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Toe structures management manual

Project: SC070056/R

Flood and Coastal Erosion Risk Management Research and Development Programme

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Miranda Kavanagh

Director of Evidence

Executive summary

This manual is intended to provide those responsible for flood defences, coastal erosion protection structures and other coastal structures with practical guidance on how to determine, monitor, assess and mitigate for the effects of toe scour. It describes the different types of toe protection structures and provides illustrations of typical designs that are often used as solutions. The case studies on beach lowering and scour management provide real examples of both good and bad practice, and discuss lessons learnt from past schemes.

Scour, in specific relation to coastal engineering projects, can be defined as 'the removal, by hydrodynamic forces, of erodible bed material in the vicinity of coastal structures'. This definition distinguishes scour from the more general erosion and notes that the presence of a reflecting structure contributes to the cause of scour. Scour that affects coastal structures can lead to partial damage, or in extreme cases, complete failure of the structure.

A comprehensive survey published by CIRIA in 1986 concluded that scour at the toe of structures represented the most prevalent and serious form of damage to seawalls in the UK. Toe scour is a serious and costly problem – moreover, it is one that is not limited to any particular environment or generally to any particular type of seawall.

Toe protection provides insurance against scouring and the undermining of a structure. It provides additional armouring of the beach or base of a defence in front of the structure which prevents waves and currents from scouring and undercutting it.

This manual presents, in a logical way, how relevant issues should be covered in the assessment, management and design of the toe of coastal defence structures. These issues include:

- monitoring of beach levels and structure condition (including links to coastal monitoring programmes) – Chapter 3;
- trigger points for action during the life of the asset (including links to 'performance features') – Section 3.4;
- maintenance and replacement of elements of the asset or of the whole structure – Chapter 4;
- environmental and sustainability issues surrounding scheme/structure planning, design and operation – Chapters 2, 3, 4 and 5;
- inputs to the assessment and design processes (including relationship to defence fragility, both for asset management and new design) – Chapter 5;
- selection of mitigation options to enhance or prolong performance of the asset – Chapter 2 and Section 5.3.

The manual is supported by appendices containing information on scour processes, methods of predicting scour and a number of case studies from around the UK to illustrate particular management approaches and draw on the experience of the techniques that have been employed.

Acknowledgements

This manual has been developed from the outputs of the Environment Agency's Toe Structures for Coastal Defences project (SC070056) undertaken by HR Wallingford, which provided much of the research base for the content of this manual.

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The project team is grateful to the members of the steering group and those who contributed time, information and images to the development of the manual.

Contents

1 Introduction

Chapter 1 introduces the manual. It gives the background to the document, outlines its objectives, its scope, intended readership and use.

Key links to other chapters
- o Chapter 2 – Toe structure types and materials
- o Chapter 3 – Asset management
- o Chapter 4 – Maintenance
- o Chapter 5 – Toe design

Who will be interested in this chapter?
- o All users of the manual

1.1 Background and context

1.1.1 A brief history

Coastal engineering dates back to the ancient world bordering the Mediterranean Sea, Red Sea and Persian Gulf where the ancient civilisations began developing harbours to aid maritime traffic, around 3,500 BC (USACE 2012).

The ancient Egyptians and Minoans developed many sophisticated port structures in the pre-Roman era, including rubble-mound breakwaters on the River Nile at Djoser (*c.*2,500 BC) and Alexandria, Egypt (*c.*1,800 BC). However, these ancient ports had a common problem that needed to be addressed; they had to be kept clear of silt in a time when dredging was unknown. This was often achieved through flushing by means of channels, tanks, sluices or diverting rivers through canals such that silt would be driven away from the harbour.

The Romans continued the evolution of coastal structures, introducing many innovations in coastal engineering including the construction of walls underwater and solid breakwaters. In the Mediterranean, they replaced many of the ancient rubble-mound breakwaters with vertical concrete walls. These colossal coastal structures could be built rapidly and required little maintenance. In some cases, wave reflection was actively used to prevent silting. In most cases, rubble or large stone slabs were placed in front of the walls to protect against undermining of the structures. The Romans were apparently the first to recognise the problem of toe scour.

In the UK, coastal defences have been constructed since at least Roman times, with flood embankments protecting areas on the Medway and the Severn Levels. The sole objective of coastal defence up until the 19th century was to protection low-lying agricultural land and the reclamation of fertile inter-tidal areas. However, by this time, the popularity of leisure pursuits at the seafront, such as bathing and promenade walking, had intensified and this led to a concerted effort to capture land and protect the coast from erosion by the construction of seawalls. This period also saw defences, predominantly vertical seawalls, built to protect critical infrastructure such as roads and railways, for example Brunel's South Devon Railway along the coastline between Dawlish and Teignmouth, while seawalls and particularly breakwaters were built for military use and to provide fishing harbours.

In the early 20th century, defences were also built to stimulate and regenerate local economies, protecting reclaimed areas. After the Second World War, many seawalls were constructed to enable the public to have better access to the coast and to protect valuable agricultural areas during a time of food rationing. The North Sea surge of 1953, which flooded many low-lying coastal settlements and agricultural land in eastern England, led to a major upgrade of flood defences around the coast.

Renewal, upgrade and construction of new coastal defences has continued to the present day, albeit with a far more enhanced system of economic and benefits justification than in the past.

Around the coastline and tidal estuaries of England and Wales there are now an estimated 2,935 km of coastal defences, which amounts to 36 per cent of the total length of the coast (Halcrow 2011).

These defences are subject to many types of failure mechanisms with crest failure, settlement and outflanking among many other forms of failure. In 1986, CIRIA undertook a comprehensive survey of coastal authorities in England, Scotland and Wales with regard to the performance of seawalls (CIRIA 1986). This included an examination of the main types of damage experienced by coastal defences. The survey concluded that erosion at the toe of structures represented, by far, the most prevalent and serious form of damage to seawalls in the UK; over 12 per cent of all seawalls reported erosion at the toe, which represented over a third of all damage reported.

Just like the Romans millennia before, toe scour, at that challenging area where the land meets the erosive force of the sea, is still recognised as a major problem for coastal defence structures.

1.1.2 What is toe scour?

Scour, in a hydrodynamic context, can be defined as 'the erosive force of moving water'. This broad definition includes erosion of sediment under any circumstances, such as beach profile change and inlet channel migration.

Scour, in specific relation to coastal engineering projects can be defined as 'the removal, by hydrodynamic forces, of erodible bed material in the vicinity of coastal structures'. This definition distinguishes scour from the more general erosion, while the presence of a reflecting structure contributes to the cause of scour.

Scour that affects coastal structures can lead to partial damage, or in extreme cases, complete failure of the structure.

The toe of a coastal defence structure is shown in Figure 1.1.

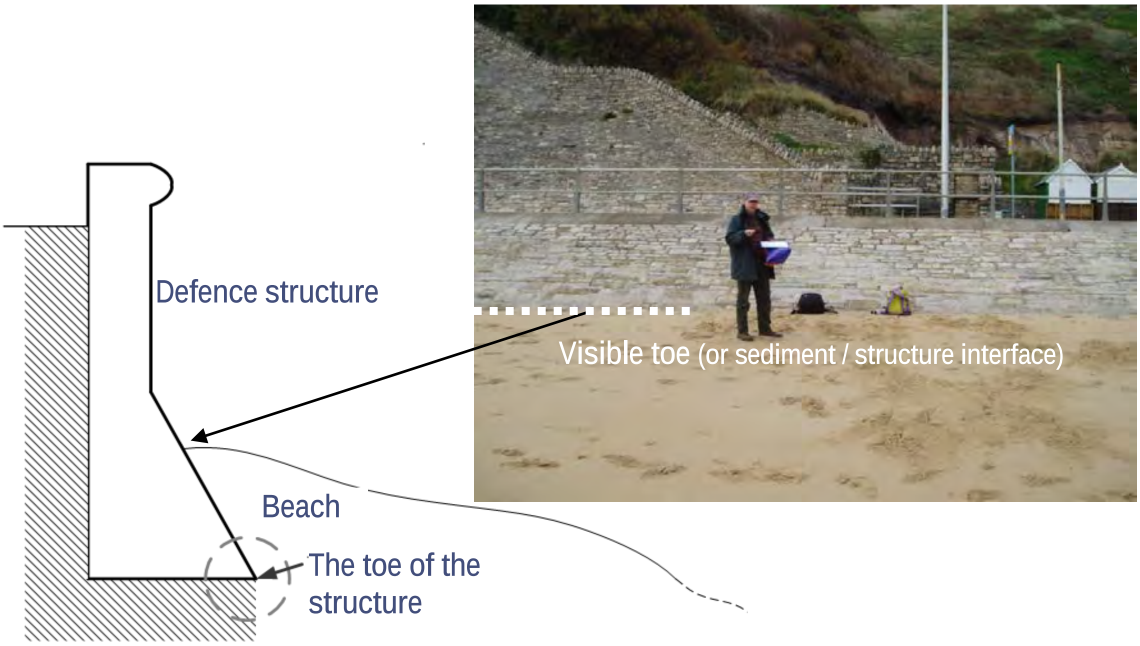


Figure 1.1 Definition of the structural toe of a defence as opposed to the 'visible' toe

Toe scour is particularly prevalent at vertical seawalls where waves, enhanced by reflections, combine with localised currents to mobilise and move sediments. This can undermine the structure leading to tilting, slumping and other failure mechanisms. Scour-induced damage also occurs at sloping-front structures when scour undermines the toe so it can no longer support the armour layer, which then slides downslope. Scour at vertical sheet pile walls can result in seaward rotation of the sheet pile toe due to pressure of the retained soil.

Structural damage or failure brought about by scour affects coastal projects in a number of ways including:

- decreased functionality;
- increased costs to repair or replace the structure (scour-related damage is often difficult and expensive to repair);
- assets protected by the structure may become at risk of flooding or erosion;
- clients and other stakeholders will lose confidence in the project's capability to perform as required.

Toe scour is a common problem that needs to be understood and considered fully in the design and maintenance of coastal structures.

1.2 Concept of toe protection

1.2.1 The need to protect the toe

Toe protection provides insurance against scouring and undermining of a structure. It provides additional armouring of the beach or base of a defence in front of the structure, which prevents waves and currents from scouring and undercutting it. Factors that affect the severity of toe scour include:

- wave breaking (when near the toe);
- wave run-up and backwash;
- wave reflection;
- the grain-size distribution of the beach or bottom materials.

The toe of a defence often requires special consideration because it is subjected to both hydraulic forces and changing beach profiles fronting the structure. Seasonal variations in beach profile will be a contributing factor in determining the type and extent of toe protection.

Toe protection is also often needed along structures that cause concentration of currents, such as training walls and breakwaters extending from the shoreline. Furthermore, highly reflective structures such as impermeable vertical walls are much more susceptible to near-structure scour than sloping rubble-mound structures.

Toe stability is essential because failure of the toe will often lead to failure of the entire structure.

1.2.2 Management criteria

Management of the toe of coastal structures should meet various criteria (for example, policy, sustainability and cost) including the need to:

- provide and maintain a level of performance;
- monitor changes in condition and performance and to gather data on these changes;
- consider the impact of extreme forcing conditions (storms, surges and so on);
- consider changes in associated shoreline or beach management practices.

The practitioner also has to work within various constraints be they political (policy), financial, environmental or simply practical.

This manual presents, in a logical way, how relevant issues should be covered in the assessment, management and design of the toe of coastal defence structures. These issues include:

- monitoring of beach levels and structure condition (including links to coastal monitoring programmes) – Chapter 3;
- trigger points for action during the life of the asset (including links to ‘performance features’) – Section 3.4;
- maintenance and replacement of elements of the asset or of the whole structure – Chapter 4;
- inputs to the assessment and design processes (including relationship to defence fragility for asset management and new design) – Chapter 5;

- selection of mitigation options to enhance or prolong performance of the asset – Chapter 2 and Section 5.3.

Environmental and sustainability issues are considered in detail throughout the manual where these relate directly to toe structures, their construction materials and related remedial interventions. Generally, as the toe is only one element of a larger structure, such issues would normally be considered at scheme scale.

1.2.3 Overview of toe structure types

The area where the land meets the sea is complex, often changing its geomorphology in relatively short timescales. This can lead to confusion over the identification of the toe – and also its type if not previously known. For the purpose of this manual, the toe of a coastal defence is defined as the physical toe of the defence structure as illustrated in Figure 1.1. This should not be confused with that which might be described as the visible toe – the interface between the beach and the structure. The structural toe of defences can of course become visible due to the removal of sediments during storms, or over long periods of drawdown, even if only temporarily.

Furthermore, the toe of structures that are located permanently below the level of lowest astronomical tide (LAT) will not be easily observed. For the most part, this manual refers to structures with emergent toe structures, that is, those that are located above LAT levels. This is partly because, historically, LAT was about the lowest level that could practically be reached for most land based methods of construction.

Because of the role that sediment and shore platforms play in the performance of toe structures, this manual also discusses the management of beaches and foreshores to a limited degree including sediment control structures such as groynes and interventions such as replenishment.

The main focus, however, is on toe structures. These structures are described further in Chapter 2, where they are considered as:

- vertical, including sheet piling, cribwork, blockwork, solid infill and toe beams, in materials such as concrete, masonry, steel and timber alone or in combination as appropriate;
- sloping, including proprietary flexible revetment systems, irregularly placed rock slopes, gabion mattresses and grouted stone (which are flexible to a degree), concrete and stone stepwork, and placed blockwork which is more rigid.

Toe structures may form an integral part of the original defence structure, being a specific element included at the design stage, or they may be supplemental (that is, added to the original structure after its construction).

1.3 Structure of the manual

Existing guidance for coastal managers includes:

- *Coastal Engineering Manual* (USACE 2012);
- *The Rock Manual* (CIRIA et al. 2007);

- *The European Overtopping Manual* (Pullen et al. 2007);
- *Beach Management Manual* (CIRIA 2010a);
- *The Use of Concrete in Maritime Engineering: A Good Practice Guide* (CIRIA 2010b);
- *Condition Assessment Manual* (Environment Agency 2006).

This manual aims to complement these documents rather than supersede them. To reduce duplication, where these other documents contain more detailed information pertinent to the design, assessment and management of toe structures, the reader is referred to these where appropriate.

1.3.1 Route map

The manual provides information on the key aspects of the management of toe structures as depicted in Figure 1.2, which summarises the contents of the manual's chapters. Throughout the manual, reference is made to case studies from around the UK. These are detailed in Appendix C.

Chapter 1 – Introduction
A general introduction to the manual

Background and context
Concept of toe protection
Structure of manual
The asset management cycle
Target audience

Chapter 2 – Toe structure types and materials
Description of the types of materials and structures used to mitigate against the effects of scour

Toe structure types
Materials
Choice of structure type and materials

Chapter 3 – Asset management
Guidance on post-scheme asset management activities such as asset performance and monitoring

Life-cycle failure modes
Planning programmes
Defining trigger/alert beach levels
Beach monitoring
Structure condition monitoring

Chapter 4 – Maintenance
Options for maintenance of different types of toe protection structures and materials and trigger levels for action

Issues associated with maintenance
Maintenance of different types of toe structure

Chapter 5 – Toe design
Illustration and description of typical designs for toe protection structures and particular considerations in the design process

Identification of problem
Design principles
Failure mechanisms
Public safety and the environment
Construction issues

Appendices
Covering physical processes; methods for predicting toe scour and detailed case studies

Scour processes
Predictive methods
Case studies

Figure 1.2 Layout of the guide

1.4 The asset management cycle

This manual provides information and guidance on the toe protection of coastal defences for practitioners, engineers, designers and managers whose responsibilities may span the life cycle of these structures. Figure 1.3 illustrates how key decision-making and management actions feature in the asset management cycle and indicates where the reader can find relevant information on particular topics in the manual.

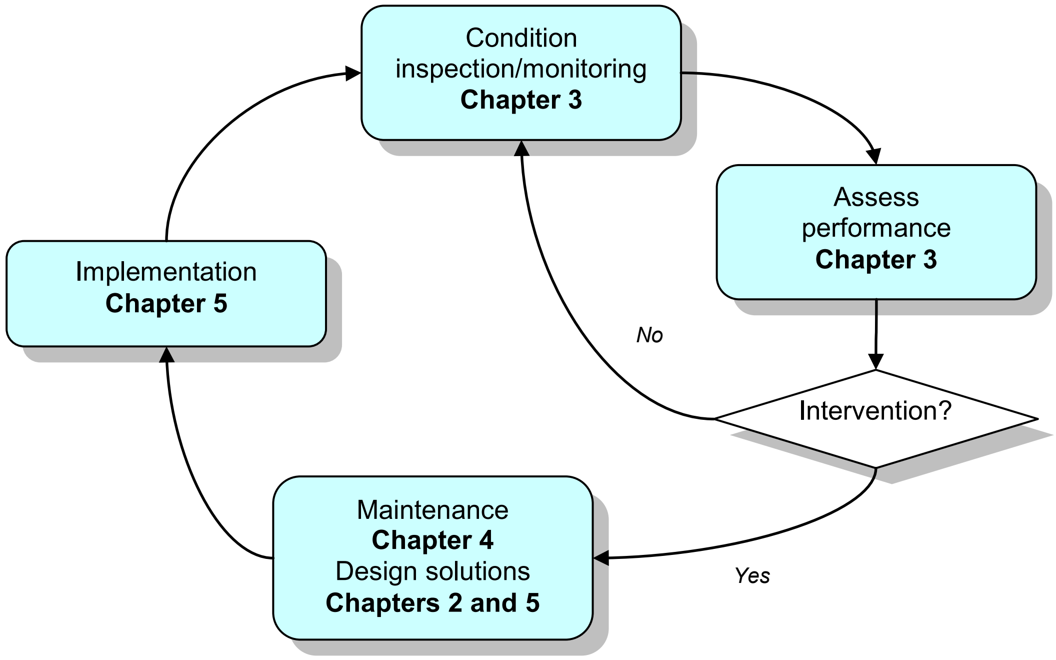


Figure 1.3 Document information in relation to the asset management cycle

It should be remembered that any intervention may modify the physical processes themselves. Monitoring of beach response should therefore continue once a scheme has been implemented.

Where relevant, the manual contains cross-references to other sections and to other documents where further or more detailed information on particular subjects or issues can be found.

1.5 Target audience

This manual is designed primarily for UK readership and practice. However, it is applicable to the management of toe protection structures anywhere. More specifically, it is aimed at all those with a direct interest in the management of flood and coastal erosion risk management assets in England and Wales, such as the Environment Agency, local authorities, drainage authorities, private owners of coastal frontages and consultants. In the UK, and specifically in England and Wales, a large proportion of this type of work is undertaken by or for the Environment Agency, and as a result the standard procedures tend to be dominated by those produced for the Environment Agency. Nonetheless, the principles set down in Environment Agency guidance are to a large extent valid for all operating authorities and so this manual draws heavily on current Environment Agency practice.

This manual is one of the suite of guidance documents covering the full range of flood and coastal erosion risk management assets. It is structured specifically to suit the practical needs of flood defence and coastal protection asset managers.

The manual is intended to provide those responsible for flood defences, coastal erosion protection structures and other coastal structures with practical guidance on how to determine, monitor, assess and mitigate for the effects of toe scour. It describes the types of toe protection structures and provides illustrations of typical designs that are often used as solutions. The case studies on beach lowering and scour management provide real examples of both good and bad practice, and discuss lessons learnt from past schemes.

1.6 Acknowledgements

This guidance draws on many sources of information. However, it is strongly guided and influenced by published outputs from the Environment Agency project, ‘Understanding the Lowering of Beaches in Front of Coastal Defence Structures’ Phases 1 and 2 (projects FD1916 and FD1927).

FD 1916, commissioned under the Joint Defra/Environment Agency Flood and Coastal Erosion Risk Management Research and Development Programme, recommended the development of a ‘toe scour’ guide to give practical guidance on the prediction of toe scour and the options for mitigating toe scour by introducing new knowledge gained from recent research and translating it into good practice.

This manual has been developed from the outputs of the Environment Agency’s Toe Structures for Coastal Defences (SC070056) project undertaken by HR Wallingford, which provided much of the research base for the content of this manual.

2 Toe structure types and materials

Chapter 2 introduces the common types of toe used in terms of structure, materials and function. The manual is intended for coastal managers and it is appreciated that most of the time they will be dealing with existing rather than new structures. This chapter describes the applicability of different toe types in such a way that it supports the choice of option when new works are planned. Importantly, it also highlights some of the advantages and disadvantages that may arise with existing structures.

Key links to other chapters:
- o Chapter 3 – Asset management
- o Chapter 4 – Maintenance
- o Chapter 5 – Toe design

Who will be interested in this chapter?
- o Asset managers
- o Coastal engineers
- o Contractors

2.1 Introduction

The toe is the seaward edge of the foundation of a coastal defence. It can have a major impact on its ability to:

- withstand beach lowering and scour;
- resist wave action;
- protect against wave overtopping.

The toe is sited in that critical location where the man-made defence meets the potentially variable beach, and where the sea dissipates its energy.

The toe can often be a sizeable portion of the defence construction and as such its effect on the human and natural environment should be carefully considered. This chapter introduces the common types of toe used in terms of structure, materials and function.

The manual is intended for coastal managers, who will mostly be dealing with existing structures rather than new ones. This chapter tends to talk about the applicability of different toe types in such a way that it supports the choice of option when new works are planned. However, it also highlights some of the advantages and disadvantages that may well arise with existing structures.

Section 2.2 presents the different types of toe construction and their use. The materials used in toe construction are discussed in Section 2.3. Toe design is discussed in Chapter 5.

2.2 Types of toe structure

In order for the toe to be secure over its design life, it needs to be founded at such a level or in such a way that it will not be undermined. It therefore needs to:

- extend to a level below the lowest expected beach level; or
- be of such a form that it can accommodate lowering beach levels; or
- be of such a form that it can be extended if the need arises.

The toe's functional design normally serves one or both of two primary functions:

- the retention of the foundation stratum, or subgrade (that is, the prevention of underscour);
- the ability to take foundation loads directly.

Table 2.1 lists the different types of toe together with a summary of their important characteristics and their applicability to different situations that may arise. In certain cases, these types can be used in tandem, for example, using concrete infill to remedy previous undermining, secured by toe piling to resist future beach lowering.

The various types of toe structure are then described in more detail.

Table 2.1 Main characteristics of different toe types

Type of toe		General applicability	Use on existing defences	Key design considerations	Hydraulic characteristics	Environmental impact	Comments
Vertical	Sheet piling – steel, concrete, timber	Exposed situations Can accommodate rise and fall of beach levels within acceptable limits. Can be used to provide cut-off of groundwater flows.	Underpinning, when used together with suitable backfill, capable of taking major foundation loads	Earth pressures, hydrostatic loads, minimum beach levels. Achievable design life limited by abrasion/ corrosion losses.	Toe face is reflective to waves if exposed above beach. Introduction of sheet piled toe can increase wave overtopping and exacerbate local beach scour.	If exposed face is high, can prevent beach access and form safety hazard.	Vulnerable to abrasion and corrosion. Requires access for piling equipment. New piling can be driven in front of existing, if necessary, to counter further beach lowering.
	Cribwork – timber or concrete frame retaining/ backfilled with rock	Mild to moderately exposed locations Can accommodate rise and fall of beach levels within acceptable limits Permeable to groundwater flows Most commonly used as a toe to a cliff.	Prevention of erosion of cliff toe	Earth pressures, retention of rock fill by cribwork, retention of earth behind, life limited by abrasion losses	Relatively absorptive if of sufficient width		Vulnerable to abrasion. Requires access for piling equipment. If materials are readily available, can be very cost-effective and easy to construct
	Masonry or concrete blockwork	Moderate to exposed situations Relatively rigid structure that requires firm foundations.	Filling scour holes, subject to good foundations	Earth pressures, potential for settlement	Toe face is reflective to waves if exposed above beach. Can contribute to scour.	More appropriate to an urban setting from a landscape viewpoint	Potentially very durable
	Rock-filled gabions	Areas of mild exposure Flexible (will accommodate	Unsuitable for major foundation loads, but can be used to fill scour	Retention of earth behind Life limited by	Relatively absorptive if rock is of adequate size	Vulnerable to vandalism Forms a personal safety hazard when	Can be very cheap solution, requiring only lightweight plant, if suitable rockfill readily

Type of toe		General applicability	Use on existing defences	Key design considerations	Hydraulic characteristics	Environmental impact	Comments
		settlement) Permeable to groundwater flows	holes, particularly if only rarely exposed to waves and abrasion.	abrasion	and grading.	gabions are broken	available.
	Concrete infill	Rigid and vulnerable to fracture if foundations are not sound	Filling scour holes and taking foundation loads, subject to good foundations	Foundation loads Achieving required strength in tidal zone	Reflective	Care is needed to achieve good appearance.	Can be in situ with a face shutter of permanent in situ facing, or bagwork can be used. Needs careful consideration in tidal conditions.
	Concrete toe beams	Rigid	Filling scour holes, subject to good foundations	Earth pressures; dead weight to retain structure behind. Requires suitable foundation.	Toe face is reflective to waves if exposed above beach. Can contribute to scour.	Care is needed to achieve good appearance.	Choice between precast or in-situ construction, dependant on scale of scheme and available tidal working window considerations. Concrete surfaces may be vulnerable to abrasion.
Sloping aprons	General note for sloping aprons: of potential use for countering the effects of beach lowering or scour. Requires greater 'land take' than vertical solutions.						
	Rock	Exposed situations Flexible construction that can accommodate settlement.	Providing a flexible layer in front of the defence that can adjust to a limited extent to accommodate the effect of beach lowering and limited	Stability against wave attack Design of underlayers to avoid settlement into the beach.	Depending on configuration, can reduce wave reflections and local scour, and also change overtopping characteristics.	Beach access and safety of beach users are important considerations.	Potentially very durable if using appropriate quality rock

Type of toe		General applicability	Use on existing defences	Key design considerations	Hydraulic characteristics	Environmental impact	Comments
			scour.				
	Flexible revetment systems	Protected and mildly exposed situations depending on weight and type of system. Flexible construction that can accommodate settlement.	Providing a flexible layer in front of the defence that can adjust to a limited extent to accommodate the effect of beach lowering and limited scour.	Stability against wave attack Ability of revetment to retain underlying material	Depending on configuration, can reduce wave reflections and local scour, and also change overtopping characteristics.		Requires careful detail at edges (edge beam or revetment excavated below lowest design beach level) to preserve overall integrity.
	Concrete slopes and steps	Exposed locations depending on strength and weight of construction	Protection to base of defence against beach lowering and scour	Stability against wave attack Requires adequate foundations.	Can modify overtopping. characteristics.	Suitable designs can readily allow beach access	Requires careful design to achieve adequate durability. Precast option may be suitable dependant on scale of scheme and available tidal working window considerations.
	Gabion mattresses	Protected and mildly exposed situations depending on weight and type of system. Flexible construction that can accommodate settlement.	Protection to base of defence against beach lowering and scour	Stability against wave attack Ability of revetment to retain underlying material	Can modify overtopping characteristics.	Vulnerable to vandalism Forms a personal safety hazard when gabions are broken.	

2.2.1 Sheet piling

Steel sheet piling is a common means of securing a defence against the threat of undermining (Figure 2.1). This technique is commonly used to stabilise foundations and reduce loss of fill materials, thus prolonging the life of the overall defence.

Figure 2.1 Underpinning of seawall toe with steel sheet piles (courtesy HR Wallingford)

Steel sheet piling is used both for new works and for restoration to secure a defence structure against undermining and instability (for example, a seawall and/or higher ground to landwards).[1] The characteristics that make steel sheet piling particularly suitable are its tensile strength and its form, which enable it to be driven to considerable depths (subject to suitable ground) without the need for excavation.

The role of sheet piling in preventing undermining is self-evident. Its role in restoring/ensuring geotechnical stability of a coastal defence is also often important. Present day design requirements (factors of safety for stability) can be stricter than those used in earlier (for example, Victorian) design and construction. When combined with long-term beach lowering, this can put considerable demands on a newly installed toe, resulting in considerably longer piles than might be needed on the basis of undermining alone. Moreover, anchor ties can be difficult and expensive to install beneath an existing structure and so heavier section cantilever piles are often used to avoid this complication. The pile section must be chosen to withstand the effects of future beach lowering and hence geotechnical loading. This, as well as corrosion, may limit the 'design life' of the structure.

Figure 2.2 shows a further example of the use of sheet piling.

[1] Examples of the use of sheet piles in toe protection schemes can be seen at the South Beach, Lowestoft, Overstrand, north Norfolk, and Fort Wall, Canterbury, case studies in Appendix C.

Figure 2.2 Sheet pile underpinning

2.2.2 Cribwork

Cribwork is a low-cost form of coast protection comprising rock-filled cages formed by piling and fabricating timber lattices into a continuous structure (Figure 2.3). In essence, they are not dissimilar to gabions except that they are made out of timber or possibly reinforced concrete. Cribwork can protect the toe of defences by absorbing wave energy and preventing scour of beach material and undermining of the structure.

Figure 2.3 Cribwork and concrete block fill, Norfolk (courtesy of North Norfolk District Council)

2.2.3 Masonry or concrete blockwork

Many of our urban seawalls, built in Victorian times, are masonry and, in many cases, it can be an attractive option for new construction. Nowadays, concrete blockwork is an alternative to stonework in terms of cost and ready availability.

The survival of hundreds of miles of Victorian masonry seawalls around the coast of Britain, many still in excellent condition, is testament to the durability of this form of construction. However, masonry is essentially rigid, reliant on the structural integrity on the outer shell of blockwork for its strength. Masonry requires a solid foundation such as a hard stratum or prepared concrete. Beach lowering, live loads on the wall from traffic and so on, leaching out of backfill, abrasion and wave impacts can all combine to damage or remove the blocks at the toe; Figure 2.4 shows an example of an extensive crack. Failure of the foundation ensues (even if only locally) and this can threaten that essential structural integrity and lead to rapid failure of the whole wall.

Figure 2.4 Extensive crack in masonry toe (courtesy HR Wallingford)

As soon as deterioration of the toe is identified, steps should be taken to repair the damage before it spreads. In this case, the use of replacement masonry is clearly an option as it offers durability and lack of visual intrusion. This is particularly important if the wall is a listed structure. However, before selecting the option, the cause of the problem (for example, whether it is the result of long-term beach lowering, a severe event or progressive attrition of the previous blockwork) should be determined to identify whether mere reinstatement of the blockwork is sufficient or whether additional measures (such as toe piling) are warranted. Care must be taken to ensure that any voids in the backfill are filled and that the replacement blocks are well founded.

Guidance on masonry walls is given in CIRIA publication B13 (CIRIA 1992).

2.2.4 Gabions

Gabions consist of steel mesh forming baskets that are filled with stones (Figure 2.5). They are available either as gabion boxes (and can be used as a flexible toe), or as mattresses (in which case they are used as a sloping apron where their flexibility allows them to accommodate beach lowering with time).

Gabions can be sufficiently flexible to fit an irregular seabed, are relatively cheap to fill and can be relatively easy to place. They are not suited to exposed coastlines and beaches with moderately aggressive sea conditions. Although they absorb wave energy, their resilience to such forces is limited becoming distorted, deformed and broken with subsequent loss of fill material. Gabion baskets can become a hazard to beach users and to wildlife when damaged/collapsed.

Figure 2.5 Gabion baskets of rock (courtesy HR Wallingford)

2.2.5 Concrete infill

Concrete infill can be used to effect repairs to the toe of a rigid (masonry or concrete) sea defence. The infill will be rigid and requires good foundation. When placed in situ, a face shutter may be necessary and the use of special finishes or facings may be required where visual appearance is an issue, particularly on listed structures. The use of concrete bagwork can be effective in filling holes where access is difficult.

Concrete infill at the toe, while being an effective means of filling voids, is often not sufficient in itself and may require other measures as well, such as toe piling.

2.2.6 Rock aprons

Rock can provide a quick and cheap method of filling in a scour hole. Rock dumps should be monitored to ensure that it continues to provide protection.

Armour rock had principally been used for breakwaters and other industrial applications, but its introduction as an acceptable material for use in UK coastal defence led to its increased use in such schemes to extend the life and to 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 modify the hydraulic performance of the overall defence (Figures 2.6 and 2.7). The latter can result in changed overtopping or different loads on the defence. However, the introduction of the rock toe has at times **increased** overtopping, so it must be carefully considered in the design process. In addition, 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.

Figure 2.6 Rock infill of scour trough, Le Dicq, Jersey (courtesy HR Wallingford)

Figure 2.7 Timber bulkhead with rock toe protection at Lepe, Hampshire

2.2.7 Flexible revetment systems

A revetment is intended to protect a slope, in this case against modest wave and current action. The protective layer may take the form of rubble, gabions (see Section 4.2.4), timber, masonry, concrete, concrete bags, concrete block mattress, concrete armour with rubble underlayers, and other resistant coverings. Using a revetment as a toe may simply be the continuation of the revetment that forms the main body of the defence below the beach/ground level or extended as a scour apron. A flexible and adaptable form of construction is often sought to cater for variations in the depth of the embankment both along its length and with time. Depending on the circumstances, a more sophisticated structural toe may be required.

There are a number of proprietary flexible revetment systems available. The systems consist of small concrete blocks which either interlock or are tied using plastic tendons to form mattresses that can be placed with a crane. Being proprietary systems their methods of application are set out in the manufacturers' literature. Some of this information is based on physical model tests, and this is to be preferred.

These flexible systems can be placed directly on a graded granular slope of a suitable particle grading, or a geotextile to retain the finer underlayer.

2.2.8 Concrete slopes and steps

The construction of an apron or steps at the base of an existing structure (Figures 2.8 to 2.10) can prolong the life and add amenity value to the defence, albeit at the expense of losing an area of the beach. This can be achieved at much reduced cost compared with rebuilding the defence entirely. Such additions to a structure will extend it seaward. There is a perceived danger that such seaward extensions of a structure will interfere with longshore sediment transport and hence reduce sediment supply to downdrift beaches. However, this must be put in perspective given the very small extent of the structural intrusion compared with the width of the seaward extent of active sediment transport.

Concrete slopes and steps have been used frequently around the UK and new techniques have been developed, for example, using a sloping asphalt apron (see Figure 2.18) to both protect the original structure against undermining and abrasion, and to reduce wave overtopping.

Figure 2.8 Extended scour apron, masonry steps and armour, St Ouens Bay, Jersey (courtesy HR Wallingford)

Figure 2.9 Concrete stepped revetment, Crosby, UK (courtesy Sefton Council)

Figure 2.10 Supplemental rock armour revetment (courtesy of HR Wallingford)

2.3 Materials

The properties of construction materials for application to seawalls and other marine structures are described in:

- *The Use of Concrete in Maritime Engineering: A Good Practice Guide* (CIRIA 2010b);
- *The Rock Manual* (CIRIA et al. 2007);
- *Potential Use of Alternatives to Aggregates in Coastal and River Engineering* (CIRIA, 2004);
- Chapter 4 of *Coastal Engineering Manual* (USACE 2012).

A substantial part of the advice given in these references and elsewhere is relevant to toe design, construction and management.

The position of the toe in a structure and hence the loading on it, together with its particular exposure to wave and tidal action, means that in relation to the whole structure, some material properties are highly significant while others are not. This section provides an overview of the relevant properties of materials with particular reference to those aspects that are especially important to toe protection.

The brief descriptions for each material cover the following key aspects where they are of particular significance:

- application;
- density;
- strength;
- flexibility;
- resistance to chemical or biological attack;
- durability (strictly speaking this is a relative measure of the ability of a given material to withstand the pressures of the environment in which it is used).

Figure 2.11 shows a seawall demarked into different zones designated 1 to 4 where Zone 1 is at the crest of the wall and Zone 4 is at the lowest level. Depending on the position of the structure in the foreshore, the toe could lie either in Zone 3 (if it is above low water) or Zone 4. These broad demarcations provide a useful reference in the discussions which follow.

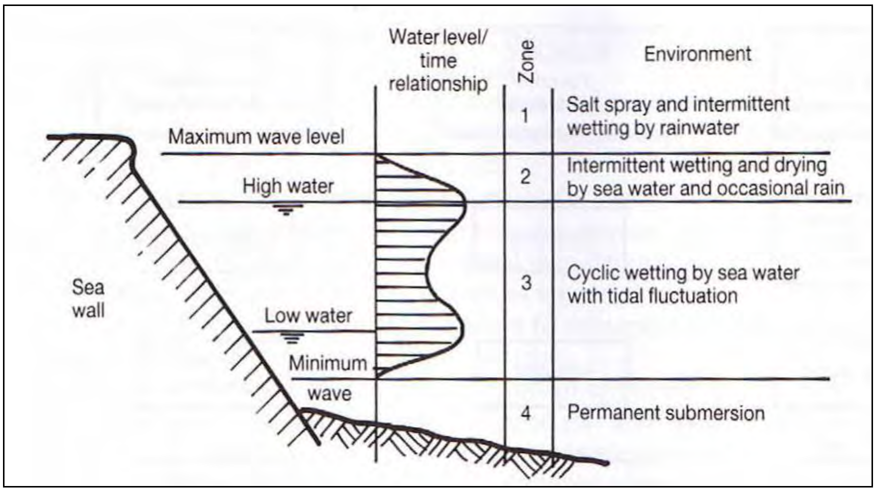


Figure 2.11 The weathering zones of a seawall (from Thomas and Hall 1992)

2.3.1 Steel

The most serious threats to steel when used in coast protection works are corrosion and abrasion.

BS 6349 sets out the standards for the design and construction of maritime structures, including the use of piling (BSI 2010).

The issue of durability is discussed in detail in the *Piling Handbook* (Arcelor 2008).

The loss of pile thickness due to corrosion only is given in the *Piling Handbook*, based on Eurocode 3, Part 5. Relevant values for loss of thickness are given in Table 2.2.

Table 2.2 Loss of thickness (mm) due to corrosion for piles and sheet piles in sea water [1]

Location of pile/sheet pile [2]	Required design working life (years) [3]				
	5	25	50	75	100
Sea water in temperate climate in the zone of high attack (low water and splash zones)	0.55	1.90	3.75	5.60	7.50
Sea water in temperate climate in the zone of permanent immersion or in the intertidal zone	0.25	0.90	1.75	2.60	3.50

Notes: [1] The values given are for guidance only.
[2] The highest corrosion rate is usually found at the splash zone or at the low water level in tidal waters. However, in most cases, the highest bending stresses occur in the permanent immersion zone.
[3] The values given for 5 and 25 years are based on measurements, whereas the other values are extrapolated.

The user should refer to the source references when assessing corrosion as it is an important and complex issue, covering not only general corrosion but also localised factors.

A particularly aggressive form of corrosion is the microbiological process known (in Britain) as accelerated low water corrosion (ALWC). In spite of its name, ALWC is not confined to low water or just above lowest tide level, and it does occur at other levels. In situations that favour ALWC, surface corrosion rates can be more than 1 mm per year. CIRIA C634 provides a comprehensive guide to the phenomenon of ALWC and its management and mitigation (CIRIA 2007).

Depending on the environment and particularly the nature of the sediments, steel loss will occur to varying degrees through abrasion. This aspect is less well quantified than corrosion as it depends on a number of variables including exposure of the steel, the prevalence of abrasive material (for example, shingle), and the level of wave activity – albeit a coarser sediment such as shingle is more likely to be encountered in a higher energy environment.

Upper rates of steel loss by corrosion and abrasion are listed by Thomas and Hall (1992). This reference indicates that, depending on the circumstances, the contribution to material loss by abrasion can be considerable, for example up to 85 per cent (of 1 mm per year) in Zone 3 (Figure 2.11) where steel is exposed to a high percentage of gravel and a severe wave climate.

A further important property of steel sheet piling is its relative impermeability. Seepage losses are generally minor and restricted to seepage through the clutches (the interconnections), which may be reduced by the presence of fine sediments.

Deep and continuous piling can intercept groundwater flow paths which might be due to tidal action (flowing both ways) or fresh ground water. The effects of this need to be assessed on a case by case basis; consequences might include a raised water table on the landward (geotechnically active) side. Mitigations might include the inclusion of weep holes or slots, or intermittent shorter piles to relieve the drainage path.

2.3.2 Rock and masonry

Rock and masonry both refer to the use of stone. The basic geological characterisation of different rock types are given by Fookes and Poole (1981). *The Rock Manual* (CIRIA et al. 2007) provides a comprehensive introduction to different rock types.

Whereas the term 'rock' is usually applied to loose stone used for armouring a sloping revetment or apron,[2] the term 'masonry' always refers to stone placed in a compact pattern, usually with a relatively smooth outer surface, often grouted (see Figure 2.12).

Masonry is often used both for armouring and to provide a decorative appearance. The principle mechanical properties of stone (hereafter referred to as rock) that make it an attractive material for use in toe construction are:

- its high compressive strength;
- its load bearing capacity;
- its high density (for inherent stability);
- its ability when placed irregularly to form a durable flexible toe.

Figure 2.12 Grouted stone/masonry (courtesy HR Wallingford)

Partially grouted armourstones combine the high resistance against currents and waves of large elements and their flexibility to adapt to ground deformations and the option of installing comparably thin layers. Stone/rock diameters of 10–40 cm and a narrow rock size distribution are best for being grouted.

Partial grouting (Figure 2.13) is a reliable and well-established method to meet the requirements for a long-lasting scour protection including sufficient permeability to avoid excess water pressure below the armour layer. Partial grouting means filling the voids in the riprap or stone layer to 35–50 per cent with a special mortar, creating a bonded layer with high resistance and high permeability and sufficient flexibility. Cement-bonded grouting materials are recommended for partial grouting (Heibaum and Trentmann 2010), although the advantages and disadvantages of using asphalt as a binder is discussed in Section 2.3.4.

[2] Examples of the use of armourstone for toe protection can be seen in the Clayton Road, Selsey, and Corton, Suffolk, case examples in Appendix C.

Figure 2.13 Grouted stone toe apron, Thames Estuary (courtesy HR Wallingford)

The strength of rock differs greatly between the various rock types, but those used for construction in the marine environment will generally be required to have good compressive strength.

The degradation of rock is due primarily to mechanical damage by way of fracture, breakage and abrasion, although ice can become a significant factor in colder climates. Fracture and breakage are more likely to occur in loose rock used for armour rather than set in masonry. In this case, the breakage of armourstones yields smaller stones which, being more susceptible to movement, become increasingly prone to further breakage and abrasion. Abrasion reduces the mass of stone used in both rock armour and as masonry. Further spalling can be due to salt attack.

Density is an important parameter if high rock weight is required (for example, for armouring). High density tends to go hand in hand with high strength and durability.

Other properties of rock relevant to its application in toe construction works are outlined below:

- **Permeability of the armour layer (rather than the rock itself).** Permeability (drainage) is an important issue and introduces a clear distinction in the application of stone when used as rock armour or as masonry. Permeability is very important in toe design as it can determine the drainage of ground water behind a structure. The groundwater level may be determined both by freshwater and seawater, the latter arising from retained tidal water or by wave overtopping onto a more porous upper part of the structure. Clearly, loose rock affords better drainage than a grouted masonry revetment. Another aspect of permeability, or porosity, relates to wave energy dissipation. For example, a rough porous rubble slope has much better energy absorption through internal turbulence than a smooth wall.

- **Compliance/flexibility** can be important attributes for toe protection where it is required to accommodate bed movement, by way of a falling (or settling) apron for instance. Clearly, this attribute cannot be enjoyed in the case of consolidated masonry.

- **Availability of rock** is an important consideration, both logistically and in terms of cost, as the source and travel distance from source can add significantly to the price. For large constructions (for example, whole revetments or breakwaters), the cost of importing large rock may be better justified than for small-scale works involving attention to the toe only. If the toe is below low water, installation may require the tipping of rock; hence there is a need to allow for generous tolerances.

2.3.3 Concrete

Concrete consists of three fundamental components: cement, aggregate (sand and gravel) and water. Different combinations of these ingredients provide for a wide range of construction materials ranging from mortars used for grouting or rendering, to high strength structural concrete.

For maritime applications, useful guidance for the design, construction, testing, repair and maintenance of many different types of such structures can be found in *The Use of Concrete in Maritime Engineering; A Good Practice Guide* (CIRIA 2010b). In particular, it highlights how achieving good quality concrete in the tidal zone is challenging, while also pointing out how necessary it is to achieve good quality dense concrete in order to achieve a durable structure in terms of both resisting abrasion and preventing corrosion of reinforcement.

Figure 2.14 Concrete toe beams (courtesy HR Wallingford)

A significant feature of concrete for use in seawalls is that it can be formed to suitable shapes which may be both functional and decorative. The latter attribute is not usually so important in the case of the toe where it is less visible than the upper parts of the structure. Concrete will more commonly be used in toe protection works to provide weight and compressive strength. Figure 2.14 shows an example of the use of concrete toe beams and Appendix C contains a case study describing the use of concrete toe beams for foundation strengthening along the seawall between Dawlish and Teignmouth, South Devon, UK.

Although the specific gravity of concrete is usually lower than that of good quality rock (for example 2.4 versus 2.6), it can be cast into larger volumes of convenient shape to match the construction details. It follows that concrete is often used where there is a need to provide ‘fixity’ at the toe without the use of piling (for example, toe beam to a single layer armour system or as passive resistance to geotechnical pressure).

Good quality mass concrete can be designed for the life of a project, for example, 100 years. Longevity of the concrete is not therefore necessarily a design life determinant, although service life can be compromised by:

- difficulties in obtaining good quality concrete if cast in situ below water;
- chemical resistance – more particularly in respect of reinforcement;
- use of aggregates that are insufficiently durable.

In the case of reinforced concrete, corrosion of steel reinforcement causes the bars to expand leading to fracture and bursting of the concrete (spalling) (Figure 2.15), possibly leading to failure. After spalling, the reinforcement is even more vulnerable to corrosion through direct exposure to air and water. This process is particularly significant in parts of the structure subject to wetting and drying, as illustrated in Zone 3 of Figure 2.11.

Figure 2.15 Reinforcement rebar corrosion and spalling of concrete (courtesy of HR Wallingford)

As time goes by and the concrete is subject to repeated cycles of wetting or submergence, chlorides from the seawater penetrate the concrete and reduce its alkalinity. The time for this penetration to seriously affect the steel depends on the thickness of cover and the permeability of the concrete (for example, penetration of 25–75 mm would typically take about ten years). The quality of the concrete is important as chlorides can penetrate very small cracks. Figure 2.16 shows an example of salt deposits with associated cracking, while Figure 2.17 shows the failure of a concrete toe beam on a Jersey beach.

Figure 2.16 Surface salt deposits and associated cracking from alkali–silica reaction in concrete (courtesy of HR Wallingford)

Figure 2.17 Failure of a concrete toe beam at St Ouens Bay, Jersey (courtesy HR Wallingford)

There are ways of reducing the ingress of seawater and consequent corrosion of steel reinforcement including:

- avoiding contact between concrete and seawater during curing;
- inclusion of pulverised fuel ash (PFA), though this increases curing time which can be problematic for intertidal construction;

- reduction in curing time with the use of rapid hardening ordinary Portland cement (OPC) as a means of reducing exposure to seawater;
- sulphate-resisting cement may be used (especially where waters are near to warm outfall discharges);
- use of various protective coatings to inhibit the corrosion process.

Clearly it is preferable to avoid the use of reinforcement in concrete wherever possible, but reinforcement may be required for certain details such as pile capping. Here, the need for having the capping (at all) could be considered; for example, it may not be needed if the capping is to remain covered by sand (but the designer needs to be aware of future risks if the capping becomes exposed).

In simple terms, good quality dense concrete will be durable. In order to achieve good quality, precast elements can produce a high level of durability and they are often appropriate in cribwork, toe beams and flexible revetments.

2.3.4 Asphalt and bitumen

Asphalt is the term given to a range of materials which comprise, in various ratios, bitumen (the binder), fillers and aggregates. Different types include:

- sand mastic (used for grouting blockwork or as a carpet below water);
- lean sand asphalt (uses include as a filter layer to upper armouring, also as underwater protection against scour);
- open stone asphalt (scour resistant revetment);
- asphaltic concrete (used for load-bearing surfaces).

Figure 2.18 shows the use of asphalt to grout a stone revetment.

Figure 2.18 Asphalt grouted stone revetment in the Thames Estuary (courtesy HR Wallingford)

In respect of the pressure of the marine environment, asphalt is generally regarded to be of poor strength, in terms of both tension and compression. Its durability is dependent on exposure. A key advantage of asphalt in toe construction is that it is relatively flexible, that is, because of its thermo-plastic properties it can accommodate ground movements better than, for instance, concrete.

The permeability of asphalt is another important consideration. This varies according to the type of mix; for example, sand mastic is impermeable while open stone asphalt is porous and relatively permeable. As with sheet piling, the permeability of the asphaltic material used, and how it is applied, is clearly important in toe protection design for the avoidance of groundwater build-up and back pressure.

A further consideration is that of abrasion resistance. Asphalt is vulnerable to abrasion in an environment where shingle is present. This can seriously affect the life of a structure.

Some further outline notes are given below in respect of the different forms of asphalt.

- **Sand mastic** is impermeable but is a weak material that can rupture under pressure.
- **Lean sand asphalt** has better permeability and is inexpensive, but depending on circumstances, may only have a short life. It could, however, be used as an interim or short term measure.
- **Open stone asphalt** can be used for the body of a wall but should not be placed underwater as boiling damages the binder; this can limit its application to toe structures in some cases. While open stone asphalt is relatively permeable to the extent that it can alleviate tidal differential, in respect of wave activity it is effectively smooth (that is, it is not a good energy absorber). It is also vulnerable to erosion. Toe scour protection using a mastic slab or stone/mastic has been used in the UK for example at Dovercourt near Harwich in Essex and in several locations on the Wirral.
- **Asphaltic concrete** is not likely to be used for toe construction.

Further detail on material compositions, properties and general guidance on the use of asphalt can be found in:

- *The Shell Bitumen Hydraulic Engineering Handbook* (Schönian 1999);
- *The Use of Asphalt in Hydraulic Engineering* (van der Velde et al. 1985).

2.3.5 Timber

Timber is widely used in jetties and other marine structures. Good compressive and tensile strength means that timber can be formed into strong lattice structures, piles, piers and platforms. Generally, the denser the wood, the stronger and more durable it is. Greenheart is classed as being of exceptionally high strength and durability.

Though less durable than concrete, the harder woods offer good durability – the life of a structure being sometimes dictated by that of the bolts and other corrodible fittings that hold the structure together than the life of the timber itself.

Timber has been used in cribwork (see Figure 2.3). Timber planks were often used as pilings to seawalls before the introduction of steel piles.

The two main causes of timber decay in the marine environment are biological attack and abrasion.

- **Biological attack.** Timber is vulnerable to damage by marine borers. The resistance to biodegradation depends on the wood species (for a list, see Thomas and Hall 1992). Two common kinds of infestation are ‘gribble’

(Figures 2.19 and 2.20) and ‘Teredo’. Gribble attacks the surface of exposed timber, mainly below mean tide, penetrating a few millimetres below the surface, whereas Teredo destroys the internal composition of the wood. Both gribble and Teredo are encouraged by clean warm water discharges and are less prevalent in more polluted environments. The more durable hardwoods used for marine construction tend to be resistant to biological attack and are thus able to provide a good service life in excess of 20 years or much longer in certain cases.

Figure 2.19 Gribble attack on Timber pile at the sediment line (courtesy of HR Wallingford)

Figure 2.20 Close up of gribble damage to timber (courtesy of HR Wallingford)

- **Abrasion.** Persistent abrasion by both sand and shingle has the effect of rounding off the corners and details of timber in the sea (Figure 2.21). As this effect tends to be concentrated close to the bed where sediments move more, the abrasion tends to be concentrated over a short length – a feature sometimes referred to as necking (Figure 2.22). Evidence of necking high above the bed can be indicative of a low beach (compared with normal or former times).

Figure 2.21 Rounding of groyne timbers by beach shingle (courtesy HR Wallingford)

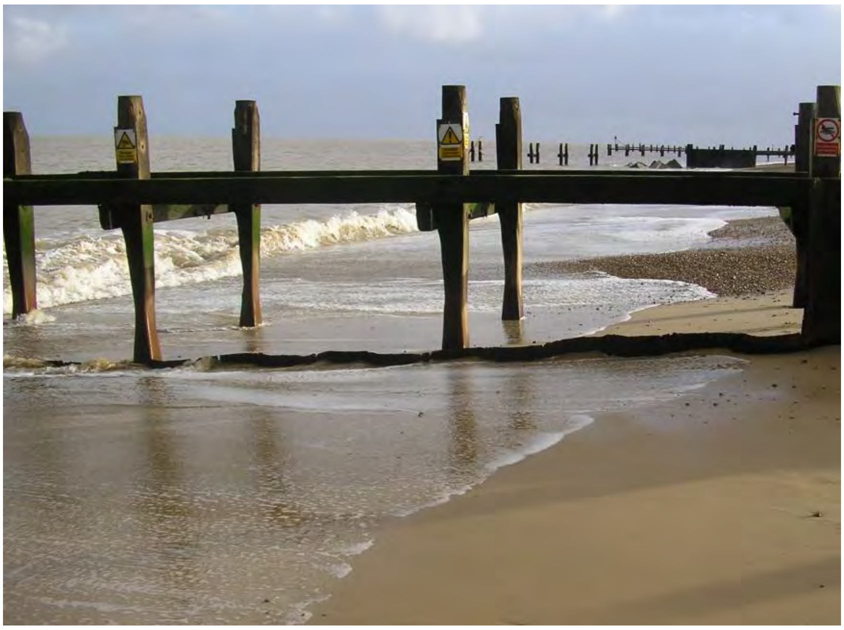


Figure 2.22 Necking in derelict timber piles (courtesy ENBE)

The specification of hardwoods for use in coastal defence needs to recognise the availability of source and the use of sustainable forest supplies. Alongside this, there may be scope to reuse existing timbers from decommissioned earlier works (for example, estuarial boat mooring piles), possibly not for main structural members, but for hand-railing or similar ancillary works.

Detailed advice on the use of timber in coastal engineering can be found in Crossman and Simm (2004).

2.3.6 Geotextiles

Geotextiles are used mainly as filters and separators in coastal construction, primarily to prevent the migration of fine sediments through a coarser over layer of rock. The design must balance drainage capacity with particle retention (a typical application is for retained sediment sizes in the range 0.06–2 mm), while satisfying load conditions for construction and service life survival.

Made from polymer filaments or fibres, geotextiles are formed into textiles by weaving or other means (for example, welding). Woven textiles tend to have a smaller range of opening sizes compared with non-woven materials.

Geotextiles can also aid the distribution of armour point loading across soft substrate conditions.

If the geotextile is to be used, it must:

- be undamaged;
- provide cover in a controlled manner;
- be laid in such a way that it is continuous over the area where it is required (for example, with sufficient overlap where sheets adjoin).

The use of geotextiles and constructability of toe details was investigated in a prototype trial at Milford-on-Sea in Hampshire (see Box 2.1).

While geotextiles have a good inherent strength, they must be selected to match the loading (for example, the placing rocks). The strength of woven materials is determined by the direction of loading, so care needs to be taken when placing in relation to the applied load (for example, rock placed on a slope). To avoid the risk of damage, non-woven geotextiles may be preferred. The loading constraints on a given material should be obtained from the manufacturer prior to specifying works.

It should also be noted that geotextile has a lower natural angle of friction than rock such that slopes steeper than 1 in 2 should be avoided.

Geotextiles are susceptible to degradation resulting from ultraviolet (UV) radiation (sunlight). This can be inhibited by using UV inhibitors and by covering the material during storage prior to installation. As part of a toe structure, the geotextile is likely to be covered once installed into the structure.

The placing of geotextile in open seas can be difficult and needs to be carefully planned. Some form of ballasting, either proprietary or bespoke, is usually devised.

Useful further information on functional design for geotextiles in hydraulic engineering is described in Section 3.16 of *The Rock Manual* (CIRIA et al. 2007).

2.4 Choice of structure type and materials

The previous two sections provide an overview of the different types of toe structure types and materials. The actual choice of a toe structure type(s) for a solution depends on a number of factors and the selection or short listing of appropriate types for a given application is the first step towards a successful design.

Factors that a designer should typically include when considering the type and materials for a toe structure include:

- geotechnical loading;

- integration with the structure it is required to support;
- design life;
- level of exposure to wave and/or current attack;
- predicted extent of future scour in front of the toe;
- visual impact, heritage aspects and so on;
- purpose (that is, new toe to new structure or new toe to old structure);
- availability of materials;
- constructability, taking into account aspects such as access, tidal working, ground conditions, groundwater, integration with any previous toe structure and environmental impacts (for example, due to excavation in the foreshore);
- cost;
- level of future maintenance.

As the combination of these factors is likely to be unique for any given site, it follows that it would be unwise to propose a generic toe structure design specification.

The designs illustrated and described in this manual are included only as case study examples and information for the benefit of the reader and are not intended as ‘off-the-shelf’ solutions. Nonetheless, some generic design principles and some tried and tested toe scour and undermining mitigation options are useful. These are illustrated throughout this manual and in particular in Chapter 5.

Box 2.1 Paddy's Gap Rock Revetment Works

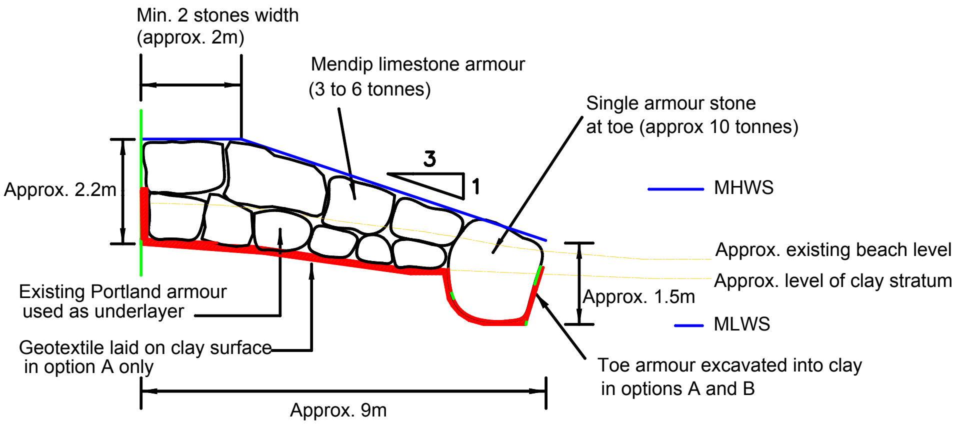
Maintenance works carried out to coastal defences at Milford-on-Sea, Hampshire, provided an opportunity to compare a number of different foundation details. The works comprised the reconstruction of a 60 m rock revetment which protects against the undermining of a concrete seawall within a groyne bay.

An existing revetment of 1–2 tonne Portland stone armour had been placed as emergency works in 1995, but performance had deteriorated and it was judged to have reached the end of its useful life by March 2003.

A new revetment was designed using existing rock as the underlayer and 3–6 tonne Mendip limestone for armour, with the toe comprising larger stones up to 10 tonnes.

Three different designs, representing varying degrees of complexity, were built with the time taken and ease of construction monitored. The performance of the different designs will be assessed during future monitoring.

The most complex design (A) is illustrated below and includes both geotextile and excavation for the toe rock. The intermediate design (B) omitted the geotextile and the least complex (C) did not include any geotextile or toe excavation.



Potential savings afforded by the designs were discussed with the contractor prior to the works being undertaken and observed on site. The toe of the structure was found to be critical since this is most affected by weather and the 'tidal window'. Construction of the slope could occur on a rising tide providing the toe had been placed properly. The time saving resulting from the omission of geotextile was relatively small (approximately 30 minutes for the 20 m section of revetment) and savings were found to be minimal since the same plant (one tracked excavator with a bucket and one with a rock grab) were required on standby for the whole of the work. The omission of an excavated toe provided a further saving of 10 minutes; however, again both excavators were still needed as the mobile beach had to be profiled even though the toe was not excavated into the clay.

The least complex section did, however, provide significantly more flexibility in construction which enabled work to be carried out during neap tides and in poor weather conditions. This provided minimal savings in the example, but could have had a significant benefit, enabling programme to be maintained, during more extensive works. However, the savings made during construction must be gauged alongside the longer term performance of the scheme.

Source: Sutherland et al. (2003)

3 Asset management

Chapter 3 provides guidance on post-scheme asset management activities such as asset performance, monitoring, data analysis, risk assessment, deterioration and trigger levels for action. It establishes a framework for practical use based on identifying the symptoms, causes and possible cures for any toe scour problems, based around a continual data monitoring programme.

Key links to other chapters:

- Chapter 2 – Toe structure types and materials
- Chapter 4 – Maintenance

Who will be interested in this chapter?

- Asset managers
- Coastal engineers

3.1 Introduction

Management of a structure toe can conveniently be considered by reference to two separate but related elements:

- structure condition and deterioration;
- dynamic changes to the beach or foreshore.

In many instances, the integrity of the structure is controlled by the dynamic changes arising from changes to the beach in combination with the forcing hydrodynamic conditions. A structure that may appear to be in excellent structural condition may be highly vulnerable to dynamic changes on the beach or foreshore, and these can impact significantly on both performance and condition.

Changes in beach slope and level, relative to any flood defence or coastal erosion protection asset, affect potential overtopping rates as well as the likelihood of undermining and failure or breaching of the asset. It is therefore important to employ a toe management system that is able to cope with dynamic changes to the foreshore. Coastal defences (including beaches) need to be managed to maintain their level of performance throughout their life cycle. This management will be required at a policy or strategic level and at an operational level.

The age of the structure is a significant consideration as some elements will begin to deteriorate more quickly than others. Elements such as joints in concrete and masonry may be vulnerable. While other parts of the structure may be continue to be in sound condition, damage to the weaker elements may lead to structure failure.

Knowledge of the original construction of the structure is crucial and access to ‘as built’ drawings provides a crucial part of the management process.

3.2 Management overview

At the **strategic level**, consideration should be given to issues such as:

- Is the standard of protection afforded by the asset sufficient and in line with the system standard overall? (That is, is the asset providing the required level of protection alongside other assets in the system or is it a 'weak link'?)
- Planning of future works and maintenance spending (that is, will the asset be refurbished, replaced or even decommissioned as part of a larger scheme or a planned change in policy?).

At an **operational level**, the management considerations are slightly different:

- Is the asset performing as designed and required?
- What needs to be done physically to the asset in order to maintain its level of performance at the required level?
- How often should the asset be inspected and how? Should it be monitored on a more regular basis than is currently the case?
- What level of assessment is required?
- What data should be collected, how, and at what frequency and resolution?

Establishing an asset management policy and strategy, together with implementation plans and operational procedures, should provide a purpose-made set of processes, tools and performance measures that will enable achievement of an optimum approach to managing assets. This strategy should identify issues such as those outlined above and seek to provide answers and solutions. Such a strategy needs to be owned at executive level, be evidence-based and be auditable in its application. For this a suitable framework can be adopted from the British Standard Institution's Publicly Available Specification for the optimised management of physical assets, PAS 55 (BSI 2008). PAS 55 is applicable to any organisation that depends upon its physical assets for the performance and continuance of its business operations.

The generic 'Plan – Do – Check – Act' framework in PAS 55 (Figure 3.1) provides a template against which industries can develop or check their own approach to the management of physical assets. The framework covers:

- **Policy and strategy** linked to corporate objectives and acceptable risk;
- **Information, risk assessment and planning** including information systems, risk identification and assessment, leading to an asset management plan with its priorities and targets;
- **Implementation and operation** focused on intervention to maintain, operate and dispose of assets including such issues as responsibility, training, awareness, communication and emergency response;
- **Checking and corrective action** including monitoring of performance and condition, asset-related failures, corrective and preventive action;
- **Management review** and audit, completing the cycle and leading on to continuous improvement.

