch case provides a working example of how predicted beach levels can be used in coastal management, when a policy decision has been made to use beach nourishment to maintain safety levels. The relatively simple benchmarking calculations are performed each time a measurement is made and could be implemented at a local level.

The Dutch use the momentary coastline (derived from the cross-sectional area of the beach profile between the dune foot and a set vertical lower datum) as their coastal state indicator for assessing the contribution of beaches to the overall flood risk. The best coastal state indicator for assessing the contribution of a beach to the overall risk of flooding or erosion in the UK has not been extensively studied. Possible coastal state indicators include the beach level at the toe of a structure, the beach level plus beach slope (as needed for calculating scour depth) and the beach cross-sectional area above a set contour. The latter approach is the one used by the Dutch. Experience with using the parametric model SHINGLE (Powell, 1990) for the cross-shore response of shingle beaches suggests that it may also be useful for UK shingle ridges.

The frame of reference approach could be applied in the UK, particularly when a managed / adaptive policy is adopted. A hypothetical example is illustrated in Figure 6.7 where a beach is the coastal defence and is to be maintained so that the probability of failure (obtained from the fragility curve or surface) is less than a set level, say 0.01 (i.e. will fail on average once every 100 years). The fragility curve will depend on a qualitative state concept (or coastal state indicator) that may be crest level or beach volume (per metre run of beach) above a certain contour.

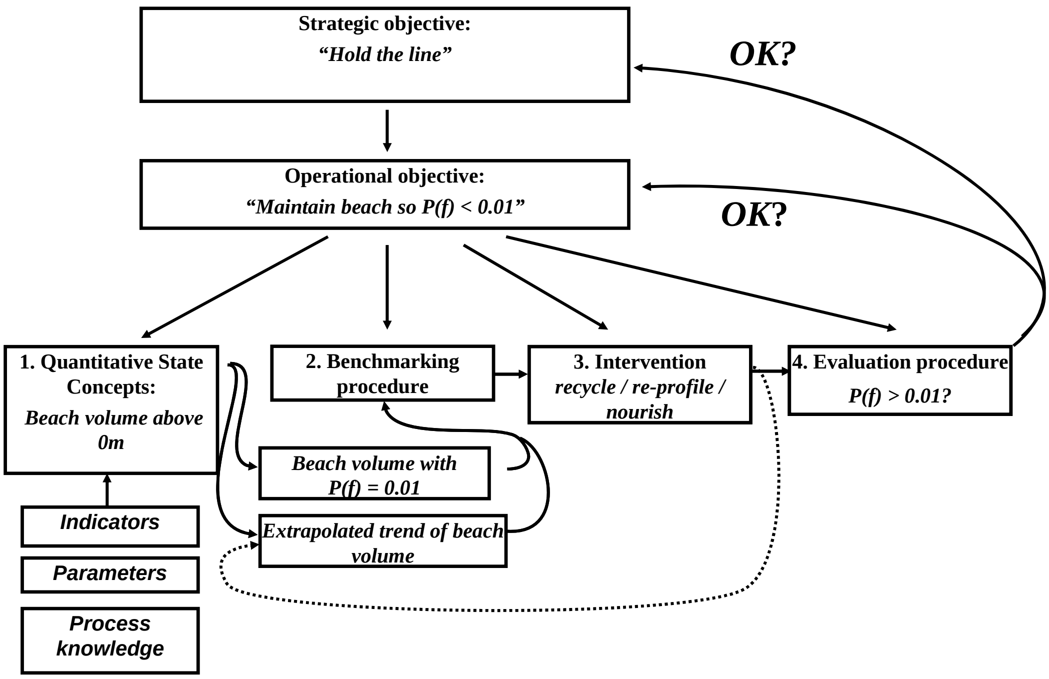


Figure 6.7 Hypothetical example of frame of reference approach applied to a UK beach (after van Koninsveld and Mulder, 2004)

The threshold value for intervention is the level or volume that gives the probability of failure, P(f) = 0.01 for present day loading conditions. The current state is determined from a routine beach profile. Intervention will take place if the value of the coastal state indicator is predicted to fall to less than the threshold level within the next year. The intervention could be recycling within the beach, re-profiling within a section or beach nourishment, if there is no spare volume within the managed unit.

The evaluation procedure would be to compare the value of the coastal state indicator after the intervention with the threshold value used in the benchmarking procedure. If the value is below the anticipated value and, in particular, if it is still below the threshold, then the intervention will have failed in its main objective. Alternatively, if the value is above the threshold the intervention may be judged a success in meeting the terms of the operational objective.

Periodically it will be necessary to check whether the intervention is not only meeting the operational objective, but also the strategic objective (to hold the line). This may be accomplished by a full application of the PAMS operational framework (Figure 6.1).

The advantages of the frame of reference approach over a full application of the PAMS framework are that once the threshold values of the coastal state indicators have been calculated (using the fragility curves in the PAMS operational framework) the calculations become simple and the results could easily be displayed on a computer (within a GIS, for example) showing where there is a surplus and where there is a deficit of sediment. Routine beach profiles are taken up to four times per year and it would be easier to apply the frame of reference approach than the PAMS Operational Framework that often. On the other hand the frame of reference approach depends on knowing the answers to questions, such as ‘which assets contribute most to flood risk?’ and ‘what it the best maintenance or improvement option (based on a multi-criteria assessment)?’ that can be answered by the PAMS operational framework. The frame of reference approach may therefore prove useful in the management of beaches and other coastal defences in the periods between SMP, when a strategic objectives has been set and there is often a favoured approach to delivering that objective.

6.5. Summary of FD1927 inputs to reliability analysis

The work performed in the Defra/EA Flood and Coastal Erosion Risk Management project, FD1927 “Understanding the lowering of beaches in front of coastal defence structures” is assisting in the development of the PAMS framework for reliability analysis in five main ways:

1. The derivation of the improved scour predictor will inform the future development of fault trees and fragility curves for coastal structures which are known to be sensitive to scour;
2. The identification of the Gaussian distribution of beach levels about a long term trend will reduce the uncertainty in the calculation of the fragility curves of coastal defences at a particular time;
3. The forecasting of changes in the mean beach level in front of a coastal defence, whether through extrapolation of historical data or numerical modelling, will allow the change in the fragility curve with time to be

calculated. This contributes to calculations of the deterioration in asset condition with time and could be used to trigger a more detailed form of condition assessment, rather than waiting for scour to be observed or failure to occur.

4. The development of a method for calculating prediction horizons will inform the duration of the forecasts of future beach behaviour (if sufficient data is available) that can usefully be used in coastal management.
5. The frame of reference approach has been identified as a useful way of linking management objectives to the technical solutions that are used to meet those objectives and evaluating their success.

7. Assessment of liquefaction risk

7.1. Introduction to seabed liquefaction

The role of scour in removing sediment from the toe of coastal structures has received much attention (Whitehouse, 1998; Hoffmans and Verheij, 1997; Sumer and Fredsøe, 2002) but the potential for the liquefaction of sediment by waves has been studied less. This situation prompted a recently completed research project on seabed liquefaction, which has been reported in ASCE (2006).

In soil mechanics, liquefaction starts to occur when the effective stress of the seabed becomes zero. A useful introduction to liquefaction is given by Sumer and Fredsøe (2002, Chapter 10). Seabed liquefaction may be caused by the passage of waves (Jeng, 1998), earthquakes and other shocks (de Groot et al., 2006a) or the rocking of coastal structures subjected to wave action (de Groot et al., 2006b). Two types of liquefaction have been observed in laboratory test and field trials, namely residual liquefaction and momentary liquefaction. Liquefaction can lead to the reduction in bearing capacity of the soil adjacent to the foundation of a coastal structure. The potential consequences of this include the seabed flowing as a liquid, reduced resistance to the slipping of a coastal structure and settlement of armour stones into the seabed.

Residual liquefaction occurs in loose sand beds due to the progressive increase of residual excess pore pressure. Under a wave crest the pressure at the bed is greater than hydrostatic, so the bed is compressed. Under a wave trough the pressure at the bed is less than hydrostatic, so the bed is dilated. This creates shear stresses in the soil, which will lead to some rearrangement of the grains and a building up of the pore pressure (dissipated by draining). If the pore-water pressure builds up to such an extent that it exceeds the overburden pressure, the soil will liquefy. Residual liquefaction hardly occurs in dense seabeds due to its high shearing resistance, which prevents the excessive build up in pore pressure. Therefore the occurrence of residual liquefaction was not investigated in the present study as coastal structures are usually founded on a medium dense to dense sand layer. However, in loose sand fills, such as may be present immediately after beach renourishment or excavation works there is a risk of residual liquefaction which may need to be examined.

Momentary liquefaction usually occurs in dense seabeds due to the damping of amplitude and the development of phase lag between the pressures at the seabed surface and lower in the bed. Under the wave trough the pressure at the bed is less than hydrostatic. This pressure decays with depth through the seabed, creating a pressure gradient. If the pressure gradient is sufficiently large it can generate more lift than the submerged weight of the soil above, resulting in momentary liquefaction, which will occur for a fraction of a wave period only (see, for example, Sumer and Fredsøe, 2002, Section 10.1.2). The rate at which the pressure decays with depth depends on the degree of saturation of the seabed: the greater the degree of saturation, the lower the rate. The pressure gradient decays most slowly with depth in a fully saturated seabed.

Here, an analytical solution for the wave-induced pore pressure response in an isotropic infinite thickness seabed in front of a breakwater, proposed by Jeng (1998) was used to study the liquefaction potential of the seabed in front of coastal defence structures subjected to various wave loadings. The liquefaction potential was determined by calculating the minimum total wave height to depth ratio that will cause the momentary liquefaction of the top 0.05m of a sandy seabed in front of a vertical seawall. Calculations were made for fine, medium fine and coarse sand with the degree of saturation between 0.90 and 1.0, for a range of water depths and a typical storm wave period of 8s. The results can be used to indicate whether liquefaction of the seabed in front of a coastal structure is likely to occur. If so, a more detailed study should be carried out.

7.2. Methodology

The analytical solution proposed by Jeng (1998) was based on Biot's poro-elastic theory (Biot, 1941) for an infinite and homogenous seabed with assumptions that the soil follows elastic stress-strain law and the fluid is compressible and its motion follow Darcy's law. The degree of saturation in the poro-elastic sediment was governed by the compressibilities of water and gas, which was proposed by Fredlund (1976). Esrig and Kirby (1977) reported that the in-situ values of the degree of saturation S_r for marine sediment normally lie in the range of 90%-100%. Intertidal sediments are more likely to have trapped air (Mory et al, 2004, 2007). Mory et al. (2007) observed from their field data that the S_r-value of sand bed on the Atlantic coast of France ranged from 94% to 100% for the top 0.5m of the sand bed. Sandven et al. (2007) describe how the gas content can be measured in soils.

The basic assumptions used in the development of the analytical solution for the problem described in this study are:

- Linear wave theory is used to calculate the wave-induced pressure on the surface of the seabed (referred to as the mudline in the geotechnical literature, even for a sandy seabed).
- The wave pressure acting on the seabed is calculated based on the combined wave height, H, from an incident wave (H_i) and reflected wave (H_r) assuming a reflection coefficient of 1.0 (so H = $H_r + H_i = 2H_i$). The maximum standing wave height (H_{max}) is assumed to be 1.6 times the water depth (d).
- The porous seabed is homogeneously unsaturated, hydraulically isotropic and has an infinite thickness.
- The compressibilities of water and gas in the poro-elastic sediment are governed by the degree of saturation.
- The soil skeleton and the pore fluid are uniformly compressible.
- Despite phase lag of the pore pressure in very fine sediments, the soil skeleton generally obeys Hooke's law, implying linear, reversible and non-retarded mechanical properties.

- The flow in the porous bed is assumed to be governed by Darcy's law.
- The porous bed is sandy and the pore pressure in the soil is a result of elastic interaction between soil and water and, thus, neglects dilation effects.
- The structure is assumed to be deeply embedded into the soil matrix (i.e. beach).
- Liquefaction is defined as occurring when the mean effective stress is equal to zero at an elevation of 0.05m below the seabed level. The depth of 0.05m was chosen as the smallest depth of liquefaction worth considering.

7.3. Theoretical solution

The analytical solution was implemented into a Mathcad calculation sheet to determine the wave heights required to cause liquefaction to the soil. The developed equations and coefficients for the analytical solution are given in Appendix A.

The effects of wave height (H) and degree of saturation of the seabed (S_r) on the occurrence of liquefaction to fine sand, medium fine sand and coarse sand beds for four different water depths (2m, 5m, 10m and 15m) were investigated in the present study. The degree of saturation of the seabed ranges between 0.9 and 1.0. The adopted permeabilities of fine sand, medium fine sand and coarse sand seabeds are 10^{-4} m/s, 10^{-3} m/s and 10^{-2} m/s respectively (Jeng, 1998). As the model is applicable to homogenous soil, constant permeability and soil stiffness are adopted in the analysis although it is expected that the permeability and the stiffness of seabed vary with depth (permeability decreases and soil stiffness increases with depth). Other typical parameters of the medium dense sand seabed adopted can be found in Table 7.1.

Table 7.1 Typical material properties of sand bed

Description	Symbol	Unit	Value
Shear modulus	G	GPa	10
Poisson's ratio	ν		0.3
Porosity	n'		0.3
Coefficient of earth pressure at rest	K_0		0.5
Unit weight of sand	γ_s	kN/m^3	18
Unit weight of water	γ_w	kN/m^3	10

For the wave condition, the wave period is fixed to be 8s and the wave lengths for four different adopted water depths were calculated using linear wave theory (see Appendix A). The wave conditions adopted in the study are shown in Table 7.2.

Table 7.2 Wave conditions

Wave Period, T (s)	Water Depth, d (m)	Wave Length, L (m)
8	2	35
8	5	53
8	10	71
8	15	82

A series of parametric studies was carried out in this study and the cases are listed in Table 7.3. A total of 12 analyses were carried out to examine the minimum wave heights required to liquefy three different types of seabed under the wave condition generated in four different water depths ($H \leq 1.6d$). The degree of saturation of the seabed, S_r, ranged from 0.9 to 1.0. Other input parameters for this study are given in Tables 7.1 and 7.2.

Table 7.3 Parametric study

Case	Seabed Type	Permeability (m/s)	Water depth, d (m)	Degree of Saturation, S_r
1a	Coarse sand	10^{-2}	2	0.9-1.0
1b	Medium fine sand	10^{-3}	2	0.9-1.0
1c	Fine sand	10^{-4}	2	0.9-1.0
2a	Coarse sand	10^{-2}	5	0.9-1.0
2b	Medium fine sand	10^{-3}	5	0.9-1.0
2c	Fine sand	10^{-4}	5	0.9-1.0
3a	Coarse sand	10^{-2}	10	0.9-1.0
3b	Medium fine sand	10^{-3}	10	0.9-1.0
3c	Fine sand	10^{-4}	10	0.9-1.0
4a	Coarse sand	10^{-2}	15	0.9-1.0
4b	Medium fine sand	10^{-3}	15	0.9-1.0
4c	Fine sand	10^{-4}	15	0.9-1.0

7.4. Results and discussion

The parametric study results for cases 1a-1c, 2a-2c, 3a-3c and 4a-4c are presented in Figures 7.1, 7.2, 7.3 and 7.4 respectively. The wave height, H, presented in the figures is the wave height of the combined incident and reflected waves. Liquefaction occurs in the seabed when both wave condition and seabed condition fall into the area above the line. For instance, the medium fine sand bed, with S_r-value of 0.98, starts to liquefy under the wave condition with wave height greater than 1.3*d* (*d* = 2m, see Figure 7.1). In Figure 7.1, it shows that no liquefaction will occur to coarse sand seabed under the extreme wave condition of $H_{max} = 1.6d$, which is the maximum wave height that can occur in water depth *d*.

Figures 7.1 to 7.4 show that the liquefaction potential increases with a decrease in permeability. The results show that the liquefaction hardly occurs to the coarse sand seabed with a water depth less than 5m (shallow water). Due to low permeability of fine sand, the fine sand seabed tends to liquefy more easily than the coarser sand bed with a higher permeability. Moreover, the wave height leading to liquefaction increases with an increase in degree of saturation. This is mainly due to the damping of amplitude of pore pressure as the amplitude of the pore pressure gradient in the seabed decreases with an increasing degree of saturation.

Generally, the wave height for the occurrence of liquefaction in fine sand seabed increases sharply when the degree of saturation is greater than 99%. No momentary liquefaction can possibly occur to fully saturated coarse sand seabed under the most severe defined wave condition (H = 1.6d).

Table 7.4 shows the minimum fully reflected wave height required to liquefy the seabed to a depth of 0.05m, for different water depths and with degrees of saturation of 90%, 95% and 100%. For the unsaturated fine sand seabed with a sea depth of 2m, the seabed could liquefy with a wave height as small as 0.4m. For the deep water case (d = 15m), the unsaturated fine sand seabed could liquefy under the wave condition with a wave height of 0.9m. No wave-induced momentary liquefaction can possibly occur in a fully saturated seabed in shallow water (d ≤ 5m).

Table 7.4 Minimum wave height required to cause the occurrence of liquefaction to seabed

Wave Height (m) / Water depth (m) / Sand Type	**2**	**5**	**10**	**15**
Coarse (S_r = 90%)	3.4	4.3	6.1	8.3
Medium fine (S_r = 90%)	1.1	1.4	2.0	2.7
Fine (S_r = 90%)	0.4	0.5	0.6	0.9
Coarse (S_r = 95%)	4.4	5.8	8.3	11.4
Medium fine (S_r = 95%)	1.6	2.0	2.8	3.8
Fine (S_r = 95%)	0.5	0.6	0.7	1.2
Coarse (S_r = 100%)	-	-	-	-
Medium fine (S_r = 100%)	-	-	-	24.0
Fine (S_r = 100%)	-	-	14.2	18.7

7.5. Simplified assessment approach

Figure .7.5 presents the minimum height of standing wave required to induce momentary liquefaction to a depth of 0.05m in a seabed with Sr-value of 0.95 at various water depths. This figure can be used to estimate the minimum wave height required to induce liquefaction to seabed. An S_r-value of 0.95 was selected in the plot because the typical air content of an inter-tidal sand bed is approximately 5% (Mory et al., 2007).

To assess liquefaction potential with Figure 7.5 first select the water depth, d, and then determine the combined wave height, H from the incident wave height, Hi, i.e. H = 2Hi. If the value of H is greater than 1.6d then H is limited to H =

1.6d. Select the most representative bed sediment grading and if the value of H is equal to or greater than the value of H on the y-axis then momentary liquefaction can occur.

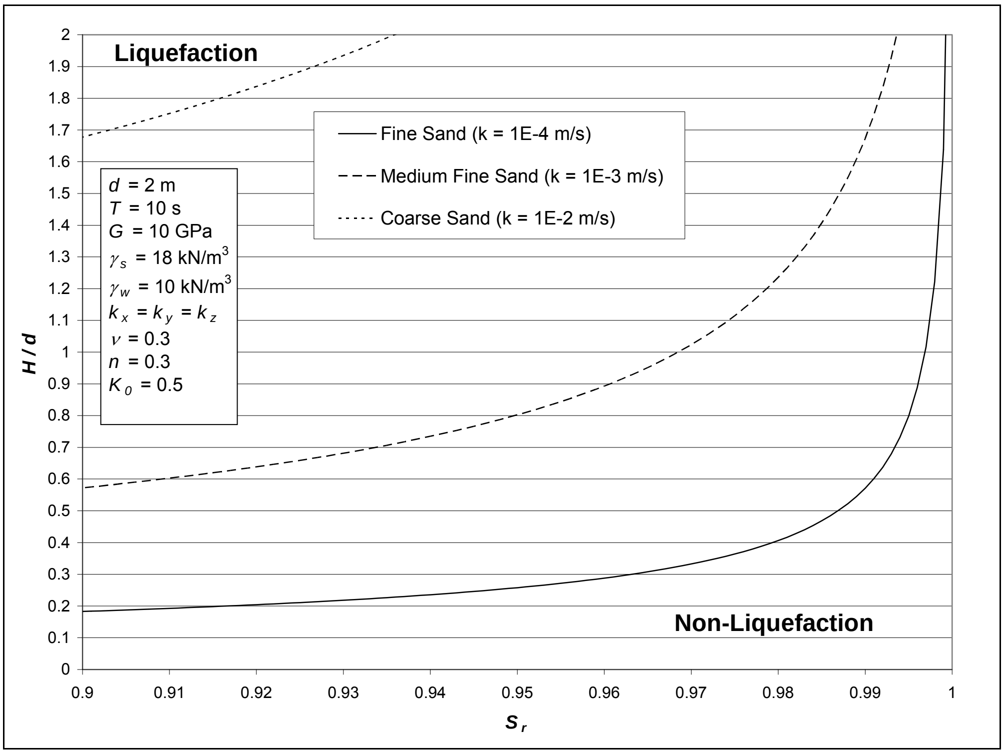


Figure 7.1 Wave-induced liquefaction potential around a marine structure founded on a saturated/unsaturated seabed subjected to various standing wave loadings with a water depth of 2m

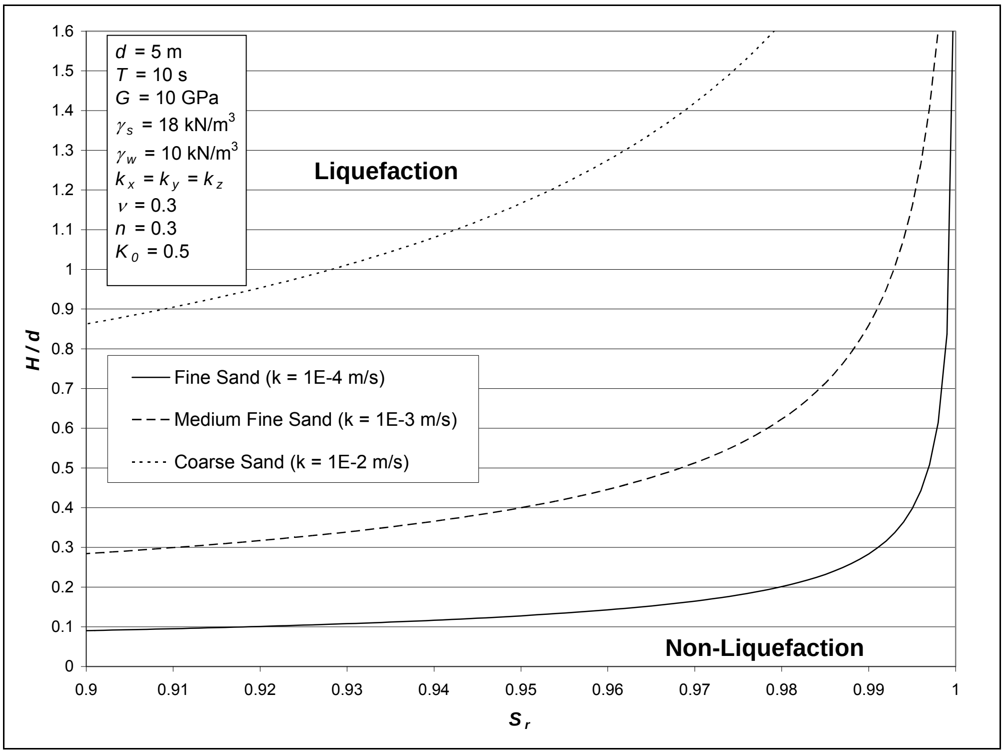


Figure 7.2 **Wave-induced liquefaction potential around a marine structure founded on a saturated/unsaturated seabed subjected to various standing wave loadings with a water depth of 5m**

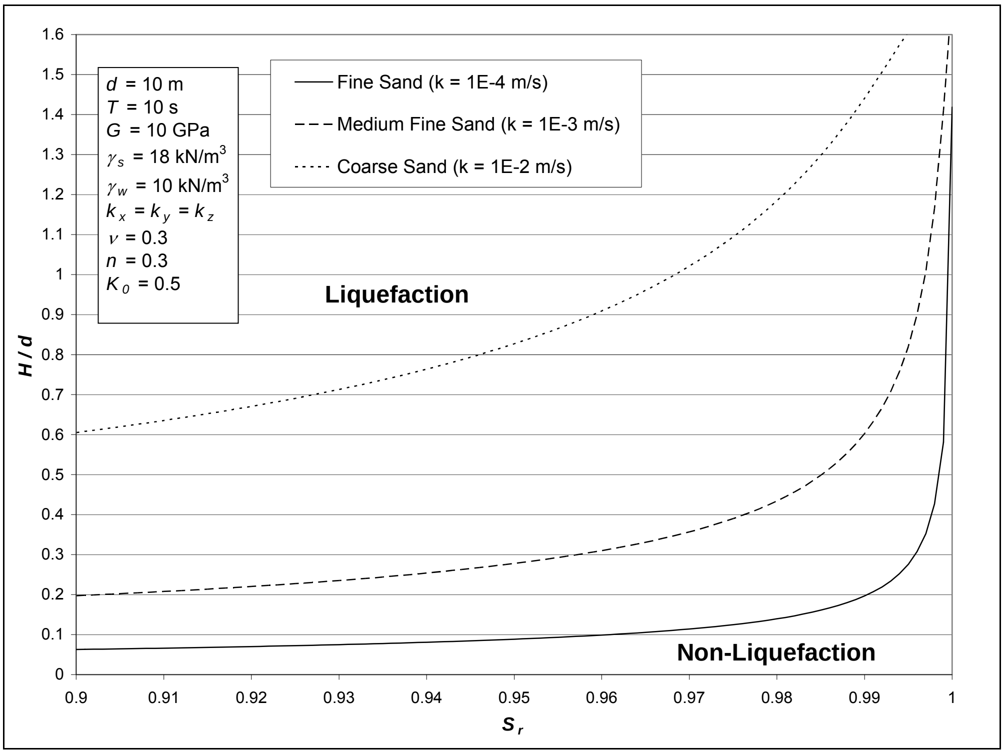


Figure 7.3 **Wave-induced liquefaction potential around a marine structure founded on a saturated/unsaturated seabed subjected to various standing wave loadings with a water depth of 10m**