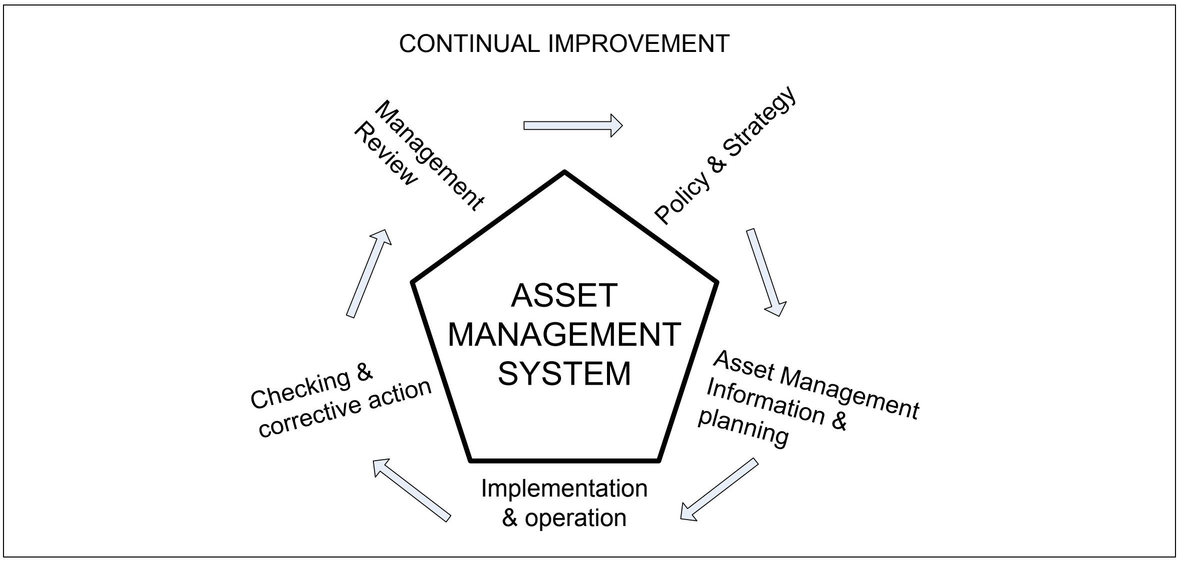


Figure 3.1 The PAS 55 Plan – Do – Check – Act framework (from BSI 2008)

Under the 'Information, risk management and planning' phase, there are some common risk-based issues that need to be considered in both the strategic and operational management of coastal defences. These issues are:

- **Precautionary approach** – the need to ensure that proper analysis is applied to decisions where there are uncertainties and/or a lack of knowledge, and where there is potential for serious or irreversible environmental harm (for example, exposing additional lives or property to flood or erosion hazards);
- **Proportionality** – the need to ensure that resources are targeted at the most significant risks and to demonstrate equity of benefits;
- **Effectiveness** – the need to provide a sound basis upon which to take consistent flood defence and coastal erosion management decisions;
- **Efficiency** –the need to take consistent and defendable decisions which allow the movement from historic and/or reactive unsustainable actions (subject to legislative constraints) to strategic proactive decisions where residual risk levels have been reduced to levels deemed acceptable and sustainable.

This manual provides information on, and analytical tools for, beach levels and structures at the toe of coastal defences that should aid asset managers or practitioners in the 'risk assessment and planning', 'implementation and operation', and the 'checking and corrective action' phases of the asset management system (AMS) cycle in Figure 3.1. The following sections discuss:

- key failure and damage types;
- risk-based assessment and reliability analysis;
- determining trigger levels for action/intervention;
- beach monitoring and surveys for data collection and the appropriate analysis of data;
- asset condition assessment;
- when to conduct surveys;

- asset deterioration.

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 to reduce the consequences of the erosion and/ or flooding problems that are being experienced. Equally there are many situations that demand immediate remedial action in order to avoid brittle structure failure and expensive repair. It should be emphasised that toe damage or failure can spread rapidly and endanger the integrity of the overall structure.

There is a need to assess the current condition and performance of coastal defences, and to consider potential future beach lowering. Such an assessment requires monitoring and analysis of survey results to identify recent patterns of change. Predictive analysis can be conducted to estimate whether beach levels may lower in the future and whether the coastal defence structures will be undermined. In addition, it is necessary to carry out calculations to assess how the defences will be affected by severe conditions (that is, high tides and large waves) for both present day and future beach levels.

The majority of toe protection works undertaken are not ‘new build’ or ‘capital’ works, but are commissioned for other reasons such as remediation, maintenance or repair, reconstruction and augmentation, or strengthening of existing defence structures. Within the context of asset management, maintenance, repair and reconstruction can be seen as stages within the management cycle which occurs throughout the lifespan of the defence structure. That is, continuous or intermittent monitoring and performance assessment will/should determine whether interventions are necessary in order to maintain the level of performance desired and offset deterioration. Figure 3.2 illustrates this as a cycle of asset management processes and decisions.[3]

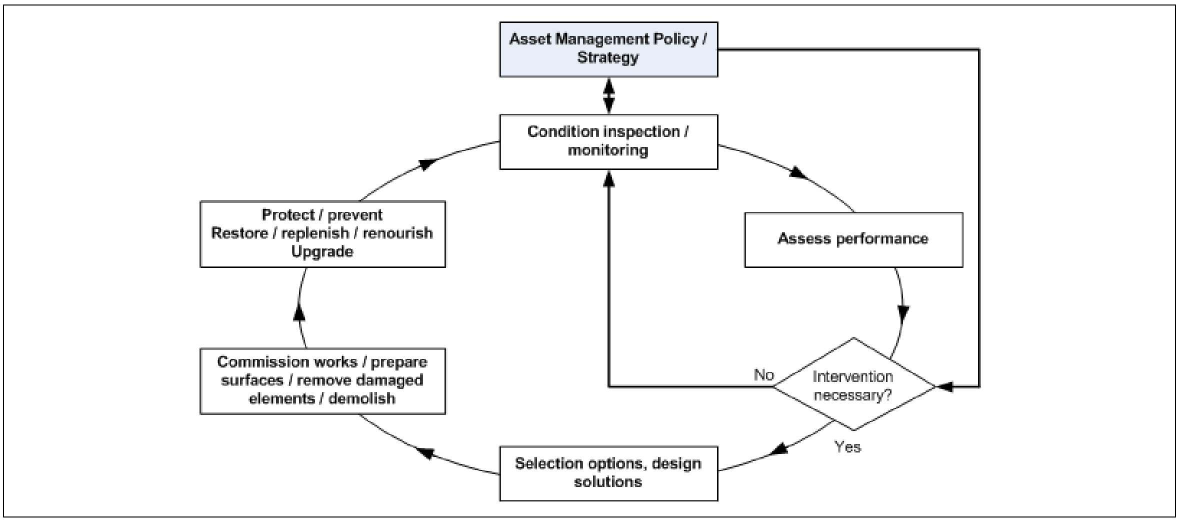


Figure 3.2 Key phases for asset management of existing coastal defence structures

3.3 Life-cycle failure modes

Potential failure modes must be considered carefully when developing a monitoring and maintenance programme. This requires a basic understanding of design principles, which are discussed in more detail in Chapter 5. The context of the design principles must be considered in relation to the in-service performance of a system under a

[3] This asset management cycle is very similar to the Frame-of-Reference for implementing coastal erosion management policy in the Netherlands (see Mulder et al. 2011).

dynamic loading. These are considered by reference to a simplified representation of the forces acting on a gravity wall illustrated in Appendix B. The calculations and approximations discussed in Appendix B require careful review to make fundamental decisions:

- How much risk is acceptable before an intervention must be enacted?
- How much lead time is required to examine and implement potential intervention options? That is, how long does it take to consider options, consult, acquire appropriate permissions, raise funds, complete designs and secure construction contracts (if required) before the risk of failure becomes unacceptably high?

Asset managers should consider such time lags before intervention is essential. The term 'trigger level' is usually used to identify the conditions of the beach and structure at which intervention might be required. The context of beach trigger levels is discussed further in Section 3.5. Appropriate trigger/alert levels must be set in relation to the time to ensuing risk of failure at an unacceptable level. The level at which the risk becomes unacceptable may be set as a matter of coastal policy. Examples of common practical examples of in-service structure damage that may require remediation are described and illustrated below.

3.3.1 Overturning and settlement of a gravity wall due to toe scour

Waves can cause scour holes at the toe of the seawall. Scour holes develop underneath the base of the wall and reduce the surface of the base of the wall, which is supported by the underlying foundation of the soil. The loading (weight of the structure, horizontal ground force and horizontal hydraulic force) of the structure thus has to be distributed over a decreasing foundation area. The developing scour hole results in a decreasing width of foundation until there is insufficient force to support the structure (Figure 3.3). If the structure has not been designed to withstand such pressures and the condition is not prevented or quickly addressed, the structure will move in response and progressively fail, or even fail completely. The point at which this is calculated to happen is known as the 'disturbing moment' or the 'ultimate limit state' (ULS).

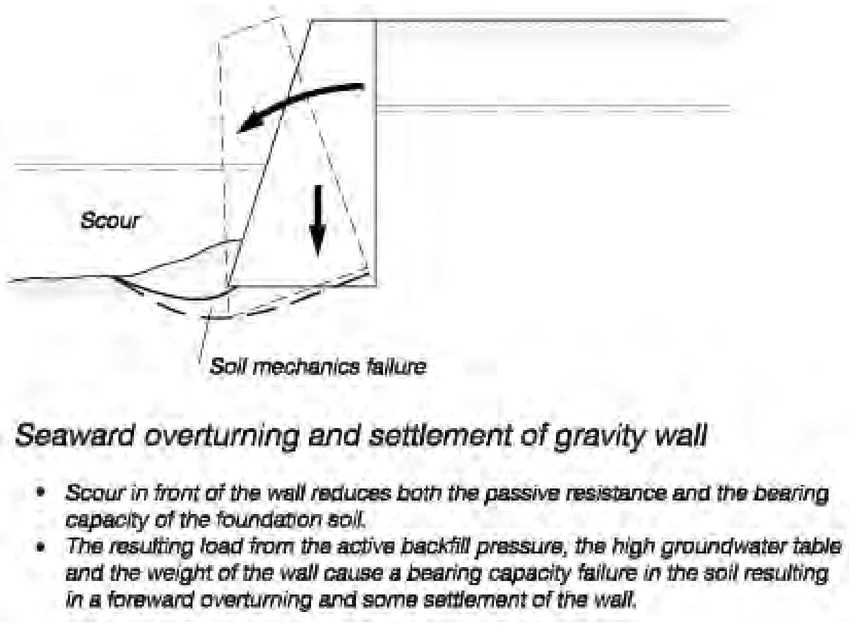


Figure 3.3 Overturning and settlement of a gravity wall due to removal of passive resistance

This ‘moment of failure’ can form the basis on which the ‘critical’ level can be set. It is, of course, not normal to wait until this ‘failure moment’ arrives before a decision is made to intervene. A factor of safety would usually be inferred from the likely rate of deterioration towards failure – the critical point. This is problematic for scour processes, as scour holes can easily form and infill again within a tidal cycle. Trigger or ‘alert’ levels for intervention (in advance of critical levels being reached) therefore need to be based upon the potential or likely depth of scour in relation to the limit state equation (Appendix B), given the preceding beach level and probability of dangerous loading conditions (that is, how likely is it that the next scour event will undermine the structure sufficiently for it to overturn or collapse).

These points are crucial as they form the basis from which subsequent calculations are derived. Obviously, the aim is to avoid and prevent instability and failure of the structure. In order to achieve this, pressures on the structure should not be allowed to fall below the level at which disturbing moments (causing instability) will occur. If the ULS is reached it may be too late to prevent failure.

Figure 3.4 shows an example of a seawall in the process of overturning.

Figure 3.4 A seawall in the process of overturning (courtesy of Black & Veatch)

3.3.2 Toe scour resulting in increased overtopping and structure sliding

Although the symptoms of a seawall failure may appear to result from other causes such as overtopping, these can often arise as a result of changes at the toe of the structure. The example below demonstrates changes in conditions at a seawall, which have arisen as a result of beach lowering:

- Beach levels reached the crest of the wall at the time of construction and the wall was well protected, with shingle, limiting wave attack on the structure toe.
- Falling beach levels have exposed the structure toe piling (which extends 7 m below the pile tops) (Figure 3.5). Although the structure continues to resist overturning and sliding, increased water depths at the structure toe allow larger waves to attack the structure.

Figure 3.5 Exposure of interlocking steel piled toe arising from beach lowering (courtesy A P Bradbury)

- Regular overtopping has occurred under even relatively benign conditions as a result of increasing water depths at the toe (Figure 3.6). Water percolation seawards of the seawall crest has caused a build-up of pressure to landwards of the seawall.

Figure 3.6 Frequent overtopping causes build-up of pressure on landward side of seawall (courtesy A P Bradbury)

- Increasing water pressures arising from overtopping have resulted in seaward displacement of sections of seawall, which have slid seawards. Although the wall does contain some drainage holes, this does not provide sufficient capacity to relieve overtopping loads.

- Repairs to the structure have been undertaken, which include construction of a rock toe to resist further lowering (Figure 3.7). Note the kink in the alignment of the wall which has arisen due to the structure failure and subsequent seaward displacement of upper wall sections.

Figure 3.7 Repaired wall showing failed realignment. Rock armour has been added at the toe (courtesy A P Bradbury)

Although structural designers usually incorporate a ‘factor of safety’ into their designs in order to reduce the likelihood of such failures, these should not be relied upon when considering reliability and stability years later in the life of the asset. This is because rates of deterioration may have changed over time. For example, varying degrees of environmental exposure may result in abrasion or damage to joints. The rate of change is often a function of the degree of exposure to wave and sediment attack, which in turn is a function of beach lowering. Occasionally changes in landward loading conditions (such as superimposed loading from new structures or vehicles, or pore water pressure) may impact on the structure.

Furthermore, factors of safety used in former times (for example, in the 19th century) would not necessarily have achieved current day standards. Factors of safety are discussed further in Section 5.4. Perhaps more significantly, the role of the structure may have changed since construction with resultant changes in loading conditions. For example, many structures originally constructed as Victorian promenades were simple structures with no significant toe formation; these structures were often constructed at the top of a wide beach and were not expected to withstand wave attack (Figure 3.8). As beaches have fallen, many of these structures have subsequently assumed the role of a seawall without further structural modification. These structures are often increasingly vulnerable to undermining as a result of falling beaches.

Figure 3.8 Promenade wall under construction (courtesy Poole Borough Council)

Specialist engineers should be consulted if critical levels are not already known, or the loadings on the structure are thought to have changed significantly or are likely to change due to other factors such as proposed new development which may vary the loading conditions.

3.4 Planning a management programme for structure toes

This section introduces the problems associated with managing the toe of coastal defence structures successfully. An outline approach is provided to assist asset managers in this task.

3.4.1 Suggested approach for the management of toe scour

For each location being considered, the engineer needs to determine a trigger or 'alert' level for intervention (see Section 3.5). This will be based on key parameters related to structural performance, beach safety and so on. Once this level has been set, a simple assessment for any section of a structure can be determined (initially) on a seasonal (for example, summer and winter) six-monthly basis, possibly supplemented following storm events. The suggested approach is outlined using the eight steps shown in Figure 3.8.

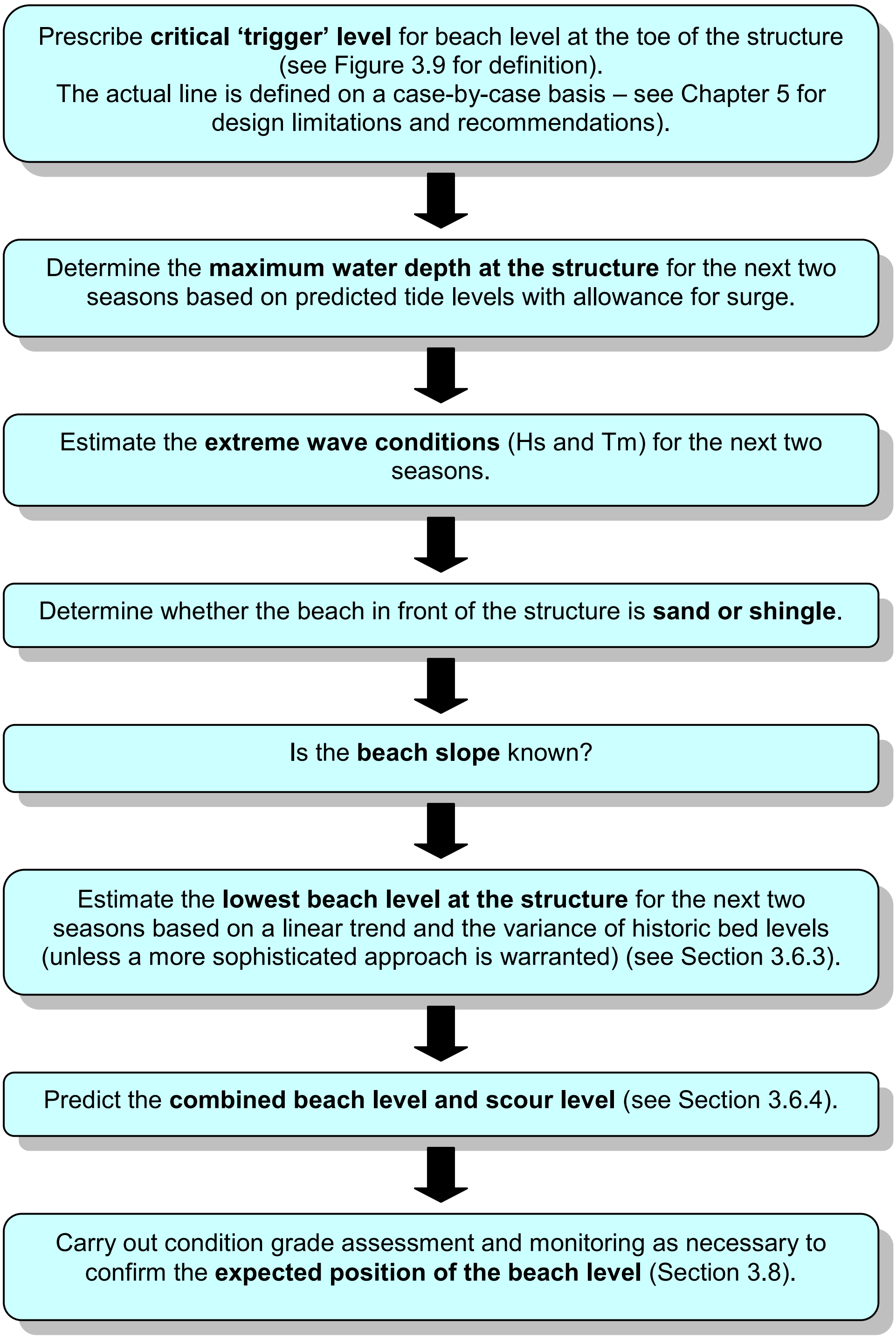


Figure 3.9 Information required to assess toe scour

Once this information has been obtained, the decision process shown in Figure 3.10 is implemented.

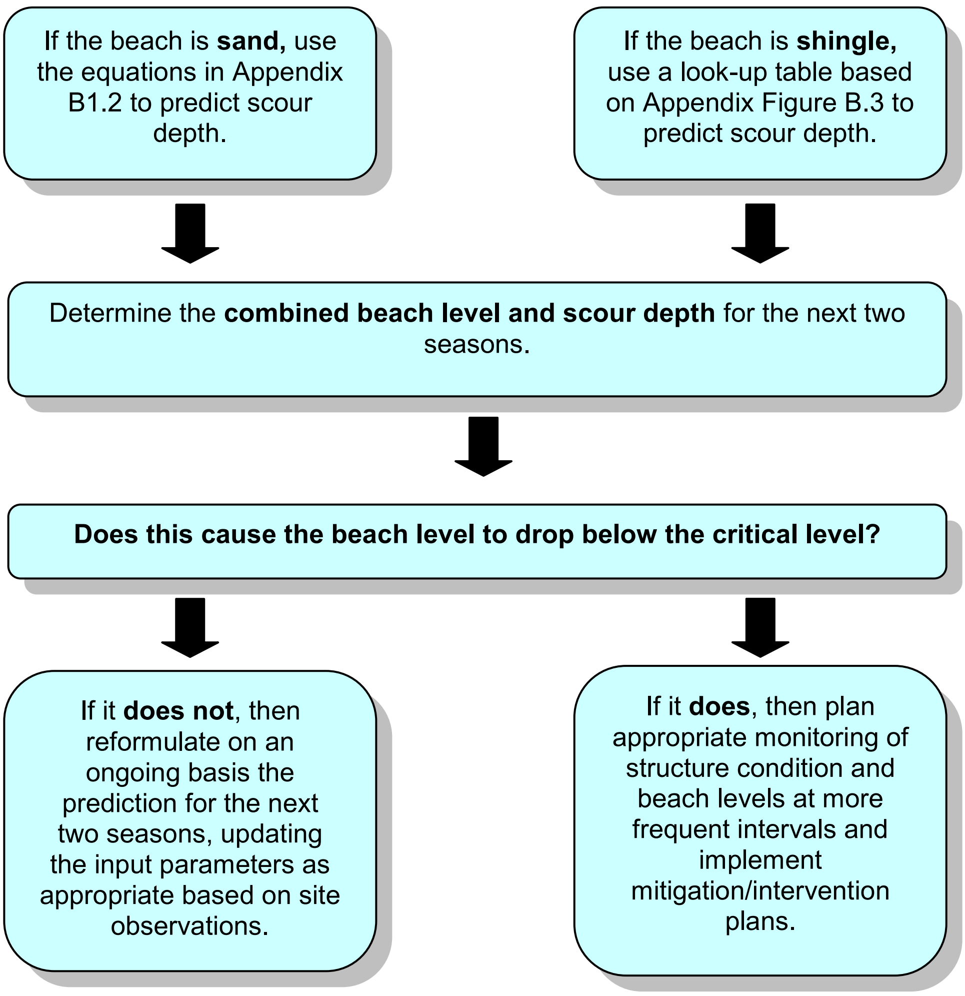


Figure 3.10 Assessment of toe scour decision process

If a more detailed assessment is required for a particular asset, the predictions can be made more frequently, given the relevant input data. With some further definition, this approach can be implemented in a probabilistic assessment (HR Wallingford 2006e).

Certain data and prediction methods are required to conduct this analysis. The following sections describe the data requirements and present the available methods for predicting scour.

3.5 Defining 'critical' and 'trigger/alert beach levels'

Establishment of a critical beach level for a structure requires an understanding of the design limits associated with the structure, that is, knowledge about the physical conditions it has been built to operate within. If sufficient information is known about an asset and the forces on it, it may be possible to determine a beach trigger level at which the probability of failure becomes unacceptably high. In some cases, it may be possible to have a set of trigger/alert levels, each with its own probability of failure. In many instances there is currently insufficient information available to determine such triggers with any degree of confidence. This is the case when as-built drawings are not available, in which case these need to be regenerated (see Section 3.6.2).

A ‘trigger/alert level’ for the beach toe level can be inferred for which the risk of excessive overtopping, or structural collapse due to loss of toe support is unacceptably high. If a beach falls below this level, then intervention will be required to reduce the likelihood of failure given the probable variation in beach level. If the trigger level can be related to some clearly defined feature (perhaps the top of the first groyne pile or a ledge on the seawall) or be clearly marked, it makes rapid assessment of the state of the beach far easier.

The examination of statistics on mean levels at the toe of a structure over a number of years, together with prediction methods and structure geometry information (level of the structure toe), provides sufficient information for the asset manager to determine the physical limits and timescale within which intervention options should be considered (that is, beach re-nourishment, remedial defence works, decommissioning and so on). This concept of the critical level is illustrated in Figure 3.11. The trigger for a more detailed asset inspection could therefore be the exceedance thresholds gained from results of the extrapolation of beach level data.

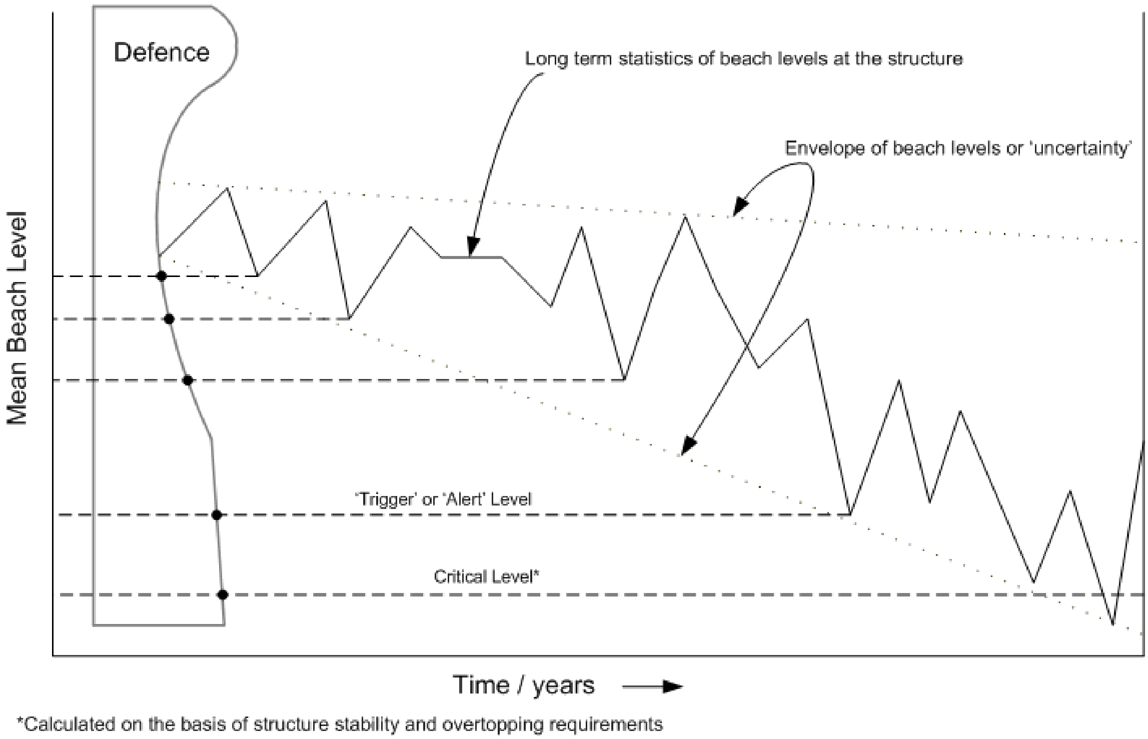


Figure 3.11 Relationship between long-term statistics on beach levels at the structure, uncertainty, and ‘trigger’ and critical levels

Such trigger levels may be based upon visual observations as well as beach profile measurements.

This approach is also valuable where there is insufficient information available to make a confident assessment of structural failure conditions. For example, exposure of the tops of toe piles may serve as a trigger to inspect more regularly when little information is available about the toe construction. Additionally, structural defects such as damage to pile tops or loss of jointing materials may also be used as indicators of a deteriorating structure. If a structure has voids caused by loss of fill due to scour, it will have an increased risk of failure and further investigation should follow – possibly using non-destructive testing.

Once the critical level for the beach at a structure has been established, it is also helpful to define an ‘alert’ level. An alert level is a point at which more detailed or frequent monitoring would be instigated, or intervention options considered and implemented (for example, beach replenishment), before the critical level is reached.

Having defined these trigger levels, beach condition can then be assessed or graded according to the volume or height of beach above the trigger level. The combination of beach level monitoring, time series data, scour depth prediction and the calculation of failure limit state can provide a representation of the relative reliability of the structure.

The following sections discuss the data required to undertake this analysis, how it may be collected and how it can be used. Examples are also provided of linear beach level analyses and trends, along with guidance on the methods of extrapolating data for the prediction of future beach levels.

3.6 Beach monitoring

The state of a beach at a particular time affects its response to loading and its ability to provide protection in the short term. Where a beach forms an integral part of the defence toe, its state will almost certainly require monitoring and maintenance to ensure that the required level of protection is sustained. Beaches and shore platforms dissipate wave energy by friction and wave breaking, thereby reducing the wave energy that reaches a coastal defence. Should beach levels fall, however, higher waves will reach a structure, which may increase the risk of failure.

Regular monitoring of beach levels allows a picture to be built up of how a beach responds to the incident wave climate at a scale of storms, seasons and years, which should inform the management policy. The remainder of this section describes how beaches and shorelines are commonly measured and monitored. This topic is dealt with in more detail by Sutherland et al, (2006a) and Section 5 of the *Beach Management* Manual (CIRIA 2010a).

3.6.1 Beach level and topography

Baseline beach surveys should enable the beach plan-shape to be described adequately and be in context with existing control structures (for example, groynes). This will assist with the understanding of sediment transport patterns and the effect of sediment control structures on sediment transport rates (and therefore beach volume). Such surveys will also assist with the determination of the best places for subsequent monitoring locations, particularly if the orientation of the beach crest(s) varies significantly along the frontage. Contour and three-dimensional (3D) measurements can be achieved by a variety of methods and tools including terrain modelling using results from a real-time kinematic global positioning system (RTK GPS) (Figure 3.12), light detection and ranging (LiDAR), photogrammetry and bathymetric surveys.

Figure 3.12 Beach survey using global positioning equipment (courtesy Channel Coastal Observatory)

The procedures used to process the data are similar irrespective of the method used to collect them. Contours can be simply extracted from the digital terrain models, generated from a grid of elevations.

RTK GPS can now provide a vertical accuracy of ±30 mm and plan accuracy to ±15 mm. For rapid coverage, GPS can be mounted on suitable 'all terrain vehicles' (Figure 3.13). For further information the reader is referred to Section 5 of the *Beach Management Manual* (CIRIA 2010a).

Figure 3.13 All terrain vehicle fitted with GPS equipment for coastal topographic surveying (courtesy of Worthing Borough Council)

Digital terrain models of beaches are produced routinely for numerous beach monitoring programmes. These surveys can be used as an excellent indicator that the structure toe is becoming under increasing loading and that more detailed attention should be given to assess increasing risks. The primary value is to identify erosion hotspots.

The sequence of colour-coded beach elevation plots shown in Figure 3.14 demonstrates clearly that the beach is becoming smaller over time. Note that the rate of change has increased with time and that the survey frequency has been increased to capture the more rapid changes, as the structure toe becomes more vulnerable. Surveys identify considerable beach narrowing at the updrift end. Much of the structure remains well protected and vulnerable areas of the structure toe can be isolated to a frontage of about 50 m, where detailed monitoring efforts can be focused.

It is recommended that detailed profile analysis should be conducted in conjunction with structure stability assessments, where the risks appear to be increasing in erosion hotspots. Changes in beach width can be also indicative of changes in beach composition and monitoring of foreshore type is important in this respect.

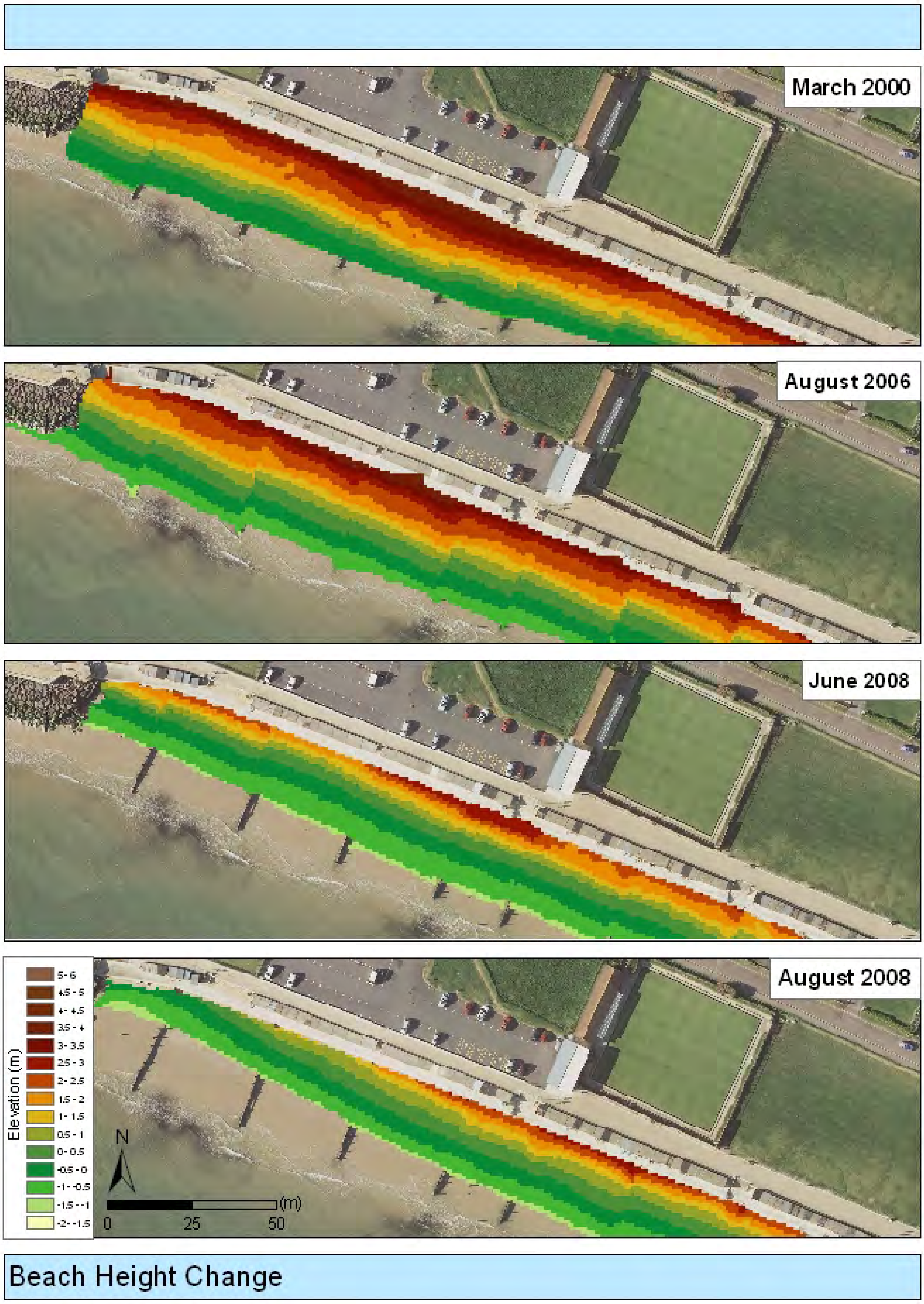


Figure 3.14 Digital terrain models showing beach evolution adjacent to seawall with changing structure alignment (courtesy Channel Coastal Observatory)

The net sediment drift direction at this site is from west to east, and this is reflected by a declining beach volume at the structure toe at the updrift end of the beach. The pattern shown demonstrates clearly the influence of an updrift change in structure alignment, which has an adverse effect on beach supply to the western (updrift) end of the beach. This change in alignment is a typical cause of falling beach elevations at a structure toe. Changes in structure alignment are highlighted generally as potential hotspots for beach loss at the structure toe and these should always be examined carefully for signs of change.

Migration of the mean high water spring (MHWS) beach contour can serve as a useful indicator that a structure toe is becoming under an increasing frequency of loading. Figure 3.15 illustrates the migration of the MHWS contour over a period of less than two years, yet shows a significant trend of migration towards the wall.

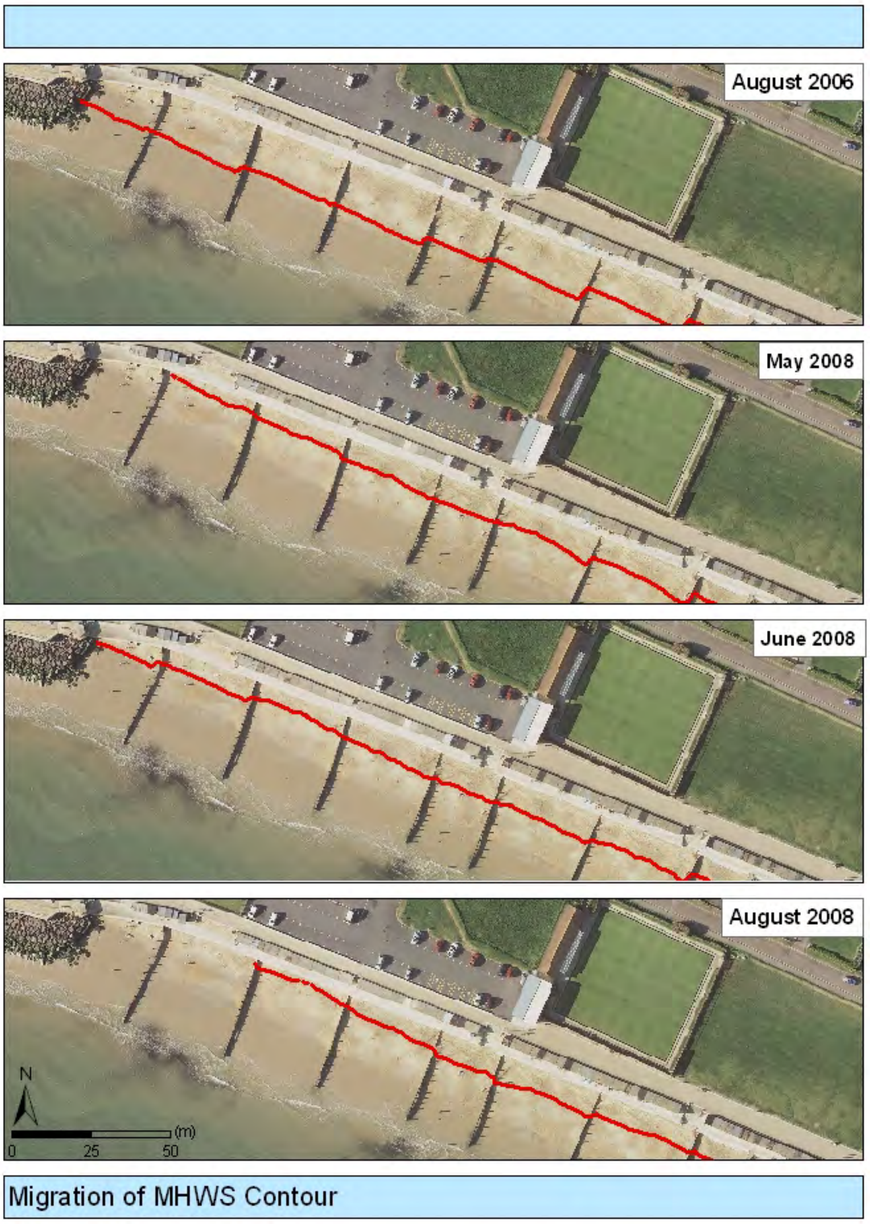


Figure 3.15 Migration of MHWS contour towards seawall (courtesy Channel Coastal Observatory)

The location of the MHWS contour suggests that 50 m of the structure frontage may be regularly under direct wave attack, in accordance with the tidal cycle. The survey frequency increased from April 2008, triggered by the rapid movement of the beach contours towards the wall (Figure 3.16). The pattern indicates that standing water will have occurred at the seawall at several stages during this period, over the upper part of the tidal cycle. Any wave activity during these periods will certainly have impacted on the wall. Parts of the wall will be offered some protection by the beach over most of the length monitored, but it is clear that the frequency of attack at the western end has increased over time. Such observations should act as a catalyst for more vigilant observations of structure toe performance. This is generally a useful indicator for assessment of overtopping risks that may be affected by toe scour.

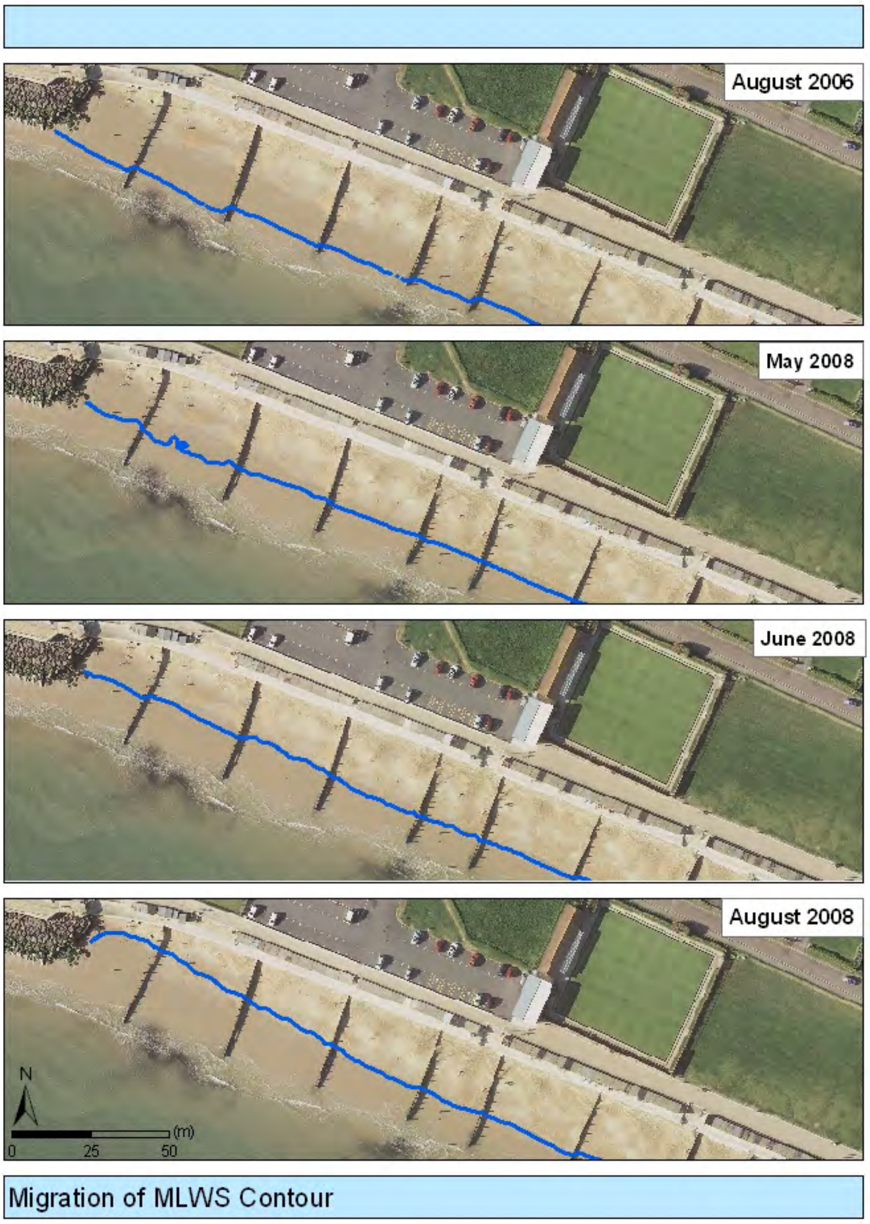


Figure 3.16 Migration of MHLS contour towards seawall (courtesy Channel Coastal Observatory)

Tidal range is significant when considering impacts of the beach on toe stability. Migration of the MHWS contour is a universally applicable indicator, irrespective of tidal range. Where the beach is narrow, typically when the tidal range is small (<3 m), the MLWS contour may also be a useful indicator. When this contour approaches the wall, the frontage will become subject to very regular wave loading and minimal support is provided to the lower part of the surface emergent element of the structure; in many cases this may mean that the structure foundations are fully exposed over the entire tidal cycle. Where available, the MLWS contour is very valuable as an indicator of the maximum beach width fronting the structure. Such information is not widely available, simply because of the practical difficulties of conducting surveys over the lowest part of the tidal cycle and is generally less useful in areas of large tidal range where the low water beach is wide.

3.6.2 Beach profiles

Beach profiles are usually surveyed perpendicular to either the shoreline or a predetermined baseline. They are used to quantify beach response to storm events, sediment recovery rates, long-term volume changes and the potential envelope of cross-shore elevations. When combined with nearshore bathymetric surveys, morphological changes across the full zone of wave influence can be assessed. Rapid appraisal of profile data (often in combination with cross-shore empirical parametric models or trigger levels) is vital to determine whether management works are needed and for establishing some of the design constraints that will have to be worked with.

The location of profile lines should be considered carefully. The ‘density’ of the lines should be sufficient to provide an adequate representative coverage of the beach for the purpose prescribed. Allowance should be made for nearshore features such as bars, banks and troughs, which induce localised variations in beach response. For the purposes of structure management, the profiles will generally need to be closely spaced (30–50 m). Where the effect of groyne systems is to be considered, it may be necessary to survey lines on either side of the groynes and at the centre of the bays. However, it may only be necessary to ‘sample’ survey within selected groyne compartments (due to the repeatability of the bay plan shape in a groyne field). Profiles taken adjacent to structures are of great value if information is needed on scour, but the exact line of the survey must be located and repeated exactly to ensure comparability.

The extent of a particular survey line is also important. The boundaries between the mobile beach toe and bed rock, and the crest and seawall/cliff should be identified where appropriate. The landward end of the survey line should extend to the structure. The seaward end should extend as far seawards as is practical; this may extend at least to the level of mean low water spring (MLWS) at sites with a small tidal range (<3 m).Supplementary shore parallel profiles are of great value – particularly at the toe of defences and the crest of beaches – to provide both a fuller coverage of data and as a cross check.

Key features to be measured on profiles

A number of key features should be surveyed on or relative to each profile:

Tidal elevation variables (relative to structure details)

These data are used as an indicator of the potential frequency of loading of the seawall under wave conditions. Structures with toe foundations that lie within the range of the intertidal zone will be subject to regular wave attack at the structure toe.

Seawall cross-section

This should describe the seawall geometry:

- crest elevation of structure;
- level of structure foundation base;
- level of bedrock base at the foundation position;
- depth of penetration of any toe piling or other toe detail;

- structure construction materials:
  - concrete grades;
  - reinforcing detail;
  - pile gauge and type;
  - fill type within seawall.

Details of the structure geometry, construction method and materials are crucial for the determination of structural stability assessment of the structure; this is a basic requirement for the determination of alarm and intervention levels. Despite this, such information is not currently available at many sites. This is a major problem for long-term structure management.

In some cases, adequate as built scheme drawings will provide all the relevant detail, but in many instances these are not available. Structures may be of any age, perhaps over 100 years old when as built drawings did not form a standard part of the design and constructions process. In many other instances, design or as-built drawings have been lost or destroyed.

Efforts should be made to store historical drawings, since these may be of significant value to structure management several decades after construction. Whereas built drawings do not exist, their regeneration is a most valuable investment for long-term structure management. The basic geometry can be measured or surveyed to provide the surface emergent structure profile. Details of the structure toe can be more problematic to determine as these are usually buried in either bedrock or beach material. Careful excavation of trial holes at the structure toe or other soil investigations can provide some of the required detail; these should ideally identify the elevation of the structure foundation base and also the depth to bedrock (if this is at a lower elevation). It may not be practical to measure such data at many sites where the bedrock is beneath thick sediments. Toe elevation data should be combined with tidal elevation data to determine the risks arising from frequency and depth of inundation of the toe area in combination with wave loading.

An alarming number of promenade type seawalls dating back over the last 100 years have been constructed on a perched foundation, which has a base well above low water and which does not close onto bedrock. The stability of such structures is entirely reliant upon the presence of the beach fronting the wall. Numerous failures have been observed over many years, usually during or following storm events when the structure toe is exposed. In the event that beach material is eroded from the structure toe, the foundation can become rapidly undermined (within a few hours). The zone between low water and the foundation base is particularly vulnerable, as water can be pumped by wave activity to scour material from beneath the foundation. Rapid loss of internal fill material can result, followed by collapse of the concrete deck membrane.

It is unsatisfactory to attempt to manage a structure without scheme drawings that detail the structure toe construction. A conservative assessment must be made based on observations and intervention measures planned in accordance with this. The safest approach is to assume no foundations are present below known levels and to determine trigger levels on the basis of these.

Where the structure toe is of a piled construction, reconstruction of the drawings is more difficult since it is neither practical nor desirable to excavate into bedrock material. A number of specialist non-destructive testing methods for example sonic testing can be used for determination of the pile length of both concrete and steel piles. This can be used to establish the depth of penetration of the toe piles. This information is needed to assess the risk of overturning, undermining or structure sliding.

Many structures have been constructed with a concrete membrane that overlies loose local material – typically local beach material or some other fill. Coring through the concrete membrane at various locations along the structure will enable identification of the type of fill material and whether voids are present beneath the surface. Cores holes should be grouted following testing to ensure that the very method of investigation does not cause a problem itself.

Bed rock profile from the seawall to seawards

The depth of the bedrock, its profile and its position relative to the structure foundations has a significant effect on the performance of the structure toe. Bedrock may not be accessible at many sites due to the thickness of surface sediments.

The bedrock profile is used, where available, to identify the solid surface adjacent to the seawall that would be exposed in the event of all beach material being removed. In some instances, changes in bed level beneath this surface arising from erosion can also be significant; this is typically the case at sites where soft or even medium clays are exposed. Such information is often difficult to collect and may not be available at many sites due to the thickness of sediments

Where construction or geotechnical records are not available, small trial holes excavated adjacent to the seawall should be used where possible to identify the depth to bedrock at the structure bedrock intersection. The beach may dry onto bedrock at some sites and this intersection of beach and bedrock can be used to provide the basis of a linear interpolation between the structure and the toe to provide an indication of the bedrock surface elevation – though this is not a wholly satisfactory method. Exposure of bedrock may occur at other locations from time to time and this can be used to build up a more complete picture of the bedrock profile over time.

Procedures linked with standard specifications for some of the regional coastal monitoring programmes require surveyors to feature code survey data; this will provide indications of bedrock elevation where this is seen on each survey. Each point has 3D coordinates and a surface type attribute. Feature coded survey data can be extracted from a number of surveys and combined over a period of several surveys to provide the basis for development of a primitive surface model of the bedrock surface. These data can be supplemented by surveying elevations of spot heights of bedrock, determined during excavations for the repair and construction of structures such as groynes. For example, this has been used effectively at Bournemouth and Milford-on-Sea to provide a more detailed bedrock surface. Where these data have been captured, it is possible to measure absolute beach volume changes by using this as a master profile rather than changes relative to an arbitrary surface (Figure 3.17). Such an approach may not, however, be feasible at many sites.

Profile surveys

Regular beach profile surveys can be used to develop an indication of changes to protection of a structure toe. All of the key descriptors identified in Section 3.5.2 are shown on the simple plot presented in Figure 3.17. The long-term profile envelope indicates the lowest and highest points that the beach has reached over the course of the monitoring.

The example shown represents changes over a 20-year monitoring period. These data can be used in combination with a trend analysis of beach cross-sectional area above a defined surface (master profile) (Figure 3.18). Where available, the master profile should be defined by the bedrock profile, thereby enabling absolute changes in beach volume to be determined; this profile cannot be defined at many sites and only relative changes can be measured. An at-a-glance assessment of the relative state of the beach to historical conditions is provided by comparing the most recent survey with the profile envelope and the mean profile.

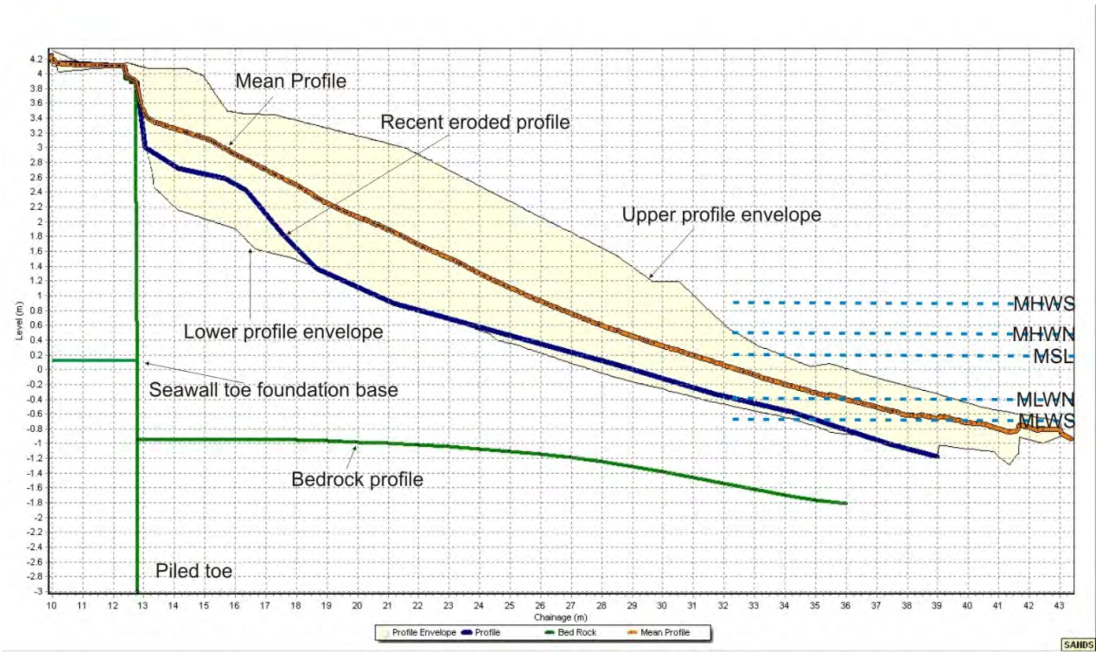


Figure 3.17 Beach profile and structure descriptors used in structure toe analysis (courtesy AP Bradbury)

3.6.3 Predicting trigger levels from monitoring data

The variations in beach levels near coastal structures at timescales of the order of tide to a year are the accumulation of the residual changes that occur during each tide. 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.

A variety of types of profile analysis can be conducted as the time series grows. Analysis of lengthy long-term trends provides the generally most useful indicator of the beach state. The record length of surveys is significant and several years of data (ideally at least 10) are required to give confidence to assessments.

Figure 3.18 demonstrates a lengthy time series during which the general trend is of erosion, but with periods of recovery. A series of storm events of varying intensity punctuate the long-term profile trend and these are associated with defined combinations of wave and tidal conditions. Wave data can ideally be derived from a nearby wave buoy in shallow water (see Figure 3.20). Alternatively, hindcast modelled offshore data can be transformed to the nearshore zone. Several of these storm conditions are highlighted in Figure 3.20. Interestingly, the most severe hydrodynamic conditions are not always associated with the greatest damage or beach loss. A number of the notable dips in beach cross-section are associated with swell wave or bimodal wave conditions that would not generally be considered to be damaging events.

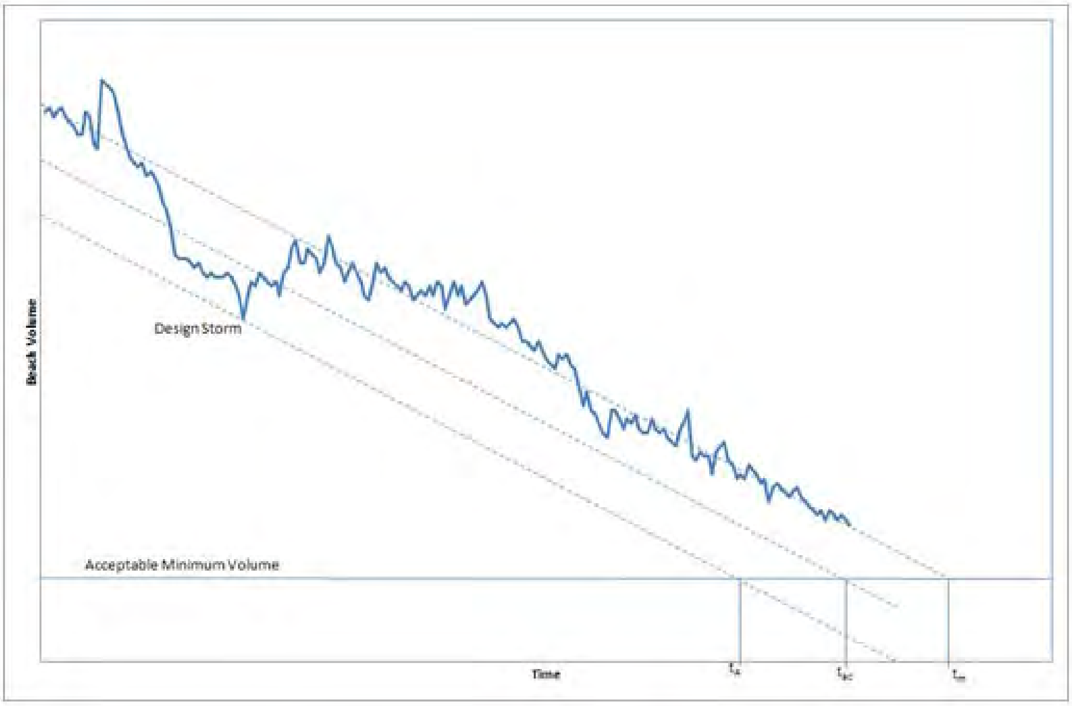


Figure 3.18 Beach profile cross-sectional trend analysis

A trend line is fitted through the data set, indicating the mean profile. The profile envelope is represented by lines parallel with the mean profile. The maximum departure from the line is associated with the design storm, which denotes the maximum departure from the envelope line. Application of a storm response line provides an indication of the anticipated maximum reduction in cross-section on a defined storm event. This is based on conventional topographic surveys and does not consider additional lowering that may take place during the storm but which is not detectable by conventional survey techniques. Additional allowance must be made for such changes (see Section 3.6.4). Using the critical cross-sections determined for each site (Section 3.5), a critical acceptable volume trigger can be added to the graph. Linear projection of the trend, envelope and storm damage lines can be used to estimate the potential timing for intervention.

Once a trend of position against time has been established, the trend can be extrapolated beyond the date of the last data point and into the future. The results of an extrapolation should be interpreted in light of the underlying principles of geomorphology and sediment transport (that is, tempered with what is actually realistic). 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.

The shape of individual profiles can provide a useful indicator of beach performance and its interaction with the structure. A shingle beach will normally form a distinct berm with a well-defined run-up crest on a healthy beach, where the dynamic equilibrium profile is able to develop fully (Figure 3.19). Where there is inadequate cross-section available for the dynamic equilibrium profile to form, the profile may be characterised by a planed off crest or even a dip at the seawall intersection. The crest invariably forms at a lower level. The implication here is that the beach and structure intersection is under wave attack. The planed surface arises from wave reflections from the seawall increasing the backwash. This observation is generally considered to provide a preliminary indication that the structure is undergoing increased loading and might merit more regular inspections.

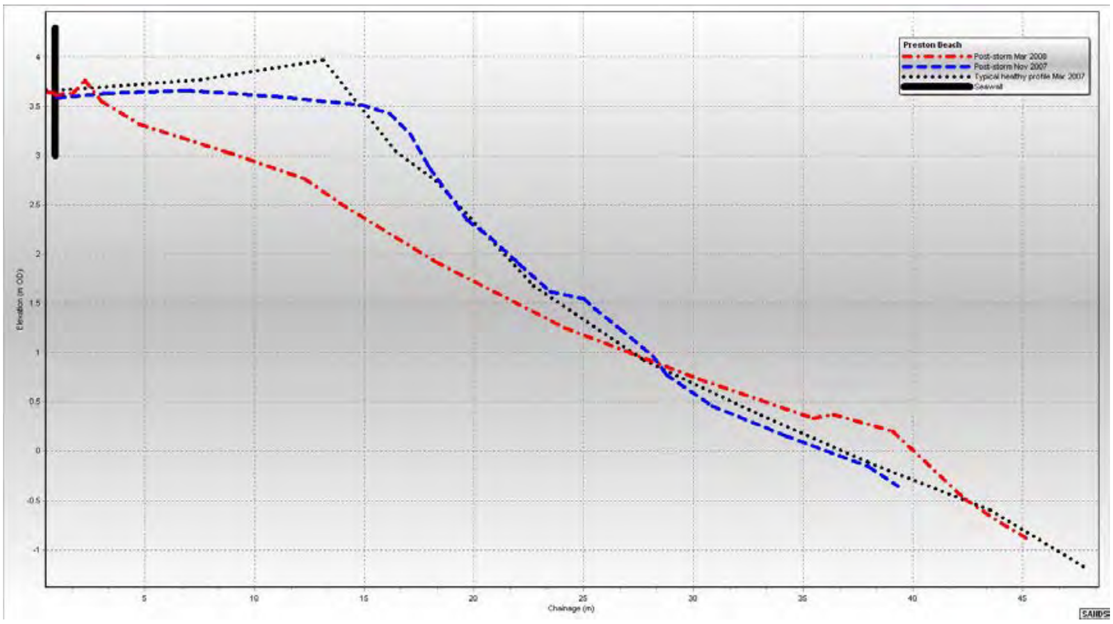


Figure 3.19 Beach profile evolution in the presence of seawall (courtesy Channel Coastal Observatory)

Post storm surveys (modified from CIRIA 2010a)

This section is concerned with the analysis of beach levels close to the toe of a structure at seasonal and shorter timescales. The prediction of beach levels on these timescales is important as they provide the initial conditions for the calculation of toe scour during a tide or storm.

The beach response to storm conditions may have both long- and short-term implications and regular post storm surveys should be conducted where possible. In the short term the surveys may act as a trigger for intervention, when the response is measured relative to defined critical conditions (see Section 3.5). They may also act to identify an alarm state, where preparation for intervention may be considered. Under some circumstances, beach lowering may reach a point where the profile is unlikely to become restored to a healthy condition naturally, as suggested by Powell and Lowe (1994). This may result in undermining and destabilisation of structures such as groynes or seawalls. The failure of the beach to recover following storm events is often due to the loss of material in longshore transport, which is accelerated as material is drawn into the subtidal zone. Alternatively, reflections from a vertical structure may make it difficult for a beach to reform. Where there is a limited longshore feed beach losses may occur from the updrift zone. It may be desirable to conduct post storm surveys in any of these circumstances.

It is beneficial to collect hydrodynamic data in conjunction with post storm survey data and to present these as an event time profile of hydrodynamic conditions (Figure 3.20). This should normally describe the whole of the event cycle including the build-up, storm peak and decay, and should where possible include integrated parameters for H_s (significant wave height nearshore), T_z (zero-crossing period), T_p (peak wave period) direction and spectral data. These should be supported by tidal profiles and wind data when possible.

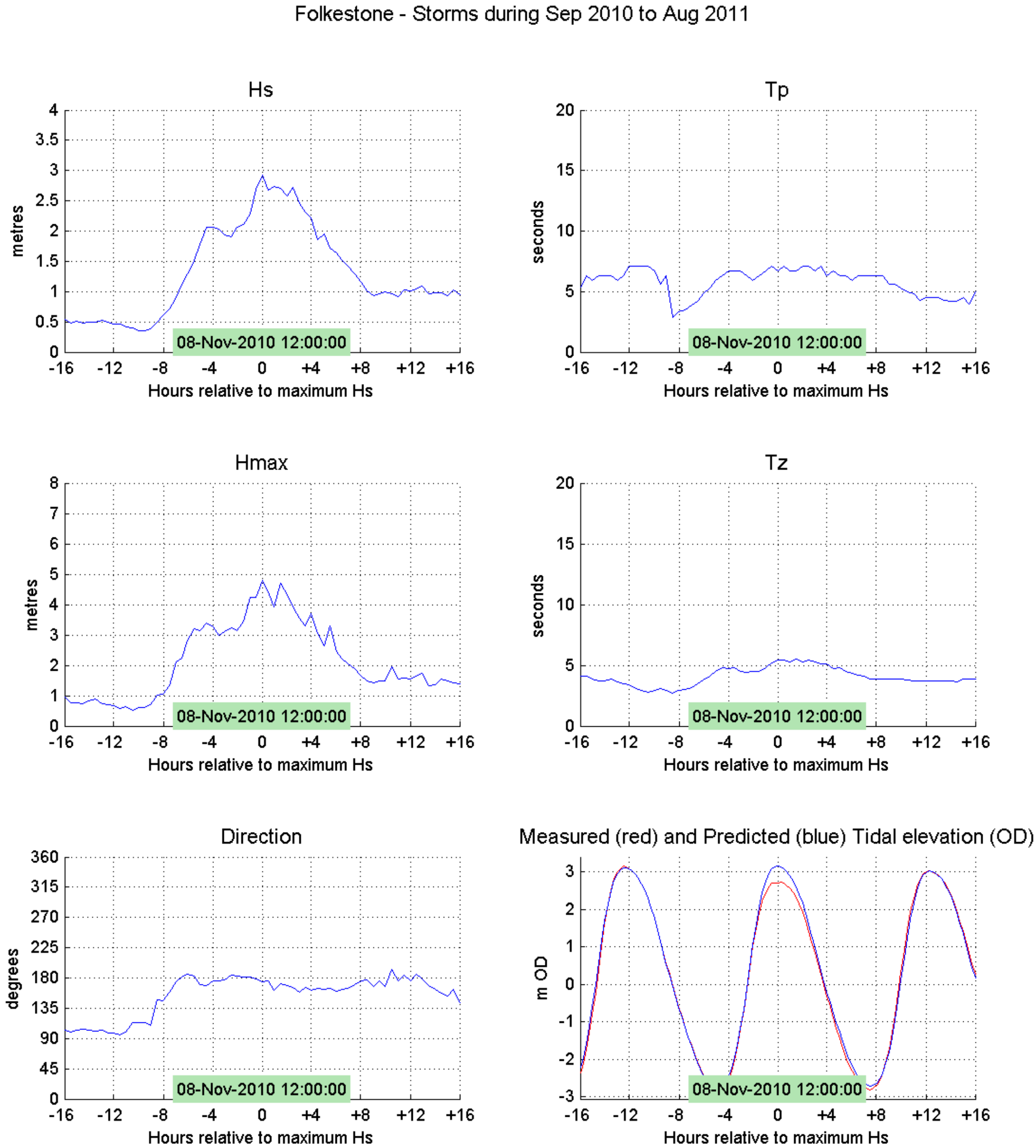


Figure 3.20 Typical hydrodynamic conditions used in the analysis of a storm event (courtesy Channel Coastal Observatory)

In some instances, profile surveys are conducted only following the most damaging of storm events, primarily to quantify damage to defence systems. However, it is also beneficial to conduct surveys following more regularly occurring storm events, perhaps with return periods of about 1:1 year. Although damage might not be expected following such conditions at most sites, it is advantageous to develop an empirical framework of measured responses in order to provide a framework of data on either side of the damage thresholds. This approach may enable the threshold conditions that cause damage and critical conditions to be defined more accurately, and also develops confidence in management procedures. This approach has been adopted within the southern regional coastal monitoring programmes, where post storm surveys are triggered by exceedance of defined threshold conditions at the network of wave buoys. The storm threshold used is typically a 1:1 year return period event, at any water level.