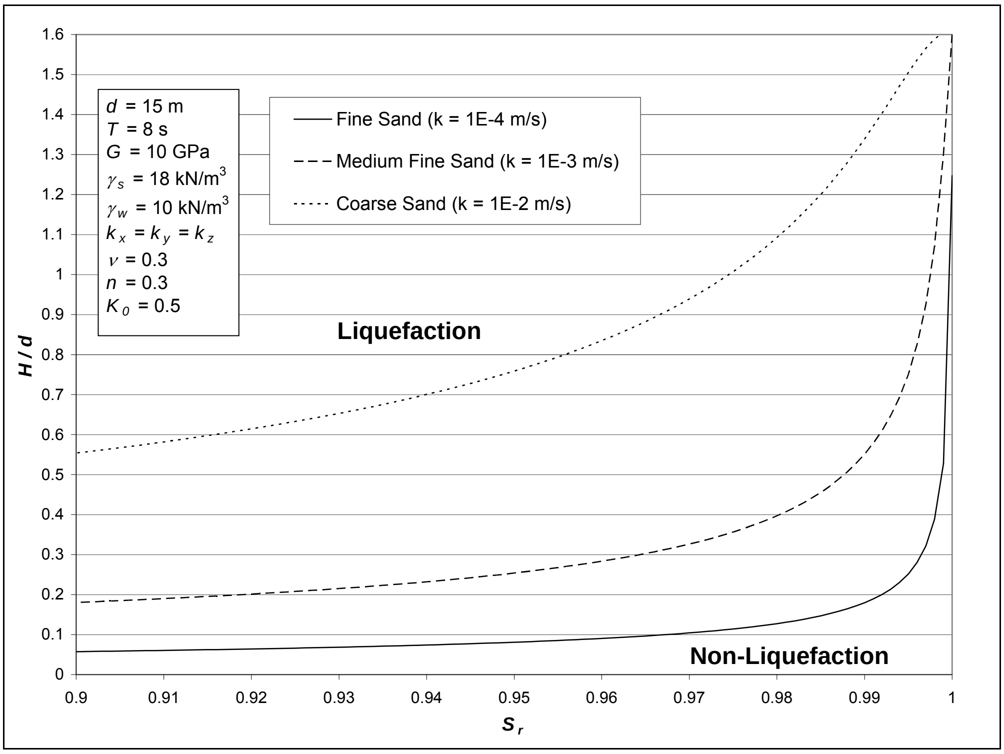


Figure 7.4 Wave-induced liquefaction potential around a marine structure founded on a saturated/unsaturated seabed subjected to various standing wave loadings with a water depth of 15m

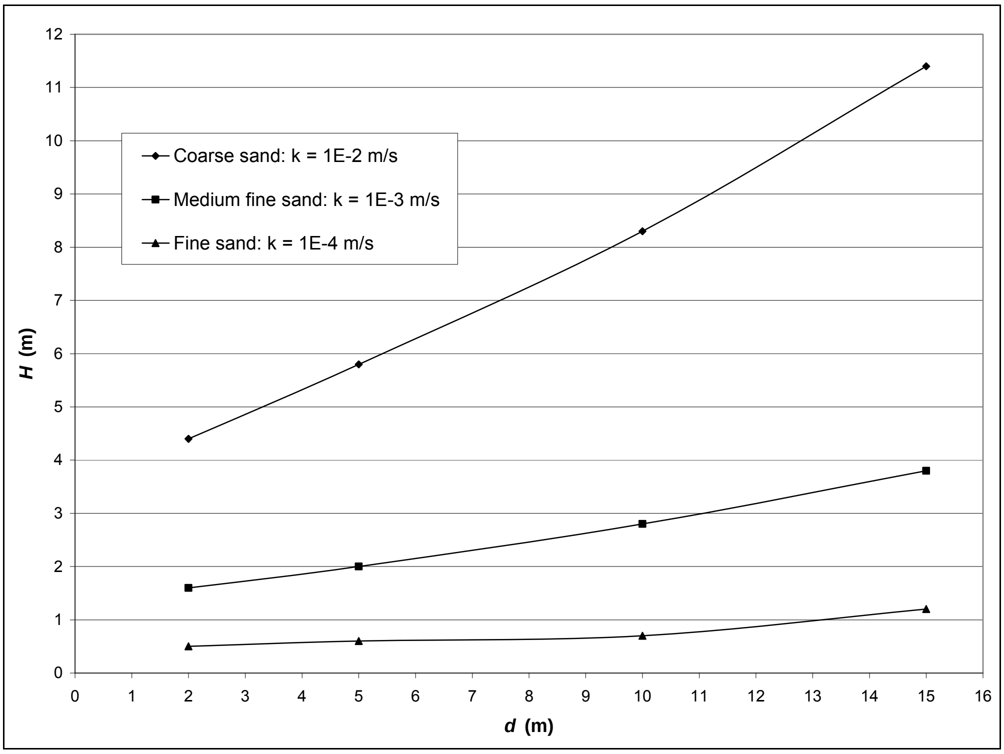


Figure 7.5 Wave heights required to liquefy three different types of sand bed with given permeability, k and with degree of saturation of 0.95, at various water depths

7.6. Conclusions

The following conclusions about wave induced momentary liquefaction can be drawn from this study:

- The likelihood of the occurrence of momentary liquefaction increases with a decrease in seabed permeability, which is associated with a decrease in grain size. A seabed of fine sand is therefore more likely to experience momentary liquefaction than a seabed of coarse sand;
- The likelihood of the occurrence of momentary liquefaction increases with a decrease in the degree of saturation of the seabed;
- The wave height required to liquefy a fine sand seabed increases significantly when the degree of saturation of the seabed increases higher than 0.995;
- An S_r-value of 0.95 is recommended for the estimation of the minimum wave height required to liquefy the seabed, in the absence of a site-specific study;
- Figures 7.1 to 7.5 can be used to provide a quick check on the potential for momentary liquefaction of the top 0.05m of the seabed. If the potential for momentary liquefaction exists, a more detailed, site-specific study can be carried out by adapting the Mathcad code developed within this project or using another liquefaction model (de Groot et al., 2006a).

8. Conclusions and recommendations for guidance

A number of distinct research studies have been carried out, which have improved the understanding of the lowering of beaches in front of coastal defence structures. These studies had four main elements, namely:

- The synthesis of existing information and approaches to predicting beach lowering and summarising the implications for monitoring. In practice the work in this project went beyond synthesising existing work and involved a number of original studies, including deriving a method for calculating the prediction horizon for extrapolated beach levels (or, indeed, any other form of prediction) undertaking an error analysis of tideline positions on OS maps and demonstrating both the Gaussian distribution of residual beach levels about a long-term trend and the seasonal variation in beach levels.
- The development of an improved scour predictor, which is suitable for use in design and in risk-based methods of asset management and which involved undertaking field and laboratory experiments.
- The provision of information on existing mitigation schemes.
- The development of an initial screening tool to assess the possibility of liquefaction in front of coastal defences.

The main conclusions are given below and are followed by a discussion of future research needs.

8.1. Development of improved scour predictors

Toe scour can reduce the level of the beach in front of a structure and increase the risk of undermining.

1. An extensive literature review and assessment of existing datasets have been used to identify laboratory test datasets that include measurements of toe scour and maximum wave-induced scour in front of vertical or sloping seawalls with wave heights sufficiently high to generate suspended sediment transport. A set of new laboratory experiments was then undertaken to extend this dataset.
2. The combined laboratory dataset was then used to derive equations representing low and high values of relative water depth for toe scour depth and for the maximum scour depth.
3. The maximum toe scour depth, $S_{t,max}$ on sandy beaches was predicted not to exceed the deep water value of significant wave height, H_{s0}, i.e. $S_{t,max} < H_{s0}$.
4. Statistical analysis gave a Root-Mean-Square error in the predicted values of relative scour depth of about 0.17. A single equation was derived to calculate toe scour depth as a function of h_t/L_m, although it has systematic and unsystematic errors. An equation was then derived as an alternative with zero systematic error and zero bias, but a slightly higher total root mean square error. The predictor takes

beach slope as well as relative toe depth into account and provides significant additional predictive capability for seawall scour in sand beaches.

5. On shingle beaches the predictor of Powell and Lowe (1994) is recommended.
6. The medium-scale flume tests, from which the sand beach toe scour predictors were derived, have used a limited range of wave heights and bed sediments. A set of large scale flume tests is therefore recommended to discriminate the effect of wave height and sediment size on scour depth.

8.2. Assessment of seabed liquefaction

Wave induced liquefaction can reduce the bearing capacity of the seabed in front of a structure.

1. An analytical solution for the wave-induced pore pressure response in an isotropic infinite thickness seabed in front of a breakwater was used to study the liquefaction potential of the seabed in front of coastal defence structures subjected to various wave loadings. The liquefaction potential was determined by calculating the minimum total wave height to depth ratio that will cause the momentary liquefaction of the top 0.05m of a sandy seabed in front of a vertical seawall. The liquefaction potential depends on the degree of saturation of the pore water in the sediment which affects its compressibility.
2. Calculations were made for fine, medium fine and coarse sand with the degree of saturation between 0.90 and 1.0, for a range of water depths and a typical storm wave period of 8s. The results can be used to indicate whether liquefaction of the seabed in front of a coastal structure is likely to occur. If so, a more detailed study should be carried out.
3. The likelihood of the occurrence of momentary liquefaction of the seabed increases with a decrease in seabed permeability, which is associated with a decrease in grain size. A seabed of fine sand is therefore more likely to experience momentary liquefaction than a seabed of coarse sand.
4. The likelihood of the occurrence of momentary liquefaction increases with a decrease in the degree of saturation of the seabed;
5. The wave height required to liquefy a fine sand seabed increases significantly when the degree of saturation of the seabed increases higher than 0.995.
6. An S_r-value of 0.95 is recommended for the estimation of the minimum wave height required to liquefy the seabed, in the absence of a site-specific study.
7. Graphs have been developed that can be used to provide a quick check on the potential for momentary liquefaction of the top 0.05m of the seabed. If the potential for momentary liquefaction exists, a more detailed, site-specific study can be carried out by adapting the Mathcad code developed in FD1927, or using another liquefaction model.

8.3. Integration into reliability analysis

The results of the research can be implemented in the existing analysis methodologies used to determine the performance of coastal structures.

1. The derivation of the improved scour predictor will inform the future development of fault trees and fragility curves for coastal structures which are known to be sensitive to scour.
2. The identification of the Gaussian distribution of beach levels about a long term trend will reduce the uncertainty in the calculation of the fragility curves of coastal defences at a particular time.
3. The forecasting of changes in the mean beach level in front of a coastal defence, whether through extrapolation of historical data or numerical modelling, will allow the change in the fragility curve with time to be calculated. This contributes to calculations of the deterioration in asset condition with time and could be used to trigger a more detailed form of condition assessment, rather than waiting for scour to be observed or failure to occur.
4. In regions where beaches are regularly monitored the use of a visual condition index for a beach should, in time, be replaced by a quantitative measure of beach performance derived from the measurements. This may require the development of suitable coastal state indicators and of methods to determine suitable threshold levels for them.
5. The development of a method for calculating prediction horizons will inform the duration of the forecasts of future beach behaviour (if sufficient data is available) that can usefully be used in coastal management.
6. The frame of reference approach has been identified as a useful way of linking management objectives to the technical solutions that are used to meet those objectives and evaluating their success.

8.4. Prediction tools, shoreline retreat, uncertainty and coastal state indicators

1. Different tools are needed to predict the response of the coastline at different scales. These tools come with different levels of reliability, accuracy, skill and required expertise. These tools may be allocated to one of four basic types: statistical analysis, process-based numerical modelling, geomorphological analysis and parametric equilibrium models. The numerical models attempt to describe fewer and fewer processes in detail as the spatial and temporal scale they are deployed over increases. The suitable time and space scales have been illustrated for the different model types.
2. In the coastal regions where the Bruun rule can be said to apply, the rate of shoreline retreat is directly proportional to the rate of sea level rise. It follows that the ratio of future shoreline retreat rate to present day shoreline retreat rate (the shoreline retreat rate multiplier) will be the same as the ration of future sea level rise rate to present day sea level rise rat. calculated using present day rates of sea level rise and regional sea level allowances (Defra, 2006). These calculations show

that shoreline retreat rates in regions where the Bruun rule applies could increase significantly – in some cases by a factor of 13 - during the 21st century.

3. The shoreline retreat rate multipliers are highest for the Northwest and Northeast of England and Scotland as this region has the lowest present day rate of sea level rise, due to isostatic rebound following the last ice age, which may also imply lower rates of present day shoreline retreat. The systems model SCAPE predicted a rather more complex response, with lower overall vulnerability to sea level rise, than the Bruun rule. Therefore the magnitudes of the shoreline retreat rate multipliers should be treated with some caution as they may well be too high. It seems probable that the shoreline recession rate will increase in many places if the rate of sea level rise increases.
4. The uncertainty in shoreline position from OS tidelines is a combination of source uncertainty, interpretation uncertainty and natural variability. Methods for calculating each of these uncertainties have been developed and example values calculated. These errors can be incorporated into analyses of historical shoreline movement, which are often used in Shoreline Management Plans and strategy studies.
5. The best coastal state indicator for assessing the contribution of a beach to the overall risk of flooding or erosion is not yet known. Possible coastal state indicators include the beach level at the toe of a structure, the beach level plus beach slope and the beach cross-sectional area above a set contour.

8.5. Beach levels – measurement, results & analysis methods

1. Advances in measurement technology have made it easier than ever before to measure beach levels at a point on a daily basis, or even more frequently.
2. Many of the possible techniques for measuring time series beach levels at a point have not yet been evaluated for the cases of beach level at the toe of a structure.
3. When beach levels have been measured at a point at a rate of at least once a day the time series have generally been short or, in the case of the Blackpool Tell-Tail data, there has been no tie-in to other data gathering programmes. The Blackpool data would have been more useful if it had been collected within an integrated beach monitoring programme.
4. There is a gap in frequency of data collection between the point measurements of beach levels through a tide (sampling about 4 times per hour) and beach profiles (collected typically 4 times per year). It is therefore impossible to determine from the data if the beach variations at the two frequencies are related, although changes in the beach level at the toe of a structure between tides are the residual of the changes within each tide. An implicit assumption that the processes are unrelated has been made in incorporating these results into the development of fragility curves.

5. A clear seasonal trend was observed in the Linconshire dataset of beach levels at the toe of the local seawalls. The trend was lower than the standard deviation about the trend.
6. Beach levels at a point in front of a structure can generally be de-trended using a simple linear least squares method, providing that neither the coastal defences nor beach management policy changed during the data collection period.
7. Further data analysis should be undertaken to ascertain if a regional approach could be taken to providing guidance on possible changes in beach levels, for use in the design of new structures, in a similar way to the regional net sea level rise allowances (Defra, 2006).
8. There is a need to establish the relationship between the behaviour of a single contour and that of the beach volume at a local level before a contour can be used as a surrogate for beach volume.
9. Beaches around Donna Nook in north Lincolnshire often showed an increase in beach volume (area under a beach profile) combined with a retreat (shoreward movement) in Mean Sea Level. Further south in Lincolnshire (between Mablethorpe and Skegness) beach volumes were found to increase as the Mean High Water advanced (moved seawards).
10. Advanced linear analyses of beach level data (such as the use of wavelets and Empirical Orthogonal Functions) and nonlinear analyses of beach level data (such as Singular Spectrum Analysis and fractal analysis) are becoming more common in academic circles. These sophisticated methods require more data of good quality than the simple linear methods require. They may also impose more constraints on the data, such as the need to be equally spaced in time and position. It will be possible to apply these methods to more areas of the English and Welsh coastlines as coordinated regional data gathering and data management programmes extend their geographical range and temporal duration.

8.6. Results obtained from analysis of beach levels

1. Beach surveys that are intended to predict the long-term trends in shoreline position should be made when the standard deviation in the beach level is low. This occurred in August for the Lincolnshire data (but in June/July for Duck, N.C., U.S.A.) and also coincided with relatively high beach levels.
2. Beach surveys that are intended to indicate how low beach levels can fall should be undertaken when the average beach level is low and the standard deviation in beach level is high. In Lincolnshire this occurred around March.
3. When a linear trend is extrapolated to provide a future prediction of beach level is has a site-specific prediction horizon. This is the average length of time over which an extrapolated trend produced a useful level of prediction compared to a baseline prediction (taken to be that future beach levels will be the same as the average of the measured beach levels).

4. Extrapolation of a linear trend fitted to 10 years of data gave prediction horizons between 0 years and 14 years.
5. Extrapolation of a linear trend fitted to 5 years of data always gave a negative skill score (i.e. a worse prediction than the baseline) so should not be used.
6. The use of extrapolated beach levels is more suitable for managed / adaptive beach management policies, rather than the precautionary approach as the latter has a longer timeframe than the longest prediction horizon.
7. The standard deviation in beach level from using 2 surveys per year was 6% different from using 10 surveys per year. The difference was approximately halved by increasing the number of surveys from 2 to 3 times per year.
8. Residual (de-trended) beach levels have a Gaussian distribution about the mean beach level, again providing that neither the coastal defences nor beach management policy changed during the data collection period.

8.7. Review of mitigation measures

The likelihood of beach lowering should have been included already in the design of any structure. In many cases the details of how a structure was constructed have been lost so, for example, the level of the structure toe is not known and the allowance made for beach lowering is not known. In some cases it will be possible to accommodate beach lowering by installing storm warning systems, delimiting areas to prevent development, increasing flood resilience, improving drainage, strengthening surfaces behind the structure to withstand higher flows or installing secondary flood defences to limit the extent of flooding.

Various approaches to reinforcing the beach or bed in front of a seawall have been tried over the years. Other techniques and structures have been applied to reduce the rate of shoreline retreat or to maintain a beach in front of a structure:

1. **Fagotting and wave breakers** have been used to reinforce shingle ridges protecting low lying areas from flooding. Most such installations are now redundant, with beach nourishment being widely used instead. The anticipated structural life of faggoting is generally low, possibly as little as 5 to 10 years, while timber wave breakers can be expected to achieve at least twice this lifespan, although they are vulnerable to being damaged by wave impact, and are prone to abrasion by beach sediments.
2. Fagotting and wave breakers may well be of some benefit as emergency works, or as short-term low-cost schemes in instances where more costly methods of protection are difficult to justify. Fagotting, in particular, is a technique that might be suitable in sheltered environments where abrasion is not a serious issue, where low cost is an overriding factor, where manual labour is available at low rates, and where the possible need for maintenance is not a major concern.
3. **Scour mattresses** are typically deployed to prevent the undermining of structures, as bed levels near them are lowered by scour caused by

the presence of the structures themselves. These mattresses, which are normally prefabricated, provide an interface between the normally solid and impermeable structures and the mobile, permeable sediments surrounding them. However, there are few instances of this type of protection being used within the intertidal beach zone anywhere in the UK. Their usage on the open coastline is generally restricted to backshore protection although there are a few examples of these being used as lightweight revetments at or above high water;

4. The availability of suitable **rock,** and greater awareness of the consequences of scour, has led to its increasing use in mitigating problems caused by beach lowering through the construction of rock blankets, toe berms and fillets. A good starting point for the design of rock (or concrete armour unit) structures in coastal engineering is the "Rock Manual" (CIRIA,1991, CIRIA / CUR / CETMEF, 2007).
5. The infilling of scour trenches, and the construction of a scour blanket or a sloping rock toe are designed to prevent the undermining of the structure. They are likely to suffer further beach lowering if the processes of beach erosion continue and this should be considered in their design. They are often installed at or below the beach level so may be covered for much of the time.
6. The construction of a more substantial rock fillet that extends partially up the height of the coastal defence structure may also serve to reduce one or more of wave run-up, overtopping, impact pressures, wave reflections and scour. Care must be taken in the design of rock fillets to ensure it does not increase run-up, overtopping and/or impact pressures by changing the way the wave break onto the structure.
7. The more substantial rock structures commonly have bedding layers or geotextiles to form an interface between the rock and the beach sediment.
8. **Detached breakwaters** are best suited for situations where the existing defences require higher levels of protection at sensitive points, i.e. over short frontages of coastline where beach lowering / scour are causing localised problems of overtopping or undermining. They may be well also suited to frontages where the wider beach formed can be justified for recreation/amenity purposes. In areas where there are large tidal ranges and/ or strong tidal currents, detached breakwaters will have more detrimental impacts than in micro-tidal or sheltered regions.
9. There are no low crested breakwaters or submerged reefs in the UK, but their applicability for mitigating problems of beach lowering or scour in front of coastal structures remains uncertain, based on the observations made elsewhere.
10. **Shore parallel sills** have been used with some degree of success in Mediterranean countries, but only in micro-tidal/moderate wave energy conditions. Their usefulness in high energy/macro-tidal conditions is yet to be proven. However, they have been successful as backshore protection, i.e. in conditions where the tidal range is not a critical factor;
11. The problems of beach lowering will be sufficiently severe and widespread in some situations, as to be worth considering a **major scheme to improve those beaches.** The direct remedy to such

problems is to import large quantities of extra beach sediment, i.e. sand or gravel, to replace that gradually lost previously, i.e. a beach recharge scheme. This approach will immediately cover over the toe of coastal structures and decrease water depths in front of them, and in many cases will improve the amenity value and aesthetic appearance of the frontage. Further details on the design and execution of major beach improvement schemes is provided in the Beach Management Manual (CIRIA, 1996a).

12. There will often be a need for ancillary works to accompany an initial “recharge” of a beaches, such as the building of groynes, monitoring and analysis of the changes in beach levels and for periodic addition of extra material in later years. These various elements are now generally identified as components of a beach improvement scheme.
13. **Groynes** are the most widely used method of controlling beach levels in the United Kingdom, where they have been used in a wide variety of situations. On shingle beaches, groynes can be used in any tidal range and under most wave conditions. On sand beaches, however, groynes are most effective in low to medium tidal ranges, because their cost can be prohibitive in areas with large tidal ranges. The spacing of groynes is related to their length. Shorter, higher and closer spaced groynes are used on shingle beaches reflecting the steeper gradients that occur on such beaches, both perpendicular and parallel to the shoreline. In contrast, groynes on sandy beaches are longer, lower and more widely spaced, typically at twice their length or more.
14. Details of the design of groyne systems, with or without beach recharge, can be found in the CIRIA Beach Management Manual (1996a). It is worth noting, however, that the design of a groyne system should not be based purely on such guidelines alone; their design always needs to be matched to local conditions.
15. Installing groynes without recharge will normally lead to problems of erosion further along the coast in the direction to which the sediment is moving, (i.e. “downdrift”) potentially extending over many kilometres. The greatest problems of erosion, however, tend to occur just downdrift of the last groyne, and may lead to the “outflanking” of a coastal structure if care is not taken to avoid this possibility. Groynes are often used in combination with beach recharge, and under such conditions model testing becomes almost mandatory.
16. Probably the most common adjustments to a coastal structure are **remedial works** such as underpinning, encasement, or addition of an apron to the wall, although there are few guidelines for design. Such works are rarely a permanent solution to the problem and hence they often have to be repeated later, either extending the protection downwards or further along the coastline. However, they do make maximum use of the existing structure which is often still sound, or can be repaired without great expense. Underpinning typically requires excavation beneath, and often behind, the face of the structure, the construction of a new and deeper “toe” and backfill of the area behind. Encasement also involves the covering of the front face of the structure, and sometimes building above its existing crest and even

over an existing back-slope, i.e. covering some or all of the original structure with a new, normally concrete, layer.

17. The construction of an apron or of steps at the base of an existing structure can prolong the life and improve the performance of that defence, at reasonable cost. However, such an intervention will not remedy the underlying causes of beach lowering. Such additions to a structure will extend it seaward, often occupying an area of the beach that previously provided an amenity area, and affecting the natural sediment transport processes in that area. There is a danger that such seaward extensions of a structure will interfere with longshore sediment transport, and hence reduce sediment supply to downdrift beaches.
18. In some situations the situation may have become so critical, that it is more cost effective to reconstruct/replace an old wall, rather than mitigate scour in front of it. Reconstruction will not remedy the underlying causes of beach lowering and may also interfere with longshore sediment transport.

8.8. Recommendations for future research

The work in FD1927 has identified the following research needs, some of which may be best suited to funding through a research council (RC).

1. Large scale flume tests of scour in front of coastal structures to confirm the scour relationships performed at a single (medium) scale (RC);
2. Tests on the stability of rock toe protection in front of coastal structures;
3. Detailed processes research and numerical model development to gain a better understanding of cross-shore sediment transport. Without this it will not be possible to produce a morphologically balanced model that can run for months to years without significantly and erroneously eroding or accreting a beach (RC). This could include the following item.
4. Combined field measurements of intertidal beach profiles and beach levels through the tide at a seawall at a timescale of months. This will relate the scour within a tide to the variations in beach level between tides to produce a better understanding of beach level variation in front of coastal defence structures over periods of weeks to months.
5. Continuing work to incorporate the results of this research into reliability analysis. This work is being undertaken within Floodsite and FRMRC and care should be taken to ensure that the outputs from these projects are taken up for general use within reliability analyses using, for example, RASP and/or the PAMS Operational Framework.
6. Work should be undertaken to attempt to quantify which coastal state indicator will best represent the contribution of beaches to overall flood risk.

8.9. Development of a toe scour manual

Consideration should be given to the development of a toe scour guidance manual, which could be produced without the need for new research and within a period of 12 to 18 months. Although certain areas could benefit from research, such as the stability of rock armour at the toe of a coastal defence, a useful guidance document could be produced by the collation of existing information. Such a manual would give practical guidance on the prediction of toe scour and the options for mitigating toe scour. It would have to complement, but not replicate, the following recent guidelines:

- Coastal Engineering Manual (U.S. Army Corps of Engineers. 2002);
- The Rock Manual (CIRIA / CUR / CETMEF, 2007);
- The European Overtopping Manual (http://www.overtopping-manual.com/)
- The Beach Management Manual (under revision);
- Condition Assessment Manual (Environment Agency, 2006).

It could also draw on the review of Sutherland et al. (2003) and on the contents of this report and other current research, as such results become available. The guidance on predicting toe scour would be based on Sutherland et al. (2003) and this report.