Events are triggered initially though an automated system of text and email alerts provided by the real-time wave buoy network. Further consideration is given to the event including a brief consultation with the relevant operating authority prior to mobilising a survey team.

Post storm surveys have been used also to calibrate, validate and extend empirical predictive frameworks used in beach management. For example, validation has been conducted of empirical models for gravel beaches using the framework proposed by Powell (1990).

It is desirable to conduct the post storm survey as soon after the event as practicable. This often means that a few days may elapse before conditions at the site are sufficiently safe for surveys to be conducted. Even then it is rarely possible to achieve the desired seaward limit of profiles because of conditions. On this basis the upper beach typically above mean low water neaps (MLWN), or perhaps higher, can be surveyed and a limited range of characteristics can be measured. It is normal to relax survey specifications in order to conduct surveys quickly after the storm event. Other considerations for planning of post storm surveys are sample locations and the performance of historical hotspot locations.

Shore parallel profiles of the beach structure intersection

Shore parallel profiles are extremely valuable tools for monitoring beach structure interaction. Profiles are conducted by measuring elevations of the beach and structure intersection (see Figure 3.12). The example presented shows the spatial and temporal variability of the beach structure intersection level (Figure 3.21). Each elevation is measured at predefined locations along the seawall; in this instance these are defined using kinematic GPS and a data logger with a stakeout control file to ensure that each point is precisely relocated on each survey.

The example presented covers a frontage distance of approximately 200 m. The time series covers a period of about seven months. It demonstrates not only the vertical extent of changes but also indicates how quickly these changes may occur. The temporal intensity is extremely high at this location; this reflects the fact that the toe of this structure was considered to be very vulnerable. The key variables plotted are the foundation base elevation, bedrock elevation and tidal levels. The structure toe becomes increasingly vulnerable as the beach elevation falls closer to the foundation base.

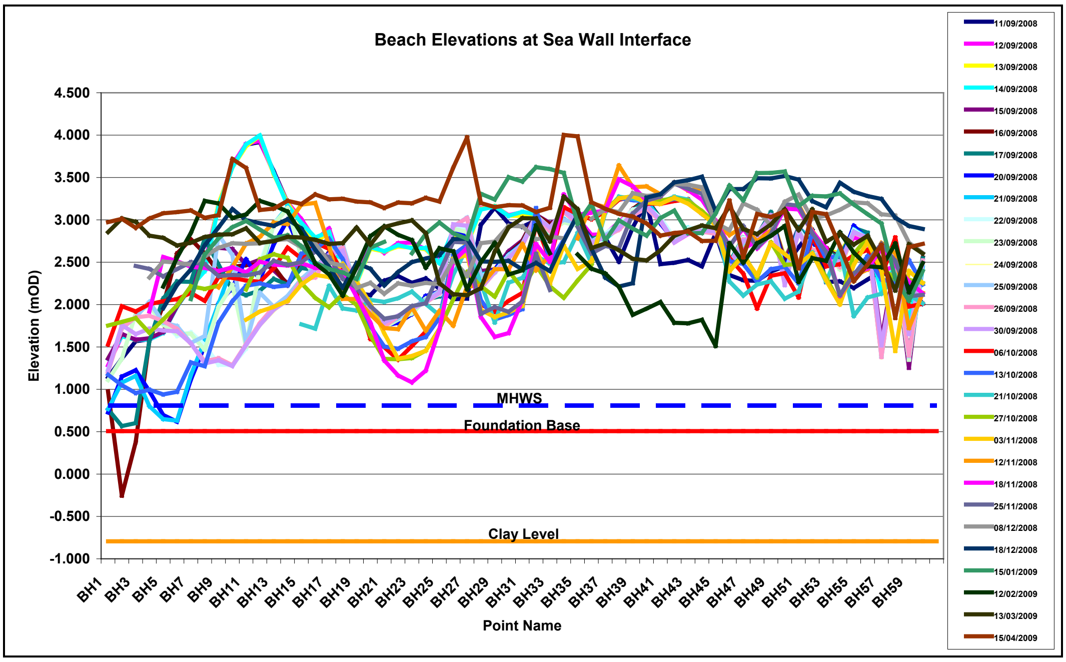


Figure 3.21 Temporal and spatial distribution of beach and structure toe intersection elevations (courtesy Channel Coastal Observatory)

An alternative approach that would achieve similar results can be achieved requiring minimal equipment. Periodic measurements of the 'dip' or vertical distance from the top of the wall to the beach using a tape can be made; these can be made repeatable by marking predefined locations on the seawall.

An alternative representation of the evolution of a single point elevation at the toe of a structure in Mablethorpe, Lincolnshire, is shown in Figure 3.22.

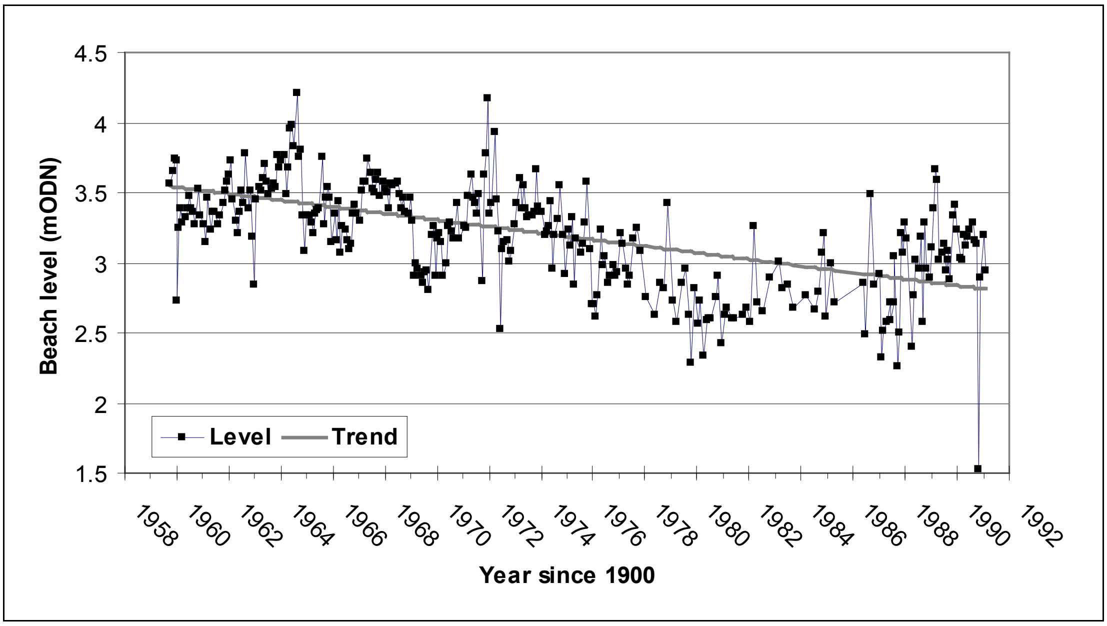


Figure 3.22 Temporal distribution of beach and structure toe intersection elevations (courtesy HR Wallingford)

3.6.4 Monitoring of scour depth

As beach levels vary throughout a tidal cycle, together with varying wave conditions, the development of toe scour is a dynamic process, highly dependent on the water level at the wall as well as the incident wave conditions. In areas of varying tidal range and wave climate, the development of a scour hole will be a variable process with periods of erosion followed by infilling and perhaps even general accretion of bed levels (Powell and Lowe 1994). The scour hole itself may therefore be a short-lived feature with no obvious evidence of its existence after a storm has declined and infilling has taken place as the tide recedes.

Significant damage can arise over a short period of time during which excursions of beach level beneath critical levels may occur. Depending on the construction type, falling beach levels over periods of just a few hours may have a significant effect on the risk of structure failure or major damage. Those structures that are most at risk under these circumstances are those constructed with an elevated foundation level (usually above MLWS), where there is significant separation between the foundation base and the underlying bedrock, and where a concrete or other hard outer structure shell encases granular fill material. This occurs at many sites within the UK.

Hence, there is a need to be able to predict the maximum vertical excursion of the scour hole during storms, as well as the more widespread and longer term processes that cause the lowering of beach/shore platforms. This is important both in the design stage of a coastal structure and in its subsequent monitoring if the risk to the future integrity of the wall is to be fully understood and timely remedial action undertaken.

As scour is frequently short-lived, the biannual beach profile monitoring typically carried out around the English coastline is unlikely to coincide with a major scour event. While there are initiatives to carry out post storm event beach profiling (Section 3.6.2), this will not capture the response at the peak of the storm. Evidence supplied by data from bespoke scour monitors suggests that a significant fraction of a scour hole can fill in within a few hours of the peak of a storm. Therefore even regular beach profiling with a spacing of a few weeks, supported by profiles collected within a day or two of each large storm, may not be enough to capture the transient phenomenon of toe scour in the field.

The deployment of scour monitoring systems that remain on-site, just in front of a toe structure operating at all water levels, for periods of weeks at a time may currently be the only realistic way of assessing the site specific variability of a beach surface with time. Figure 3.23 shows scour monitors deployed on a beach. Scour measurements can be conducted using, among other things, the following devices (Sutherland et al. 2007) installed at locations where the beach at the toe of the structure is exposed at low tide:

- HR Wallingford's 'Tell Tail' scour monitoring system (Figure 3.23) is based on a linear array of omni-directional motion sensors buried in the seabed 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 (Pearce et al. 2006; HR Wallingford 2006a) and at Blackpool (Sutherland et al, 2006a).
- Linear array of electrical conductivity meters (Ridd 1992; Cassen et al. 2005), which rely on the fact that seawater 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 developed by 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 developed by Erlingsson (1991) which used an array of infrared transmitters and backscatter detectors.

Installation and operation of scour monitors is a specialist activity and seems unlikely to find its way into routine monitoring programmes. As scour monitors are likely to be deployed for a period of weeks (or longer), a monitoring strategy could be implemented that looks for the bed lowering to the point at which short-term fluctuation from the slowly varying mean level could destabilise the defence asset. For this the likely scour depth for a given storm would have to be estimated, plus the depth of scouring that would create a risk of failure. The monitoring could then take place at least twice per year and a more detailed study or remedial action undertaken should the data show beach levels dropping below pre-determined values.

Figure 3.23 Scour monitors in operation on a shingle beach to obtain data on within tide changes in bed levels (courtesy HR Wallingford)

The somewhat limited number of historical deployments of scour monitors means that empirical statistics of performance are limited and estimation of scour is difficult. None of the deployments have taken place during particularly severe conditions and the application of results to date should be used with some caution. A reasonably typical plot is shown (Figure 3.24), which indicates intertidal fluctuations of about 0.6 m, under fairly benign wave conditions (H_s = 1.5 m). The type of pattern observed, on a sand beach, seems reasonably typical of other sites, with maximum scour occurring over the high water period and with a cyclic return over the low water period.

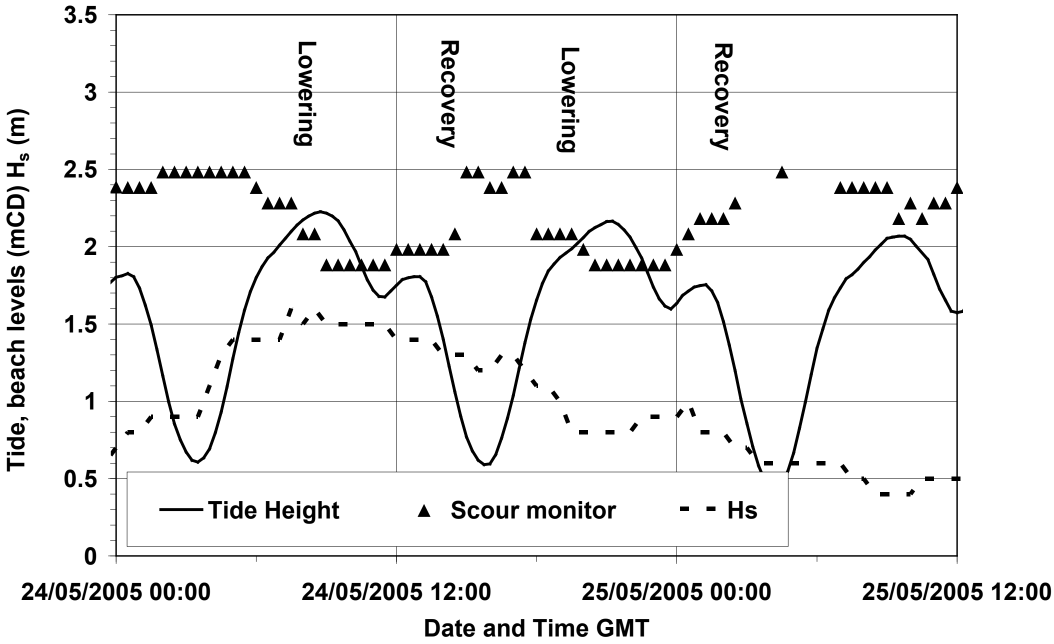


Figure 3.24 Fluctuations in sand bed levels over a tidal cycle (from Pearce et al. 2006)

Complementary laboratory work (Pearce et al. 2006) suggests similar relationships between scour depth, wave height and water depth to those shown in Figure 3.24. While the science is not yet well advanced, the preliminary empirical framework suggested by Pearce et al. (2006) appears to be supported by field scour measurements.

It is suggested that, for the purposes of monitoring, a preliminary site-specific assessment of scour potential is made using the following dimensionless formula. It is suggested that maximum scour potential equates to:

$$S_t / H_s = 0.8$$

where:

$d_t/L_m = 0.015$

S_t = maximum scour anticipated

H_s = significant wave height nearshore

d_t = depth of water at toe

L_m = wave length.

These should be calculated for extreme conditions anticipated at the site. Although based upon dimensionless laboratory tests, the significant wave height used in the development of the formula was measured in deeper water than the toe. This might typically be in the range 8–12 m depth.

Application of this framework to a reasonably typical south coast site under extreme conditions of H_s = 4 m and T_m = 7 s suggests a maximum scour of 3.2 m should occur with d_t = 1.2 m. This suggests quite alarming results which should be considered in context with other controlling variables at the site. In particular, the scour depth may be restricted by the actual depth of mobile sediment above bedrock. The actual depth of sediment may often be the governing limiting factor.

It is recommended that both the theoretical formulation and the other practical governing factors should be considered together to reach an assessment.

Actual scour monitoring has identified a maximum scour depth of about 0.9 m and this should be considered a minimum expectation at most exposed sites, with the theoretical scour potential also considered in context with the site specific geology.

Video system monitoring (based on CIRIA 2010a)

Video systems such as Argus may be used to identify changes to bed elevations in the intertidal zone, while the beach is submerged. Shore-based video systems can provide automated data collection over wide spatial and temporal scales, and during unfavourable weather conditions such as storms. Furthermore, the measurements are non-intrusive and therefore do not affect measurement results.

The Argus system (Holman et al. 1993) uses unmanned video stations, located at conveniently elevated remote locations to monitor the nearshore hydrodynamics and morphological processes, with high time and spatial resolution. The system offers cost-effective monitoring of the beach evolution, erosion and sediment movement for beaches up to 3 km in length. Measurements of morphology across the surf zone and an estimation of surface currents and wave characteristics can also be derived. The spatial resolution derived from the Argus cameras is similar to that of numerical simulations, allowing effective comparison and evaluation.

More than 50 Argus systems have been installed on a worldwide basis including a number in the UK. These cover a range of sites and systems including:

- barrier beaches (Slapton);
- surf reef performance and environmental assessment (Boscombe);
- nearshore ebb deltas (Teignmouth);
- beach structure interaction (Cleveleys);
- evolution of beaches in the presence of nearshore breakwaters (Sea Palling).

Argus camera systems are particularly effective at capturing change within the intertidal and shallow subtidal zone. This zone is the most challenging zone for data collection using alternative techniques and is also the most significant zone for beach change, when considering the toe of structures.

The images obtained by the Argus video system provide spatial and temporal resolution that enables the study of sediment transport patterns in groyne fields and the effect of storms. Profiles derived from an Argus system are compared with those using traditional methods (Figure 3.25). The large variability and advantage of the extended length of the Argus survey through the dynamic zone should be noted.

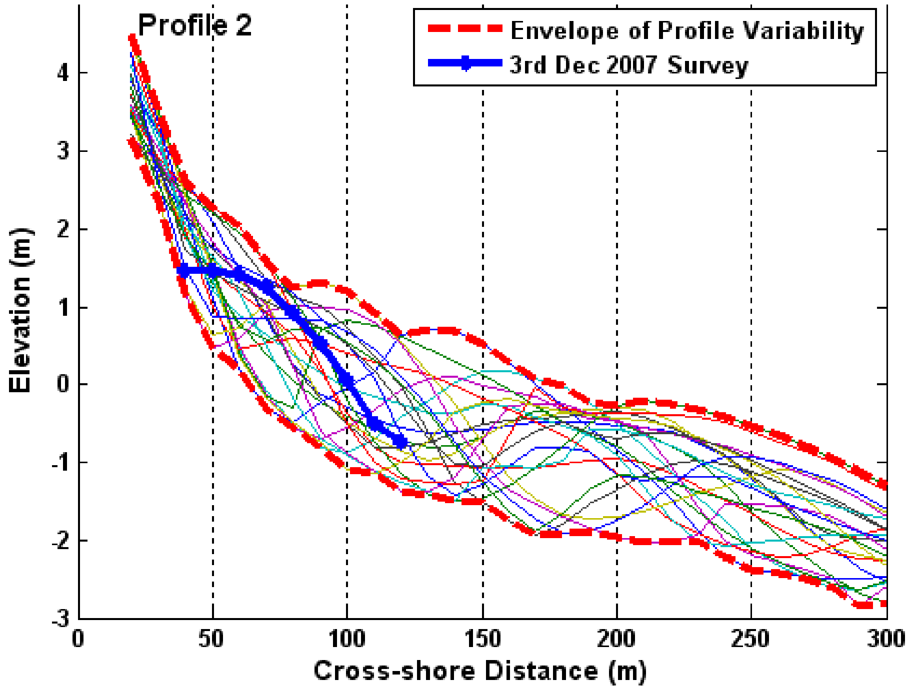


Figure 3.25 Comparison of traditional profile with Argus profile envelope (from Green and Illic 2009)

Argus is not able to detect changes on those sections of beach that are always surface-emergent and its application is limited in this context when considering changes to elevations of threshold conditions for storm damage in the subtidal zone. As the system is based on photography it can only function during daylight conditions, though this is typical of many monitoring techniques. Although the impression is given in much published literature that these systems are automated, considerable calibration and regular checking is required to ensure that the systems have not been affected by external elements such as wind. A commitment to regular maintenance and management is required.

Not all sites are suitable locations for installation of these systems. The primary requirements are for suitable elevated camera mounting locations, an appropriate power supply and access to a broadband network. High buildings may sometimes provide suitable locations, but these need to provide a good view of an extended section of beach. Towers are sometimes erected to mount cameras. Sometimes valuable data collected during storms cannot be analysed due to the conditions obscuring the images. Cameras suffer from salt spray on the enclosure lens and require regular cleaning.

As access to the cameras may be difficult, the enclosures can be fitted with a wash/wipe system to reduce the need to clean the lens manually. Enclosures made from aluminium alloy suffer from corrosion, allowing water ingress into the enclosure. Alternatively enclosures can be made from stainless steel and include clips for access rather than screw fittings. The accuracy and resolution of the images depend on number of cameras used, their height, angle and sampling frequency, and the availability of field data. They also depend on environmental conditions such as sunlight, water reflection, fog, and sea salt spray.

Successful applications of the system have been made in academic investigations. Some of these have successfully transferred the outcomes to management applications, but there is more scope for development of such automated analysis techniques.

3.7 Beach depletion and foreshore down-cutting

The performance of a beach depends largely on the volume of material present and the limits to its plan and profile changes. Where there is a net loss of sediment, beach recovery is an issue. Where there is clay beneath sand or shingle, then it is unlikely that a beach will recover naturally once the clay layer is exposed. Figure 3.26 shows an extreme example of beach depletion and foreshore down-cutting.

Erosion or changes to beaches are generally gradual (long-term) but significant erosion and lowering can occur during 'one-off' storm events. In general, failure is a result of depletion in the volume of the beach through increased longshore and/or cross-shore transport of beach sediment, or a reduction in supply of sediment onto the frontage. Changes in sediment transport are a result of changes to the wave conditions at a site and can occur for various reasons.

Figure 3.26 An extreme example of beach depletion, foreshore erosion and down-cutting – note the level of the base of the access steps in relation to the level of the beach (courtesy HR Wallingford)

The underlying geomorphology influences the performance of a beach in the lower foreshore and nearshore zones. The nature and properties of the seabed in these zones can contribute to long-term changes to the beach profile. The seabed may be formed from a variety of materials ranging from extremely durable rock to more easily eroded limestones, chalk and clays. Where durable rock platforms exist, there will be limited erosion and the impacts on the beach profile will be negligible. On softer rocks, the presence of a thin layer of mobile sediment can gradually erode the platform through abrasion. This lowering of the platform increases water depths and therefore the size of waves that can reach the toe of the beach and that of coastal structures. These conditions will lead to a loss of beach material and ultimately to failure unless steps are taken to periodically replenish the beach.

In locations where the underlying substrate is clay, erosion can accelerate as the condition of the beach deteriorates. Once the beach has effectively become a thin veneer, the underlying clay is likely to be intermittently exposed during storms and subject to erosion. The loss of clay beneath the beach profile results in a permanent lowering of the beach and increased exposure to wave attack. This process is commonly referred to as 'clay down-cutting' and can result in accelerated losses from the beach prior to failure.

Various instruments and methods have been used in studies to assess down-cutting or down-wearing of shore platforms (see Sutherland et al. 2007). One such monitoring instrument is the traversing erosion beam (TEB) as shown in Figure 3.27.

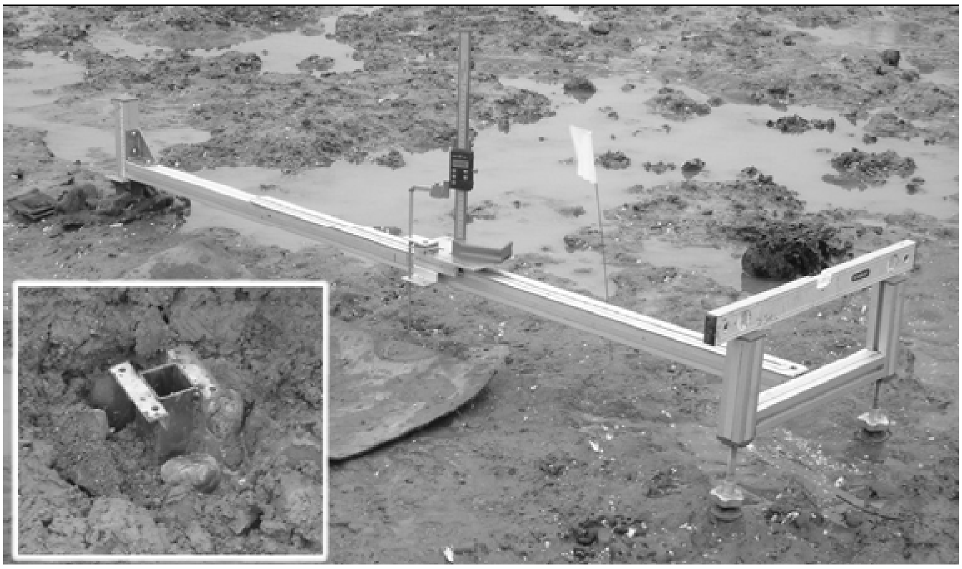


Figure 3.27 A traversing erosion beam instrument

3.8 Structure condition monitoring

3.8.1 Asset deterioration

Assessment of the defence should consider the condition and integrity of the structure itself. The material of the structure may deteriorate over time to a point at which it can no longer maintain its performance – even if external loadings do not change. A typical example would be the oxidation (rusting) of sheet piles, resulting in loss of integral strength due to loss of section thickness. As coastal structures are expected to perform satisfactorily for decades, during which time conditions can vary extensively, the assessment of condition and deterioration is an important issue. The performance of different materials when used in sea defence is discussed in Section 2.3.

Once beach levels fall below the top level of a toe structure, it is exposed to the elements of wave attack and abrasion by mobile sediments. In the case of concrete and steel piles, this also means that chemical and biological processes may also ensue.

Accelerated low water corrosion (ALWC) is a biological process of degradation that can lead to loss of section (width) of unprotected steel in the tidal zone. Chemical reactions in concrete caused by saline intrusion can contribute to crack expansion, spalling (surface flaking) and oxidation of internal steel rebars and reinforcing mesh. Abrasion of concrete by mobile sediments (Figures 3.28 and 3.29) can erode the structure and, if left un-remediated, can cause a serious loss of its mass leading to a reduction in function and performance and eventual failure (also see Section 4 on maintenance of concrete structures).

Figure 3.28 Abraded concrete toe and onset of undermining (courtesy of HR Wallingford)

Figure 3.29 Exposed reinforcing mesh due to steel oxidation and resultant cracking and flaking of concrete cover layer (courtesy HR Wallingford)

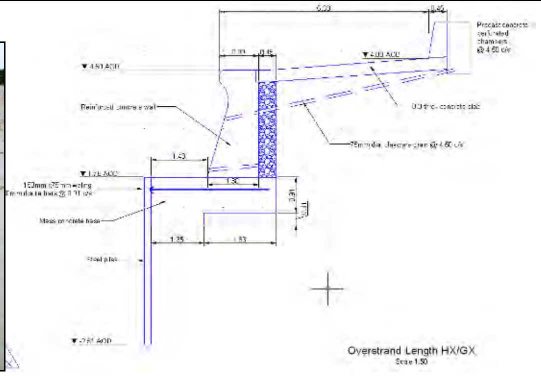
Box 3.1 presents an example from Norfolk of how estimates of asset deterioration can be determined. Such figures can be used in strategic and forward policy and financial planning. For example when capital works might be required for new structures, for planning frequent or intermittent maintenance, or for demonstrating when a reduction in the standard of service might ensue over time. Deterioration rates can be used at scheme conception simply to help establish the potential whole life costs of an asset or scheme option. Here ‘best’, ‘fastest’ and ‘slowest’ estimates are given to allow some judgement-based assumptions to be included in the determination of the most realistic rate of decline in condition.

Box 3.1 Site Description

This asset is located at Overstrand in north Norfolk.

The current defence consists of a 2.74 m high reinforced concrete wall with a 1.43 m wide reinforced concrete apron and 4.3 m long piles as scour protection. The average crest height of the wall is 4.50 mAOD. Behind the 5.00 m wide promenade at the rear of the wall, the contorted glacial drift cliffs rise to a height of 23.6 mAOD The defences were rebuilt in 1955. Observations are based on routine inspections.

The concrete wall has a condition grade of 2, tending to 3, and the steel piles have a condition grade of 3. The beach is in very poor condition lowering at a mean rate of 70 mm per year. The beach has been assigned a condition grade of 5. The sea breaks against the exposed steel piles at all high tides.



Step 1: Identify the type of asset
This is a composite structure with two different types of assets:
- concrete wall
- sheet pile.

Step 2: Identify the factors influencing the asset life
- Coastal environment
- Aggressive wave action and abrasion
- Potential structural instability resulting from the lowering of beach levels
- No maintenance of the concrete wall
- No maintenance of the sheet piles

Step 3: Identify the appropriate deterioration curves
Three deterioration rates (best, fastest, slowest) in years are used to consider the options:
- Vertical wall / Coastal / Concrete / Both / No maintenance
- Vertical wall / Coastal / Sheet piles / Both / No maintenance

	Best estimate (y)					***Fastest estimate (y)***					***Slowest estimate (y)***				
	1	**2**	**3**	**4**	**5**	**1**	**2**	**3**	**4**	**5**	**1**	**2**	**3**	**4**	**5**
Concrete wall	0	10	**30**	65	75	0	5	15	**25**	**30**	0	20	60	120	150
Sheet piles	0	**8**	**30**	**43**	**50**	0	**4**	**12**	**25**	**30**	0	**10**	**44**	**60**	**70**

The deterioration curve for the composite structure is obtained from the limiting values of the two curves above:

	Best estimate (y)					***Fastest estimate (y)***					***Slowest estimate (y)***				
	1	**2**	**3**	**4**	**5**	**1**	**2**	**3**	**4**	**5**	**1**	**2**	**3**	**4**	**5**
Composite structure	0	8	30	43	50	0	4	12	25	30	0	10	44	60	70

Step 4: Determine the deterioration curve
The fastest curve is selected as it is assumed that the asset is under severely adverse loading conditions.

	Fast estimate (m)				
	1	**2**	**3**	**4**	**5**
Composite structure	0	4	12	25	30

Step 5: Assess the current condition grade
The condition grade (CG) of the composite structure is 3.

Step 6: Forecast the expected deterioration time
The time for the asset to deteriorate from its current condition grade (CG3) to condition grade 4, which could be considered as the minimum condition grade acceptable for that structure, is 13 years (that is, from 25 to 12 years).

3.8.2 Visual condition assessment and indicators of performance

Toe structure condition monitoring, as part of a 'normal' condition inspection regime, can be hampered by the fact that toe structures are often obscured from view – they are typically either submerged or below beach level.

Where the structure is not covered by sediment, an inspection can be scheduled for a time and date when the tide is low enough for its inspection. If the structure toe is permanently covered by sediment, then there is rarely a requirement to inspect it as sediment provides a protective covering.

Inspection pits or trenches may be used if knowledge is required about the toe structure or its configuration, especially for unknown foundations.

One of the most frequent problems is the lack of knowledge about the presence and depth of toe structures, especially for older structures where engineering drawings have been lost or do not exist.

Consideration should be given to the inspection of ground beneath revetted or stepped toe revetments to assess any washout of fill material. Installation of inspection access hatches, taking core samples or installing holes for small camera probes could be prescribed for monitoring purposes.

A simple method of assessment of beach levels is to use a 'Plimsoll' type line painted on a seawall, or by 'dipping' – measuring the beach level from the top of the structure itself. A fixed line can visually indicate beach height at the wall in relation to the toe of the structure. This can provide the asset manager with a datum to record information on beach level variability over time in an inexpensive and straight forward way (see Section 3.5). Pre-determined trigger levels for beach height can be measured to flag up the need for intervention. Monitoring localised responses in this way allows beach managers to be proactive in their maintenance programme and reduces the potential for damage. Properly recorded, it also provides useful design information for future schemes.

The assessment of the condition of assets is an important process for understanding the particular state of the defence structures and the asset system as a whole. A series of snapshots of condition at a particular point in time – by repeated assessment or monitoring – can record changes in the assets over time and instigate intervention where necessary to prevent unwanted deterioration in structures or in the level of performance of the defence system.

The methods used to monitor asset condition and/or performance depend on the nature of the specific asset types. The general types of inspection and monitoring that can be applied to assets are:

- **Automated** – a monitoring system that provides feedback on asset condition and/or performance without human intervention.
- **Destructive testing** – a method of inspection that determines the condition of an asset by analysing a sample of the asset. This sample of the asset is destroyed in the process of analysis.
- **Non-destructive testing** – methods of assessing the condition of an asset without causing any damage to the asset or the removal of its components. Non-destructive testing ranges from a purely visual inspection of an asset to radiography, ultrasonic testing and a variety of other techniques.
- **Remote sensing** – a method of making detailed observations of an asset from a distance (usually a large distance). The term often refers to

observations made by Earth-orbiting satellites or low-flying aircraft. Remote sensing is inappropriate for detailed monitoring of assets (for example, the measurement of cracks and small deformations in structures) but provides a highly efficient technique for general topographical data over large areas.

Visual inspection is the only form of inspection discussed here, being the simplest form of non-destructive testing and the most widely used technique for monitoring assets – and it does not require any specialist equipment. However, this method can only assess the surface details and condition of an asset; changes to interior structure and condition are not easily identifiable until they lead to changes to the surface of the asset. Furthermore, toe structures are often designed to lie below the beach surface. Thus these structures may often not be visible at the time of assessment and therefore not observable by the inspector. These circumstances make visual assessment of toe structures particularly opportunistic – only being observable when tidal and sediment levels are sufficiently low enough.

3.8.3 Performance indicators, failure modes and performance features

Performance indicators provide evidence (visual or measured) of asset performance (at a point in time) under loading in relation to its designed or anticipated performance. Performance indicators may also, in the case of natural structures such as beaches, be referred to as 'coastal state indicators'. Possible coastal state indicators include:

- the level of the beach at the toe of a seawall for example (for undermining);
- the beach level plus beach slope;
- the beach cross-sectional area above a set contour (for overtopping).

Flood and coastal defence structures or 'assets' can fail in several ways. There are a number of well-known failure mechanisms, some of which have physical or statistical models associated with them. These performance models can be used to determine the likelihood of the type or 'mode' of failure occurring given a set of data relating to the flood defence system being analysed.

Understanding failure modes is important to performance and reliability assessment for two main reasons:

- So that the correct process-based models can be applied in the analysis of fragility in the right circumstances.
- So that indicators of failure modes can be included in condition assessments to elucidate the current condition of defences.

Failure modes display particular 'features' when they occur. For example, a sheet pile wall that is overturning will move from its nominal vertical position, leaning one way or the other. Washout of fines from beneath a structure will also result in its movement by settlement – often pre-empted by ground settlement or depression behind the structure. Such characteristics are referred to as 'performance features'.

Problems with lowering beach levels may first become apparent from complaints about access – either pedestrian or vehicular or both (Figure 3.30). Beach levels falling dangerously below the lower extent of access steps or below the seaward end of slipways (performance features) can be a handy early warning to monitor the beach closely for what could be a relatively localised short-term issue or could be signs of a longer term and wider problem.

Figure 3.30 Beach lowering at access steps (courtesy HR Wallingford)

Washout of fill materials from behind and beneath seawalls and revetments can also be exacerbated by lowered beach levels. Increased pore water pressures on the landside of the defence can compound such problems. The latter may be caused by overtopping water draining behind the wall, by water runoff from the hinterland, or by seawater flushing under the revetment toe on each high tide.

Figure 3.31 illustrates this case where, over time, a stepped concrete revetment has been effectively undermined – the fill material removed by water entering beneath the toe of the structure leading to eventual collapse of the pavement behind (Figure 3.32). However there are no visible signs of deteriorating condition of the revetment or the seawall that may have contributed to this. This appears to be an extreme case but such failures are alarmingly common. It illustrates the importance of maintaining design beach levels. If this is not possible, there should be prudent consideration of structural remediation or adaptation of the defences.

Avoidance of this type of failure is best conducted in conjunction with beach monitoring and a clear understanding of the relationship between beach and foundation levels. Further consideration of failure mechanisms is given in Chapter 5.

Although it would be rare to assess a toe structure in isolation from the rest of a defence, the toe structure may be of a different structural form and materials from the main body of the defence. For this reason the performance indicators that need to be assessed for the toe may be different as well. A concrete toe beam may, for example, show signs of cracking or settlement, whereas a toe protection built of rock armour may have no such visible signs or indicators of movement. A rock armour toe may have been intentionally designed to be 'sacrificial', that is, to settle into developing scour holes. Assessment of condition and performance can therefore be complex and requires an understanding of the purpose in design as well as structural integrity.

For the purpose of visual inspection, however, toe structures can be inspected according to the relevant materials and structure inspection guidance (Figure 3.33) in the Environment Agency's *Condition Assessment Manual* (2006). The importance of maintaining good design and construction records, as a source for comparison, is emphasised.

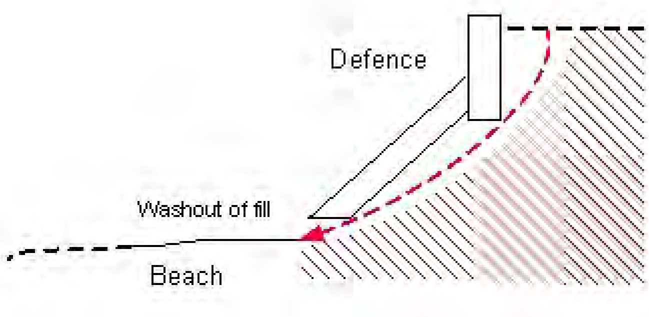


Figure 3.31 Schematic of backfill washout

Figure 3.32 Undermining and fill washout of stepped revetment and wave return wall


Seawalls
General: Defects that could reduce performance of the asset
Condition 3: Fair

Specific description: Rust staining or exposed anti cracking steel, localised spalling, lack of cover, lengthy cracking and some movement or extensive honeycombing. Minor loss of joint sealant.

Key features: Minor slumping or heave of ground surrounding structure. Significant staining. Minor cracking or spalling with exposure of surface reinforcement. Minor loss of backfill. Localised undermining or settlement. Minor cracks or holes in joints due to sealant loss.

183 Environment Agency Condition Assessment Manual

Figure 3.33 An example of a page from the 'Seawalls' section of the Environment Agency's *Condition Assessment Manual* (2006)

A particular problem with assessment of beach levels at the toe is that beach levels can rise as well as fall over short timescales, even if there is a long-term trend of beach lowering. Accounting for this in performance assessment is difficult. One option is to record the condition as normal, that is, to report the condition exactly as it is at the moment of the inspection/assessment. This level can then be assessed in the light of beach data/knowledge and trends, for example:

- long-terms trends in beach platform lowering;
- seasonal trends, for example, winter season beach lowering followed by beach level recovery in the summer;
- short-term beach processes, such as tidally induced scour, which fills again on every tide, either partially or fully.

It is therefore wise, where appropriate, to undertake an analysis of beach level variability to determine 'critical' beach levels against which beach condition assessments can be made for flood defence and coastal erosion. To facilitate ease of inspection, responsible authorities are encouraged to consider the installation of height markers (relative to the toe) on seawalls or perhaps groyne piles from which beach levels could be determined on a regular basis.

3.8.4 Detailed investigation and assessment

Visual inspection of defence structures and beaches can only be used to identify the condition of features on the surface, although these can often identify the signs of problems deeper beneath the surface or within the body of the defence (for example, rust staining on concrete surfaces reflecting oxidation of rebar within). Sometimes it may be necessary to undertake more detailed investigations to ascertain internal condition with more certainty. Such tests may be intrusive (for example, boreholes/cores) or non-intrusive (that is, by remote sensing methods).

Table 3.1 indicates some of the techniques that might be proposed to assist in the determination of the structural condition and performance of sea defence structures and beaches. Note this table is not exhaustive.

The type of investigation required is usually a decision taken by specialist consulting engineers who would provide advice on the specific method, which is best prescribed on a case-by-case basis. Hence a full account of such techniques is not repeated here.

Table 3.1 Examples of detailed intrusive and non-intrusive assessment techniques for defence structures and/or beaches

Data	Purpose
Geotechnical surveying – window sampling, trial pits, boreholes, cone penetration test, etc.	Investigation of geotechnical problems highlighted during inspection
Laboratory testing – moisture content, Atterberg limits, vane shear strength, small strain behaviour from consolidated triaxial testing and/or chemical testing	Define soil or material properties, i.e. resistance, shear strength, compressive strength and consolidation
In situ testing	Investigation of concrete surface tensile strength, resistivity, etc.
Core samples	Condition assessment, e.g. timber components
Thickness testing	For example, indicates extent of damage by corrosion or borers To investigate load capacity
Carbonation testing	Indicates extent of carbonated concrete
Corrosion testing	Applicable where chloride contamination is a problem
Dynamic load testing of piles	Investigates dynamic resistance
Static load testing of piles	Tests load/ settlement performance
Use of pressure cells and strain gauges	Determine performance of bending
Installation of and monitoring with inclinometer	Records plastic deformation changes in soil to indicate slides in foundations Used on piles to determine deflection and bending moments
Installation of and monitoring with tensometers	Measurements of pore water suction
Installation of and monitoring with observation wells and piezometers	Measurements of pore water pressures to determine soil consolidation
Magnetic, acoustic or seismic tests, e.g. ground penetrating radar	Traces anomalies in structure to infer condition or used to determine distances, e.g. pile length and decrease in concrete cover

Data	Purpose
Ecological survey	Determine animal presence for control measures
Nuclear or electrical density measurements in bore holes	Determine the quality of the foundation layer underneath a revetment layer
Magnetic, acoustic or seismic tests, e.g. ground penetrating radar	Traces anomalies in structure to infer condition or used to determine distances, e.g. detect local cavities and assessment of revetment thickness
Topographic survey to provide crest level and cross sectional geometry	Overtopping analysis to determine current Standard of Protection
Current wave and water level conditions and wave incident angle	Overtopping analysis to determine current Standard of Protection

3.8.5 When to conduct surveys

Beaches

Seasonal variability in beach levels in front of a coastal structure will affect the results of a beach monitoring programme. In practical terms, weather conditions make it difficult to plan the timing of surveys precisely. Typical monitoring programmes make provision for two regular equally spaced surveys per year, covering the summer and winter months.

While post storm surveys provide responses that often depart significantly from the typical long-term trend, they provide the most valuable data for the assessment of toe vulnerability and should be conducted as soon as possible following the storm event.

Structures

Ideally surveys of coastal structures should be conducted when they are fully emergent (that is, when the tide is below the lowest point (the toe) of the structure) so that they can be readily observed and assessed by the inspector. However, this will not be possible if the toe lies below the lowest tide level. In this case, other approaches should be considered such as employing specialist divers or even remotely operated vehicles to gather a visual record of the condition of the structure.

However, the toes of defence structures are typically (and ideally) covered by sediment, making visual inspection impossible without removing the sediment. Coastal managers should take any opportunity that may arise to enable the assessment of the toe, for example, if a beach has ‘drawn down’ after a storm event to a level that exposes the toe. The beach level may well recover quickly, and before the next scheduled (defence) inspection, so advantage of the occurrence should be taken to gather information on the condition (and type if not known) of the toe and its elements.

It may be necessary to conduct intrusive investigations (for example, trial pits) at the toe to reduce uncertainty associated with structural condition and/or stability when remedial or new works are being considered. Such activity can often be efficiently tied in with maintenance works when excavators are available.

Any landward developments that will introduce additional loadings on the defence structure will need to consider the stability of the structure – including the nature of the toe itself. If this is not known then investigations will be necessary. Similarly, if a change in the seaward conditions is forecast or planned that will affect the amount of sediment retained at the toe (for example, dredging of a nearby channel), then the likely impact on the toe and the structure should be considered.

3.8.6 Risk assessment, defence reliability and determining 'trigger levels' for action/intervention

There are two types of risk-based performance assessments that are influenced by the performance of beach levels and structures at the toe of coastal defences:

- structural failure risk assessment;
- overtopping risk assessment.

In both cases the response of the structure is strongly dominated by the beach or structure level in front of the main 'wall'. Scour in front of the seawall increases water depths, often leading to higher overtopping rates with the consequential impacts on flooding, risk to life, damage to hinterland and properties, and on erosion. The increased overtopping rates and associated wave impact forces also increase the potential for failure of the defence, although in many cases the main impact of toe scour on the failure or deterioration of defences is as a direct result of the loss of toe support – due to beach lowering or to failure of the toe structure itself.

These risk assessments have been the subject of many previous reports. In particular:

- For information on the calculation of overtopping rates and comparison with tolerable mean discharges, reference should be made to the European Overtopping Manual (Pullen et al. 2007).
- Information on assessment of defence reliability is available in many research reports, including those on the Performance-based Asset Management System (PAMS) project. The most concise summary of the issues is given by Simm et al. (2008).

A shortage of knowledge about how defences fail and variations in the characteristics of defences means that the response of a defence can never be forecast with certainty. The concept of fragility tries to capture the probability of a range of defence responses to a given load. Fragility curves for vertical coastal defences such as anchored sheet piles, cantilever walls and masonry, concrete or gabion walls contain a toe scour term (Buijs et al. 2007, Table 8). Scour predictors can be used in the calculation of fragility curves for coastal defences using the monitoring data described below.

- For **sand beaches**, the depth of scour can be predicted. 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.
- 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.

Beach level at the toe of a coastal defence varies with short-term toe scour depth, but also has a 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 failure limit state function (Buijs et al. 2007, Sections 3.2–3.4).

The long-term trend in beach level can be obtained from extrapolation of the historical data (possibly with seasonal variation). 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. 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.

4 Maintenance

Chapter 4 discusses the options for maintenance of different types of toe protection structures and materials. A matrix response summary is provided to identify potential actions to rectify common issues.

Key links to other chapters:
- Chapter 2 – Toe structure types and materials
- Chapter 3 – Asset management

Who will be interested in this chapter?
- Asset managers
- Coastal engineers

4.1 Introduction

Analysis of existing scheme performance at many sites provides clear evidence that small-scale maintenance treatment of early stage problems is far more cost-effective than allowing problems to develop. While there are often economies of scale associated with minimising mobilisation of plant and labour to conduct works, this principal does not apply to management of structure toes. Damage can develop very quickly on the toe of structures and small-scale problems can evolve rapidly to cause major failures through the various failure mechanisms. In particular, failures involving undermining of the foundations and loss of core material are particularly difficult and expensive to deal with. Indeed, in the survey carried out by CIRIA in 1986, which examined the performance of seawalls in England, Wales and Scotland, it was concluded that erosion at the toe of structures represented, by far, the most prevalent and serious form of damage to seawalls in the UK Over 12 per cent of all seawalls reported erosion at the toe, which represented over a third of all damage reported (CIRIA 1986).

The resultant cost of rehabilitation is usually extremely high. Notwithstanding these observations, there are an alarming number of structure failures which could have been avoided with timely maintenance. The fact that the toe underpins the coastal defence superstructure means that in many cases it is just not practical to reconstruct the toe without major modification to the whole defence structure.

Maintenance can be considered at two principal scales:

- minor maintenance without modification to the structure;
- toe modification by major reconstruction.

Minor maintenance might include such activities as structural maintenance of joints, maintaining safety of the tops of steel piles and maintenance of beach levels above defined trigger levels. Failure or partial failure of the toe usually requires construction of a new or modified toe. In most cases where a new toe is installed, it is built to seawards of the inadequate older structure. The design principles outlined in Chapter 5 remain relevant, although there may be some additional requirements for details to tie the modified toe to the old structure. The vast majority of toe protection works are not 'new build', but entail some form of modification to the existing structure.

Minor or regular maintenance is generally funded from the Environment Agency's or the local authority's revenue budgets. 'Capital' maintenance is more appropriate to toe modification which will probably require Flood Defence Grant in Aid (FDGiA) funding and therefore need to go through a more rigorous approvals procedure.

Similarly, maintenance works to defences that do not alter the form, profile or footprint of a defence are exempt from Marine Management Organisation (MMO) licence requirements, while works that do modify the defence, such as addition of a rock toe, do require a licence and associated consultations.

While it is normal for maintenance of the whole of the structure to be considered at the same time, this section focuses on activities related to the toe elements only.

4.1.1 Definitions

The term 'maintenance' can be interpreted in a number of ways, each reflecting different views on the scope and range of activities included. For example, the US Army Corps of Engineers' (USACE) *Coastal Engineering Manual* (CEM) (USACE 2012) defines 'maintenance' in accordance with Vrijling et al. (1995), as consisting:

> 'of the following essential elements:
>
> a. Periodic project inspection and monitoring of environmental conditions and structure response.
>
> b. Evaluation of inspection and monitoring data to assess the structure's physical condition and its performance relative to design specifications.
>
> c. Determining an appropriate response based on evaluation results. Possible responses are:
>
>   - Take no action (no problems identified or problems are minor)
>   - Rehabilitate all or portions of the structure
>   - Repair all or portions of the structure.'

This definition introduces two further terms – 'rehabilitation' and 'repair'. The CEM (USACE 2012) goes on to distinguish these two levels of intervention by saying that 'rehabilitation' implies steps are taken to correct problems before structure functionality is significantly degraded (for example, patching spalled concrete), while 'repair' implies that damage has occurred and structural functionality is already significantly reduced (for example, repairing a vertical wall).

Further distinctions can be made regarding the management approach to maintenance:

- **Pro-active or preventative maintenance** – (rehabilitation) maintenance based on the observed condition of the project.
- **Periodic maintenance –** (rehabilitation) maintenance that occurs after a prescribed time period or when a particular loading level is exceeded.
- **Reactive maintenance** – (repair) maintenance, undertaken in response to the occurrence of actual damage.

4.1.2 Scope of maintenance considered in this chapter

The term 'reconstruction' meaning the complete rebuilding or replacement of a structure is not included in the list of definitions above. Arguably, reconstruction of just the toe element could be regarded as a repair of a complete coastal defence structure. However, given that these guidelines focus specifically on the toe, reconstruction is not regarded as repair and is, therefore, not classed as maintenance. Reconstruction or construction of the toe is dealt with in Chapter 5.

The maintenance considered here includes monitoring and evaluation of the outputs of monitoring as the basis for the undertaking of rehabilitation or repair maintenance. These aspects of asset management are dealt with in Chapter 3.

The present chapter is therefore confined to the practical aspects of maintenance of toe structures by either rehabilitation or repair. Moreover, it is supposed that maintenance involves attention to present toe structures together with topping-up or additions to the host materials, but does not include replacement of a structure or the substantial introduction of new materials (see Chapter 5).

The next section looks at aspects of maintenance that are particular to toe structures such as limitation on access. Section 4.3 describes maintenance issues and remedies for various materials and forms of construction used in toe structures. As these two descriptors (materials and forms of construction) are closely allied (for example, masonry is used in revetment type construction), the sections are ordered according to material type in line with the description of materials given in Chapter 2.

4.2 Issues associated with maintenance of existing toes

4.2.1 Designing for maintenance

Maintenance requirements should be considered as early as the Project Appraisal Report (PAR) stage. Considerations at this stage will tend to focus on the financial commitment and whole life costs. Subsequent development of ideas at the outline design stage will provide important input to construction, design and management (CDM) considerations. The need to recognise and carefully consider safety during subsequent maintenance is likely to influence not only the design of the maintenance operations per se, but also design of the capital works.

In practical terms, maintenance takes place on structures that are often several decades old and thus pre-dating current safety standards, which should make adequate provision for safe maintenance. Unlike the crest of a seawall and most of the body of the wall, the toe is relatively difficult to access for maintenance purposes.

Where major maintenance to the structure toe is required, this can present an opportunity to improve access for further maintenance. For example, when a rock toe is added to replace or reinforce an existing toe, it may be possible to improve access by construction of a rock berm of an appropriate width to support a tracked excavator; this may facilitate improved access along the structure. Such works may involve additional material to those required to deliver the hydraulic and stability requirements, but will enable more cost-effective long-term repairs in the future that may not require construction of expensive temporary haul routes. Once installed, the toe might be mainly or entirely concealed from view under normal conditions of weather and beach level. The times when the toe becomes exposed to new/accumulated wear (that is, during stormy weather accompanied by beach drawdown) is not usually the best time to inspect or monitor its condition, let alone carry out maintenance. It is generally preferable therefore to eliminate maintenance of the toe until such time as a more major intervention might be required, for example, 30 years or more depending on the circumstances and residual life of the main structure.

The design of capital works provides an opportunity for planning and minimising maintenance commitment. For example, the inclusion of sacrificial allowance in the wall thickness specification for steel sheet piling may reduce long-term maintenance requirements. Clearly, such opportunities only exist in cases of new construction or major reconstruction. Funding of maintenance is typically provided from revenue budgets of the maintaining organisation, so this design stage consideration is worthwhile.

Assuming they can still be accessed, much can be learnt from the original design calculations for coastal structures. Design conditions may make allowances for corrosion, abrasion, rounding/loss of rock mass, breakage, cracking and so on (that is, damage arising from ageing and exposure to the marine environment).

Information gained from earlier records must be examined alongside any changed conditions and limited inspection of the toe in its present condition, to reassess the need for maintenance.

4.2.2 Changing loading conditions

Coastal loading by waves and currents can change over time. Wave climates are not static and a review of these is suggested for each site on a rolling 10-year cycle. Data to permit this assessment are generally becoming more widely available in the UK through the regional coastal monitoring programmes.

This means that future conditions can be different to those adopted for initial design purposes. Actual conditions can be very different to those evaluated for design simply because of the approximations and assumptions made in determining the design wave climate. Changes in the hydrodynamic effects of waves and currents will always result in changes to a (non-cohesive) sediment regime including patterns and trends of beach erosion or accretion.

Lowering of the beaches has a significant effect on the exposure of a coastal defence to wave attack. A lowered beach level at the toe can mean that parts of the structure that were previously buried become exposed to the action of the sea. In the more severe cases, this can lead to instability of the toe – and hence the whole structure – due to geotechnical pressure or undermining. The latter case could lead to the requirement to reconstruct the toe. Before this stage is reached an exposed toe is likely to require increasing levels of maintenance attention to counter the effects of direct loading and, in particular, abrasion due to water and sediment movement. The extent of abrasion and the scope for correcting it depends, among other things, on the material used. For example, open stone asphalt is highly vulnerable to abrasion while durable rock would be significantly less vulnerable. The maintenance commitment is likely to increase once the toe becomes exposed, depending on the materials used.

4.2.3 Beach management

Beach management forms a significant element of structure toe maintenance at many sites.

The most common cause of failure of seawalls is undermining of the toe, which generally arises as a result of falling beach levels. It is important to maintain the beach above the trigger levels set for the site (see Section 3.5) and this requires regular monitoring (Chapter 3). Maintenance operations may vary considerably in size, ranging from the movement or addition of a few 100 m^3 per year to more than 10,000 m^3 per year. Beach management activities may include either recycling from within the immediate frontage or adjacent sections, or by topping up using additional imported material

The approach using regular detailed baseline topographic surveys allows generation of digital terrain models and calculation of beach volumes relative to trigger levels. The approach adopted in Figure 4.1 highlights the volume necessary to maintain the required design management levels at a site fronting a seawall. The approach locates and quantifies volumetric shortfalls within groyne compartments, identified by survey. Possible recycling sources are also shown within zones containing a surplus above the required management profile. The number of truck loads required in each groyne bay is highlighted, providing the basis for logistic planning of maintenance operations. The maintenance undertaken at this site is at a very large scale relative to most UK sites. Alternatively, material may be imported from offsite sources in order to achieve the required beach levels.

On some occasions trigger levels are set on the basis of visual identification of key features on the structure toe. The example shown in Figure 4.2 has an intervention trigger when the interface between the bottom step and foundation shutter is exposed. This structure has not been designed to allow direct wave attack at this elevation on the toe.

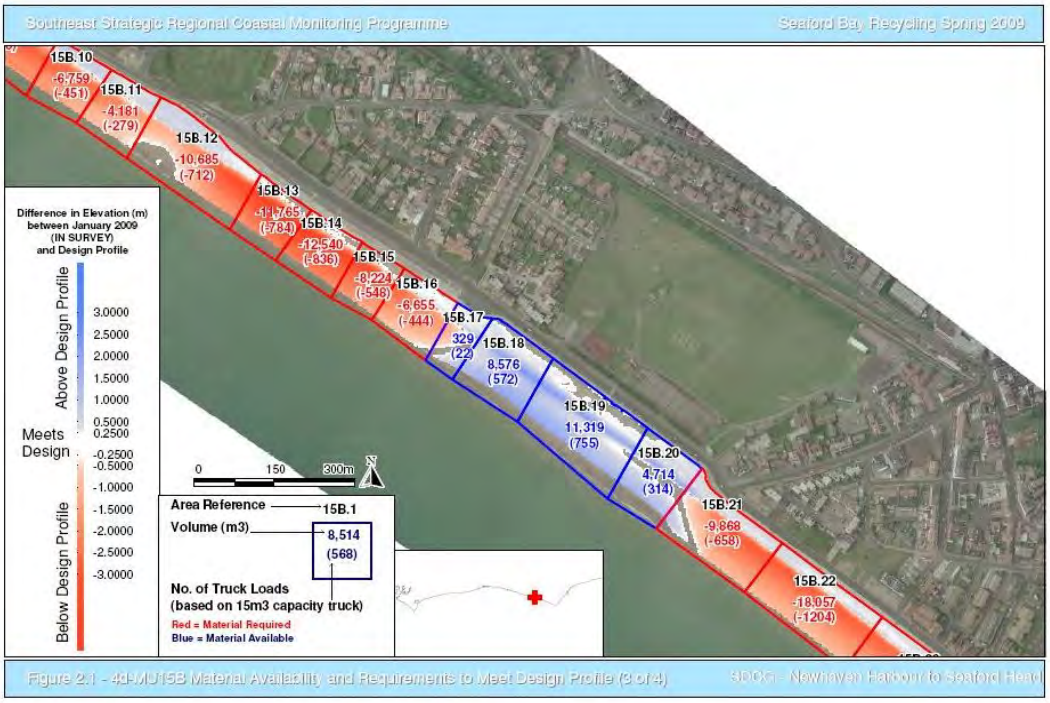


Figure 4.1 Application of monitoring data to identify beach zones beneath trigger levels (courtesy Worthing Borough Council)

Figure 4.2 Beach that has reached trigger level prior to repair and recharge (courtesy Poole Borough Council)

The crucial considerations to be made when planning beach maintenance are:

- Use projected rates of loss from monitoring to identify how much sediment is required for each maintenance operation.
- Make adequate provision for accelerated losses at hot spots.
- Add more material to updrift zones to allow for sediment transport.
- Log quantities of material added or recycled to inform long term beach management plan.
- Check the volume of material following significant storm events.

Figure 4.3 provides a valuable summary assessment of toe scour management using beach recycling and small scale recharge. Trigger levels are shown. The experience of previous toe failure at this site has been built into the maintenance plot. The experiences and linkages between beach losses and storm events can be identified. A series of small recharge operations have been conducted to maintain the beach above trigger levels and there is a clear annual trend of loss which can be used to plan for the requirements of further intervention. The vertical grey lines indicate interim recharges, which are small in size (approximately 1,000–7,000 m^3 each).

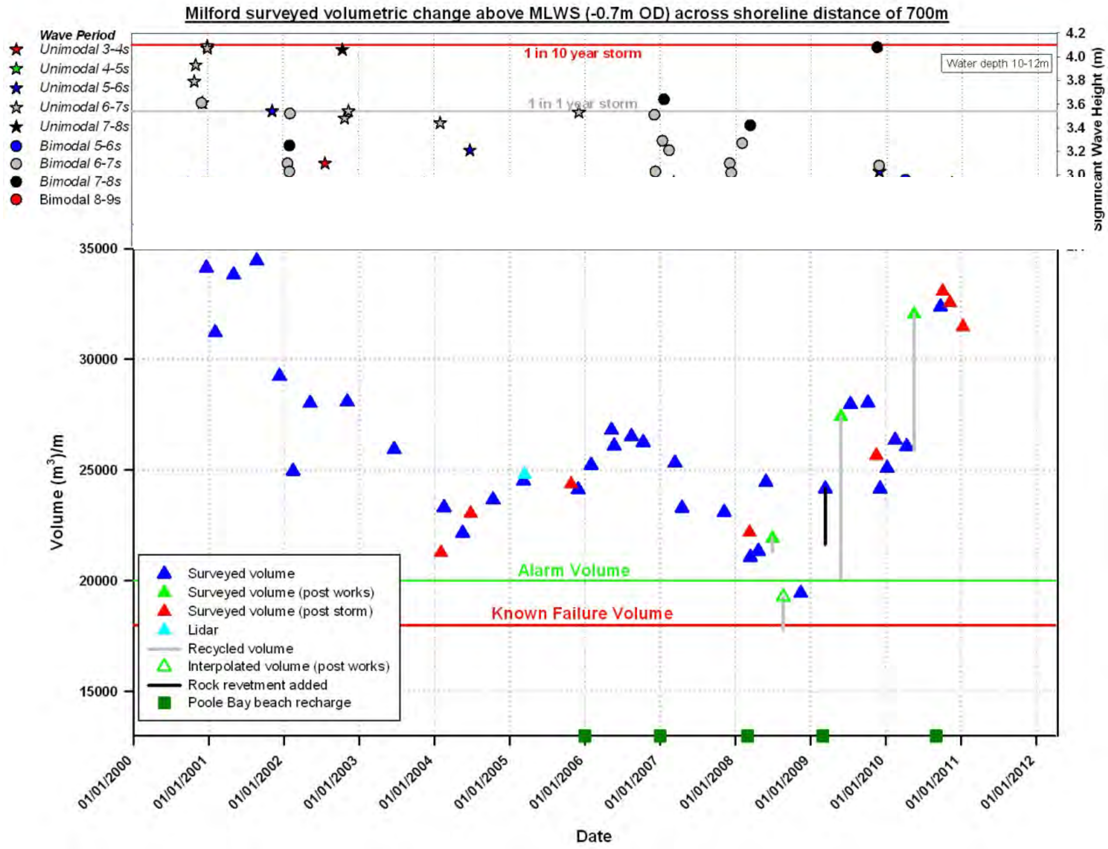


Figure 4.3 Application of monitoring data to identify long term beach performance in context with structure toe triggers (courtesy Channel Coastal Observatory)

4.2.4 Re-use of materials

The reuse of materials is generally considered to have environmental benefits. Exceptions to this might include situations where material retrieval might cause greater damage to the donor environment than if new materials were to be used from a remote source.

In the case of toe maintenance, materials for reuse are unlikely to have originated from the toe itself but may have been derived from removal or reconstruction of other works in the vicinity. Materials for reuse could include rock and steel sheet piling. In some instances, failed concrete slabs arising from toe failure have been crushed and used to provide fill material.

When reusing rock, attention should be paid to the condition of the salvaged material. If it has spent some considerable time in an aggressive marine environment (for example, tens of years), it may have become rounded, thus lessening its stability properties if required for rubble-mound type of installation. If the stones have become rounded in the marine environment but were only deployed previously for a limited period (for example, for temporary works), this could indicate inadequate durability. Provided its relative lack of internal friction is not a significant issue, partially degraded armour can often, however, be used effectively in sublayers of multilayer construction.

Extraction of piling for reuse may be achieved by use of vibratory or jacking type extractors. The *Piling Handbook* (Arcelor 2008) provides advice on pile extraction, though this is largely directed at the extraction of temporary piles. Piles previously used in permanent works are likely to introduce a number of complications, including the difficulty of relocating the original piling records, and in practical terms, the fixity caused by walings, ties and so on. Although the removal and reuse of interlocking piles provides a theoretical solution, the state of the piles is often inadequate for reuse. Clutches are often extremely badly damaged, restricting the ability to interlock. In addition, the tops of the piles are often damaged by the ends being bent or burred under the combined action of waves and sediments (Figure 4.4). Reuse of piling for maintenance implies the replacement of piles, which in turn implies the sourcing of piles that will interlock with those left in place. These complications clearly limit the reuse of piling for maintenance purposes.

Figure 4.4 Typical damage to tops of steel interlocking piles (courtesy AP Bradbury)

Stockpiling

Materials may be stockpiled for one or both of the following principal reasons:

- to offset the relatively high cost of future mobilisation each time that maintenance, using the relevant material, is carried out;
- to have materials readily available in the case of a breach or severe damage to the defence.

Both these reasons could apply to toe protection maintenance, albeit that the second one implies emergency restoration of a defence structure rather than maintenance.

Rock fill is a likely choice of material for sealing a breach. Given the circumstances under which it might be deployed (that is, during or soon after a storm), it is imperative that the stockpile is accessible using land-based plant. Moreover, the plant required to recover, transport and place the materials must also be available at short notice.

While it is often not convenient to stockpile materials at a site, arrangements may be made with nearby quarries to hold materials in reserve for emergency maintenance operations. This avoids the need for sometimes lengthy mobilisation of production.

4.2.5 Access

Access for maintenance may be especially problematic at the structure toe in cases where the toe is buried, and inspection and maintenance may not be practical on a routine basis. This may not be a major concern with new construction where well planned and controlled design eliminates the need for maintenance for many years. In these cases, intervention might be invoked on the basis of trigger levels being reached, for example, lowering of the beach to a critical level (see also Chapter 3).