8.9.1. Policy context

The response to toe scour will depend on the management strategy for the management unit involved, which will have been set in the relevant Shoreline Management Plan as either ‘Hold the Line’, ‘Advance the Line’, ‘Managed Realignment’ or ‘No Active Intervention’. If the strategy is ‘Managed Realignment’ or ‘No Active Intervention’ then the response to toe scour would not be to mitigate. Only if the strategy was to ‘Hold the Line’ or to ‘Advance the Line’ would mitigation be considered, unless temporary emergency works were approved. In a local strategy study the coastal defence options will have been considered and a preferred option selected. At the moment the selection is based on a cost-benefit analysis, although this may be replaced by a multi-criteria analysis - see below. A toe scour manual must therefore consider the policy context, within which mitigation may take place.

8.9.2. Performance context

A toe scour manual would provide guidance on the development of options to be included in the decision making process. The main design issues to be addressed are

a) The impact on the structural reliability (fragility) of defence assets. This would cross-reference the relevant sections of the Rock Manual, Beach Management Manual and Coastal Engineering Manual and would draw on Section 5 of this report for case studies and general guide to suitability and Section 6 of this report for information on how beach levels affect fragility.

b) The impact on hydraulic performance, particularly overtopping. This would cross-reference the relevant sections of the upcoming European Overtopping Manual and link overtopping to failure modes.

Other issues to be considered in the selection of options include:

- Access and amenity: the importance of these elements of the design will depend on the use of the beach and its amenity value.
- Visual impact: the importance of visual impact will depend on beach use and policy (ECUS, 2003).
- Impact on the environment: the importance of this will depend on whether the area has a designated status (e.g. RAMSAR, SAC) or if the development of the option will affect a local designated area.
- Long-term sustainability: this issue is becoming increasingly important in response to Making Space for Water. Sustainability issues include deterioration (and its effects on whole life costs), resilience and adaptation. This section would consider how approaches used during construction and maintenance could reduce the environmental impact of toe scour mitigation measures.

8.9.3. Option Selection

Approval for funding of a scheme has depended in recent years on the results of a cost benefit analysis, as described in FCDPAG3 (2000) and FCDPAG3 Supplementary Note to Operating Authorities (2003) which include recreational and environmental issues. There has, however, been a recent move towards Multi-Criteria Analysis, which includes (RPA, 2004, Table 5.17) assets, land use, transport, business development, physical habitats, historical environment, landscape and visual amenity, recreation, health and safety, availability and accessibility of services and (HR Wallingford et al., 2005) the risks to human life. There has also been recognition of the limitations of project appraisal guidance (RPA and Royal Haskoning, 2006).

The PAMS operational framework also contains a systems analysis of coastal defence system to determine the critical assets, to which most risk can be attributed and hence which will benefit most from improvement, as well as assessment of the benefits of each maintenance or improvement option. The methodologies to be used for these assessments are being developed and may, in time, come to include RPA (2004) and RPA and Royal Haskoning (2006).

A toe scour manual would not be the correct place to develop a decision support system for asset management, to determine the choice of a particular option. However, it would be an appropriate place to develop a method, possibly a decision tree, to identify technically suitable mitigation options.

8.9.4. Indicative outline content

An indicative outline contents for a toe scour manual is given below:

- Introduction
- Policy context

- Scour processes
- Estimation of scour
- Inputs to design process
- Mitigation options
    - Monitoring and accommodating
    - Ancillary works
    - Adjustment to structure
    - Major beach improvement methods
- Environmental and sustainability issues
- Identification of suitable option(s)
- Appendix 1: monitoring
- Appendix 2: case studies.

The writing of a toe scour manual would be assisted by the advice from a strong project board, which should include representatives from:

- Sustainable Asset Management Theme Advisory Group.
- Environment Agency Asset Management team.
- Coastal Local Authority.
- Civil Engineering Consultant [not the contractor].

9. Acknowledgements and conditions of use

This project was funded by the Department for the Environment, Food and Rural Affairs (Defra) under commission FD1927 as part of the joint Defra/EA Flood and Coastal Erosion Risk Management R&D Programme. In addition the project received data for free from the Channel Coastal Observatory (the strategic regional coastal monitoring programme for the southeast of England) and the National Tidal & Sea Level Facility and received considerable assistance from the University of Southampton.

Channel Coastal Observatory provided data on beach profiles, benchmarks and waves for the Southbourne field work (HR Wallingford, 2006a). The data were provided "as is" and in no event shall the Channel Coastal Observatory be liable for any damages, including, without limitation, any disruption, damage and/or loss to your data or computer system that may occur while using the data. Channel Coastal Observatory makes no warranty, express or implied, including the warranties of merchantability and fitness for a particular purpose; nor assumes any legal liability or responsibility for the accuracy, completeness or usefulness of any data, information, apparatus, product, or process disclosed; nor represents that its use would not infringe the rights of any third party.

Tide level data used in HR Wallingford (2006a, b, c) and Section 3 were supplied by the British Oceanographic Data Centre as part of the function of the National Tidal & Sea Level Facility, hosted by the Proudman Oceanographic Laboratory and funded by the Environment Agency and the Natural Environment Research Council. BODC, NERC and the Environment Agency give no warranty as to the accuracy of the data or as to the suitability of the data for their intended use by the recipient and shall not be responsible for any errors or omissions, or for the use of, or results obtained from the use of this information.

We gratefully acknowledge the assistance given by many individuals for their assistance during the course of this study, in particular:

- Andrew Pearce, David Rycroft and Gerald Müller, University of Southampton, for their assistance with the field work at Southbourne, data analysis of the field work at Blackpool, assistance with performing the physical model tests at HR Wallingford and data analysis from the physical model tests.
- Nicholas da Costa, Institut des Sciences et Techniques de l'Ingenieur de Lyon (ISTIL) France for data analysis of the Lincolnshire dataset for Section 3, performed during his internship at HR Wallingford;
- David Harlow, Bournemouth Borough Council for his help selecting the scour monitor site at Southbourne and permission to deploy equipment;
- John Shaw and Mike Pomfret, Blackpool Borough Council for permission to use the Blackpool scour monitor data;
- Uwe Dornbusch, University of Sussex (now Mouchel Parkman) for discussions of the analysis of OS tidelines;

This report is a contribution to research generally and it would be imprudent for third parties to rely on it in specific applications without first checking its suitability. Various sections of this report rely on data supplied by or drawn from third party sources. HR Wallingford accepts no liability for loss or damage suffered by the client or third parties as a result of errors or inaccuracies in such third party data. HR Wallingford will only accept responsibility for the use of its material in specific projects where it has been engaged to advise upon a specific commission and given the opportunity to express a view on the reliability of the material for the particular applications.

10. References

Allsop, N.W.H., Bruce, T., Pearson, J. and Besley, P., 2005. Wave overtopping at vertical and steep seawall. In: Proceedings of the ICE: *Maritime Engineering Journal*, ISSN 1741 7597, September 2005, pp 103-114, Thomas Telford, London.

Allsop, N.W.H., Bruce, T., Pearson, J., Alderson, J.S. and Pullen, T., 2003. Violent wave overtopping at the coast, when are we safe? In: *Proceedings International Conference on Coastal Management 2003*, pp 54-69, ISBN 0 7277 3255 2, Thomas Telford, London.

Allsop, N.W.H., McKenna, J.E., Vicinanza, D. and Whittaker, T.J.T., 1996a. New design formulae for wave loadings on vertical breakwaters and seawalls. In: *Proceedings of 25th International Conference on Coastal Engineering* (ASCE), September 1996, Orlando, ASCE, New York.

Allsop, N.W.H., Vicinanza, D. and McKenna, J.E., 1996b. Wave forces on vertical and composite breakwater. *HR Wallingford Report SR 443*, pp 1-94, March 1996, Wallingford.

Allsop, N.W.H. and Williams, A.F., 1991. Hydro-geotechnical performance of rubble mound breakwaters. *HR Wallingford Report SR183*, February 1991, Wallingford.

Allsop, N.W.H. and Wood, L.A.,.1987. Hydro-geotechnical performance of rubble mound breakwaters. *Hydraulics Research Report SR98*, March 1987, Wallingford.

Aubrey, D.G., 1979. Seasonal patterns of onshore/offshore sediment movement. *Journal of Geophysical Research* 84(C10) 6347 – 6345.

ASCE, 2006. Special Issue on Liquefaction around Marine Structures. *Journal of Waterway, Port, Coastal, and Ocean Engineering*, 132(4), Jul/Aug 2006.

Bakker, W.T., Klein Breteler, E.H.J. and Roos, A., 1970. The dynamics of a coast with a groyne system. In: *Proceedings of the 12th International Conference on Coastal Engineering,* ASCE, 1001-1020.

Bell, P.S., 1999. Shallow water bathymetry derived from an analysis of X-band marine radar images of waves. *Coastal Engineering* 37: 513 – 527.

Besley, P., 1999. Overtopping of seawalls – design and assessment manual. *Environment Agency R & D Technical Report W178*, ISBN 1 85705 069 X, Bristol, also from: http://www.hrwallingford.co.uk/downloads/projects/overtopping/overtopping_manual.pdf

Biot, M. A., 1941. General theory of the three-dimensional consolidation. *Journal of Applied Physics,* 12 (1), pp. 115-129.

Brady, A..J. and Sutherland, J., 2001. COSMOS modelling of COAST3D Egmond main experiment. *HR Wallingford Report TR115*, Wallingford.

Bray, M.J. and Hooke, J.M., 1997. Prediction of soft-cliff retreat with accelerating sea-level ris. *Journal of Coastal Research* 13 (2), 453– 467.

Bruun, P., 1954. Coast erosion and the development of beach profiles. *Beach erosion board technical memorandum. No. 44*. U.S. Army Engineer Waterways Experiment Station. Vicksburg, MS.

Bruun, P., 1962. Sea level rise as a cause of shoreline erosion. *Journal of Waterway and Harbour Division, ASCE*, 88: 117-130.

Burgess, K., Hosking, A., Loran, F., Moore, R., Lee, M., Reeve, D. and Pedrozo-Acuña, A., 2006. Risk Assessment for Coastal Erosion. In: *Proceedings, 41st Defra Flood and Coastal Management Conference*, York.

Burgess, K.A., Orford, J., Townend, I., Dyer, K. and Balson, P., 2002. FUTURECOAST: the integration of knowledge to assess future coastal evolution at a national scale. In *Proceedings of the 28th International Conference Coastal Engineering 2002*. McKee Smith (Ed), World Scientific, pp 3221 – 3233.

Burgess, K., and I. Townend, 2004: The impact of climate change upon coastal defence structures. *39th DEFRA Flood and Coastal Management Conference*, 29 June - 1 July, York UK, Department of Environment, Food and Rural Affairs (Defra), 11.12.11-11.12.12.

Carr, A.P. 1962. Cartographic error and historical accuracy. *Geography,* 47: 135-146.

Carr, A.P., 1980. The significance of cartographic sources in determining coastal change. In *Timescales in Geomorphology*, Cullingford, Davidson and Lewin (eds), Wiley, Chichester, 69-78.

Cassen, M., Abadie, S., Arnaud, G. and Morichon, D., 2005. A method based on electrical conductuivity measurement to monitor local depth changes in the surf zone and in depth soil response to the wave action. In *Proceedings of the 29th International Conference, Coastal Engineering 2004*, Volume 3. World Scientific, pp 2302 – 2313.

CIRIA (Construction Industry Research and Information Association), 1996a. *Beach Management Manual,* CIRIA Report 153.

CIRIA (Construction Industry Research and Information Association), 1996b. *Beach Recharge Materials –Demand and Resources*, CIRIA Report 154.

CIRIA (Construction Industry Research and Information Association), 1991. *Manual on the use of rock in coastal and shoreline engineering*. First Edition. pp640, ISBN: 0-86017-326-7.

CIRIA / CUR / CEFMEF, 2007*. The rock manual. The use of rock in hydraulic engineeringl*. Second Edition. Pp1200, ISBN: 0-86017-683 - 1.

Cooper, J.A.G. and Pilkey, O.H., 2004a. Alternatives to the mathematical modelling of beaches. *Journal of Coastal Research*, 20(3) 641 – 644.

Cooper, J.A.G. and Pilkey, O.H., 2004b. Sea level rise and shoreline retreat: time to abandon the Bruun rule. *Global Planet Change*, 43: 157 – 171.

Cowell, P.J., Stive, M.J.F., Niederoda, A.W., de Vriend, H.J., Swift, D.J.P., Kaminsky, G.M. and Capobianco, M., 2003a. The coastal-tract (part 1): a conceptual approach to aggregated modelling of low-order coastal change. *Journal of Coastal Research*, 19(4): 812 – 827.

Cowell, P.J., Stive, M.J.F., Niederoda, A.W., Swift, D.J.P., de Vriend, H.J., Buijsman, M.C., Nicholls, R.J., Roy, P.S., Kaminsky, G.M., Cleveringa, J., Reed, C.W. and de Boer, P.L., 2003b. The coastal-tract (part 2): Applications of aggregated modelling of lower-order coastal change. *Journal of Coastal Research*, 19(4): 828 – 848.