Where access for maintenance is necessary a number of factors need to be considered:

- **Materials to be used and/or the nature of the operation**. These will determine the type of plant needed (that is, cranes, pile driving equipment, dump trucks and so on).
- **Loads to be lifted and placed**. This will determine the capacity and reach of cranes and other plant.
- **Capacity of the promenade** to carry plant/constraints on loads and reaches, and nearness to the edge of the promenade/seawall structure. These considerations must take account of any excavation at the toe (and hence reduced passive support to the wall).
- **Access along the beach.** This may be impeded by groynes, outfalls or other structures.
- **Access to the beach from the crest of the structure**. Many aging structures did not make suitable provision for plant access to the beach at regular intervals.

As maintenance only applies to structures and defence systems that are already in place, it follows that access must have been possible at some time when the defence was originally installed or last rebuilt. Exceptions to this may result from:

- post defence construction infrastructure;
- especially in the case of very old structures, by the structure itself (for example, access formerly having been gained from the land to form a foundation, then working upwards to form a seawall);
- due to worsened (lowered) beach conditions limiting the tidal window for working.

Access to the toe is a major problem at many sites and, where access is not possible at the crest of the structure, access via temporary haul roads may be required (Figure 4.5). In this instance, access is required over a number of closely spaced timber groynes. The simplest solution is often reshaping of beach material to form a temporary access way, although this approach is often fraught with the difficulties arising from inadequate beach volume and wave action which may regularly destroy the access.

Figure 4.5 Temporary access road constructed from local beach material (courtesy New Forest District Council)

Maintenance activities may provide the opportunity to enhance access to a site. Addition of a rock toe (Figure 4.6) to support a vulnerable structure has provided the opportunity to improve access along the toe of the same structure as shown in Figure 4.5. The crest berm has been constructed to provide a suitable width to enable a tracked excavator to move safely along the top of the rock toe at all states of the tide. This approach is quite costly, since more rock is required for construction. In addition, the structure has considerable hydraulic benefits in reducing overtopping by the dissipative berm and reducing the volume of beach material required to recharge the site following completion of the toe maintenance works.

Figure 4.6 Improvement of access by construction of rock toe with wide crest berm suitable for plant access (courtesy New Forest District Council)

An example of improvement of an existing dilapidated access along the interface between a mid-19th century vertical wall and a toe revetment, at Llanfairfechan, North Wales, is shown in Figure 4.7. The later shot (to the right) shows how the access was upgraded for maintenance works that were carried out to the toe revetment.

Figure 4.7 Improvement of access by construction of concrete roadway suitable for plant access (courtesy Alan Williams)

4.2.6 Temporary works

Maintenance works often need to be undertaken in difficult conditions. Working is often required at extreme low water and any small wave activity is likely to reduce the safety of operatives. Rock armour, which is eventually used to provide an additional toe, may be used to provide a temporary bund to achieve safe working adjacent to the wall (Figure 4.8).

Figure 4.8 Temporary rock bund to provide safe working area for plant during toe repairs (courtesy New Forest District Council)

4.2.7 Budget prioritisation

The budgets available for maintenance are often limited and decisions about where monies are spent need to reflect the criticality of the each structure in the overall defence system. Prioritisation is often required to decide which structures should take precedence to be sustained at the required level of performance. This might be achieved by a risk-based assessment of the flood or erosion defence system.

What is clear is that structure toes are extremely vulnerable and can suffer a rapid brittle failure unless adequate maintenance of both the beach and structure is undertaken. Any budget assessments should consider carefully the costs of routine maintenance against the potential costs of emergency repairs following failure. Although it is difficult to provide a generic comparison for widely varying structures, recent experiences at several sites have suggested the cost of emergency repairs has exceeded £25,000 per metre run (over typical distances of 20–50 m). These very high costs reflect the fact that emergency maintenance requires:

- more complex safety arrangements;
- complex temporary works;
- dismantling of failed elements;
- working in a challenging environment to effect repairs.

It is suggested that adequate and regular planned maintenance will provide a better cost-effectiveness ratio by a factor of at least 10 against emergency works.

4.3 Maintenance of toe structures

4.3.1 Concrete

Preventing deterioration of concrete is easier and more economical than repairing concrete. Such prevention begins with construction by ensuring that:

- the proper and appropriate materials are selected;
- the mixture has the correct proportions;
- placement and curing procedures are correct for the purpose.

Particular attention should be paid to the correct selection and specification of 'marine grade' concrete mixtures for all saline and coastal applications – including toe structures. Guidance on such specifications can be found in the *Maritime Concrete Manual* (CIRIA 2010b).

The most common types of maintenance for concrete include:

- repair of cracks and spalls;
- cleaning to remove unsightly material;
- surface protection;

- joint restoration.

The *Concrete Repair Manual* (ACI 2008) contains a significant collection of concrete repair information. Topics include:

- condition evaluation;
- materials for repair;
- surface preparation;
- application methods;
- corrosion management;
- structural strengthening;
- protection methods.

Contractual guidance for measuring concrete repair work is also included.

Methods of maintenance and repair should be considered to treat wear and tear of the fabric. Such issues include:

- exposure of rebar;
- water ingress and oxidation of reinforcing bars;
- development of scour scars.

A general maintenance programme of surface treatments such as concrete, facing, patching and concrete spraying should be considered to:

- replace abraded surface cover;
- prevent water ingress;
- replace lost material from concrete structures.

Left untreated, such processes will only accelerate the rate at which these structures deteriorate.

Tables 4.1 and 4.2 give an indicative list of repair options and examples of where the techniques may be used. It should be noted that some of these also apply to the repair of masonry structures that occur frequently in the maritime environment. The tables use the following key:

- UW: generally suitable for underwater elements or parts of the structure, without specific cofferdams or limpet dams (subtidal zone);
- T: generally suitable for elements or parts of the structure in the tidal zone (between MLWS and MHWS);
- S: generally suitable for elements or parts of the structure in the splash zone (zone above highest astronomical tide);
- OW: generally suitable for overwater zones for which access is constrained;
- D: generally suitable for elements or parts of the structure in the dry.

It is essential to check:

- the compatibility of all techniques and repair products with the environment;

- the project programme;
- the required performance characteristics.

Table 4.1 Structure repair/restoration options and their applicability to the maritime environment (based on CIRIA 2010b)

	UW	T	S	OW	D
Restore structure condition and stability					
Replace element by precast element	✓	✓	✓	✓	✓
Replace element by concrete bags	×	?	✓	✓	✓
Cast in situ concrete	?	✓	✓	✓	✓
Underpinning	?	?	✓	✓	✓
Pressure grouting of voids	×	?	✓	✓	✓
Spray concrete or mortar	×	?	✓	?	✓
Supplementary ground anchorages	×	?	✓	✓	✓
Post-tensioned concrete elements	×	?	✓	✓	✓
Relieving slabs	×	?	✓	✓	✓
Restore structure performance					
Restore drainage systems	✓	✓	✓	✓	✓
Restore transitions/joints ; create joints[1]	×	?	✓	✓	✓
Recast levelling surface slab in situ	?	?	✓	✓	✓
Restore protection systems, e.g. fenders	?	?	✓	✓	✓

Notes: [1]STRRES (2007a)
✓ = generally suitable; ? = challenging; × = generally not suitable

Table 4.2 Options for repair works related to defects in concrete and concrete reinforcement and their applicability to the maritime environment (adapted from BSI 2006)

	UW	T	S	OW	D
Restore concrete					
Applying mortar by hand	×	?	✓	✓	✓
Recasting concrete	?	?	✓	✓	✓
Spraying concrete or mortar	×	?	✓	✓	✓
Replacing element	?	?	✓	✓	✓
Injection of cracks[1]	×	?	✓	✓	✓
Restoring reinforcement and reinforcement passivity					
Restoring cover: replacement mortar/concrete	?	?	✓	✓	✓

	UW	T	S	OW	D
Replacing damaged concrete	×	✓	✓	✓	✓
Re-alkalinisation: electrochemical	×	×	?	✓	✓
Re-alkalinisation: diffusion	×	×	?	✓	✓
Chloride extraction: electrochemical	×	×	?	✓	✓
Replacing or supplementing corroded rebar	?	?	✓	✓	✓

Notes: [1]STRRES (2007b)
✓ = generally suitable; ? = challenging; × = generally not suitable

Box 4.1 provides a list of useful references on concrete protection and repair.

Box 4.1 Useful references on concrete protection and repair

- *Principles and Practice of Galvanic Protection for Reinforced Concrete*, Technical Note 6, Corrosion Protection Association, 2004.
- *Handbook of Coatings for Concrete*, R. Bassi and K. Roy, Whittles Publishing, 2002.
- *Guide to Surface Treatments for Protection and Enhancement of Concrete*, Technical Report No. 50, Concrete Society, 1997.
- *Cathodic Protection of Reinforced Concrete*, Technical Report No. 36, Concrete Society, 1989.
- *Guide to the Repair of Concrete Structures with Reference to BS EN 1504*, Technical Report No. 69, Concrete Society, 2009.
- *Protection of Reinforced Concrete by Surface Treatment*, Technical Note 130, CIRIA, 1987.
- *Guide FABEM 4 Protection des bétons*, STRRES, 2007.
- *Protection des bétons par application de produits à la surface du parement*, LCPC/ SETRA, 2002.
- *Mise en peinture des bétons de génie civil*, LCPC, 1999.
- *Choix et application des produits de réparation et de protection des ouvrages en béton*, LCPC, 1996.
- *Méthodes électrochimiques appliquées au diagnostic et à la réhabilitation du béton armé concerné par la corrosion,* A. Rahanarivo, LCPC, 2005.
- *Fluctuation du potentiel des aciers dans le béton et sous protection cathodique*, I. Pepenar, G. Grimaldi and A. Rahanarivo, LCPC, 1994.
- *Concrete in Coastal Structures*, R.T.L. Allen, Thomas Telford, 1998.

Underpinning of seawall between original foundation base and bed rock

Major repairs of concrete structures are often required following undermining failures. Although concrete is weak in tension, it is possible for large areas of reinforced mass concrete walls to remain unsupported over fairly long stretches, at least on a temporary basis (Figure 4.9). This occurs commonly where the structure foundations are perched within the tidal limits and where the original foundation construction has not closed onto bedrock. Support beneath the wall is required quickly to avoid cracking and failure under tension. This may often be conducted by underpinning the wall using mass concrete to provide a new foundation.

Figure 4.9 Unsupported section of reinforced mass concrete seawall following major undermining (Chesil Cove 1962) © Stuart Morris

Maintenance operations which require underpinning are challenging and sometimes require the bespoke development of specialist tools to achieve safe working and to maximise the structural integrity of the repair. This often requires removal of failed or loose material from beneath the undermined wall. A modified excavator bucket is used to rake loose material from beneath the bridging wall while maintaining safety for operatives (Figure 4.10). Excavations should ideally close onto competent bedrock material, but this is not always possible due to the depth of beach sediments.

Figure 4.10 Preparation of foundation excavation beneath wall for underpinning, using bespoke plant to ensure safe working beneath seawall (courtesy New Forest District Council)

The detail provided in Figure 4.11 shows an underpinned toe repair with the following characteristics:

- The base footprint of the repair underpinning is wider than the original foundation.
- Drainage is secured by regularly spaced high density polyethylene (HDPE) pipe placed within the underpinned foundation.
- The void behind the wall has been filled with coarse free-draining material.
- Loose material has been removed from beneath the old foundation.
- The new foundation closes out onto bedrock (this is not always possible).
- The underpinning concrete is tied into the old structure by way of cast interlocking upstands on either side of the underpinned foundation.
- Shuttering is provided by geotextile material held in place by rock armour and shingle on the seaward face.

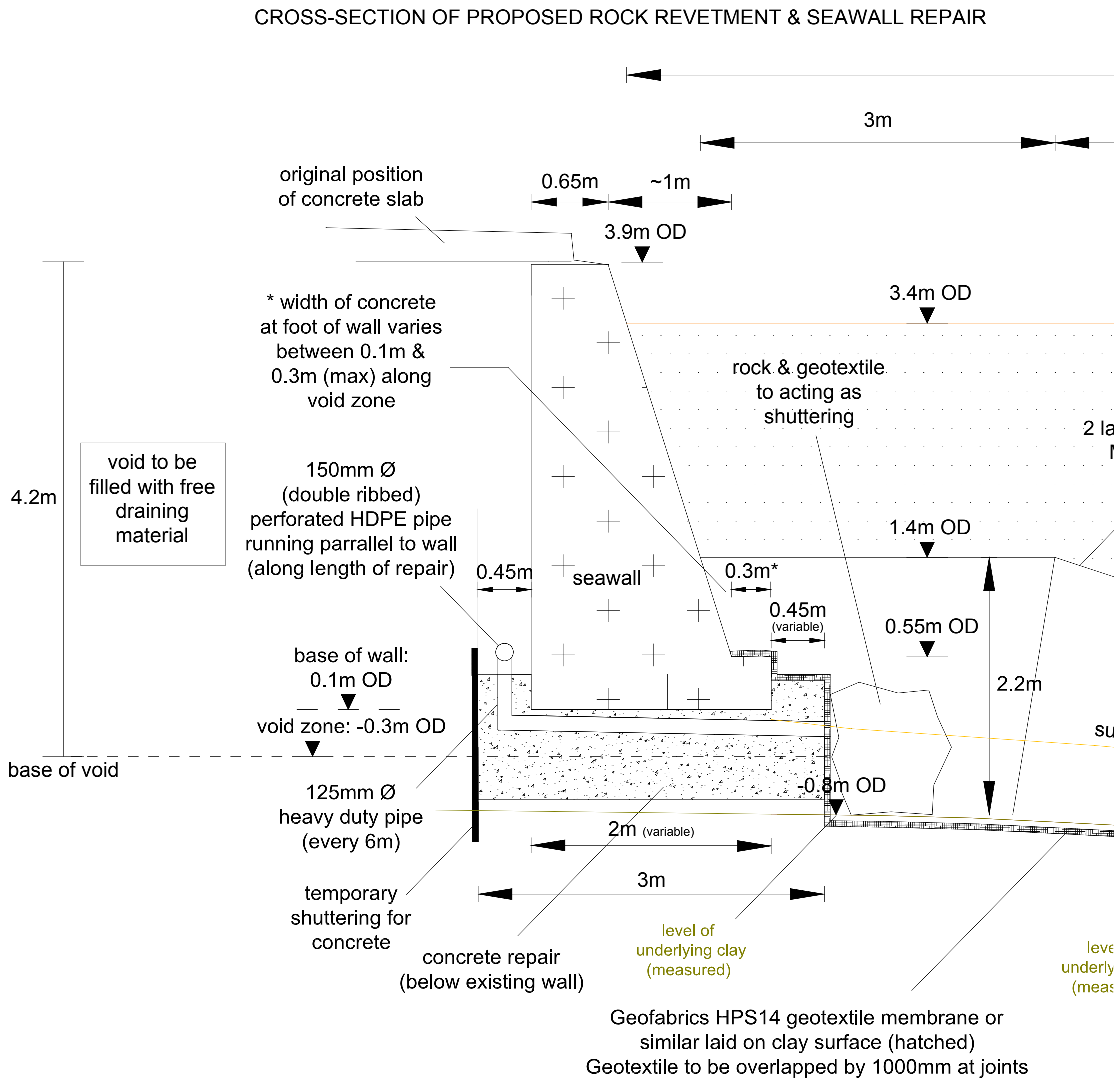


Figure 4.11 Underpinning detail for seawall foundation repair (courtesy New Forest District Council)

Shuttering the toe of underpinning foundation operations is often challenging since the toe of the structure may be subject to wave loading even during construction. In emergency situations, concrete may be pumped into voids, without manufactured shuttering, in an attempt to provide a rapid solution. This can provide an extremely expensive, untidy and not particularly strong repair since large losses of concrete may occur. Use of beach material can be used to provide shuttering bunds in this situation, although this would not be considered best practice. Ideally, shutters for repair should close tightly on adjoining surfaces, although this may not always be possible because of the safety aspects of their installation on a potentially unstable structure.

Promenade deck failures of the type shown in Figure 4.12 are fairly common and occur as a direct result of toe failure. Historical construction of mass concrete walls typically included a locally won backfilled material. This material is quite likely to have a significant fines content. Repairs of the backfill zone, following structure toe failure, should ideally be replaced with a coarse compacted granular fill that is free of fines. This will assist with drainage of water from the landward side of the structure and should prevent subsidence of the surface and formation of a void beneath decking, which may often be a cast concrete slab – although the example shown is a simple asphalt surface.

Figure 4.12 Typical collapse of promenade decking following undermining of backfilled mass concrete wall and washout of fine core (Chesil Cove 1962) © Stuart Morris

4.3.2 Rock (including gabions)

While the definitions of toe repairs used within this section do not include the addition of a new rock toe, this may often be undertaken as a large-scale maintenance option. Such treatment is regularly chosen as management option when the toe has become undermined. The details of the design of a modified rock toe are dealt with in Chapter

5. It is clear, however, that rock toe structures are far better at energy dissipation than vertical concrete structures (Figure 4.13).

Figure 4.13 Contrast between concrete toe and modified rock armour toe Folkestone Warren (courtesy Bryan Curtis)

Reprofiling rock structures

Provided access is possible, Repositioning or reprofiling of dislodged armourstone in a toe structure may be a relatively straightforward maintenance option for limited repairs (Figure 4.14).

Figure 4.14 Manoeuvring armourstone blocks in a revetment (courtesy Dean and Dyball Ltd)

Access restrictions at the toe may be more challenging, especially if material has moved from its original position seawards of the toe; in this case recovery of damaged armourstones may be difficult. It is generally more economic to add new rock to the existing profile, providing that a supply of suitable and economic of material is available. Where armourstone must be reprofiled, it is often necessary to ‘unpick’ areas of the structure to ensure that adequate interlock can be achieved from the repair. Large-scale damage may require a more substantial commitment of resources and could amount to a ‘rebuild’ rather than simply a repair.

When rock armoured toes have been constructed as part of the defence (see Chapter 5), these may regularly suffer damage (Figure 4.15). A frequent problem arises as the rock toe settles, particularly when this is placed on soft bedrock material. Settlement may result in displacement and reduced interlock of some of the armour, which may require reconstruction to maintain structure integrity. The benefit of the rock armour at the toe is that it is self-healing at the point of bed settlement and will move with the bed. This has the advantage that the structure will not become undermined. The consequence of settlement though is that more material is generally needed to top up the profile. Allowance for settlement of a falling toe is often built in to the design of a rock toe.

Figure 4.15 Armour displaced from a rock toe, Llanfairfechan, North Wales (courtesy Alan Williams)

Refilling/replacing gabions

Gabions placed in even relatively mild coastal conditions usually require extensive maintenance and are not generally recognised as efficient toe structures. The contents can settle within the gabions or may escape altogether, with contortion of the mesh or breakage (Figure 4.16). Gabions that have simply ‘settled’, which remain intact and are relatively in shape may just require topping up with fill material. Sometimes ‘leaked’ contents may be reused if they remain close by. Replacement fill material should be close to or larger than the size of the original. Gabions that have distorted badly or collapsed altogether will usually require replacement of the whole gabion unit.

Figure 4.16 Weathered gabions at East Head, Chichester Harbour, West Sussex (courtesy HR Wallingford)

4.3.1 Masonry

Regrouting/pointing

The regrouting or ‘pointing’ of joints in masonry structures is an important maintenance task. Missing joint filler allows water into the structure. If the gap penetrates through to the sublayer, then washout of fill material can ensue. The resultant voids in the sublayer can affect the response to hydrodynamic forcing on the structure, which can quickly weaken from repeated pressure changes. Loose blocks can simply be lifted or sucked out of the structure under wave or surge action and lead to further, potentially rapid, deterioration or even structural failure. Vegetation, once established in joints, can aggravate the percolation of water into the fabric of the structure (Figure 4.17) and the general weathering of the blocks.

Figure 4.17 Vegetation growth on a revetment (courtesy ENBE)

If allowed to grow, vegetation such as bushes and trees can cause deformation and movement in the structure – forcing blocks apart or out of the section (Figure 4.18). Hence vegetation should not be allowed to take root in such structures and should be cleared as necessary. Problems associated with vegetation are more likely to prevail in the higher parts of the wall structure rather than at the toe.

Figure 4.17 Vegetation colonising joints in a masonry coastal defence (courtesy HR Wallingford)

Replacement of dislodged or lost masonry blocks

Figure 4.19 shows a masonry toe before and after repointing. A durable grout should be used for pointing to ensure effectiveness and regular inspections made to identify any damage or weaknesses, and to record general deterioration in joint grouting over time.

Figure 4.19 Prior to and following maintenance of grouted masonry toe (courtesy Alan Williams)

Dislodged masonry blocks at the structure toe may act as a catalyst to rapid toe failure. Under severe storm conditions, failure of the whole defence could quickly ensue. Figure 4.20 illustrates such a situation, which if left unaddressed could cause rapid development of damage to the revetment, together with possible collapse. Major damage arising from this initially small problem would result in significantly greater costs of repair than replacing the single displaced block. Maintenance has to be undertaken soon after damage occurred, if progressive damage is to be avoided. This, in turn, implies regular inspection of defences (and inspection after storms) so that damage can be quickly observed and acted on.

Figure 4.20 Dislodged masonry blocks from revetment toe (courtesy HR Wallingford)

Collapsed, dislodged or missing blocks from toe revetments can quickly lead to more major problems, with washout of fill material and further structural instability and eventual failure (Figure 4.21).

Figure 4.21 Washout and collapse of block revetment following block removal (courtesy Canterbury City Council)

4.3.4 Steel

Sheet piles need replacement or repair where they have deteriorated over time either through rusting, abrasion and erosion of section, or through accelerated low water corrosion (ALWC). More generally, failure is manifested by holes in the piling rather than failure of the pile as a structure (Figure 4.22). In the more extreme cases, the thinning sections can become razor sharp. Remedial action should be taken immediately where corroded exposed piles pose a hazard to people or animals.

Figure 4.22 Deteriorated sheet piles

While the extreme damage shown in Figure 4.22 is generally unsuitable for repair by welding on steel plates, welding may provide a suitable repair method at locations where the damage is localised and small holes or thinning of the piles has occurred.

Damage to the tops of piles on toe structures is very common, especially when they are subject to rapid abrasion in a sediment-charged environment. Where the piles simply form a shutter to the mass concrete behind, this may not be a particularly serious issue (Figure 4.23).

Figure 4.23 Complete loss of section of sheet piles in an area exposed to sediment movement

Where damage is noted to the pile tops, reference should be made to the original design to determine the role of the piling in the structure. In those instances where the pile is simply forming a shutter, without a structural role, the piles can be maintained by burning or cutting the tops to remove sharp edges or by covering with rock armour. This is generally a safety measure (Figure 4.24).

Figure 4.24 Damaged interlocking steel piles requiring maintenance to make site safe (courtesy A P Bradbury)

The service life of sheet piles can be extended, albeit marginally, by applying protective coatings. Cathodic protection systems can also be used but these are not frequently encountered in coast defence works (unlike steel used in offshore applications, pipelines and so on), presumably because of the aggressive environment and the consequent risk of damage to the cathodes. In the toe structure especially, material loss can be substantially due to abrasion by shingle or sand.

4.3.5 Asphalt

Asphaltic revetments with open joints or fissures extending to the full depth should be repaired promptly, especially on the waterside slope. Loss of subsoil though such holes leads to the revetment settling and deforming, and eventually failure. Ideally a favourable time of year and weather conditions should be chosen to undertake such repairs (Schönian 1999).

Sand mastic asphalt repair

Holes in sand mastic revetments should be overfilled so that the grout forms a cap which binds well to the existing surface. The surface should be prepared by cleaning away sand and plant growth and pre-treated or ‘tacked’ with a coat of hot bitumen. Thin layers of repairs at the toe of the structure should be avoided where these are not bound to an appropriate base of asphaltic concrete, stone base or sand mastic.

Damage to an open stone asphalt revetment should be repaired immediately. The number of contact points between stones in the layer is limited and therefore integrity is heavily reliant on the asphaltic binder. As the production of small amounts of open stone asphalt for repair purposes is difficult and costly, stone fill grouted with sand mastic asphalt is recommended. Importantly this does not change the permeability of the revetment, although the viscosity should be well-adjusted to prevent any run out. Only loss of material from the surface can be repaired by patches of sand mastic spread over a prepared and cleaned old surface. If the damage is more serious, the section may need to be cut out and refilled again with open stone asphalt, or the existing surface may be cleaned, dried and tack coated before adding the new layer.

Figure 4.25 shows both concrete and neat bitumen being used to repair holes in an open stone toe apron at Prestatyn, North Wales. While this is not best practice and has potential impacts for the permeability of the structure if used extensively, it is clearly cheaper especially for small repairs and has been effective particularly as a stop gap. The concrete appears to have adhered reasonably well to the open stone asphalt, probably due to the surface roughness of the open stone asphalt and good preparation before the repair was effected. There has been no obvious change in the condition of the repair at the example site over a five-year period.

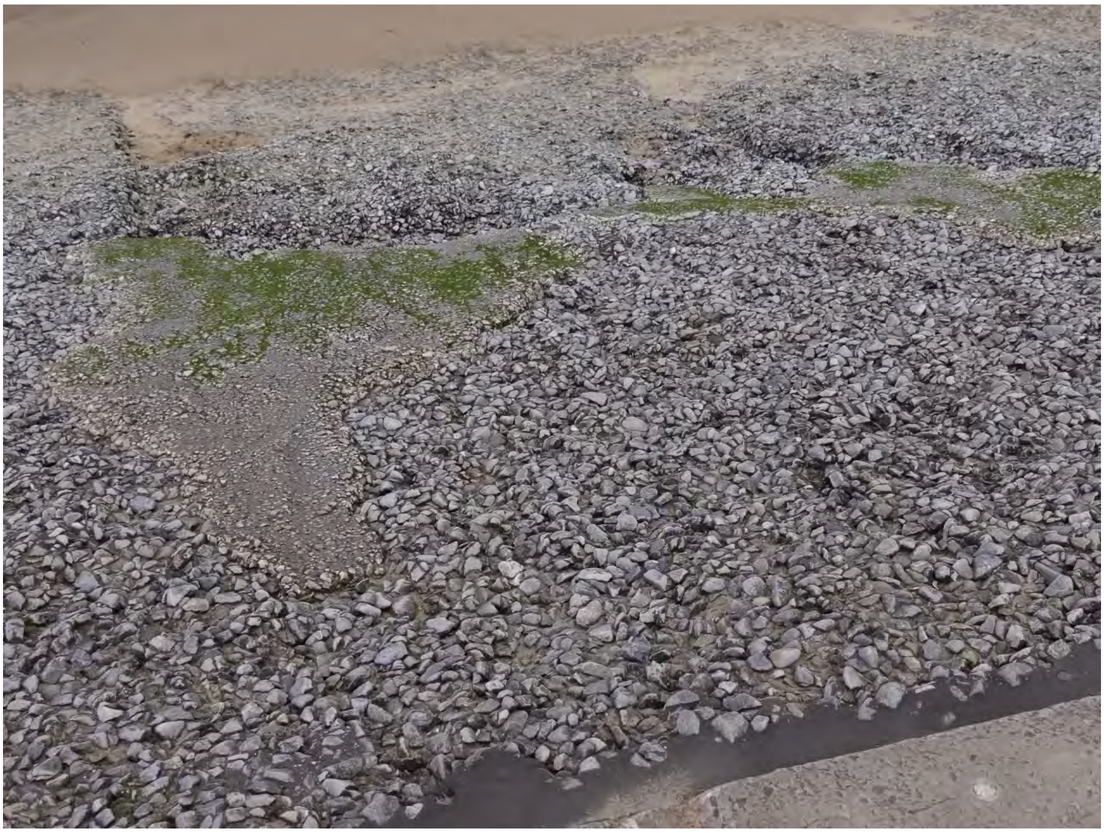


Figure 4.25 Repairs to open stone toe at Prestatyn, North Wales (courtesy Alan Williams)

General guidance on asphalt repairs can be found in the Rijkswaterstaat publication *The Use of Asphalt in Hydraulic Engineering* (van der Velde et al. 1985).

Methods of maintenance and repair for other types of asphaltic mixtures and structures can be found in *The Shell Bitumen Hydraulic Engineering Handbook* (Schönian 1999).

5 Toe structure design

Chapter 5 describes the design process with specific attention paid to how particular issues surrounding the design of the toe to coastal defences should be considered within it. New structures or significant additions are examined and awareness is given to the potential wider environmental impacts of any new toe structure. Finally, construction issues such as the practicality of timing of operations during spring tides are discussed.

Key links to other chapters:
- Chapter 2 – Toe structure types and materials

Who will be interested in this chapter?
- Contractors
- Coastal engineers

5.1 Introduction

Chapters 3 and 4 cover the management, monitoring and maintenance of the toe in the context of gaining a detailed appreciation of the environment in which the toe has to function. Chapter 3 identifies where intervention is necessary to continue to provide the required standard of defence. This chapter guides the user through the situation where the necessary degree of intervention is greater than that which can be carried out under normal maintenance (Chapter 4). It covers a range of measures from the implementation of significant repairs to designing the toe of a new sea defence.

Having established that there is a need for work, the design should progress through a logical sequence which is likely to involve:

- identification of the problem;
- project appraisal – appraisal of options from technical, economic and environmental points of view;
- design.

This manual does not give specific guidance on approvals, although guidance on environmental aspects is introduced in Section 5.3.4. In addition, when planning works to the toe the coastal manager should

- establish whether or not planning approval is necessary by consulting the local planning authority;
- contact the Marine Management Organisation as to the procedure for obtaining a marine licence, which will be required for works below high water.

5.2 Identification of the problem

Chapters 3 and 4 guide the user in the process of problem identification. These will support the identification of the possible threats to the defence standard which might include:

- flooding due to heavy overtopping;
- defence failure due to excessive overtopping;
- collapse of the defence;
- destabilisation (collapse) of the toe structure;
- undermining of the toe structure or geotechnical failure arising from low beach levels, leading to failure of the defence.

Most of these threats relate to a threshold condition being exceeded (for example, wave height). Moreover, the conditions that cause structural damage – be they impact or overtopping and so on – will also generally lead to depressed beach levels (albeit temporarily). Under these conditions larger waves can reach the defence. These conditions are likely to worsen through climate change and, possibly, longer term beach lowering. Hence, the probability of failure tends to increase with time.

In the case of geotechnical failure (undermining), the principal consideration is the long-term beach lowering (or perhaps an initial inadequacy of the defence). Transient loadings such as those due to storm action may be secondary, except in so far as they lead to short-term beach lowering and possibly an increase in hydrostatic head across the defence structure. Figure 5.1 shows an example of toe failure.

Figure 5.1 Toe failure example

It is important to identify all the possible modes of failure. The quantification of the risks, whether they are expressed as a storm return period or as a residual life, will usually be defined in the Project Appraisal or Strategy as appropriate.

A reasonable understanding of the timing and extent of the problem is an essential part of the process needed to define the scope of the subsequent design and to provide a first indication of the timing of future intervention.

5.3 Project appraisal

5.3.1 Purpose and scope

A Project Appraisal Report (PAR) is carried out when a project, or the problem, has been identified. Apart from identification of the problem, as outlined above, the PAR may well have been preceded by a Strategy Study or plan covering a greater length of coastline. In essence, a PAR is a feasibility report that sets out the technical, environmental and economic arguments for investment in a given specific project.

In England and Wales, definitive advice on the preparation of PARs for flood and coastal defence is presented in the *Flood and Coastal Erosion Risk Management Appraisal Guidance* (FCERM-AG; Environment Agency 2010) together with supporting documents. Note that several Welsh local authorities still use the predecessor to this document, the Defra Project Appraisal Guidance note (PAGN).

The guidance given by the Environment Agency and Defra is aimed at flood and coastal defence projects, in particular, those seeking central government funding. It nevertheless provides an excellent basis for approaching other types of coastal schemes (for example, for regeneration, marine and navigation purposes0.

The following paragraphs highlight some key points of the appraisal process as set down in FCERM-AG for work related to flood and coastal defence projects.

The flood and coastal risk management (FCRM) appraisal process is summarised in the *Beach Management Manual* (CIRIA 2010a), which states:

> 'What all public-funded projects have in common is that they need to be accountable and to provide a justified use of public money (demonstrating that the return on investment is higher than the alternatives and at the very least as high as might be expected from the wider basket of HM-Treasury-funded projects). It is this requirement to demonstrate accountability for investment of capital that necessitates a project appraisal. In the context of a coastal flood or coastal erosion risk management strategy there will be clear objectives that need to be taken into account within the appraisal process, which may include, but are not restricted to the following, as listed in the FCERM-AG (Environment Agency 2010):
>
> - reducing the threat to people and their property from flooding and coastal erosion;
> - delivering the greatest environmental, social and economic benefit consistent with the UK Government's sustainable development principles;
> - working with natural processes;
> - adapting to future risk and changes (for example, due to climate change);

- working with others to deliver better, more sustainable solutions that can deliver wider objectives and maximise benefits for people, businesses and the environment.'

5.3.2 Option appraisal

In the case of known or perceived falling beach levels, selection of the preferred option in the PAR will require an understanding of future levels. This will require a thorough understanding of the naturally occurring foreshore changes (see Chapter 3), together with the influence of existing or proposed structures. Future projections must take account of climate change as provided by the latest adopted advice, for example that published by the Environment Agency (2011).

Do Nothing

Having identified and quantified the underlying problem (that is, the risk of failure or of limited adequacy of the defence), the consequences of adopting a 'Do Nothing' approach are evaluated. This will usually entail prediction of defence failure and the progression of erosion and/or flooding.

Do Something

The understanding of risks and consequences arising from 'Do Nothing' provides a basis for preparing a long list of potentially viable solutions. Options for toe protection should include schemes that counter the risk in each of two distinct ways:

i. By aiming to restore and maintain satisfactory beach levels. This might include measures such as beach nourishment, recycling, control structures and so on.

ii. By restoring and maintaining the stability of the coastal defence structure against low or lowering beach levels. This might include measures such as a piled toe, rock apron and so on.

An option might include a combination of different measures (for example, sheet piled toe plus beach management plus control structures).

In the early stages of the PAR, it is necessary to set a number of primary objectives and identify any significant constraints. In the first pass it should be possible to eliminate a number of options on qualitative grounds because they clearly do not satisfy these overriding criteria (for example, use of a material that is not allowed on the given frontage).

5.3.3 Option shortlist

A shorter list of options should be appraised quantitatively in terms of the three important criteria: technical, environmental and economic.

At this more quantitative stage in the appraisal process it is appropriate to introduce the concept of 'Standard of Protection' or SoP. The SoP is expressed as the annual probability (or equivalent return period) afforded by the defence.

The SoP is usually assigned to the defence scheme (present or proposed) protecting a given risk area as a measure of its flood defence performance. Flooding might be due to overtopping or to breaching of the defence. Overtopping can be related to the combined probability of occurrence of waves and sea level. Breaching might also be related to severe overtopping (causing erosion behind the structure) or be due to some other failure of the structure including undermining. Depending on the type of defence, a breach failure of the structure body may also be calculable in probabilistic terms (for example, exceedance of threshold of significant damage to the armour layer).

For the toe structure, such a probabilistic approach may be more difficult to apply, failure being more often related to the beach level and hence the likely time to that condition happening. Fragility curves and limit state design principles may be used in the design stages of a project, but they would not normally be warranted for a project appraisal. For appraisal purposes, it would be more usual to apply sensitivity analysis. By way of example, based on a given rate of beach lowering, it might be predicted that failure of a given defence would be likely to occur in, say, year 15; sensitivity analysis could then examine the prospect and consequences of failure in years 10 and 20.

5.3.4 Environmental assessment

The weighting that the environmental impact of a proposal has on the appraisal process is highly dependent on the status of the location in question and on local and national planning policies.

Where key legislation – such as the appropriate national law relating to the Habitats Directive[4] and designations such as special protection areas (SPAs), special area of conservation (SACs) and Ramsar sites – are likely to apply to a site, it is vital that the risk of having to prevent or compensate for damage to these designations is incorporated into the project appraisal.

For example, a scheme that involves removing a designated wetland is unlikely to be acceptable (unless covered by a Coastal Habitat Management Plan (CHaMP) which has deemed otherwise) and the risk of having to protect the wetland should be incorporated into the option in the appraisal process as early as possible.

Hence an environmental appraisal should be carried out in parallel with the economic appraisal from strategy level down to detailed design. At the strategic level, this will involve a strategic environmental assessment (SEA). At project (scheme) level, in certain circumstances the environmental appraisal must be in the form of an environmental impact assessment (EIA), which includes preparation of an environmental statement.

The presence of an internationally designated site is also likely to require a habitats regulations assessment (HRA). Guidance related to FCRM including sustainability, biodiversity, heritage and landscape considerations can be found in FCERM-AG (Environment Agency 2010).

5.3.5 Economic appraisal

Economic appraisal entails the evaluation of scheme costs and benefits (tangible and intangible), the benefits being the value of damages avoided by the scheme over those that would otherwise ensue in the Do Nothing case (and in the Do Minimum case for

[4] Council Directive 92/43/EEC on the conservation of natural habitats and of wild fauna and flora

capital schemes). In FCRM, future costs and benefits are discounted to present day terms using discount factors advised in *The Green Book* (HM Treasury 2003).

As the toe of coastal defence structures is often buried or submerged, routine maintenance can be problematic. Repairs and improvements are therefore likely to be applied infrequently or only as capital works projects as and when a paramount need arises. The timing of works can have a significant effect on the discounted costs and hence the economic viability of a given scheme. Hence, the robustness of the economic argument relies heavily on the assessment of future risk. Arguably, this is more difficult to assess for the toe than for any other element of the defence structure, not least because the toe may be concealed and, in the case of a historic structure, poorly defined and understood. Sensitivity analysis should therefore form an important part of the assessment.

5.3.6 Selection of preferred scheme

In addition to the formal quantitative evaluation of economics, there is a need to take a broader view of the issues. For example, there might be overriding factors which, while not expressed in financial terms, might suggest that a scheme is eliminated or short-listed for consideration as a preference. This might include non-quantified environmental enhancement, r the robustness of the perceived merits of a given option, uncertainties and so on.

5.4 Design principles

5.4.1 Design criteria

There are some 2,935 km of built defences around the coast of England and Wales. A few kilometres, at most, of new or replacement defences are built each year, that is, significantly less than 1 per cent of the total stock of built coast defences. It is not surprising, therefore, that there is a considerable demand for assessing, installing, maintaining or extending toe structures to existing coastal defences. An important consideration in many cases is, therefore, incorporation of new toe protection works with an existing structure.

Figure 5.2 shows the seawall at Goodrington Sands, Paignton. Numerous additions to the wall since original 19th century construction can be seen, the last being the concrete/pile toe added by Torbay Council in 2007.

Design criteria can be separated into two main groups:

- those concerned with the functional purpose of the toe (for example; avoidance of undermining of the coastal defence);
- those that relate to the interaction of the toe with its environment (for example, heritage).

Figure 5.2 Goodrington Sands Seawall, Paignton (courtesy of ENBE)

Functional and performance related criteria

The toe is just one component of a complete coastal defence structure. It should be designed on the basis of a number of design criteria that will feature to varying degrees depending on the nature and use of the coastal structure.

Design is aimed at producing a structure that avoids or mitigates problems and failures, so it follows that much can be learnt from known or recurring shortcomings. CEM Part VI, Chapter 2 (USACE 2012) provides a comprehensive catalogue of failure modes of typical coastal structure types. Based on this reference, Table 5.1 summarises those failure modes connected with the toe or leading edge of the main structure.

Table 5.1 Toe related failure modes of main structure types

Failure mode	Main coastal structure type				
	Rubble mound (breakwater)	Revetment	Dike	Gravity wall	Sheet pile /wall
Sliding of armour/main structure into scour hole – undermining	✓	✓	✓		
Subsidence of armour blocks into fine material – liquefaction	✓	✓			
Instability of toe armour on a hard substrate in breaking waves	✓	✓			

Sliding of main structure due to geotechnical imbalance				✓	✓
Overturning of main structure due to geotechnical imbalance				✓	✓
Slip circle failure	✓	✓		✓	✓
Foundation settlement	✓	✓	✓	✓	✓

Note ✓ = referenced in USACE (2012)
✓ = added in Table 5.1

The failure modes outlined in Table 5.1 can be grouped together to arrive at the following generic list of functional design criteria:

- to counter the effects of beach lowering and undermining;
- to counter the effect of liquefaction at the toe (this can induce geotechnical imbalance at a vertical faced structure as well as result in subsidence of armour at a rubble toe);
- to ensure or restore the geotechnical stability of the whole defence structure – this objective includes mitigating against the risks of sliding, overturning, slip circle failure and excessive settlement;
- resistance to wave and current loading including stability of toe armour on a hard substrate.

Interaction of the toe with its environment

Impact, behaviour and environment related factors, both positive and negative, might include the design criteria listed below. This list is by no means exhaustive and each location should be carefully considered on its own merits:

- effects on hydraulic performance of the main structure;
- effects on coastal processes;
- durability – abrasion, corrosion and structural deterioration;
- public safety in construction and operation;
- effects on the natural and built environment;
- heritage and visual aspect;
- amenity - aspects concern both the beach and the structure itself;
- access both to and along the beach (pedestrian, vehicular and boats).

5.4.2 Toe structure types

Chapter 2 outlined a number of different toe structure types. It also pointed out that in many cases the project entails restoration of an existing defence structure rather than all new construction. Consequently, there are many variants on the types of toe existing and those that can be applied, utilising a range of techniques, often conditioned by the need to integrate with existing features. It is neither practical nor useful to describe the design principles of every structural permutation. Hence, for the purpose of this exercise a number of toe structures are grouped under generic headings as shown below:

- **Rubble structures type:**
  - concrete unit or rock revetment
  - tipped rock
  - rock blankets.
- **Mattress type:**
  - interlocking concrete armour
  - gabion mattresses
  - rock blankets.
- **Concrete (gravity) type:**
  - concrete apron
  - concrete foundation/underpinning
  - steps integrated with toe.
- **Sheet pile type:**
  - cut off wall
  - sheet pile underpinning.
- **Asphaltic construction:**
  - apron
  - grouting for rock or stone.
- **Cribwork type:**
  - timber and concrete cribs containing rocks
  - gabion baskets.

Table 5.2 highlights the most relevant criteria applying to each of the generic structure types. The remaining sections of this chapter outline the design principles in respect of the key criteria listed in Table 5.2.

Table 5.2 Relevance of criteria to toe structure types

Main defence structure criterion	Generic toe structure type					
	Cribwork	Rubble	Mattress	Concrete	Sheet pile	Asphaltic
Undermining	H	H	H	H	H	H
Liquefaction at the toe	L	H	H	H	H	H
Geotechnical stability	H	M	M	H	H	M
Resistance to wave and current loading	H	H	H	L	M	H
Hydraulic performance	M	H	M	H	H	M
Effect on coastal processes	L	M	M	M	M	M
Public safety	M	H	H	H	H	H
Natural environment	L	H	H	H	H	H
Heritage and Visual Impact	M	H	H	H	H	H
Amenity	L	H	M	M	H	H

Note: Relevance: **H**igh, **M**edium, **L**ow

5.5 Undermining

5.5.1 Beach lowering and scour

Fundamentally, there are two mechanisms to consider:

- Lowering of the beach due to coastal processes both in the long term (for example, sediment loss through longshore transport) and the short term (for example, storm-induced drawdown)
- Scour, induced by the presence of the coast defence structure itself (for example reflection from a seawall). Appendix A of this report and Sutherland et al. (2003) describe the processes of beach lowering in front of coastal structures and the reader is referred to these for further information on the processes.

In terms of the design of a new or replacement toe, it might reasonably be asked whether the toe design itself can influence the extent or depth scour. In this respect the arguments are similar to those that might be applied to the coastal defence structure as a whole (for example, reducing reflectivity reduces scour potential). Clearly, however, for these factors to apply to the toe structure, it has to be exposed (or only moderately covered).

In most cases, the critical condition for scour (that is, when the maximum depth of scour is reached) is likely to occur at a high water levels when the largest waves can reach the defence. Hence, unless the toe is quite exposed, it is unlikely to be a major influence on scour compared with the upper face of the defence structure. Beach levels can fluctuate widely and change rapidly under storm conditions. Previously covered toe structures may then become influential in the scouring process. In cases where the toe is substantially exposed, it should be examined according to the same principles as the main body of the wall (looking at hydraulic performance – overtopping, wave reflection and impact loads) and this should be checked during design.

5.5.2 Undermining failure

The term ‘undermining’ is not consistently defined in technical or non-technical literature. Perhaps this is because other modes of failures are likely to have occurred before a structure becomes truly undermined; see also USACE (2012). The case study from Corton, Lowestoft, in Appendix C describes a case of true undermining of a piled wall (also Figure 5.3). For discussion purposes here, and generally in these guidelines, ‘undermining’ is simply taken to mean the condition at which the beach is below the bottom of the main defence structure such that further beach lowering would lead to erosion beneath the structure.

Figure 5.3 Undermining of wartime coastal structures at Kilnsey, Holderness (courtesy of ENBE)

Some gravity structures can tolerate a measure of undermining without collapse, but the margin between some undermining and failure can be quite small and difficult to predict. Apart from exceptional cases where some undercutting has been allowed for in design, it should be assumed that undermining is not acceptable.

Figure 5.4 illustrates toe failure of a revetment due to undermining:

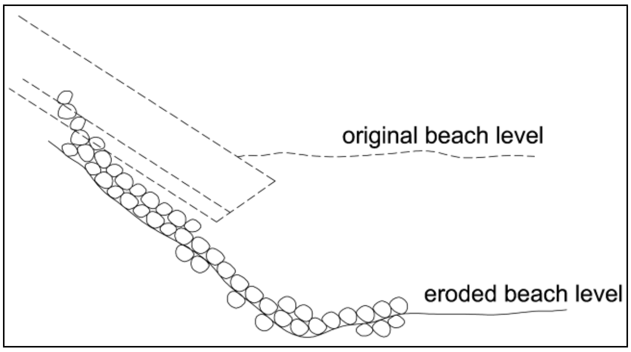


Figure 5.4 Undermining failure of a revetment

5.5.3 Mitigation

Figure 5.5 illustrates a variety of toe structures as applied to revetments armoured with rock or other proprietary type units.

Toe structures, including those depicted in Figure 5.5, mitigate against undermining of the superstructure in one of two ways:

- as a static structure, that is, of sufficient depth and inherent stability to avoid being undermined itself (for example, a stable rubble mound, concrete toe or sheet piling);
- as a flexible mattress that adapts to the lowering bed level, thus preventing undermining of the main structure (for example, various flexible mattress types and asphalt).

A major determinant in the choice between these two fundamental options is the depth of sediment and the geology at the toe.

Where a defence structure is underlain by rock or by rock beneath a shallow depth of sediment, then there is an opportunity to found a toe structure on the hard substrate. Where a stratum with limited resistance (for example, clay or weak rock) is located within a manageable depth below the mobile deposits, this may provide founding for a toe (see also the case study from Overstrand, Norfolk, in Appendix C). Episodic beach lowering would be, or at least has previously been, confined within this limited depth. However, longer term beach lowering, which might include the erosion of the underlying soft rock such as clay or marl, needs to be checked.

When considering the underlying geology, factors other than undermining are also important to the design.

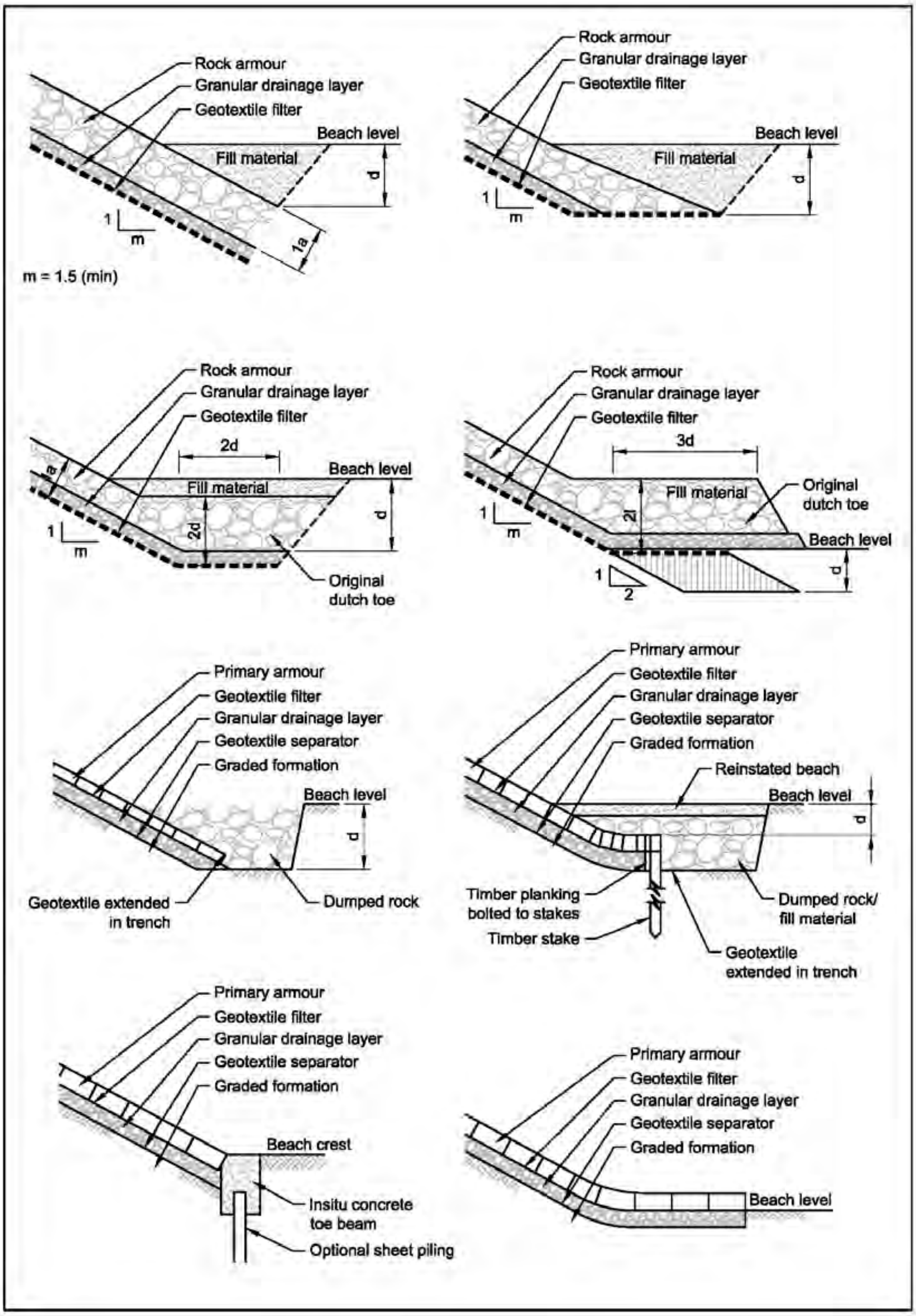
A flexible toe would be unsuitable where:

- it is required to maintain ground level for reason of providing passive support to the main structure (see Section 5.7);
- it is required to provide a rigid support for armour on the main structure face (for example, single layer armour units), unless the mattress was of such width and robustness as to eliminate the risk of lowering at its connection with the main revetment;
- its width, depth or other properties make a compliant toe uneconomic compared with a more rigid structure;

- the apron width presents an unacceptable intrusion into the recreational beach or causes damage to an area of conservation interest.

A flexible toe can, however, provide a more practical/cost-effective solution for the avoidance of undermining in some cases such as:

- where the main structure is itself of a flexible type of construction (for example, riprap slope protection – in some cases, the toe may simply be an extension of the main revetment);
- at sites where more rigid forms of construction would be impractical;
- at sites subject to large but gradual bed variation/movement.



Note: 'd'$_s$ is anticipated scour depth and '1:m' is structure slope.

Figure 5.5 Typical toe details (after McConnell, 1998)

5.6 Liquefaction at the toe

5.6.1 Consequences of liquefaction

Wave-induced liquefaction at coastal structures is described in Appendix A.

At sites that are susceptible to this effect, liquefaction presents a problem for securing the toe structure. Loss of shear strength of the bed material can lead to various negative effects depending on the type of defence structure.

For rubble structures and shallow concrete toe structures, liquefaction can result in rock or heavy concrete units sinking into the liquefied bed material – illustrated in Figure 5.6.

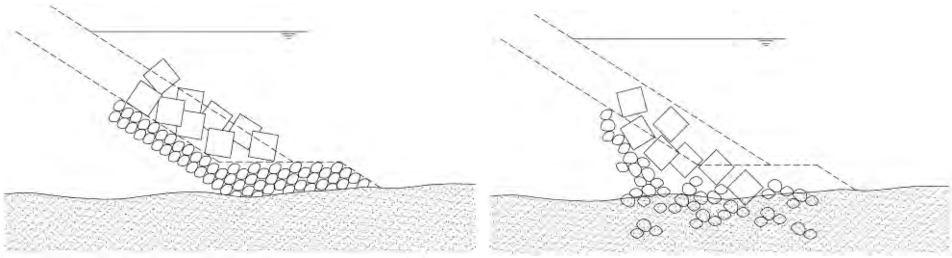


Figure 5.6 Sinking of armour units into liquefied seabed

For sheet piling and deep vertical concrete toes, the loss of shear strength on the seaward side of the structure leads to loss of passive resistance provided to the toe, thus compromising its ability to resist geotechnical loading from the active, landward side. Further to this, liquefaction can provide conditions whereby material behind the wall can flow out beneath the structure, resulting in subsidence of the area behind the wall. The tendency for this depends on the flow path, which will be shorter for certain types of construction (for example, anchored walls that are less reliant on depth of penetration than cantilevered walls).

Depending on the nature of the liquefaction, that is, whether it is momentary (and localised) or residual (and widespread), mattresses and compliant toes might in the former case offer a measure of protection to the main defence structure, or in the latter case be susceptible to sinking into the liquefied ground.

5.6.2 Mitigation

Recommendations for dealing with wave-induced liquefaction are limited at the present time. The process is not readily observed and hence, historically, it has been difficult to link failure to liquefaction when other destructive mechanisms are also at play. The following advisory notes are therefore given from a pragmatic perspective rather than being based on robust scientific evidence:

- **Existing defence structures for which a problem has been identified:**
  - For vertical wall structures where it is believed that fill is being lost through flow beneath the structure, the situation might be relieved by installing controlled and filtered drainage paths, thus providing a lesser path of resistance for outflow of water, while still retaining soil within the wall. Clearly, this is a more difficult construction operation when applied to an existing structure than it would be for a new one.

- For rubble mound structures where it is evident that the toe rocks/units are sinking into the bed, mitigation might entail reconstruction of the toe to install a bedding layer and scour apron in order to lessen the point pressures of the individual armour units. Mattress or a shallow rock apron might also be used to inhibit liquefaction at the face of a vertical wall.
- The success of these mitigations (the latter example in particular) depends on the type of liquefaction that might occur (that is, momentary or residual). Better quantification of this can be derived from Sutherland et al. (2007).

- **New defence structures:**
  - For new structures, there is clearly more opportunity to allow for liquefaction in the initial design. For example, the design of a piled wall can include a pile length allowance to counter the passive pressure lost to liquefaction. This might be preferable to installation of a scour mattress that requires future maintenance. As with scour, avoidance of liquefaction problems is likely to be preferable to cure.

5.6.3 Site Investigation

Sutherland et al. (2007) may be used to derive estimates of liquefaction depth and extent. Predictive analysis such as that referred to above requires the input of various site-specific factors as described in Appendix B.

5.7 Geotechnical stability

This section makes reference principally to British standards and practice. However, it should be noted that 'a new European suite of geotechnical design, testing and construction documents will in due course largely replace British codes and standards' (CIRIA 2008).

The European code, EC7-1, may be applied to new projects and stabilisation of existing structures but it does not deal specifically with the assessment of existing structures or reuse of existing foundations. Its application to toe protection works may therefore be limited because many cases are concerned with the risks to existing structures and/or their incorporation into new works. Nevertheless, new design may consider the use of EC7-1, in particular where important distinctions may be made between 'ultimate limit states' (states associated with collapse or with other similar forms of failure) (see Section 3.1) and 'serviceability limit states' (states beyond which specified service requirements for a structure or structural member are no longer met).

Geotechnical stability problems often relate primarily to the overall defence rather than the toe. However, they are often exacerbated or caused by problems at the toe and for this reason they are considered in this manual.

5.7.1 Sliding and overturning of the main defence structure

These failure modes arise out of an imbalance between the active geotechnical load, tending to move the structure seawards, and the passive resistance.

Sliding failure is confined essentially to monolithic type structures (Figure 5.7) while overturning failures are associated mainly with vertical or near vertical monolithic or piled defence structures (Figures 5.8 and 5.9). See also the case study from Lowestoft South Beach in Appendix C.

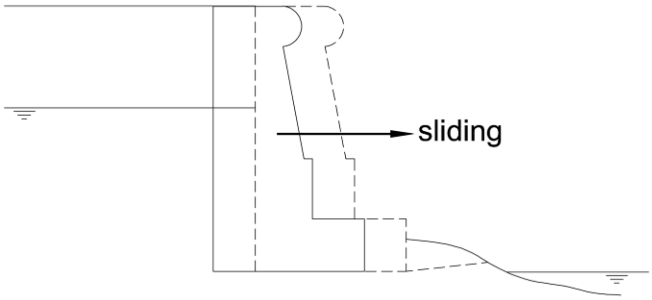


Figure 5.7 Sliding failure of gravity wall

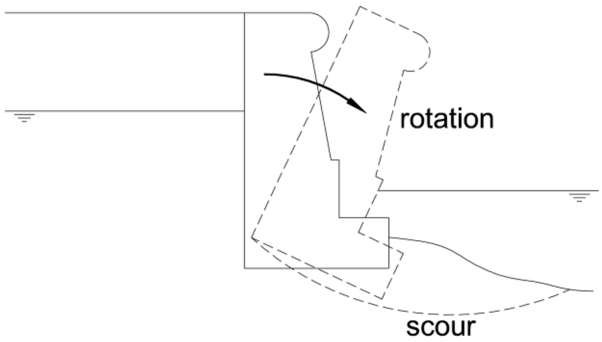


Figure 5.8 Overturning failure of gravity wall

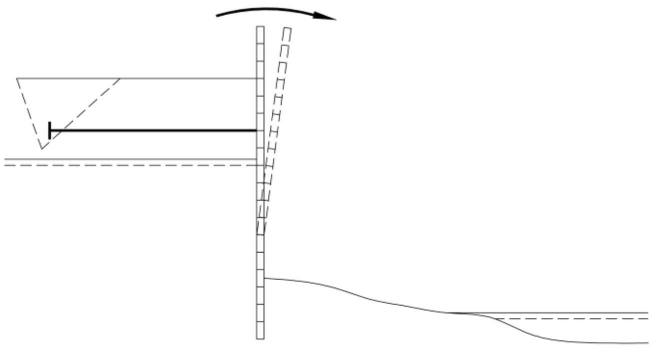


Figure 5.9 Rotation failure of sheet piled wall

Gravity walls

The potential for failure through sliding and overturning is of particular concern with regards to old seawall structures, the following being relevant factors:

- factors of safety used in former times (for example, in the 19th century) would not necessarily satisfy current standards;
- beach lowering (if prevalent) over many years or decades of service;

- possible scour induced by the presence of the vertical wall itself.

The factor of safety for sliding is defined simply as the ratio of the sum of the resisting forces compared with that of the disturbing force:

$$\text{Factor of safety (sliding)} = \frac{(P_p + F_b)}{P_a} \qquad \text{(Eqn 5.1)}$$

where:

P_a = sum of active geotechnical and hydrostatic forces (that is, on the landward side, tending to push the seawall outwards) (Figure 5.10)

P_p = sum of passive geotechnical and hydrostatic forces (that is, on the seaward side, tending to resist the active pressure and movement) (Figure 5.10)

F_b = the friction on the base of the wall, also tending to resist the active movement of the wall (equals the weight of the wall multiplied by the friction coefficient of the wall base on the ground beneath).

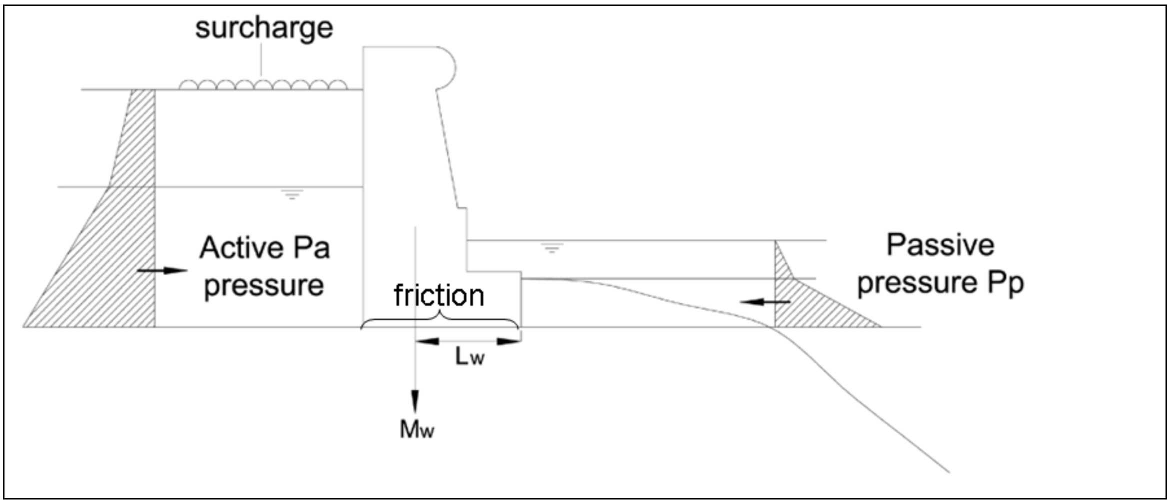


Figure 5.10 Simplified forces on a gravity wall

For overturning failure, the disturbing moment is that derived by taking the moments of the active forces about, or close to, the seaward edge of the base. The resisting moment is the sum of moments of the passive forces plus the moment of the wall weight (M_w), about the same axis. As moments are taken at or close to the horizontal line of action of friction on the base, the latter does not appear in the simple case. Hence:

$$\text{Factor of safety (overturning)} = \frac{(P_p \times \text{passive moment arm} + M_w \times L_w)}{(P_a \text{ x active moment arm})} \qquad \text{(Eqn 5.2)}$$

BS 6349-2:2010 recommends minimum factors of safety of 1.75 for sliding and 1.50 for overturning (BSI 2010).

The total geotechnical loads (both active and passive) are calculated by summing the incremental forces attributed to each layer in the soil profiles on the landward and seaward sides of the wall. The pressures imparted by each layer depend on:

- soil type, for example, fine sand, coarse sand, gravel and so on (hence, angle of friction of the soil);
- whether cohesive, and hence the inclusion of the effect of cohesion (negative on active force, positive on passive force);

- weight of overburden (weight on the layer including the weight of the layer above a given level), remembering that overburden may reduce over time through erosion;
- specific weight of the soil in the layer;
- ground water pressure (allowing for buoyancy, but added on as a hydrostatic pressure).

Each incremental soil layer has a moment arm about the base. Hence the total moment (both active and passive) is actually the sum of the increment moments on either side of the wall (the overall moment arm, as shown simplified in Equation 5.2 is therefore the average value that would equate to the total moment/total force).

The calculation of active and passive pressures can be found in most standard text books on geotechnical engineering, and is well described in the *Piling Handbook* (Arcelor 2008). This is a comprehensive subject, which is dealt with specifically and at some length by other texts and, as such, the details are not reiterated here. It should be mentioned, however, that the example above has been deliberately simplified for illustration purposes. In practical applications there are numerous possible complicating factors. Examples include:

- ground surcharges due to point loads, line loads (for example, set back wall), vehicular loads and so on;
- sloping ground;
- variable beach levels;
- variable sea levels;
- variable ground water levels;
- complex wall structures;
- inclusion of ties, struts or other supplementary supports;
- poorly understood (buried or rear face) seawall geometry;
- poorly understood properties of made ground behind a wall.

In practice, a number of simplifying assumptions have to be made. Experience of this type of analysis is therefore necessary in order to apply appropriate simplifications.

Piled walls

For sheet piled walls, the analysis for overturning failure (and design) uses the same basic principles for calculating the active and passive forces as used in the gravity wall example above. However, there are significant differences in how the two types of structure are designed or calculated to respond to the applied loads.

Whereas a gravity wall is considered to be rigid and monolithic, a sheet piled wall is treated as elastic. For sheet pile wall analysis, two possible flexure models may apply:

- **Free earth support** – in this case the pile is modelled as a simply supported vertical beam which, is assumed to be free to rotate at its toe.
- **Fixed earth support** – in this case the pile has greater penetration into the ground such that the toe end of the pile is considered as fixed (not free to rotate).