Crossman M., Bradbury A.P. and Allsop, N.W.H., 2002. Innovative rock structures: opportunities and constraints. In: Proceedings 28th International Conference, Coastal Engineering (ASCE), Cardiff, July 2002, pp 1462-1471, ISBN 981 238 238 0, published by World Scientific Publishing, Singapore.

Crossman, M., Segura-Domínguez, S. and Allsop, N.W.H., 2003. Low cost rock structures for beach control and coast protection - Practical design guidance. *DEFRA / Environment Agency R & D Technical Report CSA 6020.*

De Groot, M.B., Bolton, M.D., Foray, P., Meijers, P., Palmer, A.C., Sandven, R., Sawicki, A. and Teh, T.C., 2006a. Physics of liquefaction phenomena around marine structures. *J. Waterway, Port, Coastal and Ocean Engineering,* ASCE, 132(4): 227 – 243.

De Groot, M.B., Kudella, M., Meijers, P. and Oumeraci, H., 2006b. Liquefaction phenomena underneath marine gravity structures subjected to wave loads. *J. Waterway, Port, Coastal and Ocean Engineering,* ASCE, 132(4): 325 – 333.

Dean, R.G., Kriebel, D.L. and Walton, T.L., 2002. Cross-shore secdiment transport processes. Chapter 3 of Part III of the *Coastal Engineering Manual*, EM 1110-2-1100.

DELOS, 2005. Special Issue; Low-crested structures and the Environment. *Coastal Engineering,* Vol. 52, Numbers 10-11, November 2005.

Department for the Environment, Food and Rural Affairs, 2001. *National Appraisal of Assets at Risk from Flooding and Coastal Erosion, including the potential impact of climate change*. Final Report, July 2001.

Department for the Environment, Food and Rural Affairs, 2003a. *Procedural Guidance for the production of Shoreline Management Plans*. Report produced by a consortium led by Halcrow Group. Available from http://www.defra.gov.uk/corporate/consult/smpguidance/index.htm (page last modified 12 August 2004)

Department for the Environment, Food and Rural Affairs, 2003b. *Futurecoast*. Internet: see http://www.defra.gov.uk/environ/fcd/futurecoast.htm (page last modified 16 September 2003).

Department for the Environment, Food and Rural Affairs, 2006. *Flood and Coastal Defence Appraisal Guidance FCDPAG3 Economic Appraisal: Supplementary Note to Operating Authorities – Climate Change Impacts,* October, 2006*.*

De Vriend, H.J., 2003. On the prediction of aggregated-scale coastal evolution. *Journal of Coastal Research,* 19(4) 757 – 759.

De Vriend, H.J., Capobianco, M., Chesher, T., de Swart, H.E., Latteux, B. and Stive, M., 1993. Approaches to long-term modelling of coastal morphology: a review. *Coastal Engineering* 21, 225 – 269.

Dickson, M.E., Walkden, M.J.A., Hall, J.W., Pearson, S.G. and Rees, J., 2005. Numerical modelling of potential climate change impacts on rates of soft cliff recession, northeast Norfolk, UK. In: *Proceedings of Coastal Dynamics 2005*, ASCE, New York.

Dickson, M.E., Walkden, M.J.A. and Hall, J.W., 2007. Systematic impacts of climate change on an eroding coastal region over the twenty-first century. *Climatic Change*, in press.

Dornbusch, U., 2005. Retreat of chalk cliffs and downwearing of shore platforms in the eastern channel in the last century. University of Sussex report for Beaches at Risk project. INTERNET: available from http://www.geog.sussex.ac.uk/BAR/publish.html (page 24/07/06).

Dornbusch, U., Williams, R.B.G., Moses, C.A. and Robinson, D.A., 2007. Inter-tidal shortening along the coast of Southeast England, UK – a critique and re-evaluation. Paper submitted to *Journal of Coastal Research*.

Douglas, B.C. and Crowell, M., 2000. Long-term shoreline position prediction and error propagation. *Journal of Coastal Research,* 16(1) 145 – 152.

ECOPRO, 1996. Environmentally friendly coastal protection: Code of Practice., Government of Ireland, Dublin.

ECUS, 2003. Guidance for coastal defence design in relation to their landscape and visual impacts. CCW contract science report number 531.

Environment Agency, 2002. Risk, performance and uncertainty in flood and coastal defence – A review. *Defra / EA R&D Report No FD2302/TR1*. HR Wallingford Report SR587, January 2002.

Environment Agency, 2003a. *Coastal analysis of the Anglian Region: Lincolnshire (from Grimsby to Friskney)*. EA Report, February 2003.

Environment Agency, 2003b. RASP –Risk assessment for flood and coastal defence for strategic planning. *Defra / EA R&D Report No. W5B-030/TR1*. HR Wallingford Report No. SR 603

Environment Agency, 2006. Condition Assessment Manual. Document Reference 166_03_SD01.

Erlingsson, U., 1991. A sensor for measuring erosion and deposition. *Journal of Sedimentary Petrology* 61(4) 620 – 623.

Esrig, M.I. and Kirby, R.C., 1977. Implications of gas content for predicting the stability of submarine slopes: in Richards, A. F., (ed.), Marine slope stability. *Marine Geotechnology*, 2, pp. 81-100.

European Commission, 2004. *Living with coastal erosion in Europe – Sediment and space for sustainability.* Luxembourg office for official publications of the European Commission. 40 pp ISBN 92-894-7496-3.

Falqués, A., 2003. On the diffusivity in coastline dynamics. *Geophys. Res. Lett*., 30(21), 2119, doi:10.1029/2003GL017760.

Ferreira, O., Ciavola, P., Taborda, R., Bairros, M. and Alveirinho Dias, J., 2000. Sediment mixing depth determination for steep and gentle foreshores. *Journal of Coastal Research*, 16(3) 830 – 839.

Franco, L., Di Risio, M., Riccardi, C., Scaloni, P. and Conti, M., 2004. Monitoraggio del ripascimento protetto con barriera sommersa nella spiaggia di Ostia Centro, *Studi Costierie, No. 8.*

Fredlund, D.G., 1976. Density and compressibility characteristics of air-water mixtures. *Canadian Geotechnical Journal,* 13*,* pp. 386-396.

Gallagher, E.D., Byd, W., Elgar, S., Guza, R.T. and Woodward, B., 1996. Performance of a sonar altimeter in the nearshore. *Marine Geology*, 133. 241 – 248.

Genz, A.S., Fletcher, C.H., Dunn, R.A., Frazer, L.N. and Rooney, J.J., 2007. The predictive accuracy of shoreline change rate methods and alongshore beach variation on Maui, Hawaii. *Journal of Coastal Research*, 23(1): 87 – 105.

Halcrow (2002) *Futurecoast* [CD-ROM]. Produced for Defra by Halcrow, BGS, ABP MER Ltd., Queen's University of Belfast and University of Plymouth.

Hanson., H., Aarninkhof, S., Capobianco, M., Jiménez, J.A., Larsom, M., Nicholls, R.J., Plant, N.G., Southgate, H.N., Steetzel, H.J., Stive, M.J.F. and de Vriend, H.J., 2003. Modelling coastal evolution on yearly to decadal timescales. *Journal of Coastal Research*, 19(4) 790 – 811.

Hanson, K. and Kraus, N.C., 1989. *GENESIS – generalized model for simulating shoreline change, Volume 1: reference manual and users guide*. Tech Rep CERC-89-19, USAE-WES, Coastal Eng Research Centre, Vicksburg, Miss.

Hardacre, G. and Chester, F., 2001. Pevensey Bay Sea Defences PPP, The contractors prespective. Proceedings of the 36th DEFRA Conference of River and Coastal Engineers. Keel University. pp 07.5.1 – 07.5.7.

Hoekstra, P., P. S. Bell, P. van Santen, N. Roode, F. Levoy and Whitehouse, R., 2004. Bedform migration and bedload transport on an intertidal shoal. *Continental Shelf Research*, 24(11): 1249-1269.

Hoffmans, G.J.C.M. and Verheij, H.C., 1997. *Scour manual.* Balkema, Rotterdam. The Netherlands.

Honeycutt, M.G. and Krantz, D.E., 2003. Influence of the geologic framework on spatial variability in longterm shoreline change, Cape Henlopen to Reheboth Beach, Delaware. *Journal of Coastal Research*, Special Issue 38, 147-167.

HR Wallingford, 2004. Performance-based Asset Management System (PAMS) – Phase 1 Scoping Study. *Joint Defra / EA Flood and Coastal Erosion Risk Management R&D Programme R&D Technical Report W5-070/TR* Published September 2004. ISBN: 1-8443-2386-2. Internet: available from http://sciencesearch.defra.gov.uk (page accessed 25/02/2008).

HR Wallingford, 2005a. Performance and Reliability of Flood and Coastal Defences. *Joint Defra / EA Flood and Coastal Erosion Risk Management R&D Programme R&D Technical Report FD2318/TR1.* Internet: available from http://sciencesearch.defra.gov.uk (page accessed 25/02/2008).

HR Wallingford 2005b, Thames Estuary 2100, Flood Defence Asset Visual Condition Indexing Methodology, Technical Note, August 2005.

HR Wallingford, 2005c. Performance and Reliability of Flood and Coastal Defences. *Joint Defra / EA Flood and Coastal Erosion Risk Management R&D Programme R&D Technical Report FD2318/TR2.* Internet: available from http://sciencesearch.defra.gov.uk (page accessed 25/02/2008).

HR Wallingford, Flood Hazard Research Centre, Middlesex University and Risk & Policy Analysis Ltd, 2005. *Flod risk to people, Phase 2*. Defra/EA Joint Flood and Coastal Defence R&D Programme, Technical Report FD2321/TR. Internet: available from http://sciencesearch.defra.gov.uk (page accessed 25/02/2008).

HR Wallingford, 2006a. Beach lowering and recovery at Southbourne (2005). *HR Wallingford Technical Note CBS0726/01,* with University of Southampton.

HR Wallingford, 2006b. Design of physical model scour tests. *HR Wallingford Technical Note CBS0726/02*.

HR Wallingford, 2006c. Assessment of beach lowering and toe scour. *HR Wallingford Technical Note CBS0726/03.*

HR Wallingford, 2006d. Scour monitor deployment at Blackpool. *HR Wallingford Technical Note CBS0726/04*.

HR Wallingford, 2006e. Integrating scour research into reliability analysis of coastal structures. *HR Wallingford Technical Note CBS0726/05*

HR Wallingford, 2006f. Medium scale 2D physical model tests of scour at seawalls. *HR Wallingford Technical Note CBS0726/06*.

HR Wallingford, 2006g. Wave-induced liquefaction of sediment in front of coastal structures. *HR Wallingford Technical Note CBS0726/07* (included as Section 7 of this report)

HR Wallingford, 2006h. Mitigation methods. *HR Wallingford Technical Note CBS0726/08*

HR Wallingford, 2006i. An improved scour predictor for sand beaches. *HR Wallingford Technical Note CBS0726/09.*

IPCC 2007. *Climate Change 2007: The Physical Science Basis – summary for policymakers*. Contribution of Working Group 1 to the Fourth Assessment Report of the Intergovernmental Panel on Climate Change.

Jansen, H., 1997. POP analysis of the JARKUS dataset: the Ijmuiden-Katwijk section. *Delft Univ. Technology Fase 2 Report, Project RKZ-319*, Netherlands.

Jeng, D.-S., 1998. Wave-induced seabed response in a cross-anisotropic seabed in front of a breakwater: An analytical solution. *Ocean Engineering*, 25(1), pp. 49-67.

Larson, M., Capobianco, M. and Hanson, H., 1999a. Relationship between beach profiles and waves at Duck, North Carolina, determined by canonical correlation analysis. *Marine Geology* 275 – 288.

Larson, M., Hanson, H., Kraus, N.C. and Newe, J., 1999b. Short- and long-term responses of beach fills determined by EOF analysis. *Journal of Waterway, Port, Coastal and Ocean Engineering*, 125(6) 285 – 293.

Larson, M., Capobianco, M., Jansen, H., Różyński, G., Southgate, H.N., Stive, M., Wijnberg, K.M. and Hulscher, S., 2003. Analysis and modelling of field data on coastal morphological evolution over yearly and decadal time scales. Part 1: Background and linear techniques. *Journal of Coastal Research*, 19(4) 760 – 775.

Lawler, D.M., 1991. A new technique for the automatic monitoring of erosion and deposition rates. *Water Resources Research* 27(8) 2125 – 2128.

Li, Y., Lark, M. and Reeve, D., 2005. Multi-scale variability of beach profiles at Duck: a wavelet analysis. *Coastal Engineering* 52 (2005) 1133 – 1153.

McBride, M.W., Smallman, J.V. and Allsop, N.W.H., 1995. Vertical walls and low reflection alternatives: Numerical modelling of absorbing wave screens. *HR Wallingford Ltd Report IT 400*, April 1995, Wallingford.

McConnell, K.M., 1998. *Revetment systems against wave attack: a design manual,* Thomas Telford, London, ISBN 0727727060.

McConnell, K.J., Allsop, N.W.H. and Ethelston, D.M., 1996. Wave reflections from coastal structures: Development and application of new approaches. In: *Proceedings of the 10th Congress of the Asia and Pacific Division of IAHR* 26-29 August 1996, Langkawi Island, Malaysia.