In order to achieve fixity in the ground, a wall designed on the fixed earth support principle is longer than that designed on the free earth support method but it carries a reduced bending moment. The reduced bending moment means that a lesser pile section is required, or might have been required in the retrospective case of an existing structure.

Practical factors may indicate that one method might preferably be used in favour of another, for example:

- Limited driving depth due to the proximity of a rock head might suggest that the free earth support method be used.
- Ready availability of a low modulus section (lower bending capacity) but in ready-cut long lengths (for example, recycled piling) might be suitable for reuse if designed according to the fixed earth principle.

The free earth support method of necessity requires that the pile be propped or anchored at or near the top. A true cantilever wall must, therefore, be designed on fixed earth principles. These and related factors are important considerations to both the designer and the analyst.

The integrity of a sheet pile wall depends on two basic conditions being satisfied:

- sufficient passive moment to resist the active moment for the chosen flexure model;
- sufficient pile section capacity (known as section modulus) to sustain the pile bending moment within the limits of permissible stress and deflection; note that future loss of section through corrosion must be allowed for (see Section 4.3.4).

This detailed subject is comprehensively covered in the *Piling Handbook* (Arcelor 2008) to which the reader is referred. This reference includes several worked examples including cases for cantilevered and tied walls, using both free earth and fixed earth support principles. The *Piling Handbook* also contains design charts for simple cases that can be used for initial estimation or concept design purposes. These should be used with caution as there are many factors that can differ from 'the norm'.

As with the gravity wall case outlined above, there are numerous complicating factors involved in practical design. Experience is needed in the application of these design methods. While there are several software packages available for the analysis of these situations, it is important that they are applied by experienced practitioners in order to appreciate their limitations and produce realistic results.

The position of the toe of the pile in relation to both the landward (active) side and seaward (passive) side is a fundamental determinant in sheet pile wall design. Whereas land levels will normally remain much the same with the passage of time, clearly beach levels can vary both in the short and long term. Beach lowering would tend to impair the load resisting capacity of an older structure.

Sliding and overturning – mitigation

The previous section outlined the crucial factors that determine the stability of gravity and sheet pile defences as a result of geotechnical imbalance across the structure section (for example, due to beach lowering). Whether designing a new structure or restoring an older one, there are two possible fundamental approaches:

- retain/restore a sufficient beach level at the defence structure;

- accepting that the desired beach level cannot be achieved and installing a toe structure to support the main defence structure.

Maintaining beach level

Beach restoration might entail recharge, recycling and/or other beach management measures to regain and/or retain the required beach level. Local scour, induced principally by the seawall itself, might be mitigated using a scour apron, mattress or similar. While these toe systems might prevent local scour, they will not prevent more widespread beach lowering (for example, through long-term sediment loss), although they may be designed to accommodate a measure of beach lowering in terms of their own survival. See also Sections 5.5 and 5.6.

Where it is required to provide support to a deep structure such as a piled wall or a deep founded gravity wall, the apron would need to be of sufficient width to avoid the influence of reduced passive resistance resulting from scour occurring beyond the mattress itself. The width of the mattress, W, to achieve this condition is given by CEM (USACE 2012) for cohesionless beach deposits as follows:

$$W = d_e / \tan(45 - Ø/2) \text{ or approximately } 2d_e \qquad \text{(Eqn 5.3)}$$

where:

$Ø$ = the angle of internal friction of the beach sediment

d_e = the depth of penetration of the structure on the beach side (allowing for beach lowering as distinct from scour).

USACE (1995) also recommends that:

- for toe structures at sheet pile walls, W = not less than $2H_i$ (Eqn 5.4)
- for toe structures at gravity walls, W = not less than H_i (Eqn 5.5)

and

W = not less than 40 per cent of the water depth at the structure (Eqn 5.6)

where:

H_i is the incident wave height.

Comparison of the inequalities of Equations 5.4–5.6 with Equation 5.3 suggests that the latter condition, relating to depth of penetration, will generally be the more onerous in cases where it is required to provide geotechnical protection to a deep piled structure. It also follows that the width W could, in this case, be rather large. For example for piling with depth of penetration d_e = 6m and angle of friction of 30°, W becomes 10.4. or 12 m using the approximation.

Thus, depending on the circumstances, it may be possible for geotechnical problems due to scour to be pre-empted by the installation of a mattress or shallow blanket. The last example indicates that there may be practical difficulties in providing effective protection by these methods (for example due to size). Moreover, if a problem of geotechnical instability is already present or is likely to arise due to general beach lowering, then lightweight measures such as this are unlikely to work.

Structural mitigation

The designer must carefully consider the circumstances and requirements of each specific project.

A more engineered structure will probably be required where additional toe support is required to preserve or reinstate the resisting forces on a superstructure and this cannot be achieved by simply mitigating scour, and beach management is not feasible. Options to achieve this may take the form of:

- a deeper toe to the superstructure;
- mass concrete blocks at the toe;
- a steel sheet piled toe;
- a combination of the last two measures;
- rock placed against the main structure wall.

These toe structures will be subject to the same threats as described elsewhere for the main defence superstructure (that is, beach lowering, scour, liquefaction and so on) and must be designed for accordingly. Important aspects of design are detailed below.

Deeper toe

The most fundamental option, albeit not without practical constraints (including construction) in many cases, is simply to extend the toe to sufficient depth to avoid geotechnical instability. However, this is a measure that principally applies to the design of new structures. For existing structures, extending the toe to provide additional passive support is not necessarily straightforward:

- Underpinning of gravity structures can provide protection against undermining, but there may be difficulties in attaching sufficient mass to effect satisfactory geotechnical support.
- Lengthening piles could be achieved by attaching new pile length to the top and re-driving, but this will generally be complicated or impractical due to the presence of pile caps, walings, tendons and so on within the existing structure.

Mass concrete block

A concrete mass added to the wall may be designed as:

- an addition to a gravity wall, having the effect of improving both the sliding resistance and overturning restoring moment of the wall itself;
- an independent mass that adds to the sliding resistance of a monolithic wall and, in the case of a deep wall, provides a fixed overburden pressure and hence improved passive resistance.

Where the main wall is of sheet piling, a concrete block may similarly be used to provide friction (thrust) resistance and to improve passive resistance at the pile through increased overburden.

Steel sheet piled toe

In essence this method entails construction of a new wall to seaward of the existing structure. As such it will be subject to the same design principles as a main wall constructed in steel sheet piling. The distance between the walls will determine the interrelation of load transfer between the two – that is, the closer they are the greater the transferred active loading will be to the new wall. The surface gap between the existing wall and the new wall/toe piling will need to be armoured in some way; this may take the form of a slab possibly coupling up as a pile cap for the new construction (see also below).

Combination of concrete mass and sheet piled toe

A concrete mass placed between an existing wall and a new sheet piled wall/toe structure can also provide part of the resistance to sliding and overturning of the main wall. If the beach level is already close to the base level of the existing wall, it may be undesirable to build the new wall to a much higher and, therefore, obtrusive level. In this case, a concrete block can act both as resistance and as a thrust block to transfer active load through to the new pile. Note that in this case, the position of the thrust and its magnitude will have a sensitive effect on the new pile design – see Figures 5.11 and 5.12, and the case study from Lowestoft South Beach in Appendix C.

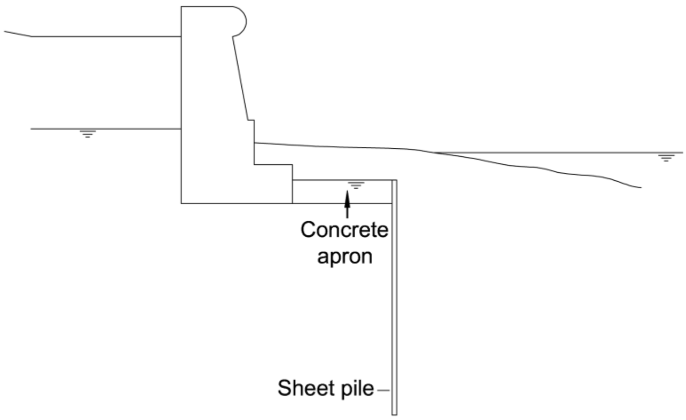


Figure 5.11 Example of thrust block and piled toe

Figure 5.12 Geotechnical support for the concrete seawall at Lowestoft (courtesy Waveney District Council)

Rock mound

A mound or fillet of rock can be used to provide passive support, both directly to the existing wall and indirectly by way of a ground surcharge on the passive side. A properly designed rock mound incorporating appropriate sublayers/geotextile can also provide scour resistance. Rock used in this way is sometimes applied as an interim measure pending installation of, or incorporation into, a long-term solution (for example, a beach recharge and management programme). Where very large rock is used, boulders should be placed against the existing wall in a manner that provides a good bearing (and is secure) while avoiding impact or pressure damage to the existing wall face.

The design of the rock should follow the principles set out in Section 5.4. The design should, moreover, make due allowance for the limited permeability of the section (that is, due to proximity of the solid interface with the existing wall). Note that short-term/interim/emergency measures may well be required and these are likely to be carried out at minimal cost or using simplified design processes, and this could translate into a minimal cross-section being used.

It is wise to design for a longer service life than the immediate urgencies might suggest (for example, 10 years rather than 2 years), as the very act of providing some relief can have the effect of changing priorities. An example is provided in Figure 5.13.

Figure 5.13 Example of rock used to provide passive support

5.7.2 Slip circle failure

Slip circle failure is a form of slope instability. Where the failure is deep seated, it can pass beneath a coastal defence structure including a piled wall (Figure 5.14).

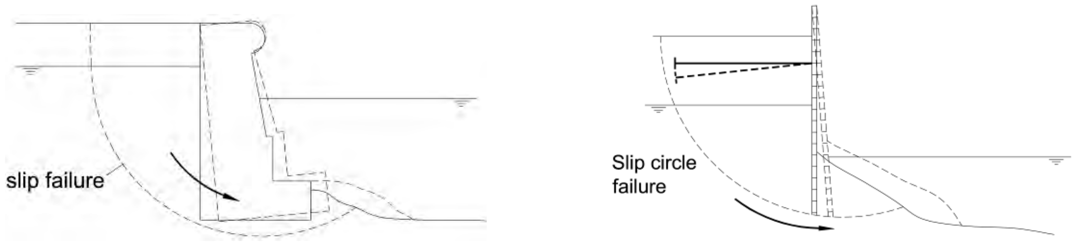


Figure 5.14 Slip circle failures

The subject of slope stability is covered by most geotechnical text books and is dealt within the context of coastal structures in CEM (USACE 2012).

Instability can occur with weak soils or when structures are placed over weak soil strata. Groundwater is a significant factor and, with a coastal structure, this can be aggravated by tidal action, landward ground being saturated by overtopping, or drainage problems.

Slope instability and slip circle failure are serious issues, the significance of which must be assessed at the outline design stage and mitigated through appropriate detailed design of the whole defence structure. Retrospective correction of slip circle failure is likely to involve major engineering and probably rebuilding of the whole defence structure. Pre-emptive measures to reduce the risk of slip circle failure may sometimes be possible but are likely to be major endeavours, for example surcharging with a deep and extensive apron, or very substantial beach nourishment. They are also likely to include measures applied to the main part of the defence (for example, re-grading the slope).

This subject is not described further in this manual and the reader is advised to refer to the reference texts for further details of the soil mechanics and analysis.

5.7.3 Foundation failure

Settlement is a function of the defence structure (type, materials, density, load distribution and so on) and the soils on which it is founded. Soft soils including weak clay, silty sand and mud are most likely to be problematic, with vertical or differential settlement of heavy structures being possible.

Settlement takes place over varying time periods:

- **Instantaneous** (applies to high and low permeability soil). This refers to settlement that occurs during or upon completion of construction.
- **Primary** (applies to low permeability soil). This relates to the gradual consolidation of the soil due to the dissipation of excess pore water and takes place over a long time period – possibly the life of the structure.
- **Secondary** (applies to high and low permeability soil). This long-term creep of the soil material is usually less than the combined effect of the instantaneous and primary settlements.

Although the toe of a given structure is located at or about the foundation level, there is little scope for mitigating settlement through the toe detail. This is because settlement relates to weight distribution of the whole structure and not just a small part of it. Where a structure is to be founded over poor bed materials, the significance and implications of settlement need to be examined on a structure-wide basis. Situations where particular consideration would need to be given to the toe would, however, include the following:

- If the toe is of a markedly different form of construction to that of the main defence (for example, a mass concrete block compared with a shallow revetment). then differential settlement relative to the main construction could occur.
- If the toe is required to support the leading edge of a primary armour layer, especially a single layer armour system placed on a steep slope, then settlement of the toe should be checked and evaluated.

Further advice on evaluating settlement can be found in CEM (USACE 2012).

5.8 Resistance to wave and current loading

Table 5.1 identified rubble mound and revetment type coastal structures as being prone to toe damage through wave or current action. This is because they are the structure types most likely to have toes of similar construction (that is, of rubble). This perceived vulnerability is also reflected in the related Table 5.2, which identifies toes of rubble construction, together with mattresses and asphaltic construction, as being most at risk.

Monolithic concrete and piled wall construction are identified as being at less risk of damage because they naturally possess a strong degree of internal strength. Nevertheless, for some applications, the evaluation of wave loads could be significant (for example, see CEM (USACE 2012) for wave forces on vertical walls) but these considerations will tend to relate to the larger superstructure. This topic is, therefore, not dealt with in further detail here.

5.8.1 Rubble mound toe structures

The key advantages of a toe structure consisting of rock are:

- flexibility, that is, the potential to accommodate changes in beach level, and scour holes;
- potential to dissipate wave energy, thus reducing wave loads on the toe and the main coastal defence structure, and also reducing the tendency for scour.

Selection of the most appropriate design concept will depend on:

- the phase in the life cycle of the coastal defence structure (for example, is it design of a new structure or an emergency repair?);
- the soil type of the beach or foreshore (rock, gravel, sand, silt, clay and so on);
- the hydraulic load conditions (waves, currents, tidal levels).

Readers are recommended to consider the design methodology for rock structures as described in *The Rock Manual* (CIRIA et al. 2007) as the state-of-the-art methodology (as of 2012). Typical toe details are also given by McConnell (1998), see Figure 5.5.

When designing a rock toe for stability under wave attack two principal failure mechanisms may be considered:

- displacement of rock;
- loss of bed material through the rock matrix.

The following summarises the design methodology for these most common failure mechanisms.

Failure mechanism: displacement of rock

For rock slopes under wave attack, formulae for rock stability are treated in detail in *The Rock Manual* (CIRIA et al. 2007), being those due principally to Hudson, van der Meer and van Gent. Empirical formulae developed by van der Meer are also presented for typical submerged toe structures, where the crest of the toe is well below the trough of the design wave. These formulae have been developed for the design of rubble mound breakwaters, but they may be applied appropriately and with caution for submerged rock toes in front of coastal defence structures as well. Some 65 pages of *The Rock Manual* are given over to describing the background, formulations and limitations of the various equations and methods that might be applied; the reader wanting to use design formulae is therefore advised to refer to *The Rock Manual.* The key features of practical coastal defence toe design are summarised below.

The relevant design parameters are:

- water depth (under the design event) at the rock toe and the elevation of the toe in relation to design sea level;
- wave conditions (wave height or statistical distribution of wave height, possibly affected by bathymetry of foreshore, wave period, and wave direction relative to the shoreline);
- current loading;
- main structure geometry (sloped structure or vertical wall, structure with or without berm, structure crest height and toe crest height);
- relative density of rock;
- expected or allowable damage level of the rock toe.

The interaction between a rock armoured toe and the incident wave field depends crucially on the relative depth of the toe, h_t/h (Figure 5.15).

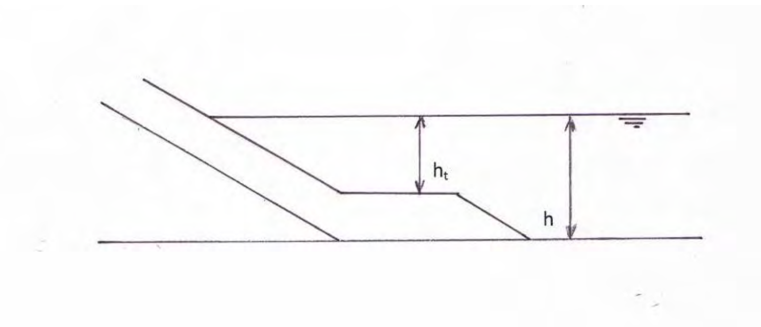


Figure 5.15 Relative depth of toe

If the h_t/h ratio is low (<0.5), that is, the toe is relatively close to the surface in relation to the ambient water depth, then the design tends towards that of an armoured slope that is partially or substantially submerged. Corrections derived by Vermeer (1986) and van der Meer (1990) may be applied that allow for different slopes in a composite structure but essentially the rock sizing design follows the general principles derived by van der Meer (1988), van Gent et al. (2004) and others for rock slopes.

If the h_t/h ratio is high (>0.5), then different principles can be applied to justify a smaller rock size due to the greater water depth between the toe and the position of greatest wave impact. The methods described in *The Rock Manual* (CIRIA et al. 2007) relate the stability number $H_s/\Delta D_{n50}$ to the ratio h_t/h where:

H_s = incident significant wave height

Δ = the relative density of the rock = $(\rho_s - \rho)/\rho$

D_{n50} = characteristic stone size

The rock sizes derived from *The Rock Manual* methods vary massively with the ratio h_t/h; for example, $H_s/\Delta D_{n50} = 6.5$ for $h_t/h = 0.8$, while $H_s/\Delta D_{n50} = 3.3$ for $h_t/h = 0.5$, thus implying a factor of nearly two-fold on D_{n50} or 7.6 on rock mass. Moreover the values thus derived relate to 0–10 per cent damage. Based on the work of van der Meer, *The Rock Manual* develops more sophisticated equations that relate rock size to the number of units displaced. *The Rock Manual* also contains relevant formulae for rock toes in front of vertical structures being derived primarily for vertical wall breakwaters and large caisson type structures.

The Rock Manual tends to focus on larger breakwater type structures where the toe is more often in deeper water and other practical considerations (for example, placing) might favour more optimal rock sizing. However, this manual on the management of toe structures advises extreme caution in applying the latter approaches (for example, those that formulate a reduction in rock size with water depth) for general coastal defence applications, for the following reasons:

- In most situations, the toe rocks will have a much reduced depth of water covering them at some stage of the tide (thus implying a low ratio of h_t/h).
- In practical terms, the toe structures of coastal defence are generally small compact features; the loss of any rock can seriously affect the integrity of the structure and can be difficult or relatively expensive to rectify.
- In many cases, the rock forming the toe will simply be an extension of armour layer comprising the upper revetment, and it may be desirable to actually increase the toe armour size or take other steps to secure the vulnerable leading edge (see below).

In cases where the toe structure is founded on a hard substrate, the stability of the leading edge of the armour is likely to be reduced due to reduced friction, especially under the action of breaking waves (Figure 5.16). These rocks are more vulnerable to movement due to the reduced friction resistance at the bed and the absence of the mutual support of rocks on the seaward side.

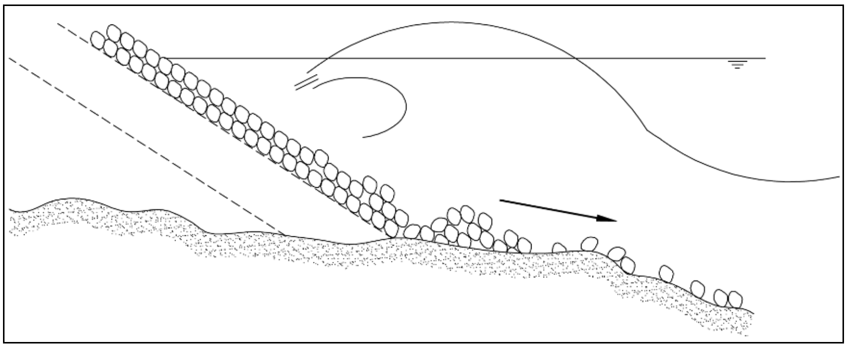


Figure 5.16 Sliding of toe armour on hard substrate

The following alternative mitigations apply to armour on a hard substrate. The first two apply equally to armour on a sedimentary substrate:

- use of oversized toe rock to counter the reduced interlock between units and reduced bed friction;
- excavation of a trench to secure the leading edge of the armour;
- use of piles (if practicable) to form a crib to secure the leading edge of the armour;
- creation of a concrete toe beam cast onto the rock bed and secured with dowels;
- anchor bolts to hold the leading armour blocks in place – see CEM (USACE 2012).

For more sheltered sites, the factors listed above would still apply to some degree.

In cases where wave activity is minor, stability considerations may be dominated by the current. Bed and slope protection for current-induced erosion has been studied over many years, the earlier significant contributions being attributed to Shields and Izbash. *The Rock Manual* (CIRIA et al. 2007) describes the historic research and more recent developments, in particular: those due to Pilarczyk (1995). In cases where turbulence may be high (for example, near to culverts or other shore structures), the formulae of Escarameia and May (1992) are given. Formulae by Maynord (1995) take into account the thickness of the stone blanket.

Physical model tests may be carried out to optimise a given design, especially in cases where the structure geometry or load conditions deviate from the valid ranges of the available design formulae.

Failure mechanism: erosion of seabed material through voids in the rock layers

Loss of bed material through the voids in rock layers can lead to a lowering of the bed beneath the rocks with consequent lowering of the rock itself until it becomes embedded – an effect akin to that of liquefaction. Unless an allowance is made for this, as for example in the case of a falling toe, it should be prevented. It can also be the case that rock armour will entrain beach sand, but under storm conditions, this will usually be washed out and sediment loss through the armour from the bed will ensue unless prevented. Erosion of bed material through a rock layer can be avoided by installing granular filters or suitably designed geotextile of appropriate specification (Figure 5.17).

Figure 5.17 Geotextile being incorporated as part of a toe structure

Granular filters may be applied where depth permits. In many cases, coastal defences require a rock size that is large relative to the depth of the structure profile, with the result that it simply becomes impractical to install a multi-layered rock system at the toe without creating an (otherwise) unnecessarily deep excavation to accommodate it. In addition, construction considerations might point to the use of larger rocks that can be placed individually, rather than small (filter) gravel sized material that has to be placed and formed to a specified depth (for example, stability of material subject to wave action). These and other factors will generally point to the need to incorporate a geotextile either as part of a layered filter system (including granular layers), or in some cases as the only separator between the primary armour and the bed.

Whether granular filters or geotextiles are used there are two fundamental criteria to satisfy for correct functional design. These are filter stability and filter permeability.

Filter stability

This relates to the perceived problem, that is, the prevention of the migration of fine sediment particles through the filter and hence through the overlying rock voids.

Using conservative principles, granular filters may be designed to be ‘geometrically tight’, requiring that the pores in the filter are too small to allow the finer bed sediment grains through. This approach is safe but can be onerous in terms of the grading of the filter, and possibly requiring two or more layers to achieve the required succession of grade ratios.

More recent research, which takes into account the actual hydraulic load on the bed layer, has facilitated the design of ‘geometrically open’ granular filters, which can be more economical. As, under normal circumstances, a geotextile cannot pass through the pores of an overlying rock layer, the requirement for filter stability only really applies to the geotextile/bed interface which can, therefore, be economically designed according to the geometrically tight principle.

Filter permeability

This relates to flow of water through the filter and hence the avoidance of excess pore pressure.

For granular filters, the criterion can be designed for either by evaluation of the pore water pressure head or, more commonly, the direct application of safe geometric ratios for successive gradings between bed, filter and rock layers.

The design of geotextile filters for permeability follows similar principles to those for granular filters except that greater consideration must be given to the longer term permeability of the materials and how this might reduce due to clogging, in particular where the bed material is silty.

The Rock Manual (CIRIA et al. 2007) details the methods and formulae for designing according to the above principles, together with the related topics of heave and piping. The manual also gives guidance on application of geotextiles in rock structures and selection of the appropriate geotextile properties (extensibility, puncture resistance, thickness and durability).

5.8.2 Rock blanket, concrete armour mattresses and asphaltic apron

This section outlines the design principles of three types of toe protection system under the action of wave or current loading:

- rock blanket or apron;
- concrete armour mattress;
- asphaltic apron.

Rock blanket or apron

An apron can limit the effect of scour induced by the main structure but cannot prevent naturally occurring beach lowering due to sediment starvation, adverse longshore drift gradient, or cross-shore movement and reprofiling. Rock size and the depth and extent of the blanket depend on the exposure (for example, open coast or sheltered estuary) and the likelihood of future beach lowering.

For a rock apron that can be exposed to open sea conditions, wave attack is likely to be the dominant design factor. The design of a blanket in this case would follow the same principles described for a rock toe (see Section 5.8.1). It is essential that the depth of the apron allows for:

- depth to accommodate sufficient number of stone diameters, and hence to be sufficiently energy absorbent, to be stable under the incident wave action;
- depth to accommodate any allowable localised scour within the apron itself;
- sufficient reserve of material to accommodate bed deformation due to general beach lowering – to this end the blanket must be designed to anticipate the most disturbed bed profile plus an allowance for the loss of some material displaced in the process of lowering;

- a margin of safety in the rock size to cater for the fact that, as it adjusts to accommodate beach lowering, the slope of the toe is likely to become steeper than that at which it was initially placed.

Figure 5.18 shows an example of a rock apron:

Figure 5.18 Example of rock apron (courtesy Jersey States Government)

The discussion above provides only an overview of this topic. For design purposes, the reader is advised to refer to the *Coastal Engineering Manual* (USACE 2012) and *The Rock Manual* (CIRIA et al. 2007).

Concrete armour mattress (or flexible armoured revetment)

These systems consist of concrete blocks joined together with cables, or attached to a geotextile sheet, to form closely spaced pattern of armour (Figure 5.19).

When placed as a revetment, they may be filled with granular material either artificially or through natural beach action – this improves the interaction between the blocks and the weight of the mattress but lessens their energy absorbing properties. When placed as a toe or buried, infill with sediment is certain.

A mattress may be laid on a sublayer of gravel or small rock, or placed over a geotextile to prevent sediment loss through the interstices.

Figure 5.19 Example of a flexible armoured revetment

The stability of the mattress depends on retention of its integrity. Some designs ignore the effect of the interconnecting tendons, taking the conservative approach that the stability of the system, ultimately, is governed by that of the individual armour units. In any case, any unanchored edges of the mattress are clearly more vulnerable to displacement than the inner area, and this is likely to be the case at the toe.

Some key advantages of these systems are as follows:

- They can be placed quickly – advantageous when working within brief tidal windows (if appropriate they can be lifted and reused elsewhere).
- They can accommodate a measure of deformation (for example, due to bed lowering) and still retain their integrity. Consequently, in certain cases they can provide an economic alternative to a rock apron where the latter would have to be of a greater volume to cater for bed lowering and hence some loss of material.
- If they are only required temporarily (for example, if they are subsequently covered permanently as a result of beach nourishment), they can be lifted out and re-laid elsewhere.

A general description of flexible armour systems may be found in *Seawall Design* (Thomas and Hall 1992). An important aspect of design concerns the permeability of the bed or embankment over which a flexible armoured system is placed. Generally, the mattresses are of low permeability and this may be further impaired by infilling with sand and so on.

If the ground over which mattress is placed is clay or silty sand (that is, of lower permeability than the mattress), then water pressure below the mattress can dissipate through it. If, however, the mattress is placed over a more permeable sediment or a granular layer, then pressure on the mattress may cause it to lift under wave action.

Lift, as just described, requires there to be a pressure differential due to wave action, as would be the case on the sloping part of a revetment. The effect would be much reduced at the toe if submerged throughout the wave passage, although piping through the underlayers or embankment could pose a similar threat.

Design rules in respect of wave loads are described by Pilarczyk (1995). The basic equation for critical stability of a semi-permeable cover layer is:

$$\frac{Hs}{\Delta D} = \frac{F}{\zeta^{2/3}} \qquad \text{(Eqn 5.8)}$$

where:

Hs = incident significant wave height

Δ = relative density of the mattress material = $(\rho s - \rho)/\rho$

D = depth of mattress

F = stability coefficient – depends on system type

ζ = breaker parameter = $\tan\alpha / (Hs/1.56Tp^2)$

Tp = peak wave period

α = ground or structure slope

However, this formula does not apply well to toe structures as it assumes there to be a realistic structure slope α, whereas a scour mattress is laid on a near horizontal bed. Pilarczyk (1995) advises the use of formulae for slope protection in which the toe is schematised by way of a mild slope of between 1:8 and 1:10. Design must also allow for some displacement of the mattress and the possibility that a more tangible slope develops as a result. The stability coefficient F must be obtained from the manufacturer of the system.

Asphaltic apron

An overview of various asphaltic materials for use in coastal defence is given in Chapter 2. For toe construction, asphaltic mastic or grouted rock is usually used.

Like a flexible armoured system, an asphaltic apron can accommodate a measure of deformation due to bed lowering. At coastal sites subject to dynamic beach behaviour, asphaltic materials may not be appropriate as rapid beach drawdown may exceed the rate at which the asphalt will deform without breaking. If appropriately applied, however, asphaltic materials can provide an economic alternative to a rock.

For design of an asphaltic toe, the reader is referred to:

- *The Shell Bitumen Hydraulic Engineering Handbook* (Schönian 1999);
- *The Use of Asphalt in Hydraulic Engineering* (van der Velde et al. 1985).

Figure 5.20 shows an early (1979) application of bitumen to a large coastal structure in the UK, which was initially constructed with a falling apron as shown in the diagram. Following beach lowering, it was discovered that there were various buried obstacles and debris which fouled the apron. To counter this, subsequent design (1987–1988) adopted a more robust design which used a 6 m long horizontal toe slab of grouted stone on a geotextile and a seawards sheet pile cut-off with some loose rock placed in front of it.

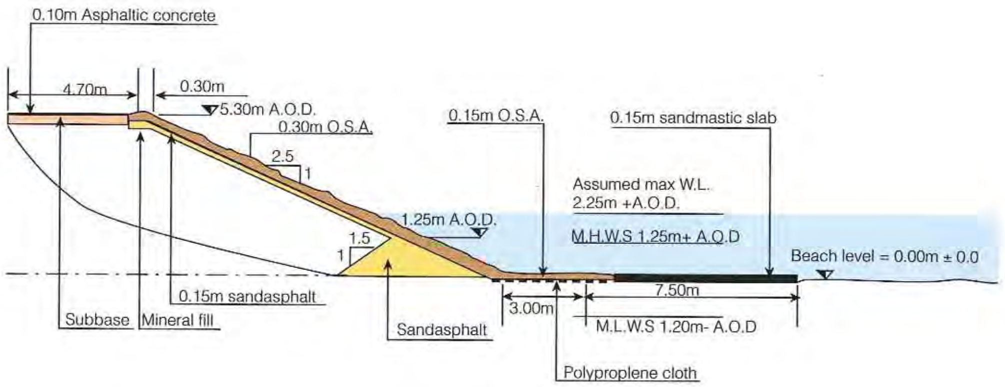


Figure 5.20 Caister on Sea, UK 1979: cross-section phase I with open stone asphalt revetment and a sand mastic asphalt toe slab (from Schönian 1999)

5.9 Hydraulic performance

Hydraulic performance refers to the reflective properties of a wall together with run up and overtopping. If the toe is covered by beach material, changes to its structure negligible impact on hydraulic performance. If the toe is uncovered under normal or extreme conditions then there could be some impact on hydraulic response which will depend, obviously, on the nature of the toe construction.

The nature of the impact will depend on the physical extent of any additions or changes to the toe.

For structural additions that are physically small in relation to the incident waves (for example, small rock fillet or sheet pile set just forward of the existing wall), the changed hydraulics will depend on the properties of the toe itself; for example, if energy absorbing, then reduced reflection and run up would likely result.

For structures that are physically large in relation to the incident waves (for example, a wide concrete platform), then the effect that this has on wave characteristics could be significant and needs to be considered in design – in addition to the properties of the toe structure itself. The effect on overtopping of locally shallower water at a seawall may be calculated using the methods described in the European Overtopping Manual (Pullen et al. 2007).

5.10 Effects on coastal processes

The term ‘coastal processes’ includes:

- the movement of sediment along the shore (longshore transport);
- the movement of sediment across the beach profile (cross-shore transport).

These movements can occur over a short timescale (for example, one tide or storm – especially the cross-shore movement) or develop over many years of accumulated change (especially the longshore movement). Gradients in longshore transport give rise to patterns of erosion or accretion while cross-shore transport during a storm can result in significant drawdown of the beach (perhaps greater than two metres’ lowering).

As with hydraulic performance, the impacts of new toe construction on coastal processes depend on the extent to which the toe is exposed under normal or extreme weather conditions. Generally, for toe protection works to linear defences, the effect of the toe on coastal processes will tend to be marginal and usually confined to cross-shore impacts. The introduction of an energy absorbing toe will tend to encourage beach retention while more reflective structures may well aggravate the scour.

5.11 Public safety

According to the HSE (2007), The Construction (Design and Management) Regulations 2007 (CDM2007):

> 'are intended to focus attention on planning and management throughout construction projects, from design concept onwards. The aim is for health and safety considerations to be treated as an essential, but normal part of a project's development – not as an afterthought or bolt-on extra'.

Designers have a duty to avoid foreseeable risks 'so far as reasonably practicable', taking due consideration of other relevant design considerations. The responsibilities of designers extend beyond the construction phase of a project. Designers also need to consider the health and safety of those who will maintain, repair, clean, refurbish and eventually remove or demolish all or part of a structure as well as the health and safety of users of workplaces.

Every project must be considered in terms of its specific circumstances. The following identifies some generic design issues:

- 'Softer' defences have the potential to fail under very extreme conditions. The damage caused can create a major danger to construction or maintenance workers, and to the public.
- Construction will often entail exposing the existing wall down to its base or thereabouts. This could pose a threat of collapse of the existing wall in the period between it being excavated and the new toe being backfilled. The design and specification of the construction run lengths must be carefully selected with a view securing the stability of the defence at all times. Generally, construction should only be undertaken in short runs. Similarly, the depth of the toe and hence excavation must also be checked in terms of the stability of the existing wall. The time during which the wall remains exposed should be kept to a minimum.
- Further to the matter of stability of the main wall structure, there is a safety risk during construction, concerning exposure to intertidal conditions. The structure is vulnerable to collapse at this point, particularly during high tides aggravated by wave action. The risks can be reduced by excluding the public from the working area and by utilising simple design that can be constructed in short runs, thus enabling construction to progress incrementally within short time windows.
- Simplicity of design (including avoiding concealed complications) will eventually also facilitate ease in demolition of the structures, thus reducing the time taken and the risks associated with this process.

In terms of public safety during service, the following general points are highlighted – note that this list is not exhaustive:

- The introduction of hard engineering structures may introduce slips, trips and fall hazards. Detailed design should carefully consider this and endeavour to reduce these risks by maintaining these features at a low level such that, under fair weather conditions, they should normally be buried. In addition, the design should endeavour to maintain the continuity of level(s) of the concrete aprons and so on, thus avoiding trip edges.
- The public are unlikely to be aware of rock or other hard structures installed below normal beach level. These can present a hazard when the beach is low and, in particular, when it reaches the critical level at which the structures are just subsurface – thus presenting a trip hazard while not being visible, or of presenting zones of unsafe ground where beach materials bridge over underlying voids in the toe. Mitigation might take the form of signs to warn the public of buried obstacles and/or of emergency response preparedness. Local emergency services would need the capability of lifting large rock in short timescales.
- Installation of a hard apron at the toe of a vertical wall increases the risk of injury in the case of a fall compared with a soft beach landing. Again, appropriate signage and possibly improved hand-railing would be desirable.

5.12 Natural environment

5.12.1 Introduction

The construction of toe structures in the marine or estuarine environment creates an artificial rocky or hard substrate within that environment. On a soft muddy or sandy shore, this is likely to represent the introduction of a habitat that is currently absent. On a rocky shore, it may represent the introduction of a substrate with different chemical properties compared with the natural rocks. While this may enhance local biodiversity, it may result in some loss, damage or change of existing habitats and species. It must be remembered that many coastal areas are covered by statutory nature conservation designations such as Special Protection Areas (SPAs),[5] Special Areas of Conservation (SACs)[6] and Sites of Special Scientific Interest (SSSIs),[7] and the introduction of a new substrate within a designated site may not accord with the site's conservation objectives.

5.12.2 Enhancement

Toe structures can offer opportunities for environmental enhancement including, for example, the provision of habitats for marine life. Guidance is given by Jensen et al. (1998), who discuss habitat creation, present suggestions to encourage colonisation of species naturally attracted to hard surfaces, and identify the types that may be attracted.

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[5] Established under Directive 2009/147/EC of the European Parliament, and of the Council, on the conservation of wild birds (the codified version of Council Directive 79/409/EEC as amended) ('the Birds Directive').
[6] Established under Council Directive 92/43/EEC on the conservation of natural habitats and wild fauna and flora (the 'Habitats Directive').
[7] Designated under the Wildlife and Countryside Act 1981, as amended.

The colonisation of plants on toe structures is dependent on the plants' ability to survive at various levels of hydration (normally linked to height above low water) or light penetration when underwater or buried by beach material. Structures at, or below, high water level may be colonised by seaweed, and the density and range of seaweeds is likely to increase at lower levels and attract a wide range of animal communities for shelter or feeding.

Under normal conditions, surfaces of concrete or quarried rock structures in the marine environment are rapidly colonised by naturally occurring micro-organisms that consume many of the dissolved and suspended substances in water. Settlement of larger organisms, such as barnacles and mussels, which can directly filter suspended matter for their food, can also occur. Grazing and browsing organisms living on rock structures devour many of the plants and animals living on the hard surfaces, creating a dynamic community with continued re-colonisation.

Fish and crustaceans can use crevices between stones and concrete blocks to avoid predators, lay eggs, or feed on organisms growing on the structure. If the structure is permanently submerged, shelters for crabs (crevices on the outside of the structure) and lobsters (galleries within the structure), and shelter for fish species such as wrasse, lumpsuckers and conger eels can all be incorporated.

5.12.3 Potential loss, damage or change

Sandy or muddy seabeds and foreshores contain a multitude of organisms (worms, crabs, molluscs and so on), many of which are important to the food chains of commercially fished species and birds (particularly in the intertidal zone). Specialised plant communities, such as saltmarshes and drift line communities, also grow above the level of mean high water neap tides. When a structure is constructed, there is inevitably a direct loss of habitat and, with it, associated species. In addition, the presence of a hard structure can modify the local wave and current climate which may give rise to sediment and, therefore, habitat changes.

5.12.4 Designated sites

Where the toe works may affect a designated nature conservation site, it is necessary to consult the statutory government's advisor on the natural environment, that is, Natural England, Countryside Council for Wales, Scottish Natural Heritage or, in Northern Ireland, the Council for Nature Conservation and the Countryside (CNCC).

5.13 Heritage and visual impact

Heritage and visual impact considerations may include the appearance of a toe structure when it becomes exposed. The relevant planning authority should be able to advise on such issues during the appraisal stage of a project and on the suitability of design alternatives during the outline design stage (Figure 5.21).

Figure 5.21 Heritage interest – the flint-faced seawall at Lowestoft South Beach

The prospect of using rock on the beach receives mixed reactions from different authorities and councils. In some cases, rock is accepted as part of coastal landscape, while at others there is a presumption against the use of rock. The use of asphaltic materials might also be resisted at locations where it has not been used before. The same applies to the use of other materials. Each project and site is unique and must be approached in terms of the specific circumstances and with regard to the wishes of local residents and the planning authority.

5.14 Amenity

Large exposed aprons or other obtrusive toe structures have a negative impact on amenity by eliminating part of the otherwise available beach area. In some cases, though, the installation of steps or an apron can improve amenity. If the apron is too high above average beach levels, however, then there may be demands for hand rails which are very liable to storm damage and corrosion. The issue is very site- and project-specific (Figure 5.22).

Figure 5.22 Toe apron proves walking platform – Paignton, Devon (courtesy of ENBE)

5.15 Construction issues

Thomas and Hall (1992) discussed construction and maintenance in the context of both new defence structures and renovation; substantially, this discussion relates equally to toe construction works. The following sections highlight the salient points.

5.15.1 Toe working in general

Works to the toe of a sea defence are often subject to tidal influence. Tidal and wave action makes such works potentially dangerous to personnel. Access to safe ground can be difficult. They are in a location that is physically aggressive (for example, due to corrosion by saline water and abrasion by wave-borne material).

While high quality workmanship and materials are often paramount to achieve a durable result, the achievement of high quality can be uniquely difficult in this environment. In these circumstances, it is crucial that those planning, managing and undertaking works at the toe are experienced in tidal and coastal working such that they are:

- aware of what is and is not achievable;
- capable of specifying and carrying out works in an appropriate manner;
- aware of the hazards;
- use plant and techniques that are suitable for the environment;
- aware of the need to keep continuously informed about forecast wind, water level, wave and other meteorological conditions, and how to access and interpret these data;
- able to plan for and manage the inevitable risks to which they and other beach users will be subject.

5.15.2 Tidal working

Tidal conditions limited the time available for in situ construction work. While the astronomical tides are predictable before any construction takes place, sea level increase or decrease due to meteorological factors can only be reliably predicted a day or so before the event. The long-term and short-term planning of operations must therefore allow for the limitations on predictions.

The danger in not completing a section in time is that the incomplete works are more vulnerable to damage by high tides and wave action. The risks of this can be reduced by limiting incremental construction to short runs and by limiting the number of different operations that must be undertaken in a tidal window. As described elsewhere, simple design usually makes for simple and speedy construction.

5.15.3 Seasonal working

The time of year that works are carried out has a major impact on the planning and logistics of construction operations. Moreover, the summer holiday period, bird migratory patterns and other seasonal factors can determine when operations may or may not be carried out.

Construction during the summer normally offers a number of positive benefits including:

- longer daylight hours;
- higher healthier beaches, which can either provide easier working conditions or sometimes make the work more difficult due to the need for a greater amount of excavation;
- reduced risk of storms and beach drawdown;

However, for seaside resorts, works on the seafront are usually avoided during the busy summer holiday period. Apart from the disturbance to the beach during toe protection works, construction also requires the use of noisy plant and can have an impact on access roads. It is often possible to fence off part of the beach and limit site traffic, which may go some way to minimising the impacts to users. Construction windows may also be restricted by ecological issues such as nesting birds.

Construction during the winter months reduces the impacts on tourism and recreational use connected with seafront use, but clearly introduces a number of limitations in terms of timing of works due to shorter daylight hours, higher risk of storm damage and so on. Careful consideration therefore needs to be given to matters such as:

- the effect of low temperature on certain construction materials, in particular asphalt and in situ concrete;
- contingencies for curtailing sections of work early should weather or other circumstances change unexpectedly;
- planning of work stages to avoid vulnerable exposed parts of the construction should access be made difficult between shifts (for example, this would apply to layered rock revetment where it is preferable to avoid leaving secondary armour exposed);
- using standard formwork, or expendable formwork that obviates the need for recovering formwork – thus speeding up operations and reducing the risk of damage over high water;
- working short sections at a time;
- restrictions imposed by special working hours agreements (to avoid disturbance to local residents or wildlife).

Should it be necessary to gain access at very low tide to perform toe protection works, then the contractor may want to take advantage of exceptionally low tides such as those occurring at the autumn and spring equinoxes.

The depth of beach and hence exposure, access and excavation needed at the toe can vary massively with the seasons. This may be reflected in major differences between beach surveys undertaken at different times. The effect of this on contract terms and price needs to be considered carefully. Available working times can have a major impact on:

- the construction techniques that are feasible;
- the quality of the finished job;
- the price.

Use of construction expertise in preparing the designs and specification for the works can thus increase quality and reduce risk, and also identify areas where the temporary relaxation of constraints needs to be considered.

5.15.4 Defence standard

The undertaking of toe restoration necessarily implies disruption to, and possibly part removal of, an existing defence. Clearly, it is important to avoid any compromise on existing defence standard during the undertaking of works. This is essential during the winter when storm surges are more prevalent. Contingency measures need to be considered and provided if necessary (for example, provision of a stockpile of materials for emergency backfilling or rock and/or beach material for protection in advance of the position of the works).

5.15.5 Sequencing construction

Construction work at the toe of a defence probably requires more considered programming than any other part of the defence structure. Elsewhere in this section it has already been mentioned that speed and ease of construction are advantageous. Depending on conditions, the use of toe sheet piling as a cofferdam can extend the tide free period considerably and equate to considerable time and cost savings. The sequencing of staged works is therefore important. The following summarises some relevant factors:

- The designer/contractor needs to consider design limits of each stage of construction, for instance, upon excavation of the toe of an existing wall. What is the residual factor of safety prior to it being restored?
- For anchored or tied sheet piling, it is clearly important to determine the sequence of backfilling and tying to avoid overloading the pile section.
- Incorporation of existing features into a new construction can save time and may serve the additional purpose of proving support or as expendable temporary works.
- With reference to ambient weather conditions, the designer/contractor should specify the striking times for formwork for concrete and bituminous materials in order that they gain sufficient strength prior to next stage.
- The use of precast units can save time, but this may not be a viable option for the toe where the detail has to cater for irregular ground conditions.
- Would the use of a cofferdam be feasible and provide advantages?

5.15.6 Access for materials and plant

The characteristics of a site and its environs, and the impact this has on access, differ greatly from site to site. For example, access through a built-up area may present certain advantages in terms of distance but have limitations on road widths or weight limitations[8] and so on, thus limiting the size or types of vehicle that can be used. Access to a site at the foot of a cliff may present the obvious problem of height difference, which might be overcome by use of cranes or by taking a longer route along the beach. Beach access and working time is more often than not limited by the tide. Moreover, the operation of plant from cliff tops needs to be carefully and conservatively examined in respect of stability of cliff edges to the loads imposed by the plant – the same applies to promenades and other elevated working platforms.

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[8] Access through built-up areas may be limited by environmental health officers.

At the construction site, access from an existing seawall crest may facilitate the placing of concrete using pumping and, within the limits of crane reach, the placing of armourstone. Access along the beach will almost certainly be needed for certain operations including excavation and placing of rock delivered to the beach. Tide permitting, access along the beach might otherwise be impeded by groynes. Depending on the circumstances, it may be necessary to create an access over or through a groyne(s), to be reinstated when no longer required.

Bulk materials such as rock may be delivered by sea but this still may require a short haul delivery by vehicles through a built-up area (if delivered to a port) or by vehicles on the beach. Suitable restrictions on public access to the beach would therefore need to be implemented during operations for health and safety reasons, and staff briefed on any specific health and safety concerns regarding safe operating conditions (for example, tidal state, weather/sea conditions).[9]

Access may also be a determinant in the choice of toe construction material. For instance, asphaltic materials can be prepared and delivered to sites that might otherwise be difficult to access with a sheet piling rig.

Access to land from the works in times of difficulty (storms and high water levels) is essential to mitigate the risk to beach users, workers and plant.

5.15.7 Placing materials

The circumstances of a given site, in particular the level of the toe works in relation to groundwater and the tide heights, can be an important criterion in the choice of construction method and material. Materials that require precise placing are less suited to situations where installation is completely underwater (this would include flexible armoured mattresses and pattern placed armour), while certain asphaltic materials cannot be placed under water due to damage to the binder.

5.15.8 As built drawings

For toe construction, it is especially important to survey completed works and prepare as built drawings. In a situation where further works may well be required in the future, it is essential that information on the structure is readily available. While in many cases the time available to prepare as built drawings can be limited by the urgency to backfill excavations, or by natural covering by the beach, nonetheless there are numerous examples where future work becomes excessively costly or risky due to lack of information on the existing structure.

[9] For further guidance on construction safety see Cruickshank and Cork (2007).

List of abbreviations

ALWC	accelerated low water corrosion
AMS	asset management system
AONB	Area of Outstanding Natural Beauty
BSS	Briar skill score
CDM	construction, design and management
CEM	Coastal Engineering Manual [published by USACE]
CG	condition grade
CHaMP	Coastal Habitat Management Plan
EIA	environmental impact assessment
FCERM-AG	Flood and Coastal Erosion Risk Management Appraisal Guidance [Environment Agency]
FCRM	flood and coastal risk management
FDGiA	Flood Defence Grant in Aid
FEPA	Food and Environmental Protection Act
FOS	factor of safety
HRA	habitats regulations assessment
HWMOT	high marks of ordinary tides
H_s	significant wave height nearshore
LAT	lowest astronomical tide
LiDAR	light detection and ranging
L_m	wave length
LWMOT	low water marks of ordinary tides
MAFF	Ministry of Agriculture, Fisheries and Food
MHW	mean high water
MHWS	mean high water spring
MLW	mean low water
MLWN	mean low water neaps
MLWS	mean low water spring
MMO	Marine Management Organisation
mODN	*metres above Ordnance Datum* Newlyn
OPC	ordinary Portland cement
OS	Ordnance Survey
PAMS	Performance-based Asset Management System

PAR	Project Appraisal Report
PFA	pulverised fuel ash
RTK GPS	real-time kinematic global positioning system
RMS	root mean square
RMSIE	RMS interpretation error
RMSSE	root mean square source error
RMSVE	root mean square variability error
SAC	Special Area of Conservation
SCAPE	Soft Cliff and Platform Erosion [model]
SEA	strategic environmental assessment
SMP	Shoreline Management Plan
SoP	Standard of Protection
SPA	Special Protection Area
SSSI	Site of Special Scientific Interest
TEB	traversing erosion beam
T_p	peak wave period
tx	zero-crossing period
ULS	ultimate limit state
USACE	US Army Corps of Engineers
UV	ultraviolet

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Appendix A: Scour processes

This appendix provides guidance on how to assess the lowering of beaches at the toe of coastal defence structures. It discusses the processes that control beach levels, including the form of the different processes and their relative importance.

A.1 Introduction

The coastlines of the world are constantly assailed by winds, waves, tides and surges, which cause coastal sediments to be transported and cliffs, shore platforms and other rocks to be eroded. As a result, our coastlines are continually changing. Under the pressure of development, mankind has for centuries attempted to stabilise our dynamic coasts through the construction of coastal defences (Rendel Palmer and Tritton 1996), often on naturally eroding coastlines. There is then a complex interaction between the coastal defence structure and the beach at which it was constructed.

Beach levels in front of coastal defence structures are continually changing, with (in the UK) a general trend for lowering, rather than accretion. Beach lowering is caused by a number of processes that take place at a range of different spatial scales and timescales, and which combine cross-shore and longshore sediment transport. The individual timescale and form of transport should not be considered in isolation. For example, storm-induced toe scour may not be a problem if the beach level at a coastal defence structure is high, so that the scour that does take place can be accommodated within the design limits of the structure, whereas if the same beach experiences a long-term beach loss, the storm-induced scour may in the longer term become serious.

The overall performance of a coastal structure therefore depends on morphological changes over a broad range of scales, as detailed below and illustrated schematically in terms of the beach profile response in Figure A.1:

- **Toe scour** (Figure A.1a) – beach levels in front of the coastal defence structure often dropping and recovering completely during the course of a single tide. Toe scour occurs over a cross-shore length of a few metres but may extend considerably further in the longshore direction. Scour at a coastal seawall or similar coastal defence structure is often referred to as toe scour because it occurs at the intersection of the beach and the structure, even though that may be at a point well above the actual toe of the structure itself.

- **Storm response** (Figure A.1b) – 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. The coherence of the longshore response will depend on how long the coastal defence structure is, and how the nearshore bathymetry, beach profile and sediment characteristics vary.

- **Inter-storm recovery** (Figure A.1c) – the beach will respond to the changing forcing conditions after a storm and variations in beach level can be observed. Recovery from storm action can take tens of tides to occur and will affect a similar longshore area to the storm response.

- **Seasonal variability** (Figure A.1d) – commonly it is 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 profiles and hence levels at the toe of a structure.

- **Inter-annual variability** (Figure A.1e) – the wave climate varies from year to year, altering the nett magnitude (and possibly direction) of longshore drift and generating erosion or accretion of the beach. The annual wave climate affects the whole coastline so its effects are felt over the scale of the sediment cell, say tens of kilometres alongshore and by of the order of one or two kilometres cross-shore.
- **Coastal evolution** (Figure A.1f) – changes in the coastal profile are driven by sea level rise and wave climate, and dominated by longshore transport controlling coastal evolution. Coastal erosion occurs over longer timescales and even larger spatial scales than beach changes due to variations in annual conditions. Coastal erosion is often associated with the lowering of ground levels caused by the erosion of a rock platform.

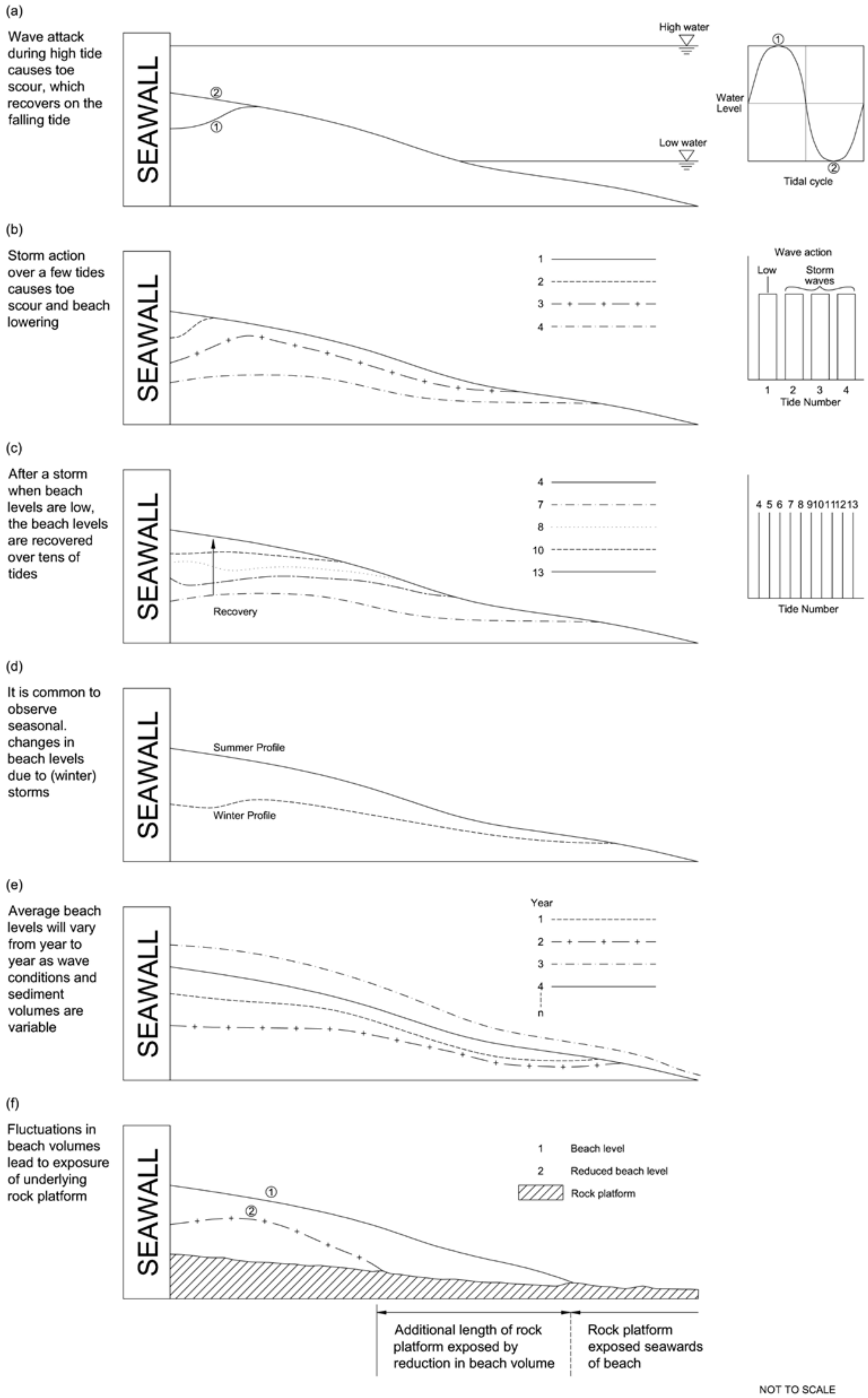


Figure A.1 Conceptual model of beach profile response in front of a seawall

A.1.1 Time and space scales

In general, the spatial scale of beach changes increases with the timescale, and longshore sediment transport processes increase in importance compared with cross-shore transport processes as the timescale increases. However, a storm surge event moving down the North Sea basin from north to south will produce a time-varying response at each stretch of coastline that it impacts upon; the integrated effect may extend for tens or hundreds of kilometres along the coast.

Figure A.2 relates to scour and erosion caused by sediment transport. Another process, wave-induced liquefaction, may also be important in the assessment of the performance of the toe of coastal defence structures as it can reduce the bearing capacity of the seabed in front of a structure. Generally liquefaction is associated with shorter length scales and smaller spatial scales than toe scour and occurs in finer sediments; it would not be expected to occur in a permeable shingle beach experiencing wave loading.

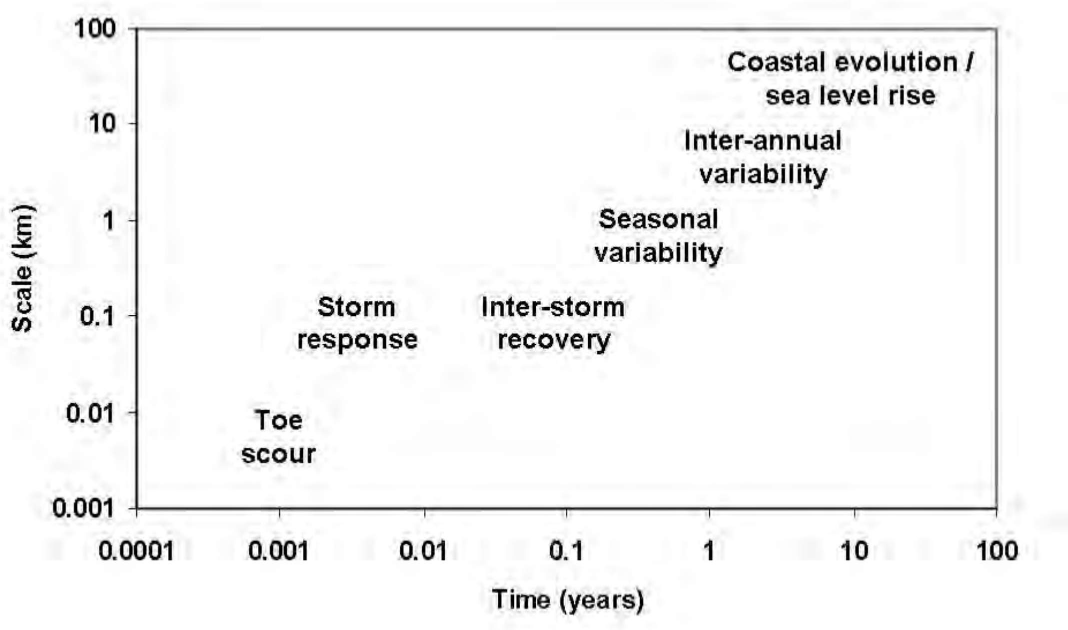


Figure A.2 Beach responses to natural forcing, indicating associated length-scales and timescales (from Sutherland et al. 2007)

A.1.2 Influence of a coastal defence structure

The lowering of beaches and/or shore platforms in front of coastal structures at different time scales and spatial scales is caused by a number of mechanisms. Some of these reflect the characteristics and geomorphological processes of the coast where the structure has been installed. They are associated with the longer time scales and larger spatial scales illustrated in the upper right quadrant of Figure A.2. The mechanisms are largely independent of the type of structure, that is, they occur whether the structure is permeable or impermeable, whether it is steep-faced and reflects waves, or whether it is more gently sloping and dissipates wave energy.

Other effects though are dependent on the characteristics of the coastal structure. For the most part, these have primarily localised effects, are short-lived and are reversible – at least on sandy beaches. They can lead to local beach lowering. In these circumstances it is necessary to assess and understand the range of beach levels in front of coastal structures in order for those assets to be managed.

It is convenient to develop a conceptual framework and scenario models and tools based on a breakdown into the following different time and space scales:

- toe scour of the beach sediment over tides and days;
- liquefaction of the beach sediment over seconds to minutes;
- beach variability over weeks and seasons, which includes storm response, storm recovery, seasonal and inter-annual variations;
- coastal erosion (long-term beach lowering) over years and decades.

There is some overlap in these categories, but it has nevertheless proved to be a useful classification as there are different processes (see Sections A.2–A.6) and modelling approaches (Appendix B) that can be broadly associated with these categories which are defined below.

A.1.3 Definitions

Scour

In the context of toe structures, scour can be defined as:

> 'the process of sediment erosion from an area of seabed in response to the forcing of waves and currents as modified (enhanced) by the presence of a structure'.

Although this mechanism affects many types of marine structure, scour processes tend to be very similar. Scour can be caused by the following processes (Whitehouse, 1998; Sumer and Fredsøe 2002):

- reflection of waves from the coastal structure, leading to increased wave action in front of the structure;
- wave breaking in front of or over the structure.

Further significant influences can arise from:

- contraction of currents along the front of a breakwater or seawall;
- generation of wave-driven currents by oblique incidence waves.

And where the structure has an end:

- diffraction of waves around the coastal structure;
- formation and shedding of vortices at the heads of coastal structures.

Thus the extent and type of scour process is dependent on:

- the wave climate and water level;
- the beach and nearshore shore profile;
- the design and position of the seawall on the shore profile.

Liquefaction

In common usage, the term 'liquefaction' refers to the loss of strength in saturated, cohesionless soils due to the build-up of pore water pressures during dynamic loading leading to a loss of effective stress. The definition of liquefaction given by Sladen et al. (1985) as is follows:

> 'Liquefaction is a phenomenon wherein a mass of soil loses a large percentage of its shear resistance, when monotonic, cyclic or shock loading is applied, and flows in a manner resembling a liquid until the shear stresses acting on the mass are as low as the reduced shear resistance.'

Beach variability

Variations in the patterns and rates of sediment transport are very common when sediments are susceptible to erosion, for example when either fine (sand) or coarse materials (shingle) are subject to wave and/or current action. These processes may lead to natural cycles of erosion and accretion irrespective of the existence, position or configuration of a coastal defence structure.

Processes such as the drawing down of beach material from the top of a beach during a storm and the gradual recovery of the beach level after the peak of the storm lead to bathymetries in front of coastal defence structures (and in natural undefended beaches) that vary in space and time. This phenomenon leads to beach variability. Figure A.3 illustrates this phenomenon where a storm event removed sediment locally to a level below the toe of the defence (right image); the subsequent image (on the left) which was taken after a calm period of weather over several months shows sediment levels have recovered considerably once more (without active intervention).

Figure A.3 Storm event sediment scour (right) and post event recovery (left) (courtesy of Peter Frew, NNDC)

Beach lowering

Although often linked, beach lowering is not necessarily the same as coastal erosion. Coastal erosion is the long-term and systematic loss of sediment from the coastal zone that occurs over periods of years. It is commonly associated with the irreversible erosion of rock, whether in the shore platform or in cliffs. In contrast, 'beach lowering' is more precisely related to the short- or long-term loss of beach materials (mainly shingle or sand) from foreshores.

Typically, however, 'beach lowering' refers to scour over spatial extents much greater (that is, across the whole shore platform) than 'localised' scour associated with particular structures.

A.1.4 Problems caused by beach lowering

Both localised scour and the more widespread beach lowering can lead to problems such as increased rate of deterioration of exposed toe structures, and the undermining of the foundations along the seaward toe of structures which can lead to partial or total collapse.

A comprehensive survey published by CIRIA (1986) concluded that scour at the toe of coastal defence/protection structures represented the most prevalent and serious form of damage to seawalls in the UK. It accounted directly for 12 per cent of the seawall failure case histories studied and was linked indirectly to a further 5 per cent of cases. Similar conclusions were drawn by Markle (1989) in the USA for rubble-mound structures. The causes of failure included:

- the removal of supporting beach material from in front of a coastal defence structure;
- a gradual dislocation of the rubble mound or blockwork foundation;
- the washing out or winnowing of granular 'fill' from behind the face of the structure;
- a modification of the wave and flow conditions in front of the structure which may, for example, increase the rate of overtopping, which in turn can lead to the erosion of the rear face of a coastal defence structure.

In addition to these, prolonged exposure of toe sheet piles (for example, as a result of scour or beach lowering) will increase the rate of corrosion and abrasion of the metal piles – thereby increasing deterioration and reducing structural strength of the toe. Exposed concrete toe beams will also be subject to wave impact forces, sediment abrasion, weathering (such as freeze–thaw cycles where these occur), and other chemical and biological processes. Damage could also be caused by boat collision or other anthropogenic means. Deterioration is discussed further in Section 3.8. Failure modes are discussed further in Section 3.3 and in Chapter 5.

The second most prevalent form of damage to coastal defence and protection structures in the UK was outflanking, where erosion occurs at the end of a seawall, allowing the removal of material from behind the structure (CIRIA 1986). Other problems associated with beach lowering and scour include the following:

- Access to the beach by steps and ramps can be made more difficult (for example as shown in Figure A.7).
- Beach lowering increases the water depth in front of the structure, allowing larger waves to reach it and potentially increasing wave forces and wave run-up on the structure. The possibility of reduced beach level, with increased water depth and potentially increased forces should be included in the design of structure toes (see Chapter 5). The potential for increased

overtopping can be calculated (Pullen et al. 2007) and, if significant, may need to be designed out or accommodated.[10]

- The potential for increased agitation of beach sediments in front of the wall or increased rate of erosion of the shore platform caused by increased wave action, and perhaps by faster tidal currents resulting from greater water depth at the toe of the structure. The erosion of cohesive shore platforms is not dealt with here, but has been the subject of research by Brew (2004) and Royal Haskoning et al. (2007).

A.1.5 Appendix contents

The rest of this appendix describes the occurrence of bed level changes seen in front of coastal defence structures:

- evidence for scour, liquefaction, beach variability and coastal erosion;
- description of processes controlling toe scour;
- description of processes controlling liquefaction
- description of processes controlling beach variability;
- description of processes controlling coastal erosion.

A.2 Evidence for scour, liquefaction, beach variability and coastal erosion

A.2.1 Evidence for toe scour

Toe scour is blamed for the failure of many coastal structures (CIRIA 1986) but toe scour holes have been infrequently observed in the routine monitoring of beach profiles (Griggs et al. 1994). Recently, however, evidence has been collected at structures in the intertidal zone (Sutherland et al. 2006a, 2007; Pearce et al. 2006), showing that scour holes can develop and substantially, or completely, fill in again during a single tide, demonstrating that simple topographic beach surveys may well miss the more severe cases of scour. Some of this evidence is presented in Box A.1.

[10] The calculation and assessment of overtopping is not discussed in depth in this manual as extensive guidance can be found elsewhere. For further information on overtopping, the reader is referred to the Eurotop Manual (Pullen et al. 2007).

Box A.1 Scour monitoring and analysis at Southbourne, Bournemouth

Two scour monitors, each consisting of eight motion sensors, were deployed at Southbourne (Bournemouth) in 2005. Under non-eroding conditions, the sensors remain buried in the beach and did not move. When a scour hole began to develop, the sensors were progressively exposed and each began to oscillate in the flow. Figure A.4 shows the elevation of the lowest oscillating motion sensor, which indicates an upper limit to the possible beach level, plotted with water level and offshore significant wave height, H_s (m) measured in approximately 10 m water depth (Sutherland et al. 2006a, 2007).

Figure A.4 shows that, as the wave height and water level rose during the morning of the 24 May, the beach level dropped by at least 0.60 m. 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 24 May, 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.

The results from Southbourne and extensive analysis of scour monitor data collected at Blackpool between 1995 and 1998 (HR Wallingford 2006a) showed 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.

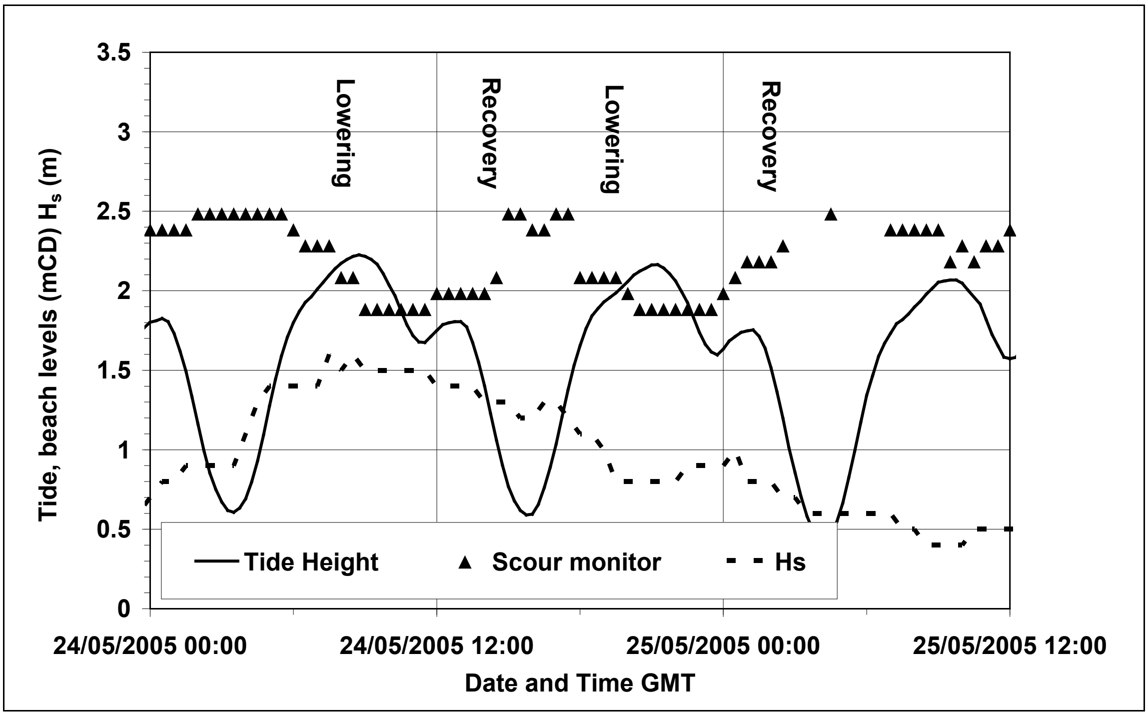


Figure A.4 Scour monitor data showing beach lowering and recovery during a tide measured at Southbourne

Storm response is the residual change in bed elevation, when the beach does not recover fully to the elevation it was at before the tide came in. The same location in

Southbourne is shown in Figure A.5 towards the end of a storm in 2006, when the beach had been drawn down and a scour trough had formed at the toe of the seawall exposing rock previously placed at the toe.

Figure A.5 Local toe scour at Southbourne on 11 January 2006 *(*courtesy of Andrew Pearce)

A.2.2 Evidence for liquefaction in front of seawalls

The authors are not aware of any engineering problems at UK seawalls that have been attributed to wave-induced liquefaction. Liquefaction is rarely observed due to its temporary nature and because it can only occur underwater.

Mory et al. (2004, 2007) used arrays of pressure sensors to identify instances of momentary liquefaction of the seabed in front of a near vertical concrete wall in the inter-tidal zone on a beach in southwest France. Liquefaction occurred to a depth of over 0.3 m over a fraction of a wave period with wave heights between around 0.7 m and 1.7 m in water depths of 1.0–1.4 m. The gas content of the pore water in seabed was also measured. The thin top layer of the seabed, disturbed by wave action, had a low gas content, but the pore water in the seabed below that down to about low tide level showed a higher gas content. This meant the pore water was compressible, which played an important role in allowing liquefaction to occur. Below low water level the seabed had a much lower gas content, so was less liable to liquefy (see Sections A.4 and B.2).

The liquefaction of a beach due to seepage, whether through a natural beach or from under a coastal defence structure, may be more commonbut is not dealt with in this section.

A.2.3 Evidence for beach variability

A time series of beach levels measured at a single point in front of the seawall at Mablethorpe Convalescent Home at approximately monthly intervals between 1959 and 1991 is shown in Figure A.6 (Sutherland et al. 2007). The trend line is the best-fit straight line through the points, which fell at an average rate of 23 mm per year during this period. Figure A.6 shows that there is a significant amount of variability about the best-fit straight line and that this variability occurs over different timescales.

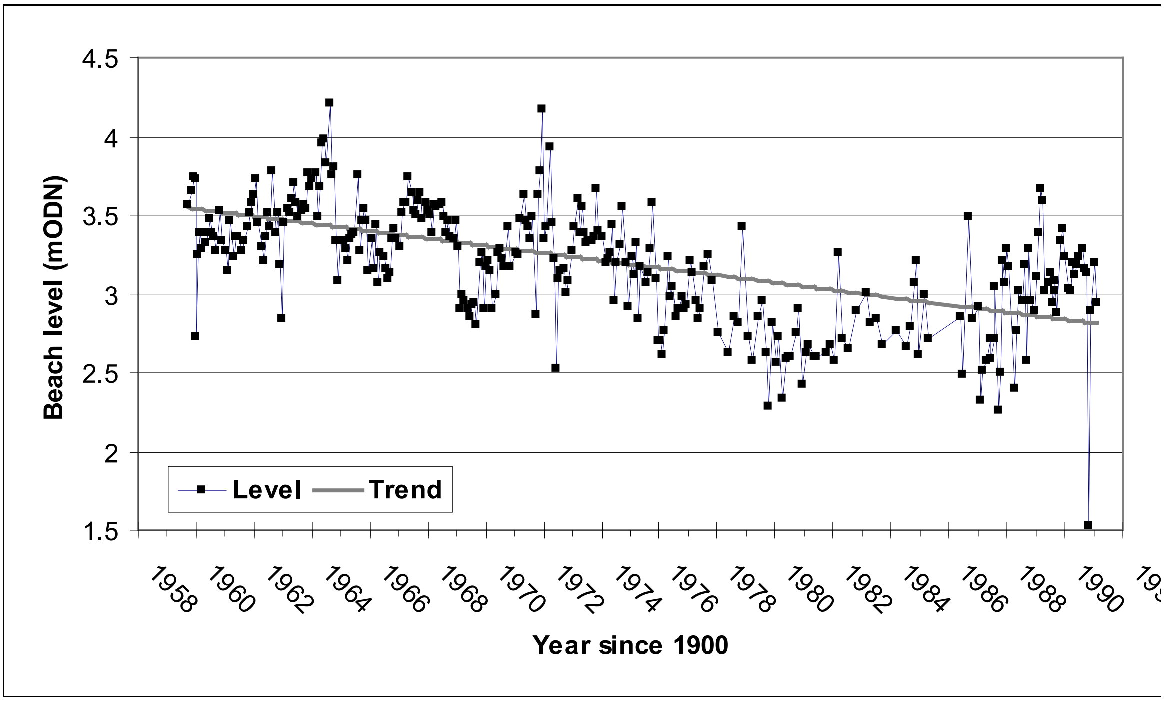


Figure A.6 Time series of beach levels in front of a seawall at Mablethorpe in Lincolnshire

A.2.4 Evidence for coastal erosion

The lowering of 'ground levels' in front of seawalls, revetments or other coastal structures is a common phenomenon not only in the UK but also around the world. In some circumstances, the beach becomes flatter and lower over a wider area in front of the structure, sometimes with the sand or gravel being largely removed to reveal the underlying rock of the shore platform (Figures A.7 and A.8). The erosion of the shore platform has been monitored for a range of coastal sites, as listed in Table A.1, where under 'source' L&C refers to Lee and Clarke (2002) while RH refers to Royal Haskoning et al. (2007). The latter measured temporal variation in the rate through a year and the average figures are quoted here. As an indication based on the results at one site, Warden Point, in one specific year, Royal Haskoning et al. (2007) found the greatest downwearing to occur in the period February to May.

Table A.1 Measured shore platform erosion rates (from Lee and Clark 2002)

Location	Rock type	Lowering rate (m/year)	Source
Saltburn-Ravenscar	Jurassic limestones, sandstones and shales	0–0.18	L&C
Isle of Thanet, north Kent	Chalk	0.025	L&C
Lyme Regis	Jurassic clays	0.1	L&C
South Glamorgan	Lias limestone	0.064	L&C
Warden Point upper platform	London Clay	0.031	RH
Warden Point upper middle platform	London Clay	0.014	RH
Warden Point lower middle platform	London Clay	0.008	RH
Easington	Glacial till	0.042	RH

Notes: L&C = Lee and Clark (2002); RH = Royal Haskoning et al. (2007)

Figure A.7 Beach and shore platform lowering, Shakespeare Cliff, Thanet

Figure A.8 Removal of beach sediments from shore platform, Penrhyn Bay

The removal of sediment may lead to the exposure and potential failure of the structure's toe, as shown in Figure A.9, which shows exposed sheet piling, some of which has been removed from the toe as a result of erosion and is lying on the beach.

Figure A.9 Example of structural damage to a seawall at Milford-on-Sea (courtesy Andrew Bradbury, New Forest District Council)

A.3 Description of processes controlling toe scour

A.3.1 Introduction to toe scour

This section outlines scour mechanisms and lists the environmental parameters that control toe scour. Some of these parameters are further investigated in other sections of the manual.

Seawall toe scour occurs when the base of the wall can be acted upon by waves, either directly, when the sea level is higher than the bottom of the wall, or through wave run-up. The presence of a structure in relatively shallow water, for example, abruptly breaks the wave and the energy is dissipated within a much smaller zone than on a natural, unimpeded beach profile. This sudden release in energy is converted into turbulence and wave reflection. The extra kinetic energy released around the toe of the seawall induces lowering of the beach at the bottom of the wall by:

- increasing local shear stress on the bed to levels exceeding the threshold for sediment motion;
- generating shock waves through the impact of waves breaking on the seawall. The pressure waves set up in the water column are transmitted to the bed and away from the wall. These high pressure gradients disturb the sediment and make it more vulnerable to erosion. Wave-induced liquefaction of bed sediments may become a contributing process at this time.
- increasing removal of the suspended sediment by longshore currents as the extra turbulence sustains sediment motion and allows it to be transported by currents; and,
- reducing sedimentation as the greater water velocity close to the seawall reduces the rate of settlement of sediment brought into the area from longshore drift.

The process of toe scour can be self-sustaining. For example, consider the situation where the beach level at the base of the seawall is above the mean high water spring tide level and therefore not vulnerable to scour under normal conditions. Once a sufficiently large storm (that is, surge water level plus storm waves) produces initial scour, a greater range of wave/water level conditions can reach the seawall and the beach level in front of the seawall may then lower progressively.

As the beach lowers further, the water table is closer to the surface, pore pressures increase and the sand can be fluidised and this increases the degree of sediment removal through backwash (Powell and Lowe 1994). Periodically, conditions may allow a recovery of the beach level if there is a sufficient sediment supply, but for narrow beaches with a sediment deficit, it may never accrete to the pre-scour level.

Further discussion about the processes of scour can be found in van Rijn (1998, 2005), Whitehouse (1998), Kraus and McDougal (1996) and Sumer and Fredsøe (2002) among others.

The following wave/water level characteristics dictate the extent and type of scour at a seawall:

- wave height;
- wave period;

- water depth at the toe of the seawall;
- beach slope;
- seawall slope;
- seawall type;
- sediment size;
- storm duration;
- angle of wave approach;
- wave overtopping.

The incident wave height, wave period, water depth at the toe of the structure, beach slope, seawall slope and seawall type determine the wave kinematics in front of a coastal defence structure and in particular the way that waves break on the structure.

The sediment size is important in determining whether sediment transport occurs and, if so, whether the sediment moves as bedload within the wave boundary layer or as suspended load in the water column.

The storm duration determines whether the scour can reach an equilibrium depth.

The angle of wave approach has an effect on the wave kinematics and in-particular the cross-shore position of wave nodes and anti-nodes (Figure A.10) and also on the wave-generated longshore currents.

The rest of this section consists of:

- an introduction to sediment transport in a non-breaking reflected wave;
- an introduction to the kinematics of waves in front of a reflecting seawall in relatively deep water;
- a description of scour caused by random waves at a vertical wall;
- a description of scour caused by random waves at a sloping wall;
- commentaries on the effect of storm duration, the angle of wave attack and overtopping on potential scour depths.

A.3.2 Sediment transport under a non-breaking wave reflected off a vertical wall

A regular non-breaking wave with wavelength L reflecting off a vertical wall generates a standing wave, as shown in Figure A.10. The incident and reflected components are in phase at the wall, so an anti-node (zone of maximum vertical amplitude) is formed. This is an area with a relatively high root mean square (RMS) surface elevation but zero horizontal velocity. On moving away from the wall, the incident and reflected components move out of phase until they are completely out of phase (when the incident and reflected surface elevations cancel each other out, so there is no surface movement but a maximum in horizontal velocity) and a node is formed. This occurs at a distance of a quarter of a wavelength in front of the wall. On moving further out, the incident and reflected components move back into phase and another anti-node occurs at a distance of half of a wavelength from the wall.

The standing wave generates steady streaming (a small net current) in the thin wave induced bottom boundary layer (Longuet-Higgins 1953, 1957). This streaming is manifested as a slow recirculating current from anti-node to node at the bottom of the bottom boundary layer and from node to antinode at the top of the bottom boundary layer – as shown in Figure A.10for the two-dimensional (2D) normal incidence case. The current at the top of the boundary layer drives a counter-rotating recirculating cell in the (much thicker) body of water above the boundary layer. This work was extended to oblique-incidence by Carter et al. (1973).

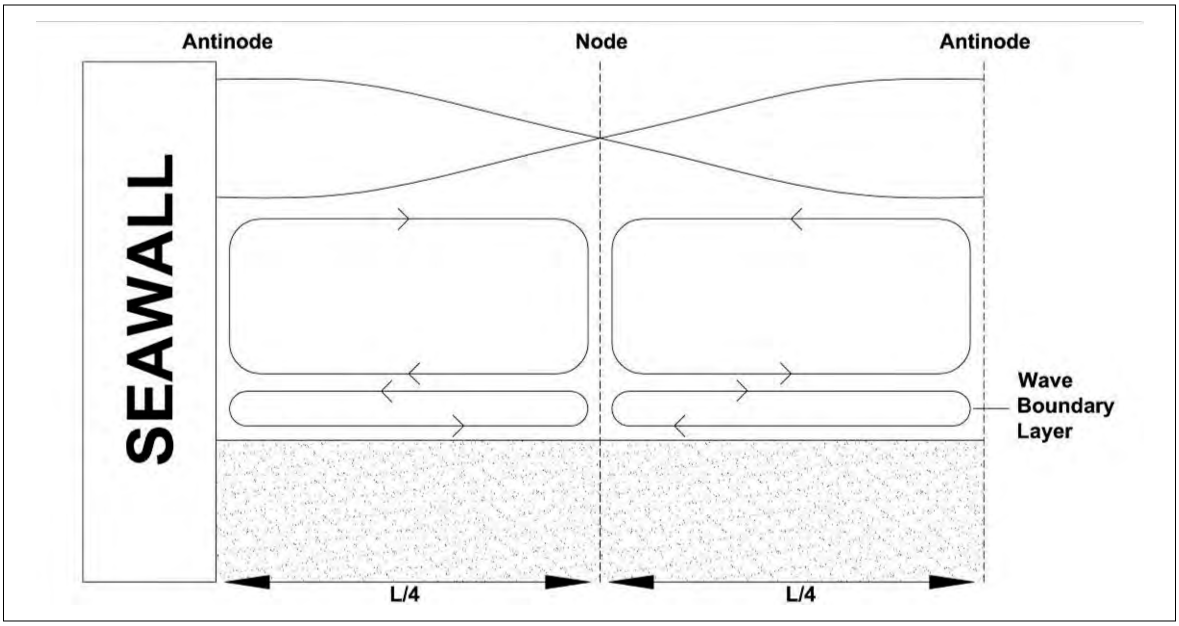


Figure A.10 Recirculating currents due to streaming under regular standing waves on a horizontal bed

If the sediment in the bed is coarse and travels close to the bottom, it will be most influenced by the horizontal movements in the bottom boundary layer, which are towards the node. The result is scouring midway between anti-node and node, and deposition under the node, as shown in Figure A.11 (lower panel). This is known as bedload or ‘N-type’ scour (Xie 1981).

If the sediment is small and is maintained in suspension, it will be most influenced by the current above the bottom boundary layer, so the net movement is away from the nodes towards the antinodes, as shown in Figure A.11 (upper panel). This is known as suspended or ‘L-type’ scour (Xie 1981). Thus the pattern of sediment erosion and accretion varies with the mode of sediment transport; bedload transport gives a different pattern from suspended load transport.

Table A.2 indicates the type of scour (‘L’ or ‘N’ type) that may be predicted for a depth-limited incident wave at a coastal structure. For simplicity, the wave height has been set to 0.8*h*, where *h* = water depth and a constant wave period of 8 s has been chosen to represent a storm wave in UK coastal waters.

Table A.2 Indicative scour pattern

Water depth (m)	Sediment grade	Typical grain size (mm)	Scour type for depth limited wave of eight seconds
2	Fine sand	0.1	L-type (suspended transport)
2	Medium sand	0.25	L-type (suspended transport)
2	Coarse sand	1	Borderline N/L type
5	Fine sand	0.1	L-type (suspended transport)

5	Medium sand	0.25	L-type (suspended transport)
5	Coarse sand	1	L-type (suspended transport)

These definitions are indicative of the behaviour that can be expected, although the mode of scour and pattern will be different with random waves and with wave breaking, and with a sloping beach – as opposed to a horizontal bed.

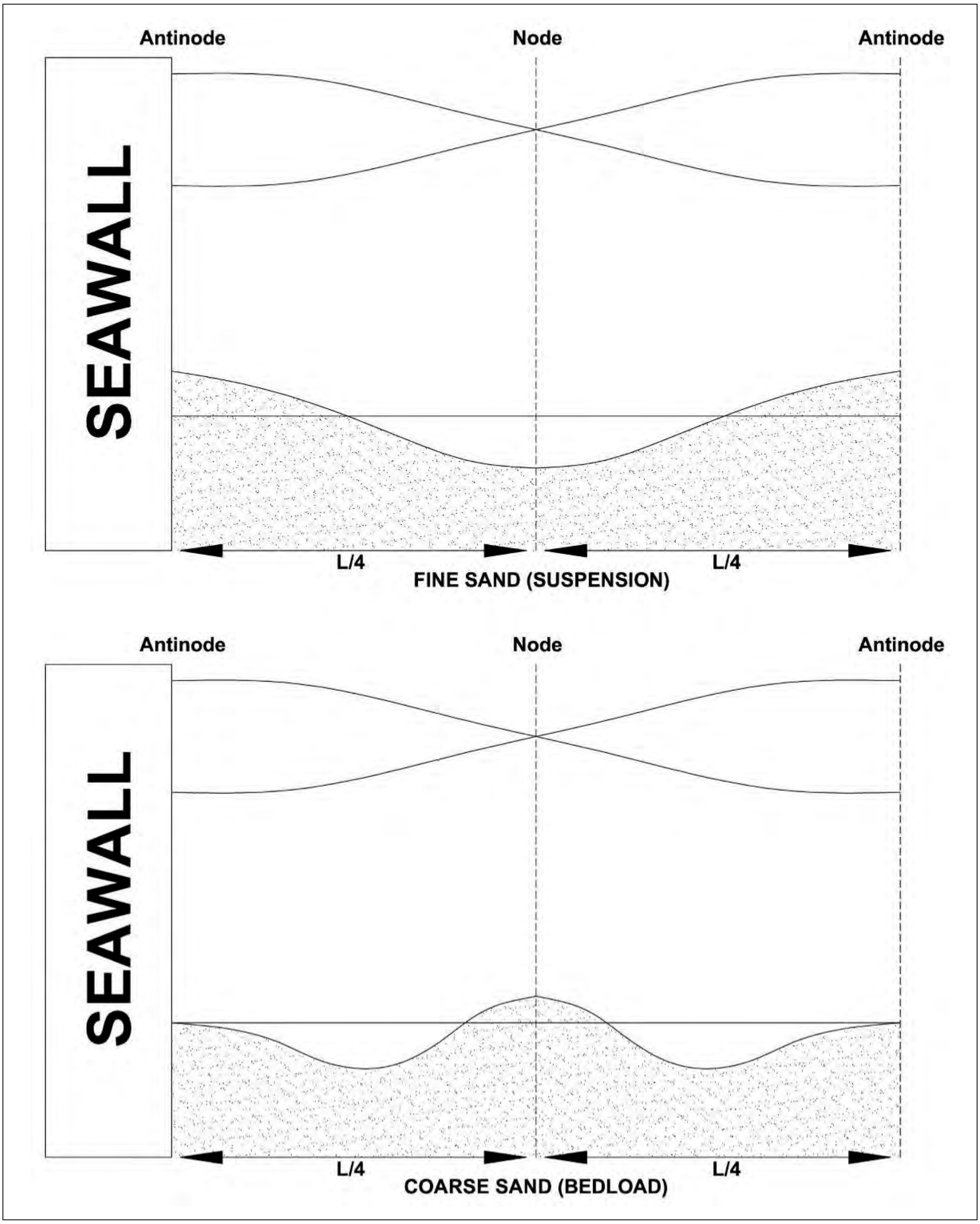


Figure A.11 Scour and deposition patterns on a horizontal bed over half a wavelength in front of a vertical seawall: Upper panel: Suspended sediment transport ('L-type'; Xie 1981); Lower panel: bedload sediment transport ('N-type'; Xie 1981)

A.3.3 Random wave kinematics in front of a coastal defence structure

Waves incident upon a coastal structure are reflected from it to some extent. The interaction of incident and reflected waves sets up a partial standing wave pattern in front of the breakwater. It is common in coastal engineering studies involving random waves to consider the random sea as the linear sum of a large number of incident component waves, plus the reflected components (O'Donoghue and Sutherland 1999). This approach ignores nonlinear interaction (such as clapotis where waves interact to form a breaking wave) but allows solutions to random wave problems to be formulated relatively easily. Each component has a reflection coefficient, given by the ratio of reflected wave amplitude over incident wave amplitude (Sutherland and O'Donoghue 1998b) and a phase shift, which relates the phase of the incident and reflected waves at the toe of the structure (Hughes and Fowler 1995; Sutherland and O'Donoghue, 1998a).

If there is a random sea state, all reflected components will be in phase with the incident component of the same frequency at a vertical wall; hence an anti-node is formed at the wall. On moving further out from the wall, each component will move out of phase at a different rate, as each has a different wavelength. Therefore the next anti-node out from the wall will have a lower RMS surface elevation than the one at the wall, and the one out from that will be lower again. Hughes (1992) showed how the spatial variation in RMS surface elevation depends on relative depth kh with $k=2\pi/L$ being the wave number, L the wavelength and h the water depth. It also depends on the narrowness of the wave spectrum. The narrower the wave spectrum, the larger the cross-shore distance over which the partial standing wave pattern will be apparent. Moreover, the shallower the water, the greater the cross-shore distance over which this phase-locking will be apparent.

The variation of RMS orbital velocity with distance from the toe of a vertical wall is shown as a function of relative depth in Figure A.12 where:

u_{rms} = root mean square horizontal wave orbital velocity at the seabed (ms^{-1})

g = gravitational acceleration (ms^{-2})

k_p (and K_p) = linear theory wave number based on spectral peak wave period (m^{-1})

T_p = spectral peak wave period (s)

H_{mo} = spectral significant wave height (m)

x = cross-shore distance from seawall (m).

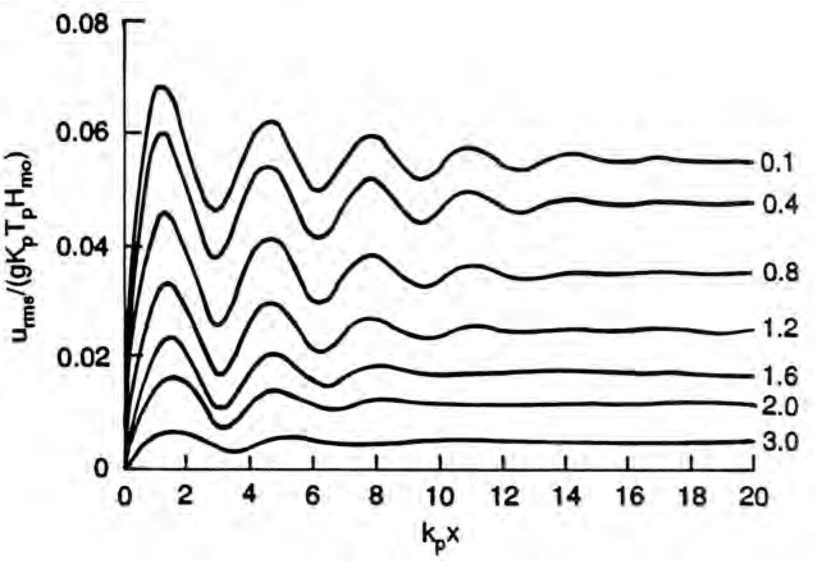


Figure A.12 Non-dimensional plot of RMS wave orbital velocity on the bed as a function of cross-shore position for different relative water depths (after Hughes and Fowler 1991)

For a sloping seawall, the incident and reflected components are already out of phase at the structure toe and reflection coefficients are lower than for a vertical wall. Both factors mean that the partial standing wave pattern generated in front of a sloping seawall is less obvious than that in front of a vertical wall (Hughes and Fowler 1995).

Hughes and Fowler (1995) and O'Donoghue and Sutherland (1999) have shown how to combine linear theory expressions for the kinematics with reflection and phase shift spectra to determine the velocities and surface elevations in front of reflecting coastal structures. These are analytical models that do not include wave breaking due to shoaling.

A.3.4 Scour in sand at a vertical wall under random waves

The pattern of scour under random waves reflects the kinematics of the waves that generate it. Bed level changes (final minus initial elevation) at the end of four laboratory experiments (after 3,000 waves) conducted with random waves, fine sand and a vertical seawall are provided in Figure A.13 (data from Sutherland et al. 2007). The wavelength in these tests was such that the scour plotted in Figure A.13 is over a distance of half of a wavelength (at a water depth of 0.4 m) to almost one wave length (at a depth of 0.1 m) – to compare with Figure A.11 – and the scour type in the model was a mixture of bedload and suspended load type. This also covers a distance of k_px to a value of 3 (at a water depth of 0.4 m) to 5.9 (at a depth of 0.1 m), which means that it covers the most intense part of the variation in wave orbital velocity plotted in Figure A.12.

In Figure A.13 negative values represent scour while positive values represent accretion. These four tests had the same initial bed profile (a 1:30 slope), the same wave period (T_p = 3.24s) and the same offshore incident wave height ($H_s \approx$ 0.2m), but different initial water depths at the toe of the seawall(h_t = 0.0, 0.1, 0.2 and 0.4m respectively). These tests show the controlling influence of water depth. A comparison has been drawn between these four tests as they resulted in very different breaking wave conditions at the wall and hence different bed profiles. Thus the scour profiles shown in Figure A.13 are for random, breaking waves in sand on a sloping beach.

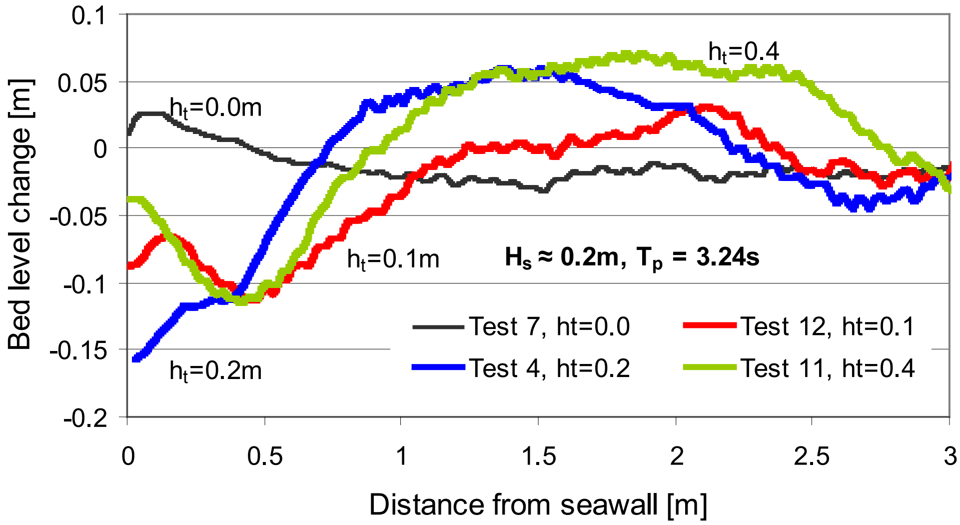


Figure A.13 Variation in final scour depth with water level for a vertical wall measured in laboratory experiments with a sand bed (from Sutherland et al. 2007)

During Test 7 (h_t = 0.0 m), the waves broke offshore and the wave energy was largely dissipated before the waves reached the wall in the swash zone. As a result there was a slight accretion at the wall but a general lowering throughout the rest of the profile. The vertical seawall was situated within the surf zone during Test 12 (h_t = 0.1 m) and some breaking occurred onto it, although most of the larger waves had already broken by the time they reached the seawall. The resulting scour profile includes a small dip at the toe of the seawall caused by turbulence and a deeper scour hole at about 0.5 m from the structure toe.

However during Test 4 (h_t = 0.2 m), the waves tended to break onto the structure and the impacts sent water high up above the seawall. In these cases, water plunging down the face of the seawall to the bed resulted in suspended sediment transport at the toe and this mechanism generated the deepest scour depths. Figure A.13 shows that the maximum scour occurred at the wall (0.158 m), with significant accretion (0.056 m) occurring 1.3 m offshore.

In deeper water (Test 11, h_t = 0.4 m), most waves did not break onto the seawall as plunging breakers, but tended to reflect from the seawall without breaking. Erosion still occurred at the seawall toe but was deepest at about $L_p/16$ from it, where L_p is the linear theory wavelength at the spectral peak wave period, calculated using the water depth at the seawall. The maximum scour of 0.117 m was significantly less than for Test 4, the plunging breaker case where the toe scour was 0.158 m.

The peak in accretion occurred in Test 11 at around $L_p/4$ or $k_p x \approx \pi/2$, where Figure A.12 indicates there will be a peak in RMS horizontal velocity. Figure A.11 indicates that accretion at $L_p/4$ from the seawall would be expected for bedload transport, yet almost two-thirds of incident waves were expected to meet Komar and Miller's criteria for suspended sediment transport, increasing to around 75 per cent when wave reflection was taken into account. This indicates that the tests were conducted in the suspended sediment transport regime, although with a greater percentage of bedload than would be expected in field cases.

The variations in scour depth with distance from the seawall are different in Figure A.11 (top, for suspended load) and Figure A.13, as Figure A.11 is for an idealised case of a flat seabed with regular non-breaking waves while Figure A.13 is for a 1:30 sloping seabed with breaking irregular waves.

A.3.5 Scour in sand at a sloping wall under random waves

Bed level changes (final minus initial elevation) at the end of three laboratory experiments (that is after 3,000 waves) conducted with random waves, fine sand, and a 1:2 (V:H) smooth sloping seawall are provided in Figure A.14. Negative values represent scour, while positive values represent accretion. These three tests had the same initial bed profile, wave period (T_p = 3.24s) and similar offshore incident wave height (H_s = 0.19m to 0.24m) but different water depths (h_t = 0.0m, 0.2m and 0.4m respectively). A comparison has been drawn between these three tests as they resulted in very different breaking wave conditions at the wall and hence different bed profiles.

In Test 27 the wave down-rush reached the sediment bed and caused the greatest scour, whereas in Test 30, the water depth was sufficient to ensure that the down-rush did not reach the bed and a lower scour depth occurred. In Test 32, the water depth at the toe of the structure was initially zero – the still water and sand beach intersected at the seawall. In this case there was accretion at the toe of the seawall, caused by sediment transport in the swash zone.

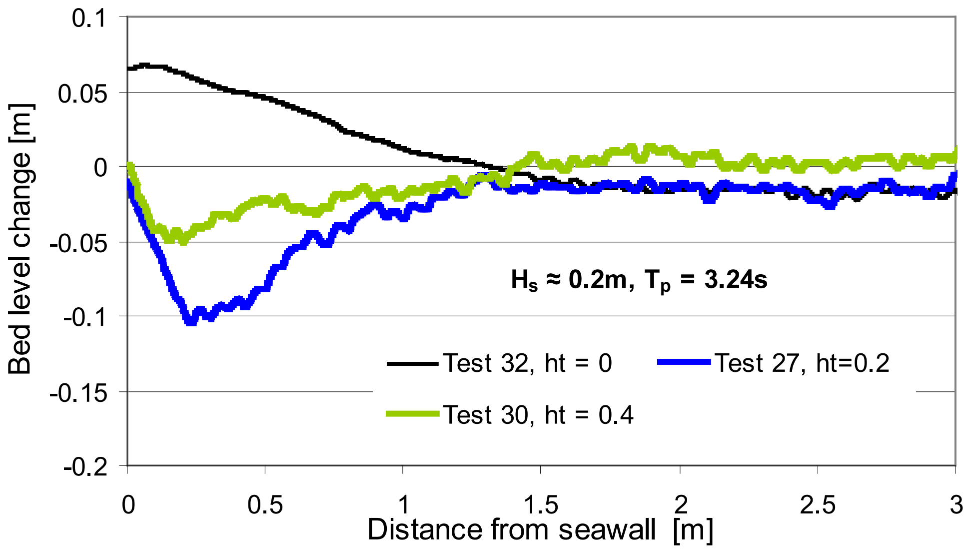


Figure A.14 Variation in final scour depth with water level for a 1:2 sloping wall (ht is initial toe water depth in m)

(Sutherland et al, 2007).

A.3.6 Scour in shingle under random waves

The scour response in shingle beaches tends to be confined more closely to the seawall than is the case with sand beaches, based on the result of laboratory tests (Powell and Lowe 1994), as shingle is relatively less mobile than sand.

As with sand, scour of a shingle beach in front of a seawall will be controlled by water depth and wave conditions. The scour profiles in Figure A.15 show how the response of the beach changes from accretionary in Types I and II, with a minor formation of toe scour at the seawall in the latter case, to scouring in Types III and IV. The toe scour in Type III is maximised owing to the particular combination of water depth and wave height occurring, and in Type IV the toe scour is reduced owing to the deeper water depth.

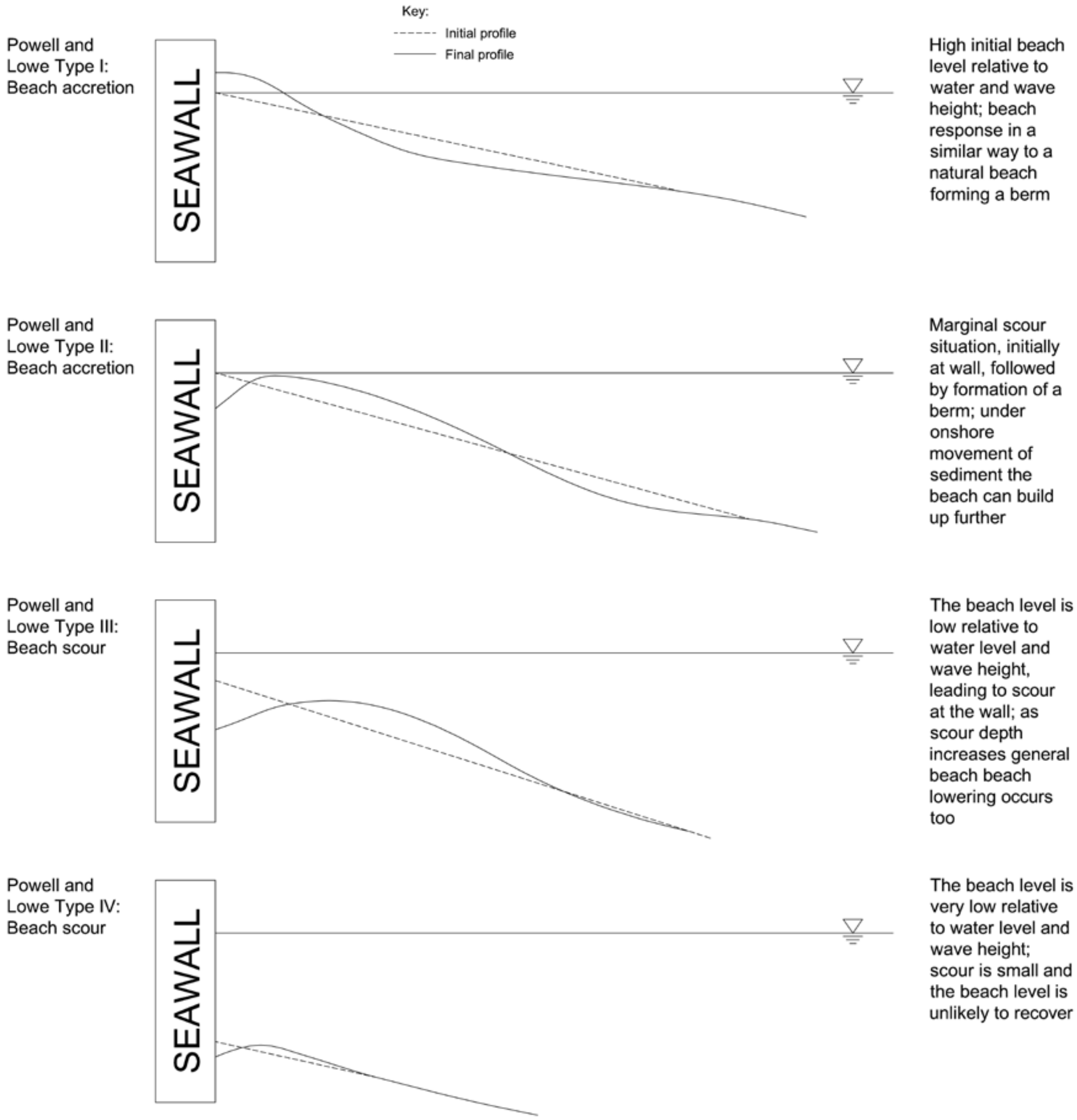


Figure A.15 Schematic diagram of beach profile and scour response under wave attack in shingle – based on model tests of Powell and Lowe (1994)

A.3.7 Angle of wave approach

When a single obliquely incident wave is reflected from a seawall, a short crested wave field is formed, which is characterised by a diamond-shaped pattern of ‘island crests and troughs’ (Hsu and Silvester 1989). Lines of island crests and troughs occur at regular intervals in front of the structure. The reflection coefficient of the seawall determines the magnitude of the island crests and troughs as a function of incident wave height, while the phase shift on reflection gives the distance from the structure to the first line of crests and troughs.

The angle at which the wave front hits the seawall has also been suggested as a factor affecting toe scour (Hsu and Silvester 1989). The depth of toe scour is expected to be greater if waves hit the wall obliquely, because the incident and reflected wave trains interfere with each other constructively, producing an interference pattern of short crested waves. Consequently, the wave height and hence scour potential at the base of the wall should be larger if the angle of incidence is oblique rather than perpendicular to the seawall. In addition, oblique waves may induce local currents parallel to the seawall, (Lin et al. 1986; Oumeraci 1994), which enhance sediment removal at the toe of the structure.

For further discussion on the effect of oblique waves the reader is referred to Section B.1.6 in Appendix B.

A.3.8 Overtopping

A further factor of importance may be the extent of any overtopping of the seawall. It is reasonable to expect that seawalls that experience heavy wave overtopping will offer less scour because the proportion of energy reflected or dissipated as turbulence at the wall will be reduced. This effect has probably not been taken into account in previous studies of toe scour for which the majority of walls appear to have been of sufficient size to limit the extent of any wave overtopping. Thus, most empirically based methods for the prediction of toe scour may be conservative if applied to low crested structures that experience regular overtopping.

There are no design relationships to take into account the overtopping influence on scour depth, although the following description is informative. Nishimura et al. (1978) studied the scour at seawalls caused by an incident tsunami (effectively a very long period wave). In this case the overtopping water returned down the face of the structure and much of the scour was caused by the flow return. They noted that:

- scour depth decreases with decreasing wave height and increasing crown elevation (as there is less return flow), although the area of serious scouring is displaced towards the seawall in this case;
- scour increases (and it occurs at the toe precisely) when the face slope is mild;
- scour decreases markedly when the water depth at the seawall increases, as less turbulence reaches the bed;
- when waves are applied repeatedly, much less scouring is induced by each successive wave.

To date most numerical models can only simulate overtopping by reducing the reflection coefficient for a given seawall profile (see Pullen et al. 2007).

However, developments in phase-resolved modelling of non-linear shallow-water waves (following, for example, Dodd 1998) have allowed wave-by-wave overtopping events to be modelled. Such models could be coupled with sediment transport and bed updating models to investigate the effect of overtopping on scour, although such work is in its infancy. Few, if any, numerical models are able to simulate accurately the turbulent dissipation occurring at the seawall.

A.4 Description of processes controlling liquefaction

In soil mechanics, liquefaction starts to occur when the effective stress of the seabed becomes zero. A useful introduction to the liquefaction of non-cohesive seabeds 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 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 and so the bed is compressed. Under a wave trough, the pressure at the bed is less than hydrostatic and 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 the high shearing resistance, which prevents the excessive build-up in pore pressure. Therefore the occurrence of residual liquefaction in front of a coastal defence structure is unlikely 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 re-nourishment or excavation works, there is a risk of residual liquefaction which may need to be examined. Residual liquefaction can persist for several wave periods and can extend over larger areas of the seabed than momentary liquefaction.

Definitions

Pore pressure: the pressure experienced by the fluid between the sand grains within the seabed.

Excess pore pressure: the difference between pore pressure and hydrostatic pressure.

Overburden pressure: pressure caused by weight of sediment and water above

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 difference 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 (Sumer and Fredsøe 2002) and only affects a limited area of the seabed.

The presence of a coastal structure will affect the potential for liquefaction in two main ways:

- Wave reflection from the structure will increase the effective wave height in front of the structure by up to a factor of two, thereby increasing the potential range of the excess pore pressures experienced within the seabed. Moreover the setting up of a partial standing wave pattern (see Section A.3) will create larger horizontal as well as vertical pressure gradients in the seabed. A method for predicting whether residual liquefaction is likely to occur in front of a coastal structure is presented in Section B.2 in Appendix B.

- Flows may be induced under the toe of the structure, which can affect the pressure in the seabed. Maeno and Tsubota (2001) carried out scale model experiments investigating the flow out of back-filling sand behind revetments due to wave loading. They showed that the cyclic seepage force which occurs around the revetment plays an important role in the flow out of the sand. Loveless et al. (1996) also demonstrated the effect that groundwater flows under a seawall could have on the scouring of a model gravel beach, even without inducing liquefaction. In the most extreme case, an onshore flow (beach de-watering) produced accretion near the seawall toe and an offshore flow (under the seawall from the landwards side) produced more erosion than occurred during the no flow test.

In cohesive material, de Wit (1995) found that the potential for fluidisation of a soft clay cohesive bed was linked to the magnitude of the wave induced stresses and the undrained shear strength of the bed. Typically if the wave-induced stresses exceeded the shear strength of the bed, the bed will fluidise. The depth of fluidisation will be controlled by the variation in depth of the forcing and the strength. For given wave conditions and soil strength profile, there will be an equilibrium fluidisation depth.

Results of surveys at Warden Point with a London Clay platform (Royal Haskoning et al. 2007) showed the presence of thin fluid mud layers overlying more consolidated deposits at various times and locations on the foreshore. Observations of extensive areas of fluid mud were found on the upper shore platform during July (2005) and these were greatly reduced in the succeeding autumn and winter. From that dataset, it is not clear that the process of mud fluidisation directly led to formation of the observed layers since the clay is not soft. It is more likely that erosion of clay took place which subsequently re-deposited through flocculation as a layer of unconsolidated mud on the platform (Royal Haskoning et al. 2007).

A.5 Description of processes controlling beach variability

The variability in beach morphology over timescales of tides to seasons depends on both longshore and cross-shore sediment transport processes. The changes in seabed level observed are the residual changes from the smaller scale processes that occur during each tide.

Sediment is moved in the cross-shore direction by wave action, with onshore motion occurring as bedload where waves become skewed (with sharp crests and long flat troughs) during shoaling and asymmetric (with steep front faces and more gentle back slopes) after breaking. The breaking waves also create setup (a local increase in water level at the shore) which drives undertow – an offshore directed return flow in the central part of the water column between the top of the wave boundary layer and the wave troughs. Sediment that has been lifted into suspension by the speed of the near-bed wave velocities and/or turbulence generated by breaking is transported offshore as suspended sediment transport by the undertow.

During a storm, offshore-directed suspended sediment transport tends to dominate and the beach is drawn down with sand being deposited near the breaker line, which may cause the formation of, or increase the size of, a breaker bar. During calmer conditions and particularly when there are relatively long low waves, such as swell, the beach material is brought back towards the shoreline as bedload.

It is therefore common to find beach levels lower in winter than in summer due to the increased occurrence and severity of storms during winter (Figure A.2d). It also follows that beach levels may show a greater variation about their seasonal mean during winter. Both these phenomena can be detected in repeated surveys of beach profiles.

Sediment is also moved in the longshore direction by waves and currents. Waves approaching the shoreline at an angle generate a net longshore current when they break, which can transport sediment along the beach. Wave action in the swash zone also drives sediment in the longshore direction as the uprush occurs at an angle to the beach while the downrush occurs more directly down the beach.

Longshore transport, also known as longshore drift, affects beach levels in front of coastal defence structures when there is a gradient in the longshore transport rate. Where the volume of sand leaving a cross-shore profile is on average greater than the volume entering it, erosion occurs and beach levels at a structure are likely to show a trend of lowering. Conversely, when the volume of sand leaving a cross-shore profile is on average less than the volume entering it, accretion occurs. This is not a common long-term trend in front of coastal defence structures as there is generally no need for a coastal defence on an accreting beach. Shorter term reversals of drift direction may lead to accretion, even when the long-term (decadal) trend is for erosion. The controlling effect of gradients in longshore transport as a cause of increasing or decreasing the sediment volume at any one coastal section conditions the beach response to storms causing cross-shore exchanges of sediment (van de Graaff 2004). The initial losses of sediment at a beach section due to longshore sediment transport processes may occur on the lower beach, possibly below low water and remote from the influence of the seawall, which means the beach has increased susceptibility to erosion due to cross-shore losses during storm events.

The effect of longshore drift on beach levels in front of defences can probably be seen most clearly in a groyned beach, where changes in wave direction can cause the longshore transport direction to change. The result is that sand builds up first against the side of one groyne with a corresponding reduction in beach level at the other groyne, then against the other side when the direction changes. The beach levels at both ends of the groyne bay are likely to vary substantially more than in the centre of the groyne bay as the plan-shape of the beach changes around that point.

A.6 Description of processes controlling coastal erosion

Many coastlines around the world are eroding as a consequence of the continual and damaging effects of winds, rainfall, waves and currents. This tendency is further strengthened by the gradual rise in sea level, allowing larger waves to travel further inshore, hence increasing their effects on the seabed and on beaches. This section considers first the erosion of non-cohesive beaches (predominantly sand) and then the erosion of shore platforms.

A.6.1 Erosion of non-cohesive beaches

Where for example a seawall is built to protect an eroding shoreline, as sketched in Figure A.16, it will not directly prevent erosion of the adjacent sections of that coastline. In this hypothetical situation, it is assumed that there is no longshore sediment transport and that the material eroded from the coastline and the shore platform is so fine-grained that it is transported out into deep water. This situation is similar to that found on the coastline of the Great Lakes in North America.

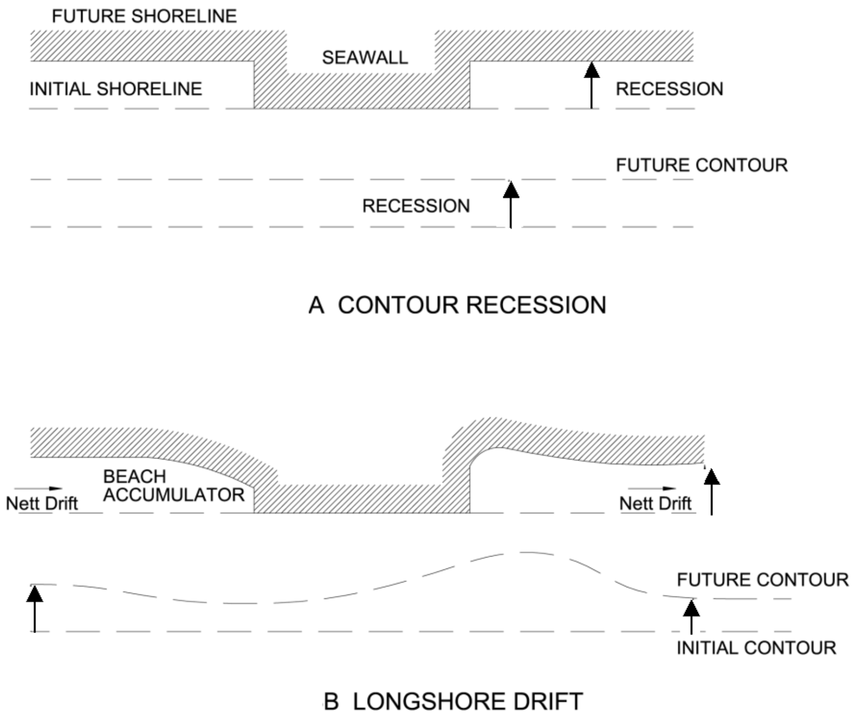


Figure A.16 Conceptual sketches of the effect of a seawall on coastal erosion

The principal cause of coastal recession in this situation is the continual erosion of the shore-platform where the wave-induced water velocities are high, particularly in the breaker zone and just outside it.

As the erosion of the coast either side of the seawall continues, the ground level either side of the toe of the wall will fall. Now assume that the seawall affects the hydrodynamic/ geomorphological processes in front of it 'beneficially' in the short term, for example, by reducing the height of the reflected waves compared to those in front of, say, adjacent cliffs. It might then be hoped that the ground levels in front of the wall would not lower as quickly as those in front of the cliffs.

However, this would require an increasingly steep longshore slope in the foreshore levels (in this case the shore platform levels) as time passes, that is, at either end of the seawall. Experience indicates that this does not occur. It is a reasonable approximation therefore to assume that the ground level at the toe of the wall will be (at best) equal to that of the ground level on either side of it. This was the advice provided in the USA by the Beach Erosion Board (1954). Griggs et al. (1991) refer to this process as 'passive erosion'. In this simple situation, therefore, the ground levels in front of the coastal structure depend on the ground levels on either side of that structure. If these continue to fall, beach lowering in front of the structure will occur.

Where groynes are built out from the shoreline, along the seawall and perhaps along the adjacent unprotected coastline as well, they can reduce the tendency for levels immediately in front of the wall to match those at either side of it, at least in the short term. Even so, the ground level at the seaward end of the groynes will be similar to that on either side of the groyne field. To maintain higher levels than would be expected without the groynes would therefore require a steeper bed/beach gradient within the groyne bays than outside.

The simple example sketched in Figure A.16 for a short length of seawall may not apply for situations, such as at Blackpool or Bournemouth, where seawalls stretch along many kilometres of coastline. A further complication arises when different types of seawall are present along a stretch of coastline, since the lowering of the beach in front of one section of an energy-dissipating seawall may be altered by the effects of adjacent, more reflective structures.

Many coastal erosion problems are a result of the interruption or alteration of the rates of sediment transport along the coastline (that is, littoral drift). If we consider the simple situation above, but now assume that there is a beach and a nett longshore drift rate, then the seawall will interfere with that sediment transport (Figure A.16B). Experience has shown that the normal effect of a seawall on longshore drift is to reduce its rate in front of the wall. Van de Graaf (2004) suggested that, as the water depth in front of the seawall deepens, the longshore transport rate will decrease. As a consequence of the differences between the drift rates in front of the wall and beyond either end of it, there tends to be an accumulation of beach sediments 'updrift' of the wall and corresponding erosion 'downdrift'. On the updrift end, the accumulation of beach sediments will tend to compensate for any trend of long-term recession of the shoreline and indeed may prevent this from occurring (for example the situation at the western end of the promenade at Sheringham in Norfolk – see Figure A.17).

Figure A.17 Accumulation of shingle at updrift end of seawall, Sheringham, Norfolk

Conversely, the interruption of longshore drift by the seawall results in greater downdrift erosion problems, at least locally, than would have occurred otherwise (for example the situation at the end of the seawall shown in Figure A.18 – Zanzibar). This localised erosion problem will be often reflected in the beach/shore/platform levels just downdrift of the wall, and the ground level contours in front of the wall can be expected to be lower at its downdrift end than at its updrift end (see Figure A.16B).

Figure A.18 Erosion at downdrift end of seawall, Zanzibar

Where a seawall prevents erosion of material from cliffs or dunes that would otherwise, by receding, have provided sediment to the beaches, there will be a further deleterious effect on the downdrift coast. Many ‘promenade’ seawalls built over the last 200 years around the coast of the UK have not only caused this situation (for example at Bournemouth – see Figure A.5 where the cliff is situated on the right of the figure) but, using more emotive language, have ‘impounded’ (Griggs et al. 1991) or ‘imprisoned’ (Owens and Case 1908) a considerable amount of potential beach sediment.

A photograph taken during the construction of the promenade at North Beach, Llandudno, for example (from Owens and Case 1908) seems to show that the shingle ridge that once ran along the beach was used as fill material for the new wall. This practice may have been commonplace in that century. In some cases, structures may have been filled with sand from the beach in front of them, hence leading to the likelihood of an ‘instant’ lowering of the beach in front of and downdrift of the wall.

Thus, it is important to note that coastal erosion at coastal defence structures constructed to mitigate against coastal erosion may experience continued erosion despite the presence of the structure, which neither adds nor removes sediment but prevents it from entering the coastal sediment transport system. As previously discussed, the interaction of waves with seawalls can cause local scour at the toe during storms, but there is no evidence to show that coastal defence structures delay the recovery of beaches if there is sufficient sediment available in the vicinity to rebuild the beach.

A.6.2 Erosion of shore platforms

A shore platform and coastal defence is similar to a shore platform and cliff system. Indeed many shore platform and defence systems used to be shore platform and cliff systems. Defences may have been added when cliff recession threatened valuable assets and while these defences can halt the erosion of the subaerial cliff, they do not stop the erosion of the foreshore, as illustrated in Figure A.19.

Figure A.19 **Erosion of a cohesive shore platform at the toe of a seawall**

Thorough reviews of the mechanisms for the downwearing of shore platform and cliff systems have been provided by Nairn and Willis (2002), Walkden and Hall (2005), Royal Haskoning et al. (2007) and Trenhaille (2009).

The following list of relevant erosion and weathering processes is based on that in Royal Haskoning et al. (2007), which contains more information and references on each process:

- **Abrasion by mobile non-cohesive sediment**. Waves and currents can move sand and gravel across the surface of a shore platform, leading to abrasion. Abrasion will cease if the sediment becomes deep enough or the fluid velocities low enough to ensure that there is no motion of the particles in contact with the shore platform.

- **Mechanical wave erosion**. The surface of a shore platform will erode if the shear stress generated at its surface is higher than its threshold shear stress for erosion. This can occur under non-breaking waves, but will be enhanced by wave breaking, particularly under plunging waves when the turbulence reaches the seabed. Rapid rates of erosion have been noted in the Canadian Great Lakes just in front of cliffs and seawalls where reflections enhance the turbulent energy dissipation.

- **Biological processes**. Boring organisms can weaken the surface layers of a shore platform (for example, upper 10,mm), making it more susceptible to mechanical erosion. This is perhaps more the case where the physical rates of erosion are low, as this allows extensive colonisation to take place. It is also affected by the tidal range as this affects the duration of submergence.

- **Desiccation and wetting**. Repeated wetting and drying can cause the upper layer of the shore platform to desiccate, which may lead to the upper surface desiccating and cracking.

- **Physico-chemical effects in clay**. Salt seeping and diffusing into a clay shore platform from seawater may improve its resistance to erosion.
- **Freeze–thaw cycle**. Shore platforms may freeze when exposed to air temperatures below zero, but then rapidly thaw when the tide comes in causing frost damage (assuming that the sea remains unfrozen). It is rare for the freeze–thaw cycle to cause much damage in the UK, but it is an important mechanism in colder countries.
- **Softening following the removal of overburden**. Shore platform that is newly exposed by the erosion of a cliff is subject to a lower load than it was previously and may expand and lose strength.
- **Softening due to pressure fluctuations caused by waves**. The pressure at the surface of the shore platform will vary as waves pass overhead and this repeated increase and decrease in pressure may soften the surface of the rock.

Consider the case of a vertical seawall with its toe in the shore platform, but where a sand beach is normally present, as sketched in Figure A.20. If a storm were to occur when the beach level was already low (probably in winter), then it may be possible for a section of shore platform in front of a seawall to be exposed by the scouring away of the sand beach. The exposed section of shore platform may then be eroded through abrasion or excess shear stress, causing a local lowering of the level of the shore platform. When the storm subsides, the scour hole is likely to fill up with sand (see Section A.2.1) leaving little trace of the presence of a scour hole in the sand and none of the erosion of a shore platform. If the non-cohesive beach were to be completely lost, the same location may be eroded more than nearby locations due to the increased levels of turbulence.

The interaction also occurs as a three-dimensional process due to the formation of a patchy veneer of sand and gravel on the platform, migration of sand bars and spatial variations in biological engineering.

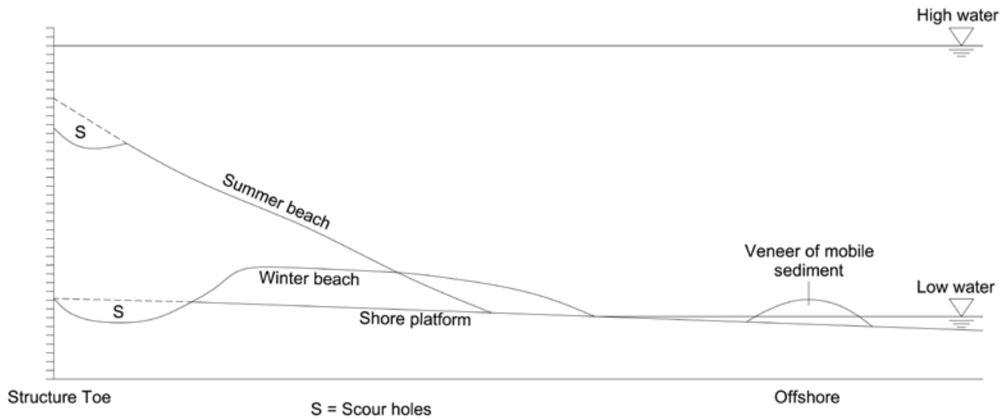


Figure A.20 **Seawall founded in cohesive shore platform with beach**

Appendix B: Predictive methods

B.1 Methods for predicting toe scour

B.1.1 Introduction

This section describes the process by which scour in front of seawalls can be predicted for sand and shingle beaches. Previous studies have reviewed a range of existing scour prediction methods and refined and developed various aspects of them (see Sutherland et al. 2003, 2007; Royal Haskoning et al. 2007). Although numerical cross-shore profile models have been used to model wave induced toe scour (Powell and Whitehouse 1998), they still lack some of the main processes involved and so the most common approach to predicting toe scour is through the use of empirical predictors fitted to experimental data. This section contains the most up-to-date:

- predictor for toe scour depths caused by irregular waves at a vertical seawall in a sand seabed;
- predictor for toe scour depths caused by irregular waves at a vertical seawall in a shingle seabed;
- commentary on how to compensate for oblique angle waves and sloping seawalls.

B.1.2 Prediction of toe scour depths at vertical seawalls in sand seabed

The new empirical equations for the depth of scour at the toe of a vertical seawall presented below have been developed using an extensive database of new and previously published laboratory data (HR Wallingford 2006b, Sutherland et al. 2006b) (see Figures A.13 and A.14 in Appendix A for typical scour profiles).

All the laboratory data was collected in wave flumes using irregular waves with a TMA or JONSWAP spectral shape and a constant water depth through each test. Each test lasted for 3,000 waves or until equilibrium was reached. In the majority of cases, a fine sand with median diameter d_{50} = 0.13 mm (Fowler 1992) or d_{50} = 0.11 mm (HR Wallingford 2006b) was used and the incident offshore wave height was approximately 0.20 m. The initial bathymetry for each test was a smooth sloping sand bed. The scaling of the tests was designed to ensure the key processes were represented in the modelling.

Field data on beach lowering and recovery during a tide has also been collected (HR Wallingford 2006a, 2006c). These data were collected in situations with constantly varying water levels and wave heights. The field data have been plotted with the laboratory data in Figure B.1, which shows that the field data generally have lower scour depths than the laboratory data. This difference is believed to have been caused by the use of wave height, wave period and scour depth measured at high tide and which are therefore valid for only a small duration. The field scour did not have sufficient time to reach an equilibrium scour depth. Nevertheless, in some cases the field measurements of scour did approach the values measured in the laboratory.