Minikin, R.R., 1952. *Coast erosion and protection Studies in causes and remedies.* Chapman and Hall Ltd, London.

Möller, I., 1997. Statistical methods for characterising long-term beach profile morphodynamics: theory and application to beach profile data from Duck, North Carolina. *HR Wallingford Report IT 454*.

Mory, M., Michallet, H., Bonjean, D., Piedra-Cueva, I., Barnoud, J.M., Foray, P., Abadie, S. and Breul, P., 2007. A field study of momentary liquefaction caused by waves around a coastal structure. *Journal of Waterway, Port, Coastal and Ocean Engineering*, ASCE*,* 133(1): 28 – 38.

Mory, M., Michallet, H., Abadie, S., Piedra-Cueva, I., Bonjean, D., Breul, P. and Cassen, M., 2004. Observations of momentary liquefaction caused by breaking waves around a coastal structure. In *Proceedings 29th International Conference Coastal Engineering 2004*. McKee Smith (Ed.), World Scientific, 4204 – 4214.

Motyka J .M. and Brampton A.H. Coastal Management - Mapping of littoral cells. *HR Report SR 328*, January 1993.

Murphy, A.H. and E.S. Epstein, 1989. Skill scores and correlation coefficients in model verification. *Monthly Weather Review*, 117:572–581.

Murray, A.B. and Ashton, A., 2003. Sandy-coastline evolution as an example of pattern formation involving emergent structures and interactions. In: *Proceedings of the International Conference on Coastal Sediments 2003*. CD-

ROM published by World Scientific Publishing Corp. and East Meets West Productions, Corpus Christi, Texas, USA. ISBN 981-238-422-7.

Nairn, R.L. and Southgate, H.N., 1993. Deterministic profile modelling of nearshore processes. Part 2. Sediment transport and beach profile development. *Coastal Engineering*, 19: 57-96.

Ordnance Survey, 1997a. *Consultation Paper 3/1997 – Postional Accuracy of Large Scale Data and Products*. Ordnance Survey, Southampton.

Ordnance Survey, 1997b. *Land-Line User Guide, Reference Section*. Ordnance Survey, Southampton.

Ozasa, H. and Brampton, A.H., 1980. Mathematical modelling of beaches backed by seawalls. *Coastal Engineering*, Volume 4, pp 47 – 63. doi:10.1016/0378-3839(80)90005-8

Pearce, A.M.C., Sutherland, J., Müller, G., Rycroft, D. and Whitehouse, R.J.S., 2006. Scour at a seawall- field measurements and physical modelling. In: *Proceedings, 30th International Conference on Coastal Engineering,* San Diego, USA. J. McKee Smith (Ed), World Scientific, pp 2378 – 2390.

Pearce, A.M.C., Sutherland, J., Obhrai, C., Whitehouse, R.J.S. and Müller, G., 2008. Seawall toe scour: modelling and prediction. Coastal Engineering (in preparation).

Pelnard-Considère, R., 1956. Essai de théorie de l'évolution des formes de ravage en plages de sable et de gâlets. Proceedings *4th Journees de l'Hydraulique*, Question III, rapport 1, 74-1-10, Paris.

Powell, K.A., 1990. *Predicting short term profile response for shingle beaches*. Hydraulics Research Wallingford Report SR219. February 1990.

Powell, K.A. and Lowe, J. P., 1994. The scouring of sediments at the toe of seawalls. In: *Proceedings of the Hornafjordur International Coastal Symposium*, Iceland - June 20-24. Edited by Gisli Viggosson - pp 749 to 755.

Ramsay, D.L. and Brampton, A.H., 1995. Coastal Cells in Scotland. *HR Wallingford Report EX 3176.*

Ranasinghe, R. and Turner, I., 2006. Shoreline response to submerged structures: A review. *Coastal Engineering*, Vol. 53, No 1, pp65-80.

Reeve, D., 2006. Explicit expression for beach response to non-stationary forcing near a groyne. *Journal of Waterway, Port, Coastal and Ocean Engineering*, ASCE, 132(2), 125 – 132.

Ridd, P., 1992. A sediment level sensor for erosion and siltation detection. *Estuarine, Coastal and Shelf Science* 35, 353 – 362.

Risk and Policy Analysts Ltd (RPA), 2004. Evaluating and multi-criteria analysis (MCA) methodology for application to flood management and coastal defence appraisals. Joint Defra/EA Flood and Coastal Erosion Risk Management R&D programme, R&D Technical Report FD2013/TR. Internet: available from http://sciencesearch.defra.gov.uk (page accessed 25/02/2008).

Risk and Policy Analysts Ltd (RPA) and Royal Haskoning, 2006. *Developing and evidence base for improving appraisal guidance.* Joint Defra/EA Flood and Coastal Erosion Risk Management R&D Programme, Technical Report FD2019/TR. Internet: available from http://sciencesearch.defra.gov.uk (page accessed 25/02/2008).

Robinson, C., Baldock, T.E., Horn, D.P., Gibbes, B., Hughes, M.G., Nielsen, P. and Li, L., 2005. Measurement of groundwater and swash interaction on a sandy beach. Proceedings of *Coastal Dynamics '05*, ASCE, Barcelona.

Rosati., J.D., 2005. Concepts in sediment budgets. *Journal of Coastal Research*, 21(2) 307 – 322.

Różyński, G., 2005. Long-term shoreline response of a nontidal, barred coast. *Coastal Engineering* 52, 79 – 91.

Różyński, G. and Jansen, H., 2002. Modeling Nearshore Bed Topography with Principal Oscillation Patterns*. Journal of Waterway, Port, Coastal and Ocean Engineering,* ASCE, 128(5): 202 - 215

Ryan, A., 1999. *Has anything really happened to the beach? Assessing the positional accuracy of OS mapping used to determine the geomorphological changes at Porlock Bay in Somerset*. Unpublished MSc thesis, Department of Geography, University of Portsmouth.

Sandven, R., Husby, E., Husby, J.E., Jønland,J., Roksvåg, K.O., Stæhli, F. and Tellugen, R., 2007. Development of a sampler for the measurement of gas content in soils. *Journal of Waterway, Port, Coastal and Ocean Engineering*, ASCE, 133(1): 3 – 13.

Sayers, P.B., Goulby, B. and Johnson, D., 1999. Real-time hazard forecasting: A review of implementation and two years of operation at Samphire Hoe, Dover. Proc. Ministry of Agriculture Fisheries and Food Conference for Coastal and River Engineers, Keele, UK, 1999

Shennan, I. and Horton, B., 2002. Holocene land- and sea-level changes in Great Britain. *Journal of Quaternary Science* 17: 511-526

Silvester, R. and Hsu, J.R.C., 1997. *Coastal Stabilisation*. Advanced Series on Ocean Engineering – Volume 14, World Scientific. .

Southgate, H.N. and Beltran, L.M., 1996. Self-organised processes in beach morphology. In: *Proceedings of the 8th International Conference Physics of Estuarine and Coastal Seas*, The Hague, NL. 409 – 416.

Southgate, H.N. and Brampton, A.H., 2001. Coastal Morphology Modelling: a guide to model selection and usage. *HR Wallingford Report SR 570,* June 2001.

Southgate, H.N. and Möller, I., 2000. Fractal properties of coastal profile evolution at Duck, North Carolina. *Journal of Geophysical Research,* 105(C5): 11489-11507.

Southgate, H.N. and Nairn, R.L., 1993. Deterministic profile modelling of nearshore processes. Part 1. Waves and currents. *Coastal Engineering*, 19: 27-56.

Southgate, H.N., Wijnberg, K.M., Larson, M., Capobianco, M. and Jansen, H., 2003. Analysis and modelling of field data on coastal morphological evolution over yearly and decadal time scales. Part 2: Non-linear techniques. *Journal of Coastal Research*, 19(4) 776 – 789.

Stive, M.J.F., 2004. How important is global warming for coastal erosion. *Climatic Change,* 64: 27 – 39.

Stripling, A. and Panzeri, M., 2007. Submitted to *Proceedings ICE, Maritime Journal.*

Sumer, B.M., and Fredsoe, J., 2002. *The Mechanics of Scour in the Marine Environment*. World Scientific, Singapore, 536 p.

Sutherland, J., Brampton, A., Motyka, G., Blanco, B. and Whitehouse, R., 2003. Beach lowering in front of coastal structures: research scoping study. Defra/EA R&D Report FD1916/TR1. INTERNET: available from http://sciencesearch.defra.gov.uk (page accessed 25/02/2008).

Sutherland, J, Brampton, A and Whitehouse, R, 2006a. Toe Scour at Seawalls: Monitoring Prediction and Mitigation. In: *Proceedings of the 41st Defra Flood and Coastal Management Conference,* pp. 03b.1.1 – 03b.1.12.

Sutherland, J. and Gouldby, B., 2003. The vulnerability of coastal defences to climate change. Proc. ICE, *Water and Maritime Engineering*, 156(WM2): 137-145.

Sutherland, J., Peet, A.H. and Soulsby, R.L., 2004. Evaluating the performance of morphological models. *Coastal Engineering* 51, pp. 917-939.

Sutherland, J, Obhrai, C, Whitehouse, RJS and AMC Pearce, 2006b. Laboratory tests of scour at a seawall. In: *Proceedings of the Third International Conference on Scour and Erosion,* Amsterdam, 2006. CURNET, Gouda, The Netherlands [CD-ROM] ISBN 90-376-0503-6.

Taylor, J.A., Murdock, A.P. and Pontee, N.I., 2004. A macroscale analysis of coastal steepening around the coast of England and Wales. *The Geographical Journal*, 170(3) 179 – 188.

Thomas, R.S. and Hall, B., 1986. *Seawall design*. CIRIA/ Butterworths, London.

U.S. Army Corps of Engineers. 2002. *Coastal Engineering Manual*. Engineer Manual 1110-2-1100, U.S. Army Corps of Engineers, Washington, D.C. (in 6 volumes).

Van Koningsveldt, M. and Mulder, J.P.M., 2004. Sustainable coastal policy developments in The Netherlands. A systematic approach revealed. *Journal of Coastal Research,* 20(2) 375 – 385.

Van Koningsveldt, M. and Lescinski, J., 2007. Decadal scale performance of coastal maintenance in the Netherlands. *Shore and Beach* 75(1), 20 – 35.

Van Rijn, L.C., Walstra, D.J.R., Grasmeijer, B., Sutherland, J., Pan, S. and Sierra, J.P., 2003. The predictability of cross-shore bed evolution of sandy beaches at the time scale of storms and seasons using process-based Profile models. *Coastal Engineering* 47: 295 – 327.

Walkden, M.J.A. and Hall, J.W., 2005. A predictive mesoscale model of the erosion and profile development of soft rock shores. *Coastal Engineering* 52, 535-563.

Whitehouse, R., 1998. *Scour at Marine Structures: A Manual for Practical Applications*. Thomas Telford Publications, London, 198 p.

Whitehouse, R.J.S., Hearn, S., Waters, C.B. and J. Sutherland, 2000. Data report on measurements by HR Wallingford at Teignmouth UK (1998–1999). *HR Wallingford report TR105*, November 2000, Wallingford.

Whitehouse, R.J.S., Spearman, J.R., Townend, I.H., Pethick, J.R. and Cooper, N., 2006. Review and formalisation of geomorphological concepts and approaches for estuaries. Report prepared for Defra and Environment Agency Joint Modelling and Risk There. Report FD2116/TR2. [Available from http://www.hrwallingford.co.uk/projects/FD2116/index.html].

Wijnberg, K.M. and Terwindt, J.H.J., 1995. Extracting decadal morphological behaviour from high-resolution, long-term bathymetric surveys along the Holland coast using eigenfunction analysis. *Marine Geology,* 126: 301 – 330.

Winant, C.D., Inman, D.L. and Nordstrom, C.E., 1975. Description of seasonal beach changes using empirical eigenfunctions. *Journal of Geophysical Research,* 80(15): 1979 – 1986.

Wijnberg, K.M., Aarnikof, S.G.J., van Koningsveld, M.V., Ruessink, B.G. and Stive, M.J.F., 2004. Video monitoring in support of coastal management. . In:

Proceedings of the 29th International Conference Coastal Engineering 2004. McKee Smith (Editor) World Scientific Press, 3136 – 3148.

Woodruff, P.E. and Dean, R.G., 2000. Innovative erosion control technology in Florida. In: *Proceedings 27th ASCE International Conference on Coastal Engineering,* Sydney, Australia, pp3583 – 3859.

Zhang, K., Huang, W., Douglas, B.C. and Leatherman, S., 2002. Shoreline position variability and long-term trend analysis. *Shore and Beach* 70(2) 31 – 35.

Zuzek, P.J., Nairn, R.B. and Thieme, S.J., 2003. Spatial and temporal considerations for calculating shoreline change rates in the Great Lakes Basin. *Journal of Coastal Research* Special Issue 38, 125 – 146.

11. Bibliography

Aird, N.P., Davidson, M.A. and Marino-Tapia, I.J., 2004. Physical processes associated with onshore sand bank migration adjacent an estuary mouth. In: *Proceedings of the 29th International Conference Coastal Engineering 2004*. McKee Smith (Editor) World Scientific Press, 2519 – 2530.