Also plotted on Figure B.1 is Equation B.1 (Equation 12 of Sutherland et al. 2007), which is intended to serve as a conservative predictor of scour depths and which may be used in the absence of site-specific information on beach slope. The scour depth is scaled with, and less than, H_s which is the unbroken offshore significant wave height:

$$\frac{S_{t\max}}{H_s} = 4.5e^{-8\pi(h_t/L_m+0.01)}\left(1-e^{-6\pi(h_t/L_m+0.01)}\right) \qquad [-0.013 \le h_t/L_m \le 0.18] \qquad \text{(Eqn B.1)}$$

where:

S_{tmax} = maximum toe scour depth (m)

h_t = water depth above the beach level at the toe of the structure (m)

$L_m = gT_m^2/2\pi$ = linear theory wavelength based on mean wave period T_m (s) and acceleration due to gravity g (assumed to be 9.81 m/s^2).

The range of validity of Equation B.1 is given in the square brackets in terms of h_t and L_m, which is the wavelength.

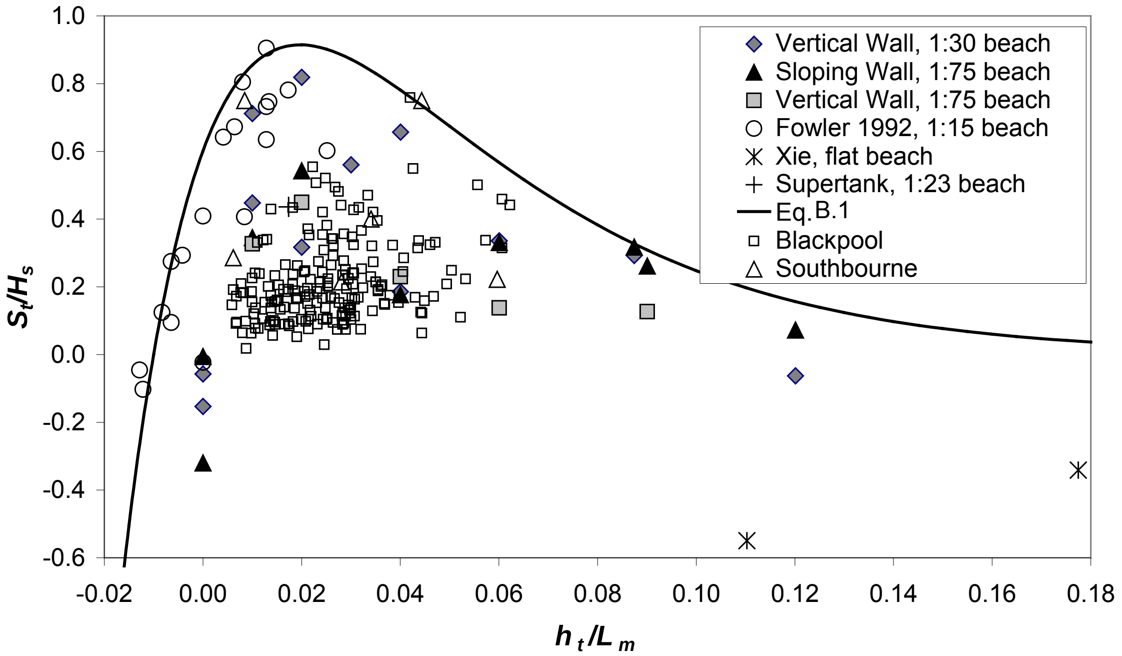


Figure B.1 Measured and predicted laboratory and field data of relative toe scour depth in sand (Sutherland et al. 2007)

HR Wallingford (2006f) showed that the relative toe scour depth from the laboratory experiments depends on the beach slope and is given by:

$$\frac{S_t}{H_s} = 6.8\left(0.207\ln(\alpha)+1.51\right)e^{-11.7\pi h_t^*/L_m}\left(1-e^{-6\pi h_t^*/L_m}\right) - 0.137 \quad [-0.015 \le h_t/L_m \le 0.12] \quad \text{(Eqn B.2)}$$

where:

S_t = scour depth at the toe of the structure (m)

H_s = incident significant wave height (m)

α = beach slope (radians)

h_t^* = water depth (m) above the beach level at toe of structure including effect of wave setup calculated using the equation of Holman and Sallenger (1985)

h_t = water depth above the beach level at the toe of the structure (m).

The range of validity of Equation 3.2 is given in the square brackets in terms of h_t and L_m, which is the wavelength: $L_m = g\,T_m^2/2\pi$ = linear theory wavelength based on the spectral average wave period.

Equation B.2 is plotted with the measured data in Figure B.2, 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. Equation B.2 has zero bias and systematic error and predicts the highest toe scour depths relatively well. In Figure B.2, Equation B.2 is plotted for the range of h_t^*/L_m for which data was obtained at that particular beach slope. The water depth h_t* includes a correction for wave set up which will be most influential at low value of h_t.

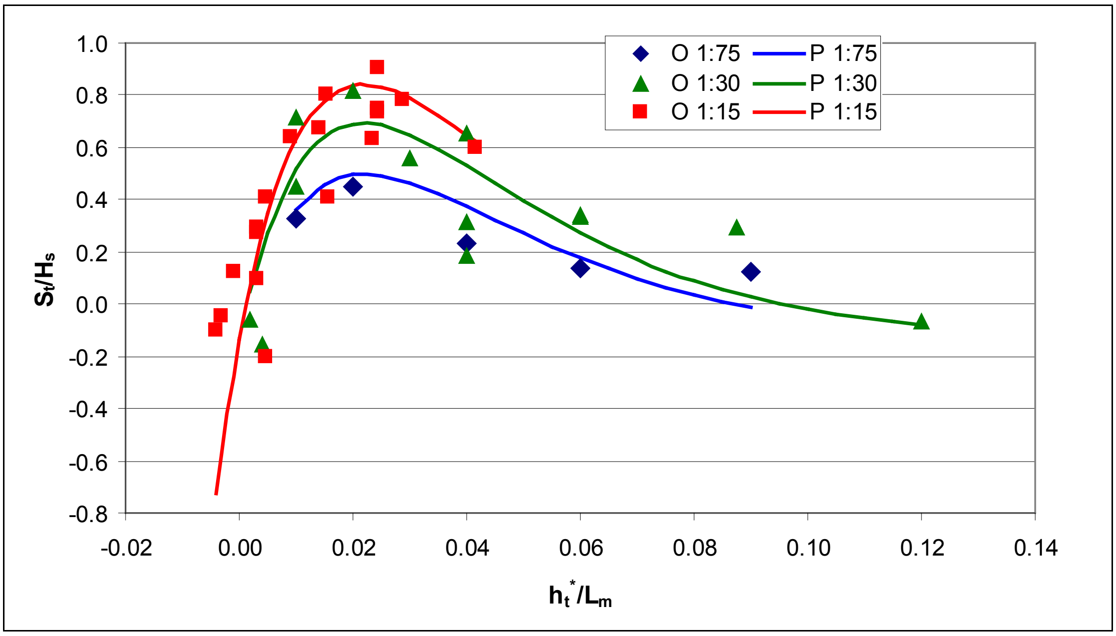


Figure B.2 Measured and predicted (Equation B.2) relative toe scour depths as a function of relative toe depth in sand (Sutherland et al. 2007)

The best-fit straight line (fitted to that predicted against measured data) 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 are relatively low errors for the high relative scour depths, which are likely to be the most important, while the largest errors in the predictions occur for negative observed scour depths (that is, accretion at the toe of the structure). However, these cases are relatively unimportant – at least as far as the stability of a structure is concerned.

B.1.3 Storm duration

The duration of the wave/water level conditions is also an important control on toe scour development. Scour is not an instantaneous process – the trough deepens over a number of waves. Powell and Lowe (1994) used physical wave flume models of a coarse grained beach to demonstrate how scour develops until a quasi-equilibrium is obtained within about 3,000 waves. There was rapid initial scour that declines exponentially towards the equilibrium depth. In Type I and II and Type III scour (Figure A.15 in Appendix A) approximately 55 per cent and 45 per cent respectively of the quasi-equilibrium scour depths formed in the first 100 waves; the development was slower for Type IV, requiring about 500 waves.

Similar trends are also apparent for sand beaches, though results from model studies (McDougal et al. 1986) suggest that the scour hole is slower to develop, with equilibrium unlikely to be achieved within a realistic storm/water level duration. This is supported by the result contained in Powell and Whitehouse (1998). The experimental tests of Sutherland et al. (2007) indicated that the average timescale of the scour was such that 95 per cent of the equilibrium scour depth would be reached after about 2,500 waves, although there was considerable scatter in the timescales derived. For typical storm mean wave periods of 6–8 seconds, this would take between about 4 and 5.5 hours to achieve.

The use of Equation B.2 is therefore recommended for predicting potential scour depths in the field. If the duration that the environmental conditions are expected to hold for is less than 3,000 wave periods, the expected scour depth may be reduced by a factor determined from Equation B.3 for the time variation of scour depth.

$$S(t) = S_e(1 - \exp(-t \, / \, T_s) \qquad \text{(Eqn B.3)}$$

where:

$S(t)$ = scour depth at time t (m)

t = time since start of scour process (s)

S_e = equilibrium scour depth (m)

T_s = timescale for scour (s).

McDougal et al. (1996) suggest T_s = 3100T, with T the wave period. Xie (1981) suggested that, for fine sand in suspension, the equilibrium scour depth would be reached in 6,500–7,500 wave periods for H/L >0.02 and in 7,500–10,000 wave periods for H/L <0.02. Powell and Lowe (1994) found that for a shingle beach the equilibrium scour depth was reached in about 3,000 waves.

B.1.4 Prediction of toe scour at vertical seawalls in shingle beaches

Scour depth in shingle beaches can be predicted using the parametric plot of Powell and Lowe (1994) reproduced as Figure B.3 (as also used by Whitehouse 1998), which was based on an extensive set of laboratory tests conducted with normally incident irregular waves that broke on a 1:7 slope shingle beach, with a vertical impermeable seawall. The method is valid for 5 mm <d_{50} <30 mm (modelled at 1:17 scale). (Figure A.15 in Appendix A shows some schematic scour profiles.)

Figure B.3 shows contours of S/H_s plotted on a graph with axes of relative depth, h_t/H_s and relative wave steepness, H_s/L_m, where:

- h_t/H_s: relative water depth
- h_t is the initial water depth at the wall
- H_s is the extreme unbroken deep water wave height
- H_s/L_m: wave steepness
- L_m is the mean wavelength of the unbroken wave (using $T^2g/2\pi$)
- S: scour depth after 3,000 waves.

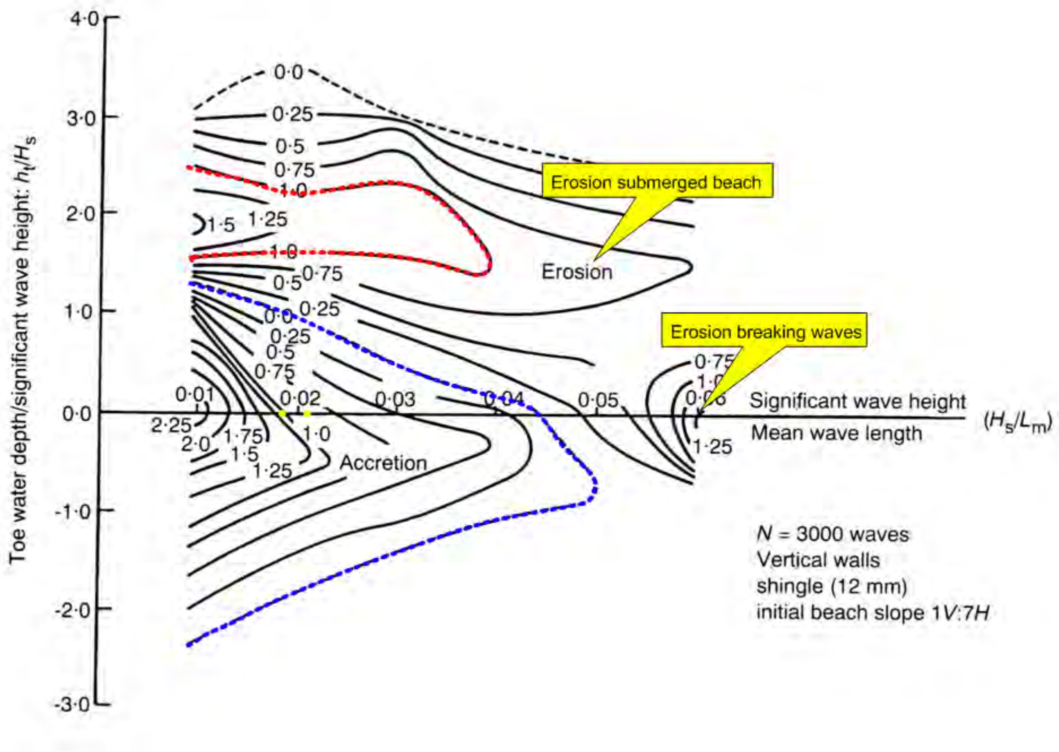


Figure B.3 Prediction diagram for contours of S/H_s scour (erosion) and accretion at vertical seawalls with shingle beaches (after Powell and Lowe 1994)

To select the worst possible scour, look at the dimensionless scour values for all h_t/H_s values below the maximum relative water depth corresponding to the wave steepness, H_s/L_m, and select the greatest relative scour height, which can exceed H_s. The plot gives the scour after 3,000 waves; a correction must be used to predict scour for time intervals other than 3,000 waves – see Section B.1.3.

B.1.5 Effect of a sloping or permeable seawall

The effect of a sloping wall on scour depths has been investigated by several authors, including:

- Powell (1990) noted that, for impermeable sloping structures of 1:1.5 to 1:2, there is no significant reduction in scour depth compared with that at a vertical wall. However, reducing the slope of an impermeable structure to 1:3 reduced the scour hole depth by 25 per cent to 50 per cent. Powell also noted that rock armour revetments generally showed less susceptibility to local scour and may even show accretion.

- Powell and Lowe (1994) showed a reduction in scour depth of almost 65 per cent in a shingle beach when a vertical wall was replaced by a sloping wall of 1:1.25. The scour depth was reduced by about 80 per cent for a 1:2 slope and there was accretion at the structure toe for a 1:3 slope. A rubble mound coastal defence showed no scour at its toe.

- Sumer and Fredsøe (2002, Figure 7.17) quantified the effect of wall slope in the non-breaking case (d/L >0.05) and showed that scour was reduced by about 80 per cent (60 per cent) for a wall slope of 30° (40°) above horizontal (compared with the scour from a vertical wall).

- Sutherland et al. (2006a) compared the maximum scour depths and the toe scour depth at a 1:2 (27° above horizontal) sloping impermeable wall to those at a vertical impermeable wall for four different offshore wave conditions and water depths with H_{si}/h_t = 0.5–1.0, where H_{si} is the incident significant wave height and h_t the toe water depth. The results are shown in Figure B.4 and show no systematic reduction in scour depth with wave height. In these cases, the downrush from the highest waves was reaching the seabed in some cases, which caused scour to occur.

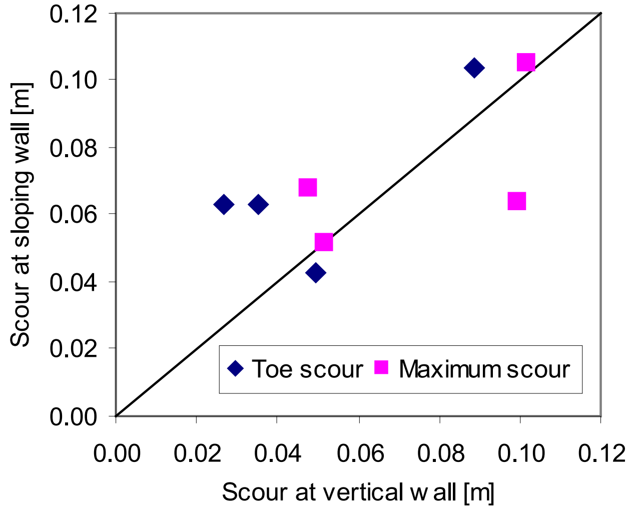


Figure B.4 Comparison of scour depths in sand at a 1:2 sloping wall and at a vertical wall for the same offshore wave conditions (Sutherland et al. 2006a)

In shallow water, the depth of scour is controlled by waves breaking on the wall and turbulence reaching the seabed. Under these circumstances the effect of reducing the seawall slope can be insignificant. It is only when water depths at the toe of the structure are sufficient to prevent turbulence reaching the seabed that a systematic reduction in scour depths with wave height can be expected. Moreover, for a sloping seawall, the antinodes will not occur exactly at the seawall, as there is a phase shift on reflection (Sutherland and O'Donoghue 1998a) so the position of deepest scour may change.

B.1.6 Effect of oblique waves

Following on from the discussion on the angle of wave approach in Section A.3.7, waves that are incident at an oblique angle to a structure will reflect from it causing a pattern of island crests and troughs that, for regular waves, will propagate along the front of a seawall. The steady streaming in the wave boundary layer may transport bedload along the front of the structure. If the coastal structure is in sufficiently shallow water that the waves are breaking, a longshore current will be generated which can also transport sediment that has been mobilised by the wave action along the face of the structure. Both circumstances may substantially increase the scour depth, particularly if a tidal current also flows along the face of the structure. Under these circumstances, a physical model or possibly a coastal area numerical model may be needed to assess the potential for scour.

B.2 Methods for predicting liquefaction

An analytical solution for the wave-induced pore pressure response in an isotropic infinite thickness seabed in front of a breakwater has been developed by Sutherland et al. (2007) based on the approach of Jeng (1998). It has been used to study the liquefaction potential of the seabed in front of coastal defence structures subjected to various wave loadings. The results can be used to indicate whether liquefaction of the seabed in front of a coastal structure is likely to occur. If the results are considered significant to the structure, a more detailed study should be carried out.

The liquefaction potential was determined by calculating the minimum total wave height to depth ratio that will cause the momentary liquefaction of the top 0.05 m of a sandy seabed in front of a vertical seawall. The analytical solution for an infinite and homogenous seabed was implemented into a Mathcad calculation sheet to determine the wave heights required to cause liquefaction to the soil. Details of the assumptions used to derive the analytical solution, the developed equations and coefficients are given in Appendix A of HR Wallingford (2006c).

The effects of wave height (H) and degree of saturation of the seabed (S_r) on the occurrence of liquefaction to a fine sand bed in a water depth of 5 m is shown in Figure B.5 for a typical storm wave period of 8 s. The degree of saturation of the seabed ranges between 0.9 and 1.0. 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 90–100 per cent. 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 per cent to 100 per cent for the top 0.5 m of the sand bed.

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 1.3d (d = 2 m, see Figure B.5).

Table B.1 shows the minimum fully reflected wave height required to liquefy the seabed to a depth of 0.05 m for different water depths and with degrees of saturation of 90, 95 and 100 per cent. For the unsaturated fine sand seabed with a sea depth of 2 m, the seabed could liquefy with a wave height as small as 0.4 m. For the deep water case (d = 15 m), the unsaturated fine sand seabed could liquefy under the wave condition with a wave height of 0.9 m. No wave-induced momentary liquefaction can possibly occur in a fully saturated seabed in shallow water ($d \leq 5$ m).

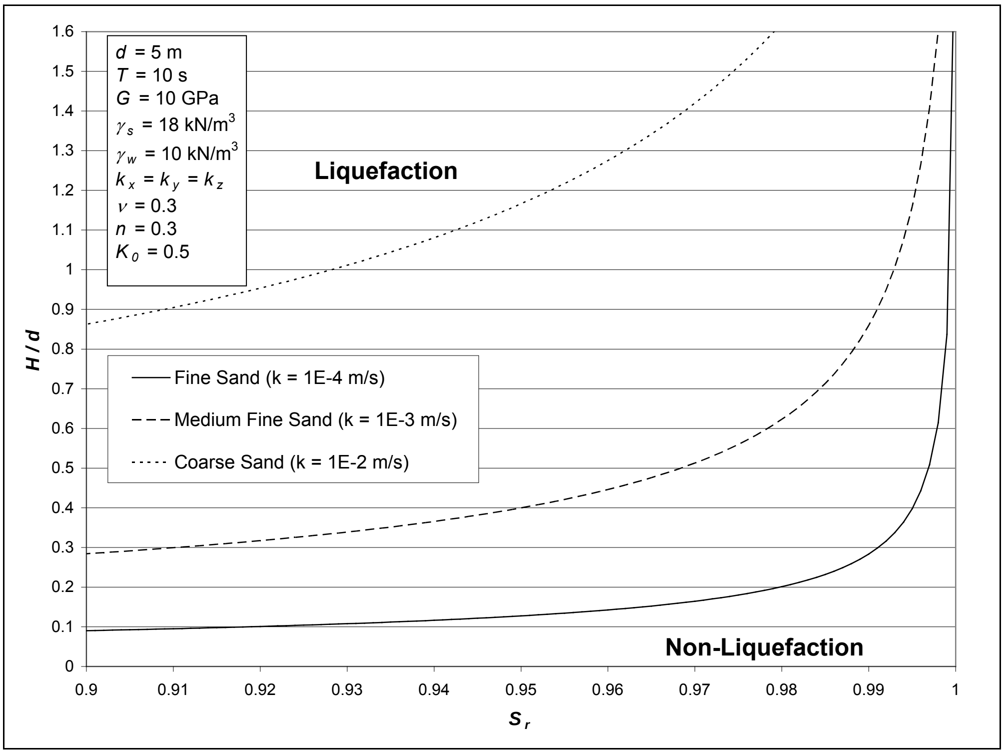


Figure B.5 Wave-induced liquefaction potential around a marine structure founded on a saturated/unsaturated seabed subjected to various standing wave loadings with a water depth of 5 m (Sutherland et al. 2007)

Table B.1 Minimum wave height required to cause the occurrence of liquefaction to seabed

Sand type	Wave water depth (m)			
	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

B.2.1 Simplified approach for assessment of liquefaction

Figure B.6 presents the minimum height of standing wave required to induce momentary liquefaction to a depth of 0.05 m in a seabed with an S_r 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 per cent (Mory et al. 2007).

To assess liquefaction potential with Figure B.6, first select the water depth, *d*, and then determine the combined wave height, *H*, from the incident wave height, H_i (that is, $H = 2H_i$). If the value of *H* is greater than 1.6*d* 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.

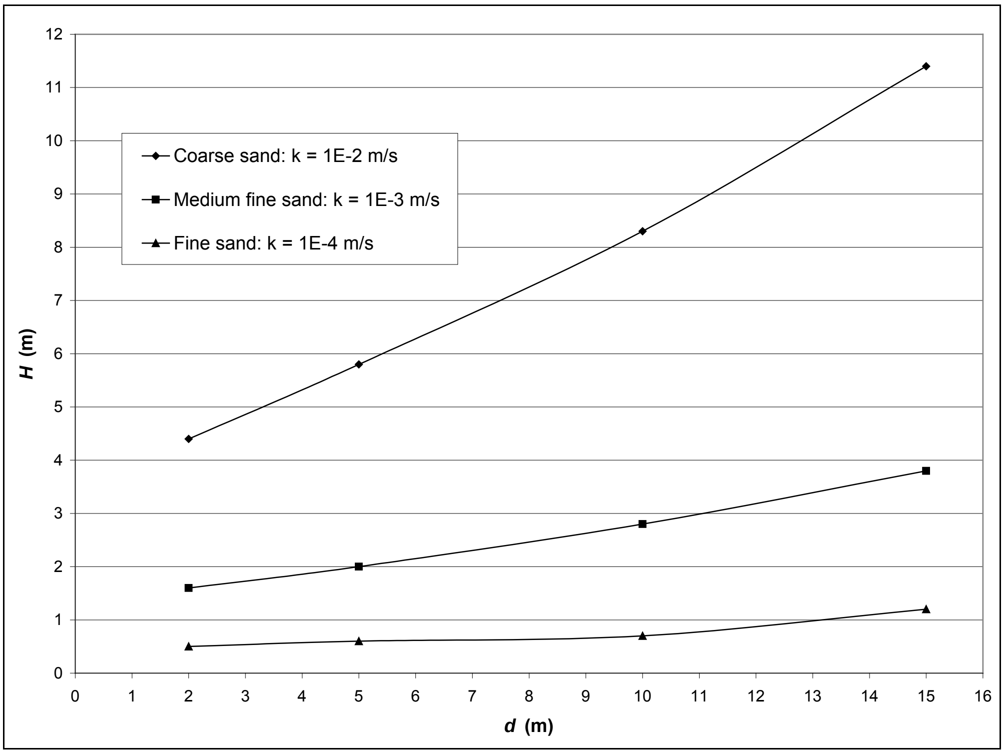


Figure B.6 Wave heights required to liquefy three different types of sand bed with given permeability, *k*, and with degree of saturation of 0.95, at various water depths (Sutherland et al. 2007)

B.2.2 Guidelines and recommendations on liquefaction

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.
- In the absence of a site-specific study, an S_r value of 0.95 is recommended for the estimation of the minimum wave height required to liquefy the seabed.
- Figure B.6 can be used to provide a quick check on the potential for momentary liquefaction of the top 0.05 m 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).

B.3 Methods for assessing erosion of soft rock

Scour in sand beaches front of reflective coastal defences has been studied extensively in recent years (Sumer and Fredsøe 2002; Sutherland et al. 2006a, 2007). Beach lowering in cohesive foreshores has been studied and management guidance given by Royal Haskoning et al. (2007). Little or no attention has been paid to the possibility of scour in cohesive seabeds in front of reflective structures. The possibility of this is explored below. First, a way of establishing spatially varying pattern of wave kinematics in front of reflective coastal structures is discussed; this follows the approach discussed in Section A.3.3. The mechanisms for eroding soft and hard cohesive seabeds are then summarised.

B.3.1 Constant erosion above threshold shear stress

HR Wallingford has developed a simple method for predicting the erosion in a seabed in front of a reflecting structure. This takes the reflective wave kinematics method of Section A.3.3, driven by wave conditions, and calculates time series of the spatial distribution of bed shear stress in front of the structure. The erosion rate is set at a constant rate for every hour when the bed shear stress was greater than the threshold value.

Example time series of bed shear stress and scour depth at a point are shown in Figure B.7 for a soft clay bed and relatively long period waves. The shear stress distribution varies spatially in front of the structure due to variations in the velocity field. It follows that the erosion profile varies spatially as well, as shown in Figure B.8, where the time taken to reach scour depths of 0.5, 1.0 and 2.0 m is plotted against the distance from the structure toe.

The advantage of this approach is its simplicity and speed of operation. A significant disadvantage is that it is almost certainly unrealistic to assume a constant rate of erosion once the threshold value has been passed. An erosion formulation more like that given in Section B.5.3 is likely to be more realistic.

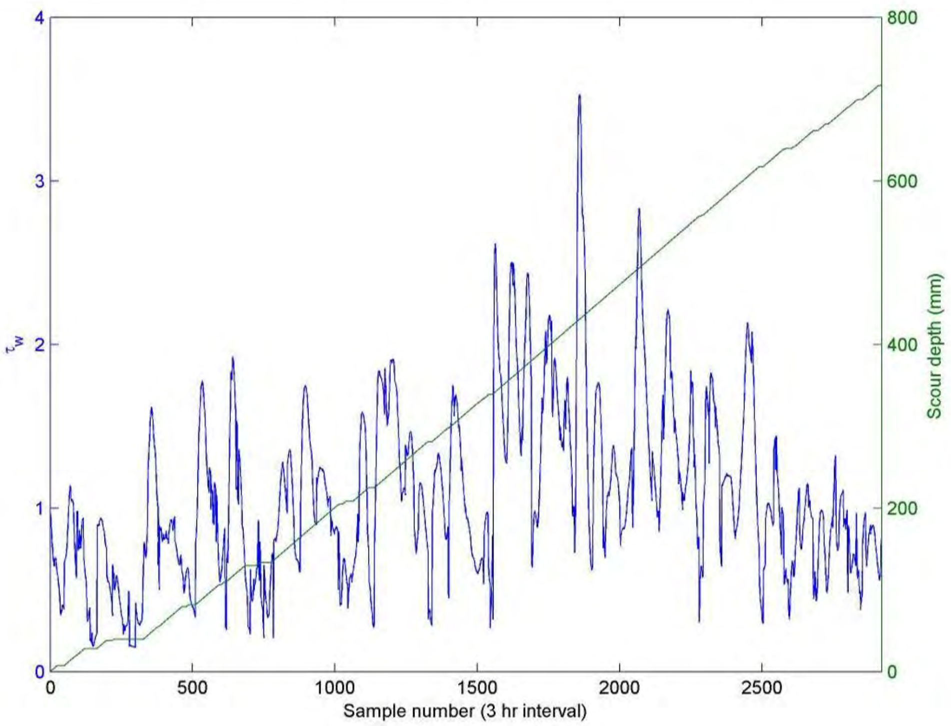


Figure B.7 Time series of bed shear stress and scour depth assuming a constant erosion rate above the threshold (Whitehouse, 2006)

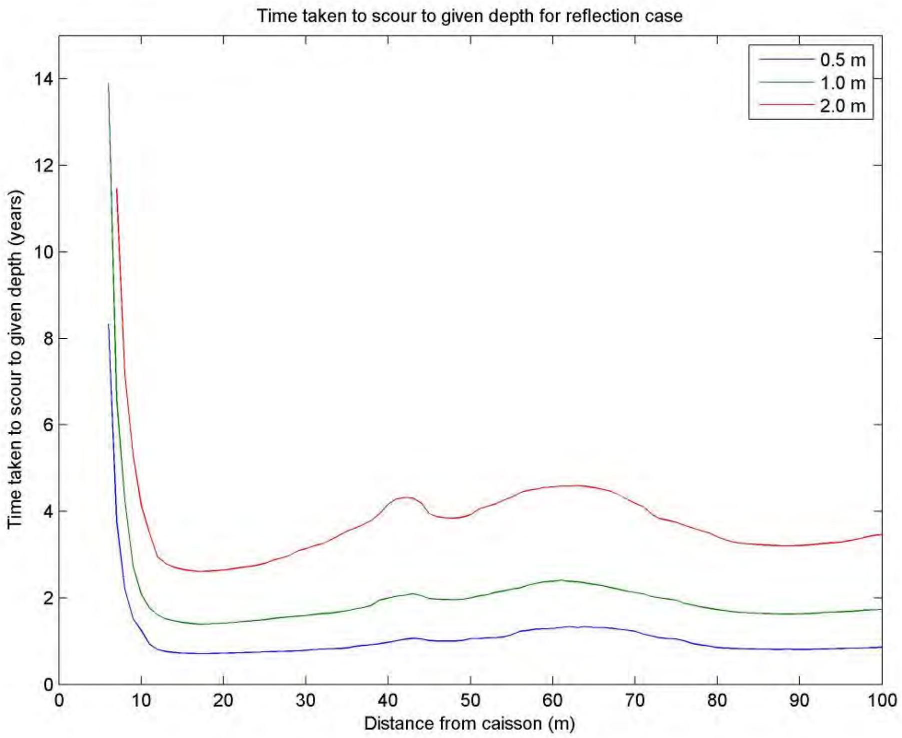


Figure B.8 Time taken to scour to given depth for reflection case

B.4 Erodibility index approach

Annandale (1995, 2006) proposed and presented a methodology that compares the stream power, P, to the ability of the soils to resist scour defined by an erodibility index, K.

Calculation of the scour depth in a multi-layer soil (seabed) consists of five steps:

1. Calculation of the erodibility index, K.
2. Calculation of the stream power required for erosion to occur, P_R, which is a function of the erodibility index.
3. Calculation of the available stream power at the undisturbed (zero scour depth) seabed, P_a.
4. Calculation of stream power at the base of each layer of the seabed, P_n, by applying a reduction factor to P_a based on the relative scour depth, S/S_{max}, where S is the depth of the base of each layer ($0 < S \leq S_{max}$) and S_{max} is the maximum scour depth (independently determined for non-cohesive fine sand – for example using Equation 3.1). The form of the reduction factor can be expressed as $P_n = a.P_a.exp(-b[S/S_{max}])$ where a is the amplification of the stream power at the seabed level caused by the presence of the structure, b controls the rate of decay of stream power with depth and both a and b have been fitted to data. An alternative form, $P_n = P_a.exp(b[1-S/S_{max}])$ has been proposed but not tested, with b another fitted constant and $0 <= S/S_{max} <= 1$.
5. Comparison between P_n and P_R for all layers down to S_{max}. Scour can occur for all layers where $P_n \geq P_R$.

In practice, if $P_a > P_R$ (the stream power required for the erosion of the top layer of the seabed) then the entire seabed will erode. In this case we assume that the base of the scour hole will fall with the eroding seabed. The results produced can be summarised for generic soil types as shown in Figure B.9. This shows the variation of relative scour depth on the *y*-axis with wave forcing quantified on the *x*-axis, the following plausible behaviour is observed:

- relative scour depth is less than the scour depth that would be experienced in non-cohesive fine sand;
- relative scour depth increases with wave forcing;
- relative scour depth decreases with increasing soil stiffness.

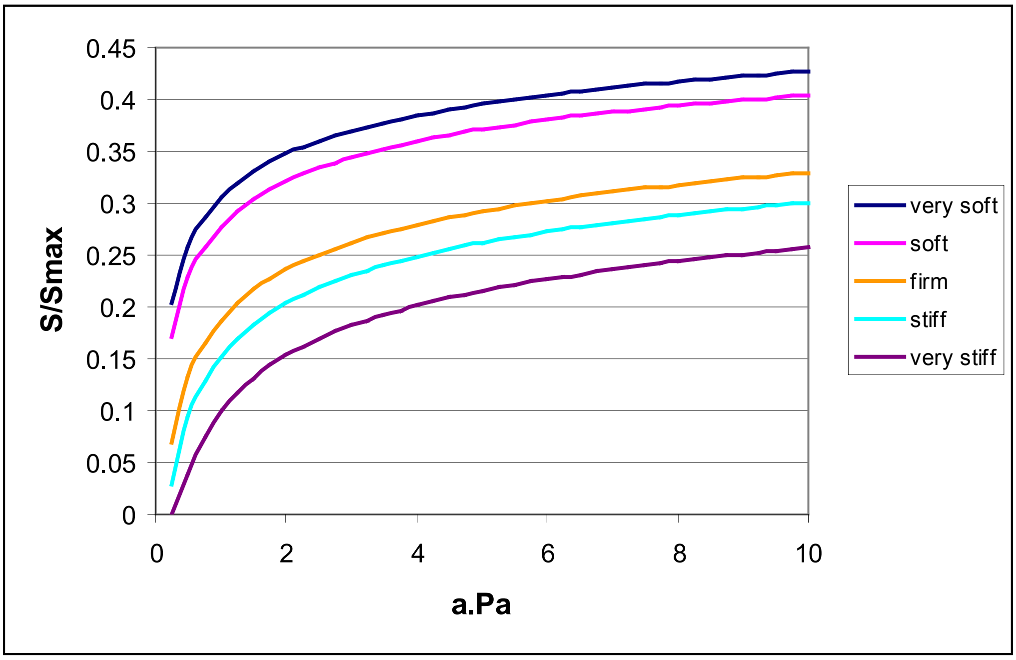


Figure B.9 Curves of reduction in scour depths in soft rock

This approach provides a good screening tool for evaluation of scour risk in cohesive soils and soft rock. The limiting factor in its application is the ready availability of standard geotechnical data for the soil at the toe of the structure. The case studies in Appendix C were not found to have comprehensive information on soil conditions. The application of this method requires careful consideration in the light of case study information where soil parameters are well known; this leads to the conclusion that scour risk evaluation for new design work or maintenance work would benefit from the addition of appropriate soil parameters such as vane shear strength and soil structure during site investigation. This method has been used for offshore marine structures (Harris et al. 2010)

B.5 Shore platform erosion models

A shore platform and coastal defence is similar to a shore platform and cliff system. Indeed many shore platform and defence systems used to be shore platform and cliff systems. Defences were added when cliff recession threatened valuable assets and, while these defences can halt the erosion of the subaerial cliff, they do not stop the erosion of the shore platform.

There is still considerable debate over the relative contributions of bed shear stress, abrasion and turbulence to the erosion of cohesive clay coasts. Thorough reviews of the mechanisms for the downwearing of shore platform and cliff systems have been provided by Nairn and Willis (2002), Walkden and Hall (2005) and Trenhaille (2009). Potentially useful methods include the ones outlined below.

B.5.1 COSMOS

The model of Nairn et al. (1986) related downcutting to wave-induced shear stress and to the intensity of wave breaking (which effects the turbulence and velocity fluctuations at the bed). Observations indicate that downcutting increases towards the shore and so cannot be due to shear stress alone.

B.5.2 SCAPE

The Soft Cliff and Platform Erosion (SCAPE) model of Walkden and Hall (2005) models the erosion of the shore platform based on the wave power and a vertical shape function. The erosion model is a variation of model developed by Kamphuis (1987) of the rate of erosion as:

$$E = \frac{F}{R} = \frac{H_b^{3.25} T^{1.5} \tan\alpha}{R} \qquad \text{(Eqn B.4)}$$

where:

H_b is the breaking wave height

T is the wave period

α is the average slope across the surf zone

R represents material strength and some hydrodynamic constants.

Walkden and Hall (2005) modified Kamphuis' approach to give:

$$\frac{dy}{dt} = \frac{F}{R} f_1(w - z) \tan\alpha = \frac{H_b^{3.25} T^{1.5}}{R} f_1(w - z) \tan\alpha \qquad \text{(Eqn B.5)}$$

where:

y is the retreat distance

F the erosive forces under random waves

f_1 is a shape function that describes the variation in F with elevation, z below the time-varying water level, w

α is the platform slope, which is a function of elevation.

The volume of material eroded is calculated by integrating the erosion over the vertical shape function at each time-step, then time-stepping through a tide. The shape function f_1 was determined by dividing the erosion rate of some physical model tests by the beach slope and interpolating.

An example application of SCAPE to the platform elevation change in front of a coastal structure is shown in Figure B.10. This figure demonstrates that (at least for this specific application, which is for a cliffed coast) continued erosion of the cliff leads to the release of sediment onto the foreshore which maintains the platform elevation above 1 m (line labelled

‘Natural coast’). Where the scenario of a seawall is introduced (line labelled ‘Engineered coast’) the platform is predicted to undergo a progressive reduction in elevation, in this case by 0.7 m over a period of 100 years. The average rate of 0.007 m/year is in the same ballpark as the observed rates in Table B.1. The model results were averaged between 10 m and 20m from the cliff toe or seawall toe, and not at the toe itself.

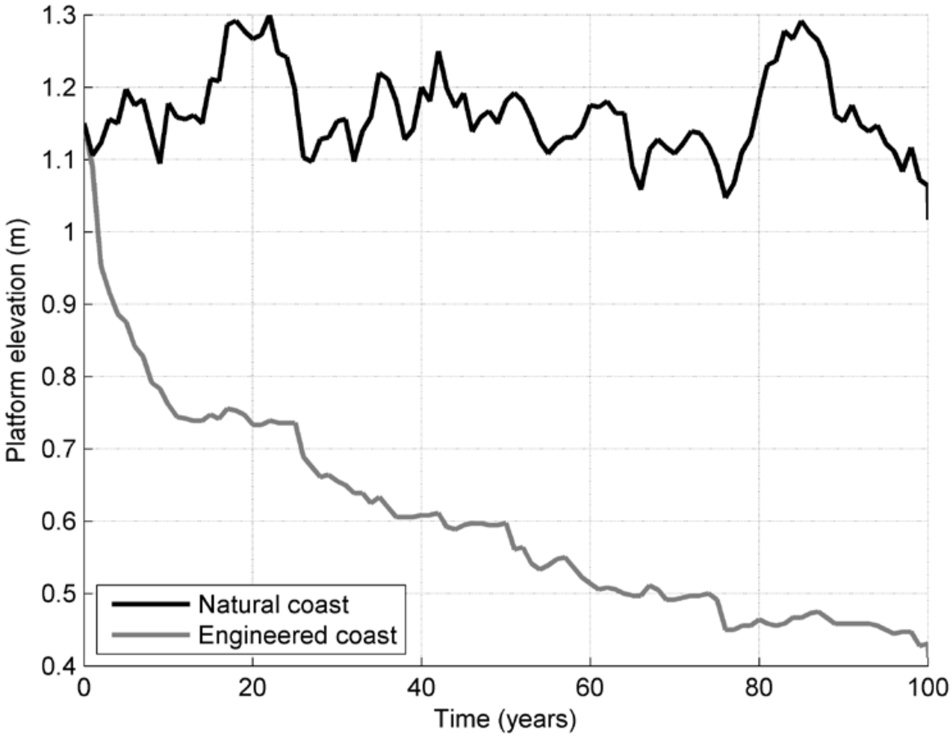


Figure B.10 Downcutting of shore platform predicted by SCAPE model (Walkden and Rossington 2009)

B.5.3 Trenhaille

Trenhaille (2009) presented a model of the erosion of soft rock coasts based on his experience of modelling hard rock coasts. He included erosion by three main mechanisms;

- Erosion of bare clay by excess shear strength
- Erosion by abrasion
- Erosion by wave impact.

These mechanisms are described below.

Erosion of bare clay by excess shear strength

Trenhaille (2009) relates the erosion of bare clay surfaces to the excess shear stress using:

$$E_{ss} = N_o K_{ss} (\tau - \tau_c)^p \qquad \text{(Eqn B.6)}$$

where:

E_{ss} is the erosion (ma^{-1}) by a single wave type at a single level

N_o is the number of waves of that type at each tidal level

K_{ss} is a calibration constant, the dimensions of which depend on the value of p (when p = 1, the dimensions of K_{ss} are $m^2kg^{-1}s$)

p is a calibration coefficient (assumed dimensionless) with typical values of 0.81 (Amos et al, 1992) or 1 (Zeman 1986)

τ is the bed shear stress (Nm^{-2})

τ_c is the critical bed shear stress (Nm^{-2}), which depends on the clay content and the shear strength

Trenhaille (2009) noted that for the Canadian Great Lakes, τ_c varies between 0.5 Nm^{-2} and 20 Nm^{-2} and performed model runs with 5 Nm^{-2} and 20 Nm^{-2}. The exponent p was set to 1 and K_{ss} was set to 2.4×10^{-7} $m^2kg^{-1}s$.

Erosion by abrasion

Clay surfaces with sediment on them can be abraded. The mechanisms for abrasion are not well understood – it is not even clear if abrasion is more effective under a thinner, more mobile layer or a thicker less mobile layer, although abrasion will stop when the layer is sufficiently thick to become immobile at the seabed.

Trenhaille (2009) relates abrasion to the ratio between the sediment thickness, ζ_t, to the maximum thickness of sediment that can be moved by a given wave, ζ_{tmax} as follows:

$$E_a = N_o K_a \frac{\zeta_{t\,max}}{\zeta_t} \qquad \text{(Eqn B.7)}$$

where:

N_o is as defined above

E_a is the abrasion (ma^{-1}) achieved by a single wave at a single tidal level

K_a is a coefficient to convert the sediment thickness ratio to abrasion.

A minimum value of ζ_t = 0.01 m is set to prevent excessive erosion as ζ_t tends to zero.

Trenhaille (2009) uses the equation of Sunamura and Kraus (1985) for the maximum thickness of sediment that can be moved by a given wave in the surf zone, namely:

$$\zeta_{t\,max} = 81.4 d_{50} \left(\theta_b - \theta_{cr}\right) \qquad \text{(Eqn B.8)}$$

where:

d_{50} is the median sediment grain size

θ_b is the Shields parameter at the breakpoint

θ_{cr} is the critical (threshold) Shields parameter, taken to be 0.04.

The conversion constant for abrasion, K_a was set to 1×10^{-6}. Note that the only relationship between the erosion rate by abrasion and wave conditions is through the Shields parameter at the breakpoint. As wave heights increase, it can be assumed that the Shields parameter at the breakpoint will increase, so the maximum thickness of sediment that can be moved will increase and so the abrasion rate will increase.

Equation B.8 is useful for exploring the role of sediment veneer thickness in protecting the shore platform. Typically under storm conditions, the Shields parameter in Equation B.8 can reach values of order one and probably more; this is much larger than the threshold value of 0.04. This means the sediment is very mobile and Equation B.8 predicts that the maximum thickness of sediment that can be moved by the wave is of the order 10–100 times d_{50}. For typical values of d_{50} on beaches of fine sand (0.1 mm) and coarse sand (1 mm), this indicates a depth of movement of 1–10 mm for the fine sand and 10–100 mm for the coarse sand.

By way of comparison we refer to the data collected on beach veneer variability by Royal Haskoning et al. (2007) at Warden Point in Kent. In July 2005 the beach was found to be very thin and formed from a mixture of sediment; overall it comprised approximately a 100 mm thickness of sand sized sediment, with shell fragments and pebble sized material. A follow up survey in February 2006 showed that the veneer had been largely removed and the platform was exposed between individual pebbles. While there was no measurable change in platform elevation in this period, it does confirm the thickness of beach material that can protect the platform and which can also be removed leading to exposure of the platform surface.

Erosion by wave impact

Mechanical erosion of clay by wave impact was calculated by Trenhaille (2009) as:

$$E_{bf} = N_o K_{bf} \left(S_F - S_{Fcr} \right) \qquad \text{(Eqn B.9)}$$

where:

E_{bf} is the recession (ma^{-1}) from a single wave and water level condition

K_{bf} is a wave erosion calibration coefficient

S_F is the stress exerted by the surf (kgm^{-2}).

The surf stress is given by:

$$S_F = \left(0.5\gamma \frac{H_b}{0.78} e^{-\chi S_w} \right) \sin^2 \varphi \qquad \text{(Eqn B.10)}$$

where:

γ is the specific weight of water (kgm^{-3})

H_b is the wave height at breaking (m)

χ is a dimensionless surf attenuation coefficient (set to 0.01)

S_w is the width of the surf zone (m)

φ is the local beach slope.

The term $\sin^2\varphi$ reduces the surf stress for lower slopes, with φ limited to a maximum of 50°. Initial calibration runs indicate a threshold for excess surf stress of between S_{Fcr} = 50 kg m^{-2} and 500 kgm^{-2} depending on the resistance of the material. A conversion factor of $K_{bf} = 1 \times 10^6$ was found to give suitable erosion rates.

This model is expected to be able to give similar outputs to the time series graphs shown in Figure B.10.

B.6 Methods for predicting beach levels over timescales of a tide to seasons

The variations in beach levels near coastal structures at timescales of the order of the tide to a year are the accumulation of the residual changes that occur during each tide. These changes can occur at a range of timescales, as shown by Figure A.2, of beach levels at the toe of a seawall at Mablethorpe (Lincolnshire). For example, 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 timing of beach surveys.

This section is concerned with the analysis of beach levels close to the toe of a structure at seasonal and shorter timescales. The prediction of beach levels on these timescales is important as they provide the initial conditions for the calculation of toe scour during a tide or storm.

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). 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.

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.

Until it happens, however, a more common approach to assessing the variability of beach levels at these timescales is through the analysis of beach monitoring data. Box B.2 demonstrates how an inter-annual and seasonal trend may be fitted to time series of beach level data collected at a point in front of a coastal structure.

Box B.2 Mablethorpe – investigating long-term beach trends and residual levels

The best-fit line of the form given in the equation below was fitted to time series of measured beach levels in front of coastal structures at seven locations in Lincolnshire stations (HR Wallingford 2006b, Section 3.1.5) including the time series from Mablethorpe shown in the figure below.

$$Z(T) = a - bT + c\sin(2\pi/T) + d\cos(2\pi/T) \qquad \text{(Eqn B.11)}$$

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 B.11 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 (22 mm) and peaked in October. The average profile had an amplitude of 110 mm and peaked in September, with its lowest value coming in March.

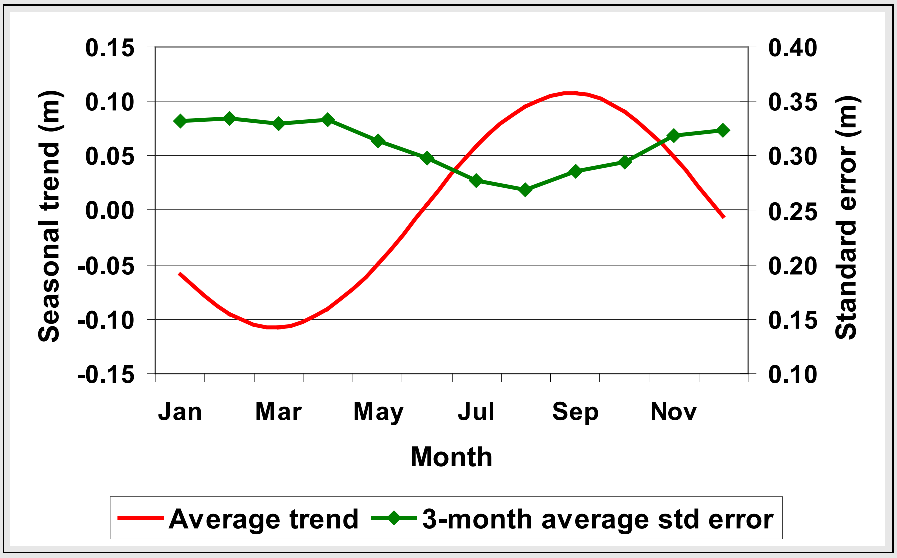


Figure B.11 A best-fit seasonal trend from Lincolnshire stations

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.

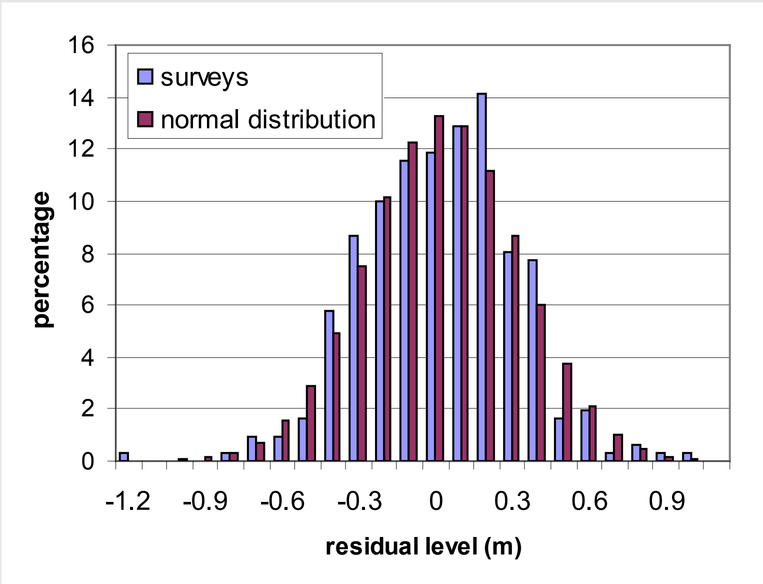


Figure B.12 Measured and Gaussian distribution of residual beach levels at Mablethorpe convalescent home

Residual beach levels are obtained when the long-term trend is removed from a time series of beach levels. The seven 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 2006b, Section 3.1.3). The plot for Mablethorpe is shown in Figure B.12.

B.7 Methods for predicting coastal erosion

The first places to look for an indication of whether there is a long-term problem of coastal retreat at a location are as follows:

(a) Local Shoreline Management Plan (SMP). If this is from the second round of SMPs, it should contain predicted changes for three epochs: 0–20 years, 20–50 years and 50–100 years for no active intervention and with present management scenarios.

(b) FutureCoast[1] CD ROMs, which contain the analysis of historic Ordnance Survey map tidelines, as well as statements on coastal 'behaviour systems' and local scale 'shoreline response', which describe the future evolutionary tendency.

(c) Local strategy studies, which may have modelled the coastal evolution of a smaller stretch of coastline in more detail than the SMP.

(d) National Coastal Erosion Risk Mapping project.

(e) Long-term records of beach levels in front of a coastal defence.

If a long-term record of beach levels in front of a structure is available, such as the Environment Agency's biannual beach surveys carried out in Anglian Region for the last 10 years, then long-term trends in mean beach level in front of the structure and in intertidal beach volume should be calculated. If these values show a statistically significant decrease in mean beach level with time, existing trends should be projected forwards to identify when the structure may become vulnerable to the additional effect of local toe scour, should recent trends continue.

If there is a systemic problem of long-term coastal erosion at the location of a coastal defence, beach levels at the structure are almost certain to have a long-term trend towards lower values. This will have implications for the stability of coastal defences. 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 or position of the shoreline. Methods for doing this are discussed in the guidance for producing Shoreline Management Plans (Defra 2006b) and Sutherland et al. (2007, Chapter 3).

Changes in shoreline position can be related to beach level at the toe of a structure through knowledge of the beach slope. The SMP guidance (Defra 2006b) includes a comparison of the following methods for analysing shoreline interactions and responses:

- extrapolation of historical data (covered here in Sections B.7.2 and B.7.3);
- numerical modelling (covered here in Section B.7.6);
- geomorphic extrapolation (covered here in Section B.7.7);
- parametric equilibrium models (covered here in Section B.7.8).

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 determine the most likely position. To obtain more site-specific data or data for shorter periods than in (a)–(d) above, the following methods may be used.

[1] Developed by Halcrow (2000–2002) on behalf of Defra and the National Assembly for Wales.

B.7.1 Shorelines

Ordnance Survey (OS) maps have shown tidelines since the introduction of the first OS one inch to the mile maps in 1801. OS maps therefore provide up to about 200 years of shoreline positions collected at different known epochs (although the practice of recording the date, or at least the year, of a survey of tidelines has ended with the move to digital mapping, as the date of survey is not an attribute stored with the resulting tideline in OS digital maps).

Previous maps tend to be less reliable but can still be useful for indicating the form of the geomorphology. The shoreline position is mapped more accurately on larger scale maps, however, so historical trend analysis (for example, Whitehouse et al. 2009) often starts with the first County Series of OS maps published between 1843 and 1893 at scales of 1:2,500 and 1:10,560. The County Series was also the first set of maps to include high and low water marks of ordinary tides (HWMOT and LWMOT) – earlier maps had included high and low water marks of spring tides. Subsequent map series have continued to use HWMOT and LWMOT, so the use of County Series maps onwards ensures a consistency in the definitions of tidelines used to analyse shoreline change.

The representation of tidelines in OS maps is discussed in some detail in HR Wallingford (2006d), which contains an error analysis that can be used to assess the reliability of the trends identified (see also Sutherland et al. 2007). This error analysis is summarised in Box B.3.

Aerial photographs have been used by Ordnance Survey and in some SMPs to illustrate geomorphologic features and how they change with time. Beach profiles can also be obtained from photogrammetry, as can a detailed topographic map. They are not, however, maps and offsets may be apparent between overlapping images which can necessitate the use of automated software to correct the distortion (Moore 2000; Leatherman 2003). Georeferenced orthorectified aerial photographs can be incorporated within a geographical information system (GIS) to provide the basis for displaying features. Overlaying photographs from different periods allows the changes in identifiable features to be plotted. It is also possible to use satellite photographs for the same purpose, particularly since the launch of more accurate satellite photographic services such as IKONOS in 1999 and Quickbird in 2001.

Care should be taken when combining shoreline positions from different sources as some may be proxy-based (that is, measure a discernible feature on the beach) while others may be based on a vertical datum (that is, measure the position of a fixed contour).

Box B.3 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 (HR Wallingford, 2006d, Section 3.5; Sutherland et al. 2007, Section 3.2).

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. A suitable root mean square source error (RMSSE) for a tideline is 3.3 m for the OS County Series and 2.8 m for National Grid maps, including the digital Mastermap series.

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. Four components of the interpretation uncertainty have been identified:

1. Truncation of levels in Admiralty Tide Tables to nearest 0.1 m.

2. Surveys can be taken when predicted high or low water is within ±0.3 m of the target level.

3. Surveys can be taken within ±0.5 hours of high tide.

4. The root mean square (RMS) vertical error in determining the instantaneous position of the tideline, which should have been surveyed in calm conditions.

The four errors are assumed to be independent, so are combined to give typical values of RMS interpretation error in level of 0.23 m for high tide and 0.29 m for low tide (although these values are likely to increase with tidal range). HR Wallingford (2006d) demonstrates how the calculations can be made for a specific site. The vertical RMS errors can be converted into a horizontal RMS interpretation error (RMSIE) using the beach slope.

Natural variability reflects the dynamic changes in the shape of the beach that occur in response to changes in waves and water levels. The root mean square variability error (RMSVE) for this figure should be obtained for each site by analysing beach profiles.

The sources of error are summarised below:

1. RMSS for 1:2500 scale mapping decreases from 3.3 m for County Series maps to 2.8 m for National grid maps. Mastermap mapping is taken to have the same error as National Grid mapping.

2. 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. RMSVE can be determined from beach profiles. As an example, in Lincolnshire between 1959 and 1991, the RMSVE at MHW varied between 0 m and 8 m, while that at MLW varied between 10 m and 23 m. 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 this equation:

$$RMSTE = \sqrt{RMSSE^2 + RMSIE^2 + RMSVE^2} \qquad \text{(Eqn B.12)}$$

The range of expected values will then be about four times the RMS total error (at 95 per cent 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 6–10 m.
- MLW on a National Grid map with a 1:30 slope would have a RMS total error of 14–24 m.
- MLW on a National Grid map with a 1:100 slope would have a RMS total error of 31–37 m.

So, for example, two surveys of MLW (if on a 1:100 slope) could be up to 150 m 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 MLW 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.

B.7.2 Extrapolation of historical data

The prediction of future shoreline positions or beach elevations at the toe of a coastal defence by the extrapolation of a historical trend is one of the most common methods in use today. The main components of the method are:

1. Collect historical data from different times.
2. Determine the coefficients of a best fit trend of shoreline position or elevation against time.
3. Extrapolate the best fit line into the future.

Methods for determining the coefficients of a best-fit line are discussed next, followed by examples of extrapolation.

B.7.3 Determination of the best-fit trend

The most common form of historical trend analysis involves fitting a simple linear trend to 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 jack-knifing, 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. (2007) recommended that weighted methods should be used if uncertainties are understood, but not otherwise. The ordinary least squares, re-weighted least squares, jack-knifing 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.

There are a number of advanced linear and nonlinear data analysis methods that can be used to analyse the long-term prediction of beaches. The linear methods include using correlation, Fourier series, random sine functions, wavelets, empirical orthogonal functions, canonical correlations and principle oscillation patterns (Larson et al. 2003; HR Wallingford 2006e, Section 3.2). Non-linear analysis methods include singular spectrum analysis, multi-channel singular spectrum analysis and fractals (HR Wallingford 2006e, Section 3.3). The more advanced data analysis methods rely on having a large quantity of regularly-sampled, accurate data. The use of such analyses will become more common as the amount of data collected by organised regional coastal observatories increases, but in the meantime they are mainly research tools.

Beach level time series data can be statistically analysed to give an indication of the rate of change of **elevation** and hence of erosion or accretion. Measured rates of change are often used to predict future beach levels by assuming that the best-fit rate from one period will continue into the future. The historical trend is then extrapolated to give predictions of future beach levels, which can be used by a coastal engineer to predict when a trigger/alert or damage level may be reached. Alternatively or in addition, long-term shoreline rates of change can be determined using statistical analysis of **cross-shore** position versus time data. Box B.4 outlines a range of methods that can be used to undertake a standard linear analysis of beach level data.

Box B.4 Linear analysis of 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 B.13.

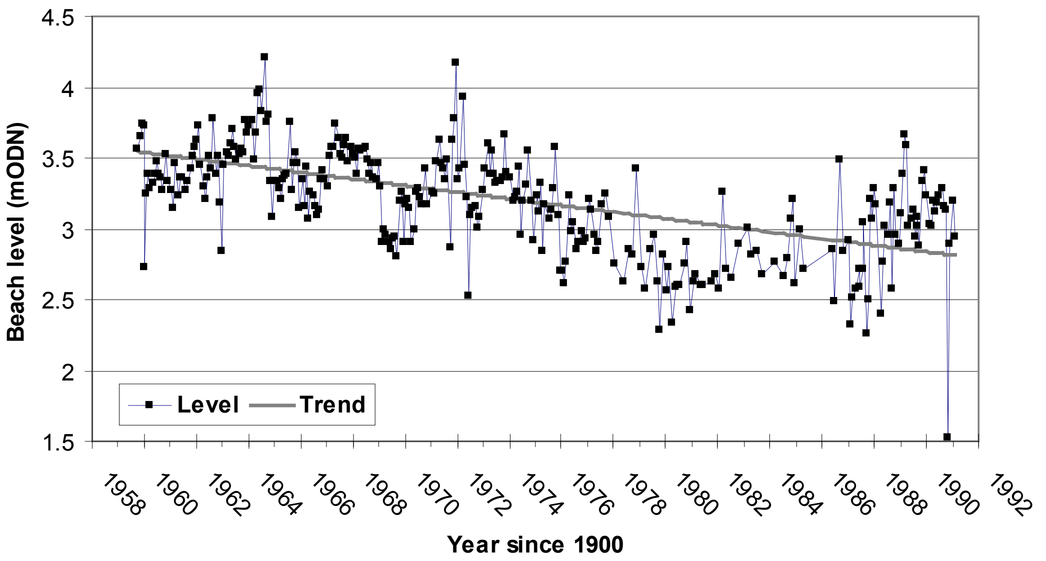


Figure B.13 An example of time series beach levels

A linear trend fitted to a time series of beach levels gives 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.

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 in which the data was collected.

B.7.4 Extrapolation of trend to future dates

Once a trend of position against time has been established, the equation and its fitted coefficients can be used to extrapolate the trend beyond the date of the last data point and into the future. Any such extrapolation depends on future conditions being similar to past conditions. The results of an extrapolation must be interpreted in light of the underlying principles of geomorphology and sediment transport (Whitehouse et.al. 2009).

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. The use of confidence limits and their limitations are illustrated in Box B.5.

Box B.5 Example of the extrapolation of beach survey data with confidence limits

The use of an extrapolated trend to hindcast beach levels is illustrated 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 per cent confidence limits were calculated on the assumption of a Gaussian distribution of residual beach levels (see Box 3.2). Figure B.14 shows that only three out of the 92 measured beach levels between 1970 and 1980 fell outside the 95 per cent 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 per cent confidence limits.

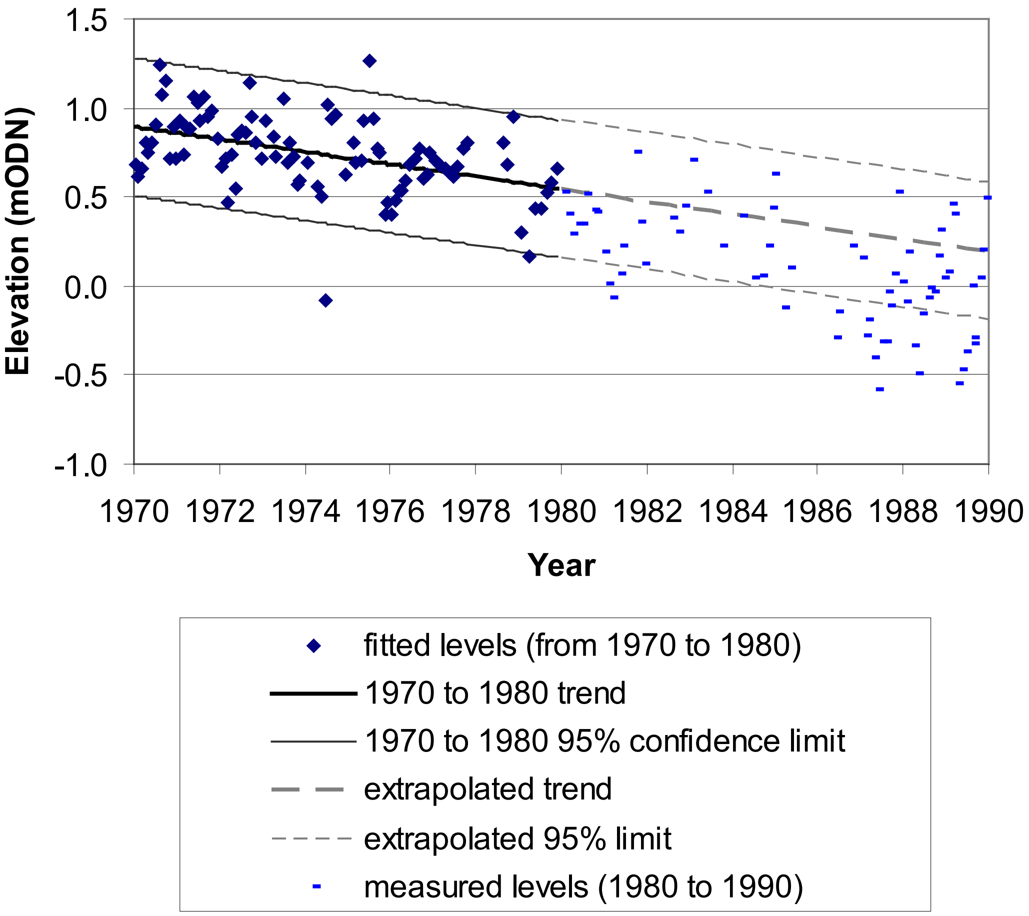


Figure B.14 Linear trend in beach levels at Boygrift outfall from 1970 to 1980 and extrapolated trend from 1980 to 1990 plotted with measured beach levels

and 95 per cent confidence limits

The usefulness of an extrapolated best-fit trend in beach levels as a predictor for future beach levels has been examined using 30-year long datasets of beach levels at the toe of the seawalls from four locations in Lincolnshire (Sutherland et al. 2007). The data were divided into 10-year long sections starting from 1960. At each location, a least-squares best-fit straight line was fitted to each 10-year section and the rates of change in elevation are shown in Table B.2. For a 10-year trend to be useful as the basis for predicting beach levels over the following 10 years, the rates of change from successive decades should be similar and should ideally be close to the 30-year average rate of beach level change, which is also given. Generally, in Table B.2 they are not. Only one of the 10-year rates of change is within ±100 per cent of the previous one and only three are within ±200 per cent (out of eight combinations). Only five of the 12 decadal rates were within ±100 per cent of the 30-year rate.