Ashton, A., List, J.H., Murray, A.B. and Farris, A.S., 2003. Links between erosional hotspots and alongshore sediment transport. *Proceedings of the International Conference on Coastal Sediments 2003.* CD-ROM published by World Scientific Publishing Corp. and East Meets West Productions, Corpus Christi, Texas, USA. ISBN 981-238-422-7.

Aubrey, D.G., 1979. Seasonal patterns of onshore/offshore sediment movement. *Journal of Geophysical Research* 84(C10) 6347 – 6345.

Balsillie, J.H., 1986. Beach and coast erosion due to extreme event impact. *Shore and Beach* 54, 22 – 37.

Balsillie, J.H., 1999. *Volumetric beach and coast erosion due to storm and hurricane impact*. Florida Geological Survey Open File Report No 78, Tallahassee, Florida, 37p.

Boruff, B.J., Emrich, C. and Cutter, S.L., 2005. Erosion hazard vulnerability of US Coastal Counties. *Journal of Coastal Research*, 21(5) 932-942.

Bradbury, A., Mason, T. and Barter, P., 2005. Large-scale multi-agancy strategic beach monitoring of the South-East coast of England – provision of data and analytical tools. In: *Proceedings of the Florida Shore and Beach Preservation Association National Conference on Beach Preservation Technology*, Hilton San Destin, Florida, USA. INTERNET: available from www.fsbpa.com

Brady, A.J. and Sutherland, J., 2001. COSMOS modelling of COAST3D Egmond main experiment. *HR Wallingford Report TR115*, Wallingford.

Burchart, H.F, Hughes, S.A. (2003) Fundamentals of Design. Manual Part VI, Design of Coastal Project Elements Chapter 5 in *Coastal Engineering Manual*, 1110-2-1100, US Army Corps of Engineering, Washington, DC.

Cowell, P.J., Thom, B.G., Jones, R.A., Everts, C.H. and Simanovic, D., 2006. Management of uncertainty in predicting climate-change impacts on beaches. *Journal of Coastal Research*, 22(1) 232-245.

Dean, R.G., 1991. Equilibrium beach profiles: characteristics and applications. *Journal of Coastal Research*, 7(1) 53-84.

Dong, P. and Chen, H., 1999. A probability method for predicting time-dependent long-term shoreline erosion. *Coastal Engineering* 36, 243 – 261.

Edina, 2006. Digimap: Historic Map Collection. Internet: at http://edina.ac.uk/digimap/description/historic.html. Page accessed 16/05/2006.

Edwards, C., 2001. Date of survey and accuracy of cliff coastline in OS digital data. Letter from C. Edwards to Dr Uwe Dornbusch, 07 March 2001, quoted from BERM final Report: Technical Report (Dornbusch, 2002).

Environment Agency, 2003. *Coastal analysis of the Anglian Region: Lincolnshire (from Grimsby to Friskney)*. EA Report, February 2003.

Evans, E., Ashley, R., Hall, J., Penning-Rowsell, E., Saul, A., Sayers, P., Thorne, C. and Watkinson, A. (2004) Foresight. Future Flooding. Scientific Summary: Volume I, Future risks and their drivers. Office of Science and Technology, London. Available from http://www.foresight.gov.uk/Previous_Projects/Flood_and_Coastal_Defence/Reports_and_Publications/Volume1/Chapter6.pdf

Galgano, F.A., Douglas, B.C. and Leatherman, S.P., 1998. Considerations for shoreline position prediction. *Journal of Coastal Research*, 14, 1025-1033.

Halcrow, 2004. Poole Bay and Harbour Strategy Study, Technical annex 3, historic shoreline evolution. Internet: available from http://www.bournemouth.gov.uk/residents/Environment/Flood_and_Coast_Defence_Options.asp. Page accessed 18/05/2006.

Hammar-Klose, and Thieler, E.R., 2001. *Coastal Vulnerability to Sea-Level Rise: A Preliminary Database for the U.S. Atlantic, Pacific, and Gulf of Mexico Coasts.* U.S. Geological Survey, Digital Data Series DDS-68, 1 CD-ROM

Holman, R.A., Sallenger, Jr. A.H., Lippman, T.C. and Haines, J.W., 1993. The application of video image processing to the study of nearshore processes. *Oceanography* V6(3).

Johnson, D.A., 1905. *Instructions to field examiners*. Ordnance Survey publication.

Kingston, K.S. and Davidson, M.A., 1999. Arttificaial neaural network model of sand bar location for a macro-tidal beach, Perranporth, UK. In: *Proceedings IAHR symposium on River, Coastal and Estuarine Morphodynamics* (Genoa, Italy) 227 – 236.

Kraus, N.C., Isobe, M., Igarashi, H., Sasaki, T.O. and Horikawa, K., 1982. Field experiment on longshore sand transport in the surf zone. In: *Proceedings of the 18th International Conference on Coastal Engineering*. ASCE, 970 – 988.

Kriebel, D.L. and Dalrymple, R.A., 1985. *A northeaster risk index*. R&D Coastal Engineering, Newark, Delaware, 33p.

Koukoulas S., Nicholls R.J., Dickson M.E., Walkden M.J., Hall J.W., Pearson S.G., Mokrech, M. and Richards, J. 2005. A GIS tool for analysis and interpretation of coastal erosion model outputs (SCAPEGIS). In: Proceedings of Coastal Dynamics 2005, ASCE, New York, *in press*.

Leatherman, S.P., 2003. Shoreline change mapping and management along the U.S. East Coast. *J. Coastal Research*, Special Issue 38, pp 5-13.

Lee, J.H., Takewaka,S., Sakai, T. and Takano, S., 2004. Use of X-band radat for wave and beach morphology analysis. In: *Proceedings of the 29th International Conference Coastal Engineering 2004*. McKee Smith (Ed.) World Scientific, 2681 – 2693.

May, S. K., Kimball, W. H., Grady, N., and Dolan, R., 1982, CEIS: The coastal erosion information system. *Shore and Beach*, vol. 50, p. 19-26.

Moore, L.J., 2000. Shoreline Mapping Techniques. *Journal of Coastal Research*, 16(1) 111-124.

National Research Council, 1990. *Managing coastal erosion*. National Academies Press, ISBN 0-309-04143-0, pp 204.

Niwa Scientific, 2006. *Cam-era: video-monitoring New Zealand's environment*. INTERNET: www.niwascience.co.nz/services/cam-era/tech, page accessed 30/08/2006.

Oliver, R., 1996. Taking to the water: some examples of Ordnance Survey mapping of the coast. *Sheetlines,* No 45: 9-27, Charles Close Society.

Oliver, R., 2005. *Ordnance Survey maps: a concise guide for historians*. Second Edition, The Charles Close Society, London. ISBN 1 870598 24 5.

Ordnance Survey, 1882. *OS 307 Instructions to Surveyors*. Ordnance Survey, Southampton.

Ordnance Survey, 2003. ISB Project 195 – ICZMap Project Manager's project completion report. Pp44.

Ordnance Survey, 2006a. Historical Map Data. Internet: at http://www.ordnancesurvey.co.uk/oswebsite/products/historicalmapdata/. Page accessed 16/05/2006.

Ordnance Survey, 2006b. Positional Accuracy Improvement. Internet at http://www.ordnancesurvey.co.uk/oswebsite/pai/. Page accessed 16/05/2006.

Ordnance Survey, 2006c. MasterMap User Guide. Version 6.1.1, April, 2006. Internet: available from http://www.ordnancesurvey.co.uk/ Page accesses 16/05/2006.

Parker, 2003. The difficulties in measuring a consistently defined shoreline – the problem of vertical referencing. *Journal of Coastal Research*, Special Issue 38, 44-56.

Pilkey, O. H., and Davis, T. W., 1987. An analysis of coastal recession models: North Carolina coast. In: D. Nummedal, O.H. Pilkey and J.D. Howard (Editors), *Sea-level Fluctuation and Coastal Evolution*. SEPM (Society for Sedimentary Geology) Special Publication No. 41, Tulsa, Oklahoma, p. 59-68.

Pope, Pope, Reed, West, Lillycrop, 1997. Use of an airborne laser depth sounding system in a complex shallow water environment. In: *Proceedings Hydrographic Symposium XVth International Hydro Conference.*

Reeve, D.E., 2004. Acounting for variability in coastal prediction. In: *Proceedings of the ICE, Maritime Engineering*. 157 (MA 1) 35 – 47.

Reeve, D.E. and Fleming, C.A., 1997. A statistical-dynamical method for predicting long-term coastal evolution. *Coastal Engineering* 30, 259 – 280.

Ruggiero, P., Kaminsky, G.M. and Gelfenbaum, G., 2003. Linking proxy-based and datum-based shorelines on a High-Energy coastline: Implications for shoreline analyses. *Journal of Coastal Research*, Special Issue 38, 57 – 82.

Sims P., Weaver, R.E. and Redfern, H.M., 1995. Assessing coastline change: a GIS model for Dawlish Warren, Devon UK. In: *CoastGIS '95: International Symposium on GIS and Computer Mapping for Coastal Zone Management*. R. Furness (ed). Cork, Ireland, February 1995.

Spivack, M. and Reeve, D., 2002. A stochastic model for shoreline evolution. In: *Proceedings 26th International Conference on Coastal Engineering,* , ASCE. 3433 – 3437.

Steetzel, H.J., 1991. A model for profile changes during storm surges. In Proceedings of *Coastal Sediments '91*. pp 618 – 630.

Steetzel, H.J., 1993. Cross-shore transport during storm surges. PhD dissertation, Delft University of Technology. Pp 242.

Stive, M.J.F., Aarninkhof, S.G.J., Hamm, L., Hanson, H., Larson, M., Wijnberg K.M., Nicholls, R.J. and Capobianco, M., 2002. Variability of shore and shoreline evolution. *Coastal Engineering* 47 (2002) 211-235.

Tetzlaff, D.M., 2005. Modelling coastal sedimentation through geologic time. *Journal of Coastal Research* 21(3) 610 – 617.

Thieler, E.R., and Hammar-Klose, E.S., 1999. *National Assessment of Coastal Vulnerability to Future Sea-Level Rise*: *Preliminary Results for the U.S. Atlantic Coast.* U.S. Geological Survey, Open-File Report 99-593, 1 sheet. Available online at: http://pubs.usgs.gov/of/of99-593/

Thieler, E.R., and Hammar-Klose, E.S., 2000a. *National Assessment of Coastal Vulnerability to Future Sea-Level Rise: Preliminary Results for the U.S. Pacific Coast.* U.S. Geological Survey, Open-File Report 00-178, 1 sheet. Available online at: http://pubs.usgs.gov/of/of00-178/

Thieler, E.R., and Hammar-Klose, E.S., 2000b. *National Assessment of Coastal Vulnerability to Future Sea-Level Rise: Preliminary Results for the U.S. Gulf of Mexico Coast*. U.S. Geological Survey, Open-File Report 00-179, 1 sheet. Available online at: http://pubs.usgs.gov/of/of00-179/

Thomas, S. and Ridd, P.V., 2004. Review of methods to measure short time scale sediment accumulation. *Marine Geology*, 207, 95-114.

Townend, I., and K. Burgess, 2004: Methodology for assessing the impact of climate change upon coastal defence structures. *Proceedings of the 29th International Conference on Coastal Engineering 2004,* 19-24 September; Lisbon, Portugal, World Scientific Publishing, 3593-3965.

US Army Corps of Engineers, 2004. *Regional Morphology Analysis Package (RMAP): Part 1. Overview.* US Army Corps of Engineers Engineer Research and Development Centre Regional Sediment Management Demonstration Program Technical Note ERDC/RSM-TN-16. Internet: available from http://www.wes.army.mil/rsm/ accessed on 25/10/05.

Wiegel, R.L. (2002a) Seawalls, seacliffs, beachrock: what beach effects? Part 1. *Shore and Beach* 70(1) 17-27.

Wiegel, R.L. (2002b) Seawalls, seacliffs, beachrock: what beach effects? Part 3. *Shore and Beach* 70(3) 2-14.

Wiegel, R.L. (2002c) Seawalls, seacliffs, beachrock: what beach effects? Part 2. *Shore and Beach* 70(2) 13-22.

Winterbotham, H.S.L. (1934). *Professional Papers New Series No. 16 – The National Plan (The ten foot, five foot, twenty five inch and six inch scales)*. HMSO, London.

Wozencraft, J.M., 2003. Sensor fusion for advanced airborne coastal mapping. In: *Proceedings of the International Conference on Coastal Sediments 2003.* CD-ROM published by World Scientific Publishing Corp. and East Meets West Productions, Corpus Christi, Texas, USA. ISBN 981-238-422-7.

Zhang, K., Douglas, B.C. and Leatherman, S.P., 2001. Beach erosion potential for severe Nor'easters. *Journal of Coastal Research*, 17(2) 309 – 321.

Zhang, K., Huang, W., Douglas, B.C. and Leatherman, S., 2002. Shoreline position variability and long-term trend analysis. *Shore and Beach* 70(2) 31 – 35.

Ergon House
Horseferry Road
London SW1P 2AL

www.defra.gov.uk

Department for Environment
Food and Rural Affairs