Table B.2 Rates of change in elevation in front of seawalls for different periods

Period	Rate of change (m/year)			
	Convalescent Home	Bohemia Point	Boygrift Outfall	Chapel Point
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

In this example, the 10-year rates of change in beach level provided little predictive capability for estimating the change in elevation for the following 10-years, let alone for the planning horizon that might need to be considered for a coastal engineering scheme. However, they may still provide a useful prediction over a shorter time interval. In order to determine how far ahead a trend can be extrapolated and still provide a useful prediction of future beach levels, its prediction horizon can be calculated.

The prediction horizon is the length of time over which a predictive technique produces on average a better prediction of future beach levels than a simple baseline prediction. The quality of a prediction is determined using a skill score (Sutherland et al 2004), which is a non-dimensional measure of the accuracy of the prediction relative to the accuracy of a baseline prediction of future beach levels. The most commonly used skill score in morphodynamics modelling is the Brier skill score (Sutherland et al. 2004, 2007) described in Box 3.6. A common baseline prediction of future elevations is that they will not change.

Box B.6 Brier skill score

The Brier skill score (BSS) is a non-dimensional measure of accuracy of prediction compared to a simple 'baseline prediction' and is determined by:

$$BSS = 1 - \frac{\sum_{1}^{n}\left(Measured_i - \Pr edicted\right)^2}{\sum_{1}^{n}\left(Measured_i - Baseline\right)^2} \qquad \text{(Eqn B.13)}$$

A resultant BSS score of 1 is a 'perfect' prediction of the extrapolated data. A score of 0 means the prediction is the same as the baseline prediction. A score less than 0 indicates that the prediction is worse than the baseline prediction. 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 BSS has been used to calculate the skill of coastal profile and coastal area models (Sutherland et.al. 2004) by comparing measured and predicted bathymetries at one point in time, using the baseline assumption that the final bathymetry was the same as the initial bathymetry. It has also been used to compare measured and predicted time series of beach levels at a point in space (Sutherland et.al. 2007) where the BSS was calculated as a function of the duration of the prediction, then averaged in bins of equal duration.

B.7.5 Procedure to establish an average prediction horizon

The concept of the prediction horizon was derived from meteorology, where it is used to assess how far in advance weather forecasts can be made. It was applied to the prediction of future beach levels using a linear trend fitted to historic data by HR Wallingford (2006c, Section 3.1.2) where the Brier skill score at each point in time was calculated as a function of the duration of the prediction (see Box B.7). The skill scores were ordered by the duration of the prediction and sorted into bins of equal length of time. The BSS were averaged for each bin to give the mean skill score as a function of the duration of the prediction.

An example of this for beach level data at the toe of a seawall in Lincolnshire is shown in Figure B.15, where the Brier skill score is plotted against duration of prediction for seven locations. The prediction horizon is the duration at which the average BSS decreases to zero.

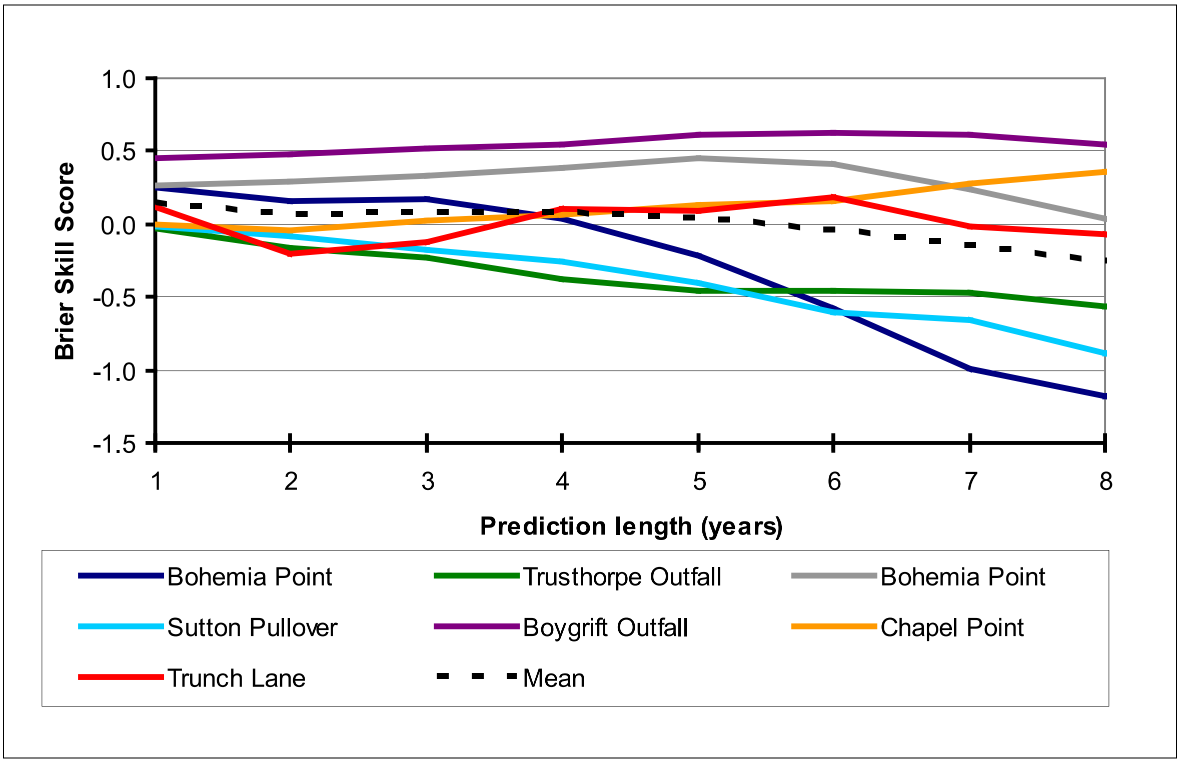


Figure B.15 Brier skill scores versus time for Lincolnshire profiles based on linear trends fitted to 10 years of data (Sutherland et.al. 2007)

The extrapolation of the best-fit trend in historical 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 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 B.7.4, 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, informing toe management and performing multiple interventions.

Box B.7 Determination of prediction horizon

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 (2006e, 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, one year (that is, all results with duration between 0 and 1 year, 1 and 2 years, and so on).

Calculate the Brier skill score for each bin, i, using the equation below.

$$\mathrm{BSS}(i) = 1 - \frac{\langle (x - y)^2 \rangle}{\langle (x - B)^2 \rangle} \qquad \text{(Eqn B.14)}$$

B.7.6 Numerical modelling

A considerable amount of research has been carried out over the last two decades 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 (2006e):

- one-line models for sand beaches;

- coastal profile models for sand beaches;
- coastal area models for sand beaches;
- systems model SCAPE for soft cliff and platform erosion (with a sand beach) – see Section B.5.2 for example application.

The approximate limitations and applicability of the types of existing numerical models are illustrated in Figure B.16. Coastal tract models are based on sediment budgets. Figure B.16 shows that the numerical models attempt to describe fewer and fewer processes in detail as the spatial and temporal scale over which they are deployed increases.

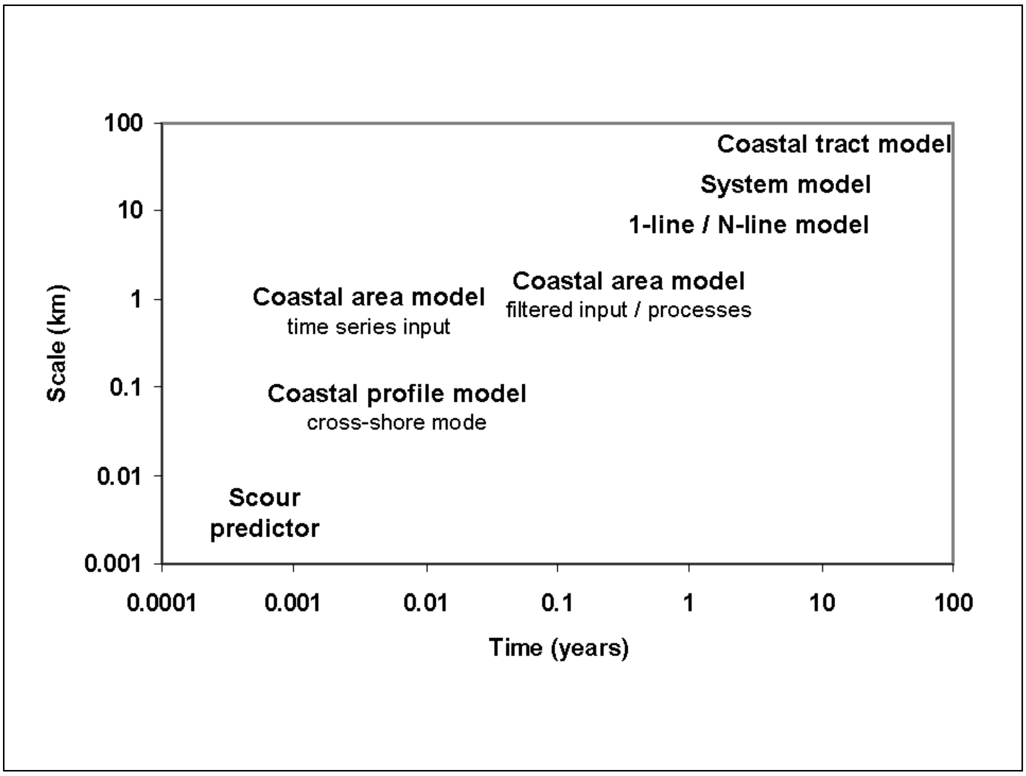


Figure B.16 Indication of spatial scale and length of prediction for different numerical model types

B.7.7 Geomorphic extrapolation

Geomorphic extrapolation is an expert-led, feature-focussed assessment of morphological behaviour and response, such as those provided for some scenarios by FutureCoast (Halcrow, 2002). Although a consistent methodology can be applied, this approach relies on expert judgement and so a range of outcomes is possible. Further information is available in Whitehouse et.al. (2009).

B.7.8 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.

The two main equilibrium beach concepts commonly used to predict coastal morphology are:

- the Bruun rule for coastal retreat (Bruun 1962);
- log-spiral coastlines (Silvester and Hsu 1997).

Bruun (1962) proposed Equation B.11 for the equilibrium shoreline retreat, *R*, of sandy coasts that will occur as a result of sea level rise, *S*.

$$R = S\frac{L}{h+B} \qquad \text{(Eqn B.15)}$$

where:

L is the cross-shore width of the active profile (that is, cross-shore distance from closure depth to furthest landward point of sediment transport)

h is the closure depth (maximum depth of sediment transport)

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, $S \times L$, 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). An example of how the Bruun rule can be used to calculate potential changes in shoreline retreat rates is given in Box B.8.

Box B.8 Relative shoreline retreat rates using Bruun rule

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 future rates of sea level rise, including the effects of isostatic change, can be obtained from Defra (2006b) while the present day rate is given by adjusting the global rate IPCC (2007) for regional isostatic changes (Defra 2006b). These were combined to give the shoreline retreat rate multipliers shown in Table B.3. The Bruun rule indicates that shoreline retreat rates could increase significantly – in some cases by a factor of 13 – during the 21st century.

Table B.3 Shoreline retreat rate multipliers for different time spans

Region	Shoreline retreat rate multipler				
	1961–2003	1990–2025	2025–2055	2055–2085	2095–2115
East and south-east England	1.0	1.5	3.3	4.6	5.8
South-west England and Wales	1.0	1.5	3.5	5.0	6.3
North-west and north-east England	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. However, both Cooper and Pilkey (2004a, 2004b) and Stive (2004) cautioned against its use due to its simplicity and restrictions. The Bruun rule is likely to be particularly inadequate in regions where there is a significant variability in the longshore transport rates (Dickson et al. 2007).

Log-spiral curves have been fitted to characterise the equilibrium plan-shape of a sandy beach between two hard control points with a dominant wave direction (Silvester and Hsu 1997). The control points may be headlands or beach control structures. If, in particular, new structures are planned, the equilibrium beach shape should be calculated to see how close it comes to any coastal defences at the back of the beach.

Appendix C: Case studies

Case study	Location	Structure type	Issue
C1	Ael-Y-Bryn, North Wales	Rock armour toe revetment	• Erosion at the toe of a cliff and slippage of cliff face • Reduction of beach volumes
C2	Corton, Suffolk	Rock armour toe protection and revetment	• Beach lowering leading to undermining • Structural failure
C3	South Beach, Lowestoft, Suffolk	Sheet pile wall and concrete thrust block/beam	• Beach lowering
C4	Holme Dunes, north Norfolk	Beach drainage	• Erosion of dunes due to a change in coastal processes
C5	Overstrand, north Norfolk	Sheet pile wall and stepped concrete apron	• Failure of apron due to corrosion, leading to cliff erosion
C6	West End, Dovercourt, Essex	Open stone asphalt and geotextile	• Shoreline retreat leading to potential erosion of embankment and release of landfill
C7	Teignmouth to Dawlish Railway, Devon	Concrete stepped toe beam	• Loss of beach material leading to undermining of structure
C8	Felixstowe, Suffolk	Sheet piling, beach recharge and control structures	• Failure of groynes, leading to rapid reduction in beach levels, resulting in undermining of structure
C9	Clayton Road, Selsey, West Sussex	Rock armour toe and geotextiles	• Beach lowering and toe scour, caused by wave reflections, leading to risk of structural failure

Case study	Location	Structure type	Issue
C10	Fort Wall, Canterbury, Kent	Encasement and rock armour toe	• Beach lowering leading to structural failure (overturning)
C11	Prestatyn, North Wales	Beach recharge, rock groynes and sloping apron	• Wave and tidal induced scour
C12	Colwyn Bay, North Wales	Rock berm and rock groynes	• Beach lowering, leading to heavy overtopping and foundation instability
C13	Rhos-on-Sea, North Wales	Rock armour breakwater	• Beach lowering
C14	Penrhyn Bay, North Wales	Beach recharge and fish-tail groynes	• Beach lowering and overtopping
C15	Sandbanks Peninsula, Poole, Dorset	Rock groynes and rock toe protection	• Beach lowering caused by groyne failure and complex tidal current regime
C16	Seaford, East Sussex	Beach recharge	• Beach lowering leading to undermining and structural failure
C17	Selsey Bill, West Sussex	Concrete armour units and a rock berm	• Beach lowering
C18	Sidmouth, Devon	Offshore breakwaters and a rock groyne	• Beach lowering leading to wave overtopping
C19	Portobello Beach, Edinburgh	Beach recharge and timber groynes	• Beach lowering leading to wave overtopping

NB: Case studies C11–19 were sourced directly from Appendix 1 of *Beach Lowering in Front of Coastal Structures – Research Scoping Study* (Sutherland et al. 2003) and are thus presented in a different format to case studies C1–10.

C1 CASE STUDY: Ael-Y-Bryn, North Wales

Courtesy of Conwy Council and Coastal Engineering UK Ltd

C1.1 Identification of the problem

The site is located just to the east of the Little Orme headland, Llandudno, North Wales (see Figures C1.1 and C1.2).

Figure C1.1 Site location

This case study concerns coastal protection to the small development known as Ael-y-Bryn. Reference is also made to the adjacent development of Craigside (Figure C1.2). Properties at Craigside were built much earlier than those over the shorter 170 m frontage of Ael-Y-Bryn.

The coast here consists of cliffs of about 15–20 m in height, fronted by a beach consisting of sand, shingle and rounded limestone boulders derived from earlier cliff erosion. The beach at the toe of the cliff face has a marginal depth of covering over the underlying clay that forms the cliff. Table C1.1 gives some data extracted from Coastal Engineering (2001).

Table C1.1 Relevant levels abstracted from Coastal Engineering (2001)

Frontage	TP	Top of wall level (mODN)	Beach level (mODN)	Base/clay level (mODN)	Beach material above clay horizon
Craigside	1	6.790	5.690	4.140	Coarse sand and shingle
Craigside	6	6.790	4.130	3.800	Shingle
Craigside	11	6.800	4.370	3.340	Shingle, small boulders
Ael-Y-Bryn	14	-	2.715	2.315	Sand, gravel and small boulders
Ael-Y-Bryn	15	-	2.775	2.175	Coarse sand, cobble and small boulders

The Craigside properties were protected from the erosive action of the sea by a vertical masonry retaining wall. Most of the Ael-Y-Bryn properties were built between 1974 and 1980, the six houses closest to the cliff edge being built subsequently after 1983. As a condition for building these later properties, the developer was required by the (then) local authority, Aberconwy Borough Council, to provide coast protection. These measures entailed regrading the cliff and constructing a revetment. According to local observers, the revetment was formed by pushing the indigenous boulders from lower down the foreshore up to the toe of the cliff (Figure C1.3). Larger boulders were set at the bottom of the mound, with smaller boulders at the top.

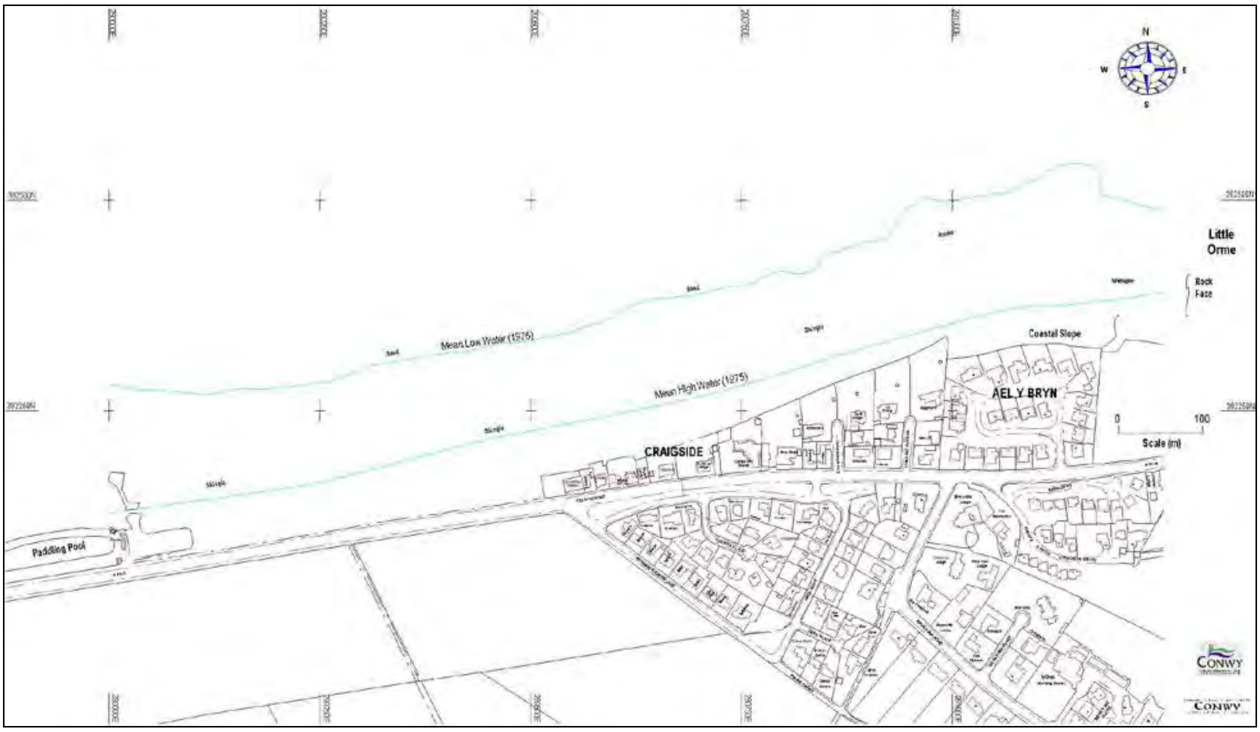


Figure C1.2 **Location plan showing Ael-Y-Bryn, Craigside and the paddling pool area**

Figure C1.3 Coast protection measures constructed as a condition of the development

Llandudno Bay (the bay between the Little Orme and the Great Orme) is shielded from some directions of wave attack, but exposed to waves from northwest through to northeast. The incident wave climate gives rise to both easterly and westerly migrations of beach sediments between the two headlands. Coastal Engineering (2001) describes how the bay is effectively a closed cell in terms of drift. In spite of the cellular nature of the bay, comparison of historic and recent plans showed there had been a depletion of sediment, and that the cliffs at Ael-Y-Bryn had been slowly eroding. Even after the installation of the developer's revetment at Ael-Y-Bryn, continuing erosion was evident, being manifest by way of erosion at the toe of the cliff and slippage of the cliff face.

Examined in the 1990s, records showed that there had been a significant reduction in the beach volume at Craigside. This was, at the time, incorrectly perceived to be due to coast protection measures to the west, including construction of a groyne. Subsequent monitoring demonstrated that other factors were at play; in particular, the shallow revetment, having a crest level of 5–6 mODN was readily overtopped by storm waves on high tides (1 in 1 year sea level was calculated to be 4.63 mODN). It is also possible that the alleged removal of boulders from the lower shore had reduced the capacity of the foreshore to attenuate wave energy, thus resulting in greater impact higher up the cliff face.

As part of the initiatives for this frontage, Coastal Engineering UK recommended an extension of a monitoring programme which had begun for other parts of the frontage in 1997. Monitoring identified no real problems with the vertical walled defences at Craigside. The wall was adequate and groundwater monitoring showed no variation with tide level. The principal concern remained with Ael-Y-Bryn and, in particular, how to protect the cliff.

C1.2 Appraisal

The Shoreline Management Plan (Sub-cell 11a, Great Orme's Head to Formby Point) published in 1999 advised a 'hold the line' policy for the whole of Llandudno Bay. The outcome of a review reported in the project appraisal, Coastal Engineering (2006), stated that the then existing policy for the specific Ael-Y-Bryn frontage was one of 'no active intervention'. However, the residents had been expressing concerns over the adequacy of the defences for over 15 years; moreover, the distance between the boundaries of the properties and the cliff edge was by now only 15–20 m.

Given the nature of the infrastructure at risk, the indicative standard of protection for the frontage was 0.5–2 per cent when expressed as annual probability. By comparison, the actual risk of serious erosion was calculated to have an annual probability of 20 per cent. The stated objective of the project appraisal was, therefore, 'to provide an appropriate level of coastal defence to cliff top properties at Ael-Y-Bryn, Llandudno, threatened by erosion'.

Following the breaching of defences at Towyn and elsewhere on the North Wales coast in 1990, a number of initiatives involving combinations of private and public sector funding were considered; this included a possible scheme to restore the standard of coast protection afforded to the cliff top houses at Craigside and Ael-Y-Bryn. The prospect of 50 per cent of the funding being provided by the residents, with the balance coming from the Welsh Assembly Government , enabled a financial means by which a coast protection scheme could be promoted.

In addition to the 'do nothing' option, as required to determine scheme benefits, three main options were examined:

- linear defence consisting of a re-armouring of the cliff face;
- linear defence consisting of a vertical wall, similar to that at Craigside;
- beach recharge and management.

Apart from being significantly the most costly option, beach recharge was rejected partly due to concerns over who would be responsible for its continued management. Of the two linear defence options, re-armouring of the revetment was the preferred one, yielding a benefit/cost ratio of around 3.

Consultation was carried out with each of the six residents benefitting from the project, and with the Countryside Council for Wales (CCW). Though initially hesitant in endorsing the use of rock, CCW agreed to the scheme, given that the locally occurring rounded boulders would be reused in the works.

C1.3 Outline design and consents

A ground investigation was carried out including trial pits (Figure C1.4) and boreholes. However, the latter proved problematic due to the percussion tool hitting boulders.

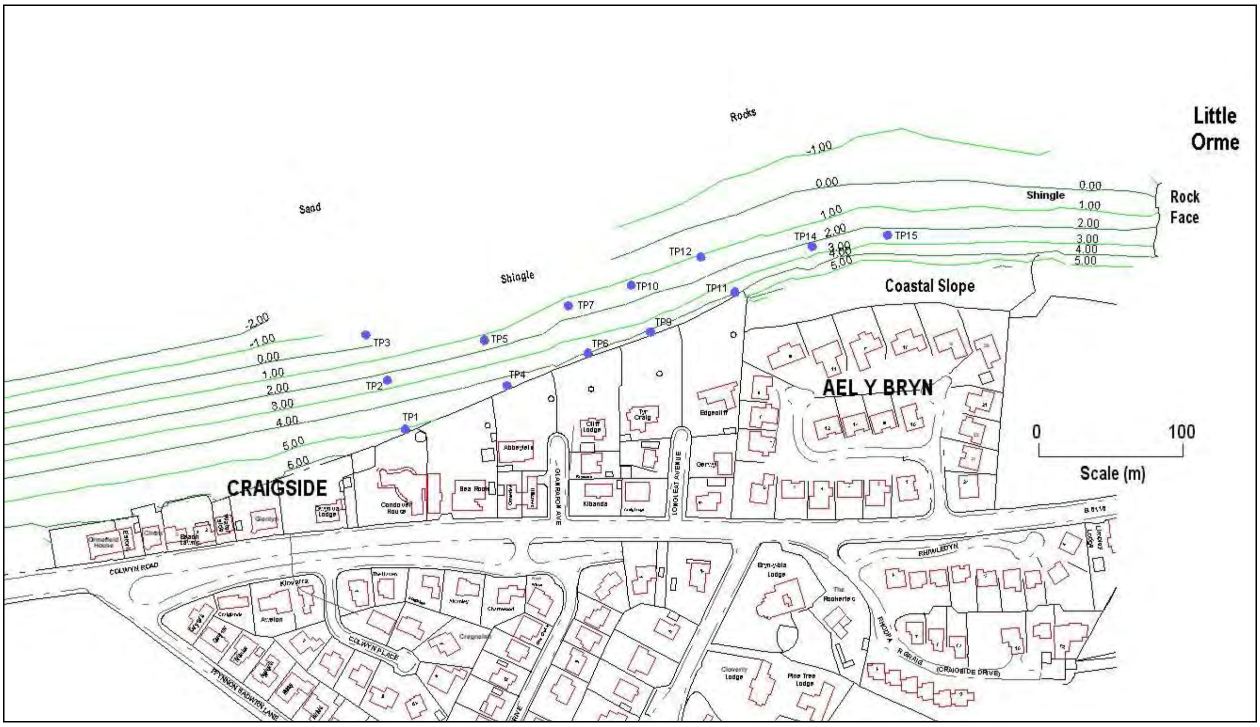


Figure C1.4 Position of foreshore trial pits

The capital works scheme would consist principally of cliff regrading and a rock revetment laid over geotextile. Subject to maintenance, the proposed scheme would provide a Standard of Protection against erosion damage of 1–4 per cent annual probability and it would have a scheme life of 50 years.

Consents were obtained as follows:

- planning approval (Town and Country Planning);
- approval by CCW in respect of works being carried out within a SSSI (the Little Orme had been designated as a SSSI noted for terrestrial features, which was extended in 2001 to the foreshore at Ael-Y-Bryn in respect of the boulders on the lower foreshore);

- from Conwy County Borough Council in respect of the Coast Protection Act 1949;
- approval from the then Department for Environment, Transport and the Regions (DETR) in respect of the Coast Protection Act, Section 34 (interaction with navigation);
- Food and Environmental Protection Act (FEPA) licence.

The last two consents were dealt with under a single application to Defra's Marine Consents Unit.

C1.4 Detailed design

The detailed design was prepared by the local authority. Figure C1.5 shows a typical cross-section which comprised:

- re-grading and reseeding of the cliff face;
- a crest of level of +6.8 mODN;
- a revetment consisting of 1–3 tonne rock rip-rap laid at a 1 in 2 slope on geotextile placed on the cliff face;
- a toe, buried into the beach to about 1 m depth or less if underlying clay was encountered.

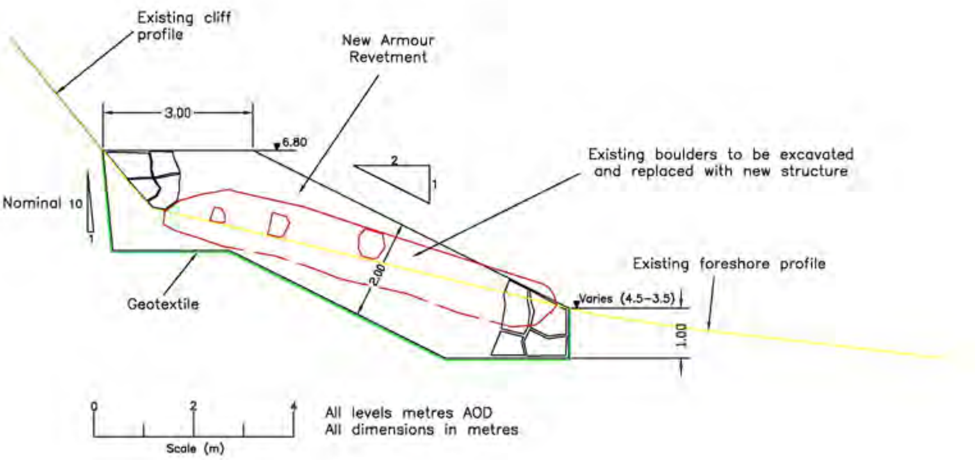


Figure C1.5 Typical cross-section

C1.5 Construction issues

The capital works scheme was procured through competitive tender. Further to the reuse of the local rounded boulders, rock was obtained from local quarries. This limestone had been used elsewhere where it was known to be performing well after 30 years' service. Given its position at the foot of the cliff, access to site needed careful consideration. To facilitate access, a temporary haul road was created from the paddling pool (see Figure C1.2), the latter area also being allocated for the contractor's compound. Figure C1.6 shows the regraded cliff face.

Figure C1.6 Regraded cliff face

The construction works were at a high elevation in the beach profile but still required shift working in line with the tidal window and weather conditions (see Figure C1.7).

Figure C1.7 Construction in progress

C1.6 Post construction Issues

The completed scheme is shown in Figures C1.8 and C1.9. Beach surveys and an inspection of the site are carried out twice a year; there is an annual report. No problems have been encountered with the toe of the revetment. Public perception of the scheme is good.

Figure C1.8 The completed scheme

Figure C1.9 Aerial view of the project (note Craigside vertical wall to the west of the revetment)

C1.7 Lessons learnt

Working with the residents at Ael-Y-Bryn yielded a satisfactory outcome. The residents made a significant contribution to the cost of the works, amounting to some £130,000 (roughly £10,000 per property), the remaining 50 per cent of the cost being met by the Welsh Assembly Government.

It is acknowledged that the crest level might need to be topped up in the future to sustain the standard of protection but the adaptive nature of the works allows for doing this. The toe is not expected to be a problem.

C1.8 Acknowledgements

We would like to acknowledge the invaluable advice and assistance provided by Dyfed Rowland of Conwy County Borough Council and Alan Williams of Coastal Engineering UK Ltd in the preparation of this case study.

C1.9 References

Coastal Engineering, 2001. *Craigside/Ael Y Bryn frontages Llandudno*, Coastal Engineering Consultancy Services for Conwy Council, September 2001.

Coastal Engineering, 2006. *Ael Y Bryn, Llandudno, proposed coast defence improvements: project appraisal report.* Coastal Engineering UK Ltd for Conwy County Borough Council, August 2006.

C2 CASE STUDY: Corton, Suffolk

Courtesy of Waveney District Council

C2.1 Identification of the problem

Corton village is located on the Suffolk coast, about five miles north of Lowestoft (Figure C2.1). Up to about 20 m in height, the soft coastal cliffs that back the narrow beach at Corton are rich with fossil remains and are of geological importance. Corton was the 19th century home of the Colman (mustard) family at a time when the local economy was based around rural activities; Corton has since become a popular seaside holiday resort.

Figure C2.1 Location of Corton

In 1870 the Colman family had concrete and timber coastal defences built to protect their property between the middle and the southern end of Corton. Relics from these early defences are still evident today (Figure C2.2).

The seawall built prior to the works described in this paper originated from 1960 and 1967 and consisted of a steel sheet piled toe behind which was a mass concrete berm and a raked concrete slab covering the lower cliff face (Figure C2.3). In addition to the new seawall, timber framed/steel sheet faced groynes were installed along the frontage.

Figure C2.2 **Detached concrete relics of 1870 Colman coast defences**

Figure C2.3 **Typical 1960–1967 toe and revetment structure (note date of photograph)**

In 1986, 50 metres of the 1960s seawall failed due to undermining. By this time, the groyne field was also in poor condition due to abrasion of the steel piles. The failed section of seawall was rebuilt together with two groynes; otherwise only routine maintenance was applied. However, the shoreline continued to retreat and further collapses of the seawall occurred in November 1999 and April 2000 (Figure C2.4). It appears that the beach level dropped below the toe of the sheet piles but that the metal ties restrained them close to the top, resulting in the piles kicking out at the toe (Figure C2.5). Where scour penetrated behind the piles, the whole structure collapsed with the concrete berm articulating over the piling.

Uncertainties as to the form and scale of the long-term solution led to a decision to carry out immediate holding measures, thus giving the necessary time to properly consider and implement more major reconstruction work. The holding measures, carried in the autumn 2000, consisted of a double layer of 3–6 tonne rock placed along the failed defence line. In spite of these measures, further collapses occurred at other part of the frontage in the 2000–2001 winter requiring further holding repairs in the autumn of 2001.

Figure C2.4 **Failure of the seawall in winter 2000–2001**

Figure C2.5 **Rotation of the toe piling (photograph taken after the 1999 failure)**

C2.2 Appraisal

The Lowestoft Ness Shoreline Management Plan 3B (1996) had advised a coastal defence policy of 'Hold the Line', although the economic case was reported to be marginal. The subsequent Strategy Study (Halcrow 1998, adopted 1999) had concluded that Hold the Line at Corton was not sustainable in the long term but might be viable in the short term. The Strategy Study did not, however, consider the intangible benefits of defence such as recreation and access to the beach.

The Flood Hazard Research Centre at Middlesex University was therefore commissioned to carry out a specialised study on intangible benefits – the Corton Village Study. Taking the findings of the Corton Village Study into account, the Project Appraisal Report prepared by Halcrow (completed 2002) examined a range of options based principally around two defence methods, rock revetment and beach recharge with groynes, in combination with variants on the level of intervention and period for holding the line. Beach management options were significantly more expensive than those using a rock revetment. The recommended preferred option was to Hold the Line to year 20 using a rock revetment, to be followed by management of an eroding coast; the scheme included beach access to be maintained for 20 years. The preferred option had a benefit/cost ratio of 1.11.

The PAR enjoyed a high level of consultation with locally elected members and the local community. The understanding and awareness brought about by this liaison resulted in the preferred option being generally accepted as a reasonable compromise between the competing objectives. The use of rock was not challenged during consultation.

Further important consultation took place with Natural England with particular reference to the geological SSSI. This consultation influenced the design of the rock revetment such as to allow sufficient wave overtopping to prevent vegetation of the lower cliff face where it was required to keep geological features exposed.

C2.3 Outline design

Figure C2.6 shows a schematic of the outline design.

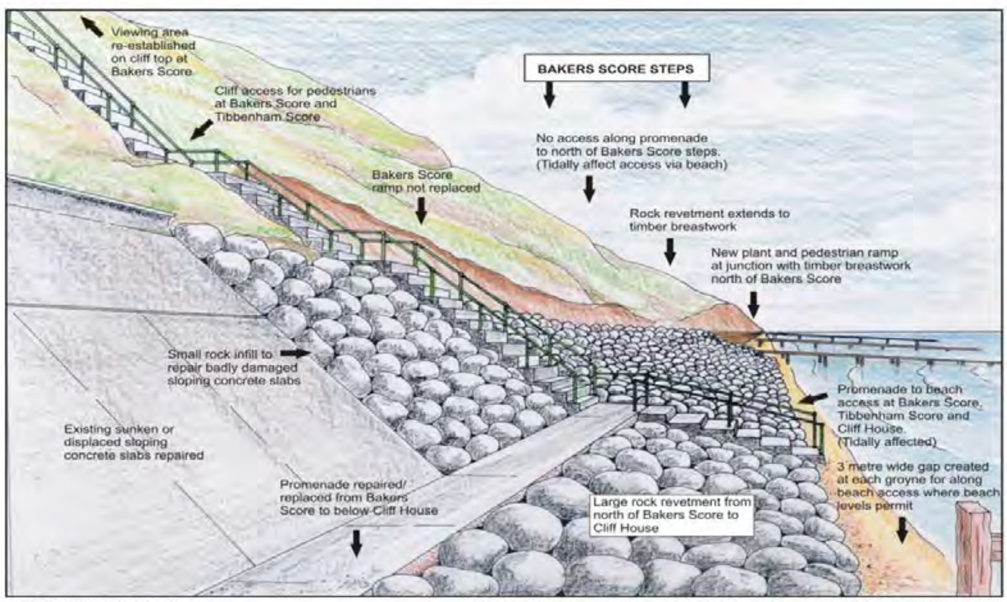


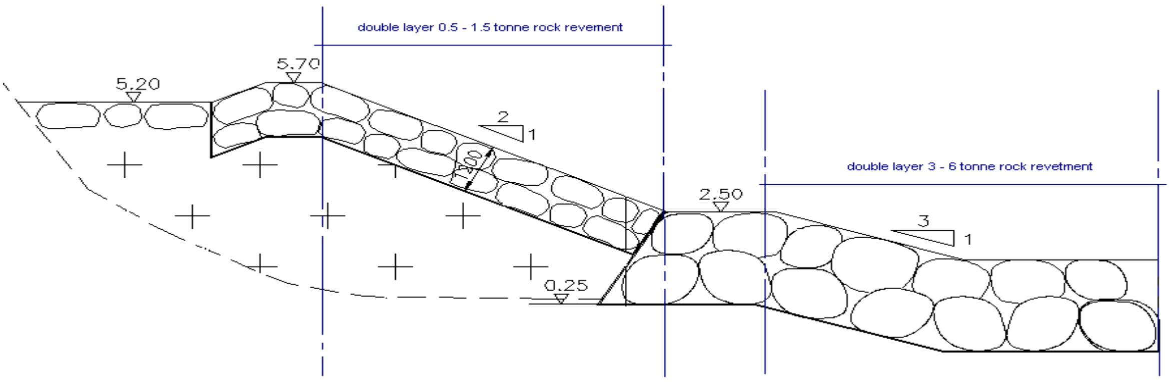
Figure 5: **PREFERRED SCHEME OPTION AT BAKERS SCORE**

Figure C2.6 **Schematic of the outline design**

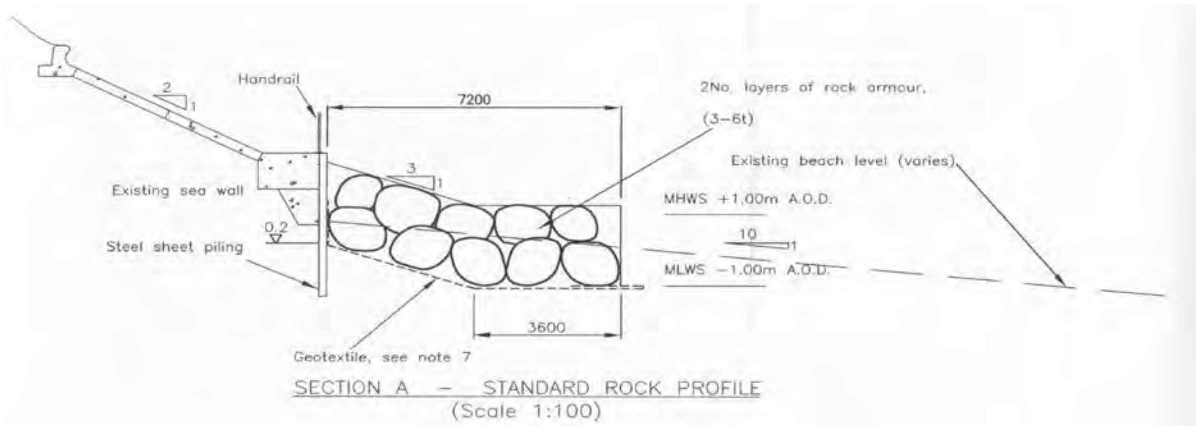
C2.4 Detailed design

The original design concept supposed that the toe would extend to -1.8 mODN. Between the tender and procurement stage, however, there was a worsening of coastal conditions which suggested that better protection was needed for the cliff part of the works. Consequently, in rebalancing the design to retain a similar overall cost, the toe level was raised to -1.0 mODN though, understandably, this raised concerns about increased future risk of toe damage. To counter this, future costs for subsequent maintenance repairs were included in the strategic budget. Further to this, steps were taken to secure the leading edge of rocks as safeguard for the time when the toe became exposed; the geotextile was wrapped around the toe rocks and held in position by the next line of rocks (a so-called Dutch Toe). However, wave attack whilst the toe was still exposed resulted in the fabric being torn and this idea was therefore abandoned.

Figure C2.7 shows two design cross-sections through the revetment. The upper section shows the cross-section of the new defence where the original construction was totally destroyed (see Figure C2.4); the lower section shows the typical section to the south of this where, partly for cost reasons, the original cliff apron slab survived and was incorporated into the section.



Full reconstruction over northern part



Revetment fronting remaining 1960s structure

Figure C2.7 Design cross-sections

C2.5 Construction issues

The construction works were put out to tender in August 2002 and J T Mackley & Co. Ltd was appointed to undertake the construction in February 2003. In the interim period they worked with Waveney District Council (WDC) in the development of cost saving design modifications.

At the time of works construction, the beach level had fallen significantly below that anticipated at the time of tendering. This required a new plan for undertaking rock placing and resulted in an extended period for completing the works.

Rock was delivered to the beach by barge (Figure C2.8). Figure C2.9 shows rock recovery operations. During rock recovery, an excavator sank into an isolated area of extremely low-bearing capacity ground; it remained there for 17 days by which time it was a write off (Figure C2.10).

There were concerns regarding the topping up of the rock revetment installed in 2000. The project costs did not allow for removal and replacement of all the rock; moreover, removal of the rock presented a serious risk to safety as it was providing support to the failed wall behind. Various options were considered, it being concluded to concentrate on discrete areas where voids could be filled with imported rock, and individual rocks replaced as thought necessary.

The completed scheme is shown in Figure C2.11.

Figure C2.8 Rock delivery

Figure C2.9 Rock recovery

Figure C2.10 Excavator loss

Figure C2.11 Finished works

C2.6 Post construction issues

A storm in December 2003 damaged the upper concrete slab over about 200 m of the southern frontage. The rock revetment did not suffer damage. The repairs to the existing slab part of the structure had, knowingly, been minimal in this area owing to the need to contain the scope of works within the available finance. The damage was subsequently funded and repaired.

C2.7 Lessons learnt

The project dealt with the potentially emotive topic of the managed withdrawal of coastal defence, albeit that continued defence and maintenance of beach access was assured for 20 years. This sensitive issue was appropriately handled through high level consultation and co-operation between the stakeholders and the public.

C2.8 Acknowledgments

We would like to acknowledge the invaluable advice and assistance provided by Paul Patterson, Waveney District Council, in the preparation of this case study. The case study is substantially derived from the paper by Patterson et al. (2004).

C2.9 References

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Halcrow, 2002. *Corton village coast protection project appraisal.* September 2002.

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C3 CASE STUDY: South Beach, Lowestoft

Courtesy of Waveney District Council

C3.1 Identification of the problem

By 2005, Waveney District Council (WDC) had become increasingly concerned about falling beach levels at Lowestoft South Beach (Figure C3.1). The area of particular concern was 200 m long in an area known as ‘Children’s Corner’ (Figure C3.2). The lowered beach levels presented a threat to the seawall and jeopardised the amenity value of the site. Given the urgency of the situation, WDC was keen to study and implement short-term remediation works (minimum five years’ service life) in advance of an eventual longer term strategic solution.

Figure C3.1 Location map – Lowestoft, Norfolk

There were two types of seawall in the area of concern (Figure C3.3):

- a concrete wall (90 m northern half) dating from 1922 (actually an encasement of the original old flint wall);
- an old flint wall (110 m, southern half) dating from c.1880, with a concrete core of variable quality, faced with grouted flint cobbles.

For the purposes of this case study, only the old flint wall is discussed, though the same approach and fundament design solution was applied to both.

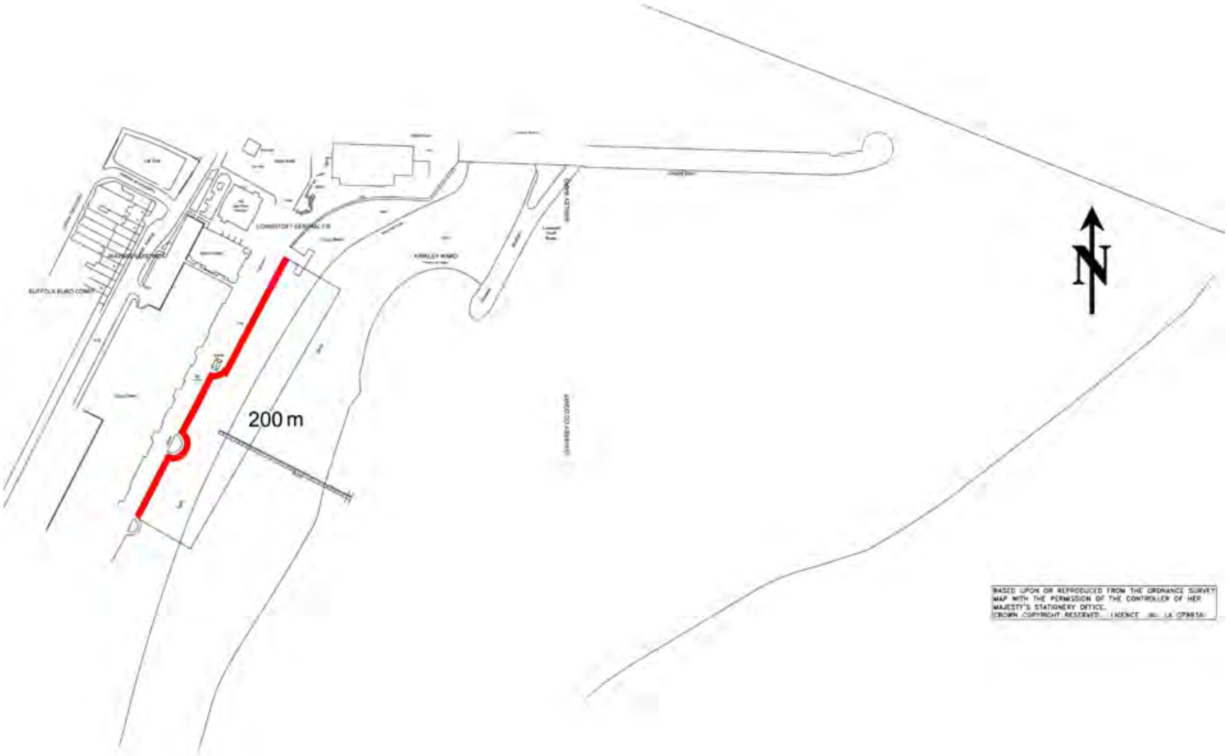


Figure C3.2 **Location of the seawall and toe protection works, Lowestoft South Beach**

Figure C3.3 **Old Flint Wall and concrete wall**

C3.2 Appraisal

An appraisal study was commissioned whose principal objectives were to:

- review the existing assumptions on risk and the nature of potential defence failure;

- identify and assess viable response options leading to the selection of a preferred solution.

Children's Corner and particularly the northern corner had historically suffered low beach levels. This was due, in part, to the more intrusive aspect of the promenade at its root with the harbour's South Pier, coupled with the concentration of incident and reflected wave energy. Other factors, including the influence of offshore banks on nearshore wave climate and the impact on the harbour on littoral drift, were examined in the appraisal study.

The risk of wall failure was evaluated in accordance with standard geotechnical procedures for a gravity wall. The required factors of safety (FOS) for overturning and sliding were defined as follows:

$$\text{FOS (overturning)} = \frac{\text{Resisting moment about wall toe}}{\text{Disturbing moment about wall toe}}$$

$$\text{FOS (sliding)} = \frac{\text{Resisting horizontal force}}{\text{Active horizontal force}}$$

BS 6349 (1988) recommended minimum factors of safety of 1.5 for overturning and 1.75 for sliding.

The principal variable in the analysis was the assumed level of the beach. Sea level had a lesser effect on the result. For the purposes of the appraisal, two beach levels were examined:

(i) The level of the bottom of the Old Flint Wall (0.36 mAOD), that is, the level at which the wall would become undermined

(ii) The height of beach needed to provide sufficient passive resistance to ensure stability in accordance with the factors of safety shown above.

Site investigation showed that the Old Flint Wall was rather variable in cross-sectional geometry (Figure C3.4). The results of the geotechnical analysis are shown in Table C3.1.

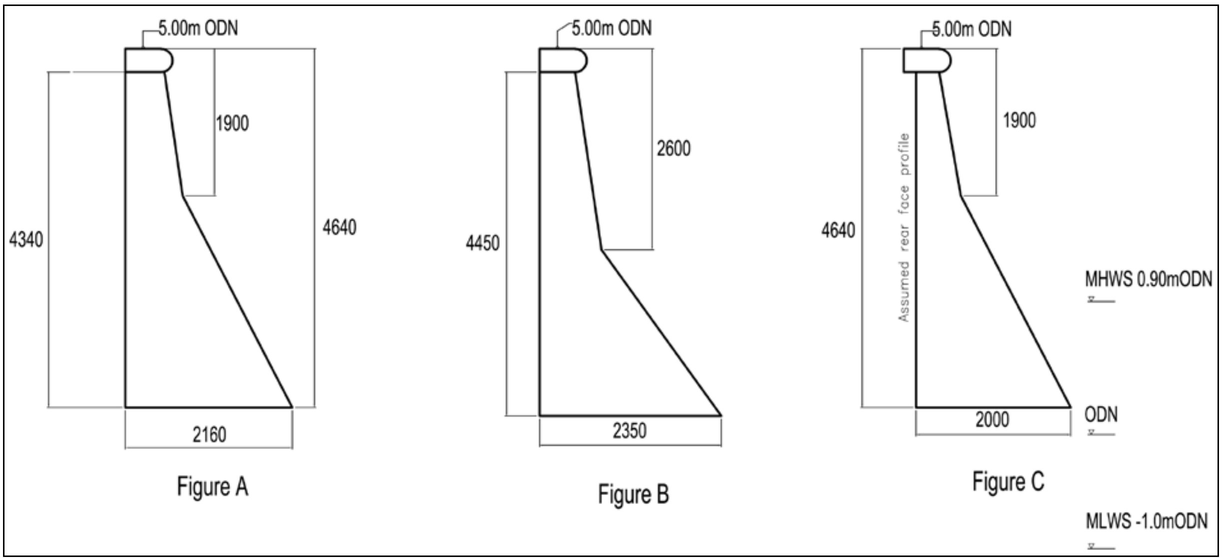


Figure C3.4 Old Flint Wall sections

Table C3.1 Factors of safety for the Old Flint Wall

	Beach level (mODN)	FOS overturning	FOS sliding
Sections A and B			
Case (i)	0.36	0.60	0.64
Case (ii)	2.17	1.50	1.75
Section C			
Case (i)	0.36	0.46	0.54
Case (ii)	2.25	1.50	Not calculated

In the case that the beach level was drawn down to the base of the wall (that is, to 0.36 mODN), the wall would fail in respect of both overturning and sliding. This low beach level is unlikely to be reached by 2010 given the outlook for beach lowering. Nevertheless, levels close to this critical point could be encountered.

The calculations showed that the wall needed a beach level above 2.17 mODN (that is, approximately 1.81 m above the base of the wall) to ensure stability (to required factors of safety) in respect of overturning. In the case of resistance to sliding, the lowest acceptable beach level was less onerous at 1.15 m above the base of the wall. In conclusion, and being mindful of the expected beach trends, the risk of instability (to 2010) was considered to be moderate. However, it was also considered prudent to allow for the possibility that any short-term mitigation might be required to secure the wall for a longer period (for example, 15 years rather than five). This prospect significantly increased the likelihood of a failure and possibly a breach of the unprotected wall within the given time horizon.

The appraisal examined a range of options for mitigating the risk to the Old Flint Wall including measures directed at maintaining a higher beach, alongside measures directed at securing the wall in the case that the beach continued to fall:

- beach nourishment (recycling);
- beach nourishment (imported);
- short timber groyne;
- Armorflex revetment;
- stepped concrete seawall with piled toe;
- concrete surcharge;
- sheet piled toe;
- rock toe;
- concrete armoured toe;
- ground anchors.

Having given due consideration to the technical merits, environmental factors (including amenity) and financial issues, the preferred option was the use of a sheet piled toe to secure the wall against geotechnical instability and undermining.

C3.3 Outline design

The fundamental design consisted of a six-metre long steel sheet piled wall (length selected for efficient cutting and use of delivered pile lengths), together with a concrete thrust block (apron) cast in situ between the toe of the Old Flint Wall and the newly driven sheet piled wall. The outline design was based on the conventional methods for cantilever sheet pile design. The loadings are listed in Table C3.2.

Table C3.2

Type of pressures	Loading
Active	• Lateral load transmitted through the thrust block, being the net load after friction losses from the old wall and the thrust block were removed. • Active geotechnical loads induced by the vertical loadings of the thrust block and the Old Flint wall. • Active geotechnical loadings induced by the soil profile on the landward side. • Water differential.
Passive	• Passive geotechnical resistance induced by the soil profile on the seaward side.

The outline design provided the basis for consultation. Consultations were undertaken as follows:

- Environment Agency re need for a notice under the Coast Protection Act 1949 and consent under Water Resources Act – neither needed in this case;
- Marine and Fisheries re FEPA licence – not needed in this case;
- planning consent with particular reference to heritage and amenity;
- land ownership.

The scheme was classed as maintenance and the consents were very straightforward given the nature and extent of the works.

Both outline and detailed design were carried out in compliance with CDM objectives and regulations. Among other matters this identified the potentially increased risk due to falls from the promenade edge onto the concrete thrust block (apron) when it becomes exposed.

C3.4 Detailed design

Figure C3.5 shows the detailed design cross-section. Particular issues and areas of detail considered within the detailed design included the following.

The outline design included a pile cap (a concrete wrap over including steel reinforcement) which was eliminated in the detailed design due to:

- potential frailty;
- eventual corrosion of reinforcement;

- difficulty in forming it in areas of detail (for example, around the circular ramp sections).

In the simplified design, the concrete thrust block was either simply made to fill the void up to the level of the pile, or it was shuttered back from the pile edge at a higher level in order to yield a sufficient mass of concrete (and hence contributory friction resistance to the Old Flint Wall).

Drainage was included by way of holes cut in intermittent pile pans, augmented by allowing every fifth pile to be curtailed at 3 m depth (instead of 6 m) – the reduced passive resistance was allowed for in the design.

The plan included details of complicated features. In particular the toe detail around the circular ramps was examined with particular reference to the radius of curvature (actually the angular compliance at clutches) of alternative pile types.

C3.5 Construction issues

Figures C3.6 and C3.7 show the new toe detail during construction. Particular issues that arose during construction included the following.

Excavation of the old wall toe revealed considerably greater variability in the toe depth than had been anticipated on the basis of the earlier trial pits; the problem was overcome by varying the size and hence mass and friction resistance of the thrust block, keeping the piling the same. This apparently straightforward alteration required careful consideration as the block's depth, width and top level were all constrained within practical limits.

Flowing groundwater emerged over a small section of excavation; this was countered by increasing the drainage across the pile section as noted above.

One of the ramp sections was damaged (but did not collapse) during excavation; here the priority was to continue to secure the toe and deal with the 'topside' damage subsequently.

C3.6 Post construction issues

Following construction there has been a need to manage the increased hazard of a fall from the promenade edge onto the concrete apron when it is exposed. This has been achieved by banking up the sand to cover the apron. Although the prospect of an increased risk was recognised earlier in the project, the issue was aggravated by the need to increase the depth of the concrete apron in places to counter the (subsequently revealed) shorter depth of the old seawall.

C3.7 Lessons learnt

Possibly the most significant lesson to emerge from this project was the need for early (during detailed design stage) and more comprehensive trial pits to better determine the variability of the depth of the existing wall. Discovery of the extent of the toe variations during construction led to the need for rapid decision-making and re-analysis of the design section while the works were in progress. In the event, once the likely extent of the variations was realised, a number of additional pits were dug in order to pre-empt further discovery and to enable the ongoing works' details to be planned accordingly.

Further to the above point, when working with old and poorly understood structures such as the Old Flint Wall, it would be worth using enhanced factors of safety. This would then offer greater flexibility in any adaptive engineering that might ensue following discoveries made during construction.

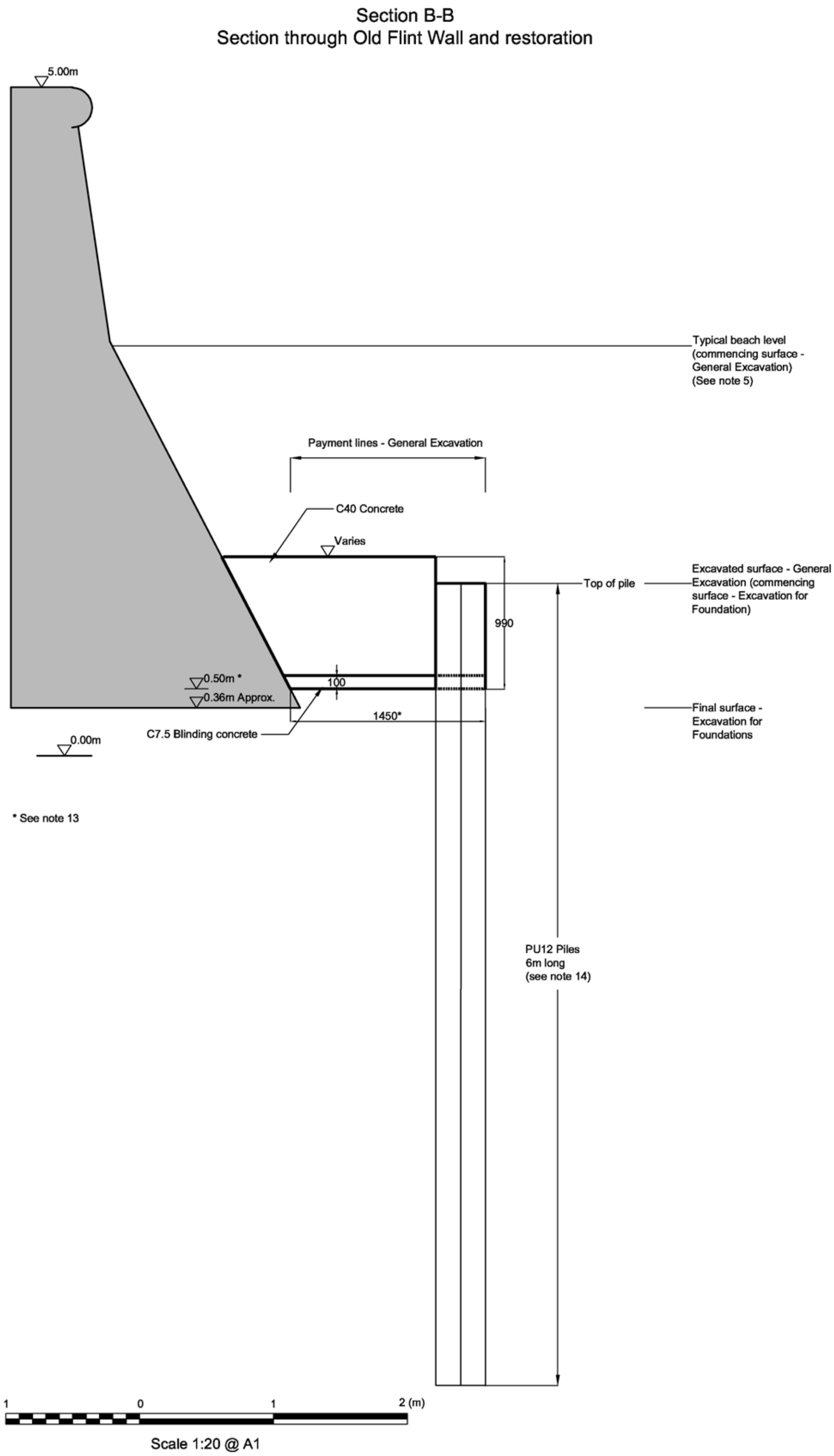


Figure C3.5 Old flint wall design cross-section

Figure C3.6 Toe piling being installed (at concrete wall section)

Figure C3.7 Shuttering for concrete thrust block (apron) – old flint wall section (note water ingress and exposure of the toe of the existing wall)

C3.8 Acknowledgements

We would like to acknowledge the invaluable advice and assistance provided by Paul Patterson, Waveney District Council, in the preparation of this case study.

C4 CASE STUDY: Holme Dunes, Norfolk

Courtesy of the Environment Agency

C4.1 Identification of the problem

Holme Dunes are located about 5 km to the east of Hunstanton on the Norfolk Coast (Figure C4.1). The dunes provide a natural flood defence to a number of properties, agricultural land and the Holme Dunes National Nature Reserve. The nature reserve, managed by Norfolk Wildlife Trust, is a designated SSSI, SPA, Ramsar site and Biosphere Reserve noted for its dune system and freshwater marshes. Figure C4.2 shows the site photographed in 2009 (note the more recent brushwood dune protection).

Figure C4.1 Location of Holme Dunes

In the mid-1990s, concern was expressed about the perceived erosion of the dunes in the area referred to as the Firs (named after the large property just inland of the dunes). A report[11] by the Department of Marine Sciences and Coastal Management at the University of Newcastle commissioned by the Environment Agency suggested that erosion was due to larger waves reaching the toe of the dunes, noting that this appeared to be linked to recent (1991–1996) evolution of the ebb delta at Thornham. It was thought that loss of tidal volume of the inlet had resulted in a weakening of the ebb delta which, in turn, had resulted in a reduction in protection to the shore due to changes in wave refraction and shoaling.

C4.2 Appraisal

The North Norfolk Shoreline Management Plan had recommended that property should be protected in the short term by strengthening the eroding dune system using construction that was acceptable environmentally and financially. The economic benefits of coast protection at the site were very limited, thus implying that any scheme would have to be low cost. There was, nevertheless, a statutory requirement to protect

[11] Holme Dunes: coastal Processes and Geomorphology Study

the designated conservation interests. A formal detailed appraisal was suspended pending the identification of suitable options that might, at the outset, satisfy these fundamental criteria. Options considered included:

- recycling of sand and shingle from a nearby bar (Gore Point), but this was not allowed because of potential damage to habitats there;
- beach recharge, as well as detached breakwaters and groynes, but high costs relative to the limited benefits precluded these more expensive options.

The Environment Agency worked with English Nature and Norfolk Wildlife Trust in formulating a potentially viable option. The concept of beach drainage as a means of shore stabilisation came to light following discovery of the principle in Denmark. Invented by the Danish Geotechnical Institute, the system was licensed, designed and marketed by MMG Beach Management Systems (UK). The prospect of a workable system at Holme was attractive, not least because it appeared to offer a solution that entailed no structural interference with the dunes. However, should a permanent installation to be installed there would be royalty payments to make in respect of the patent rights held by MMG. Given that the method was still highly innovative with only a limited history of applications at this stage (there had been only one other application in the UK), it was decided to carry out a trial for which MMG kindly agreed to waive the royalties.

Figure C4.2 The site in 2009 – note the recently installed brushwood dune protection

C4.3 Outline design

The design was prepared by MMG for the size of perforated pipe, geotextile wrapping, depth of pipe and pump requirements. Its 'Beach Management Systems' brochure described the theory of beach drainage thus:

> 'with a sandy beach, lowering the water table in the beach face eliminates buoyancy factors and vastly reduces the lubricating effect between the grains, thus restoring the frictional characteristics of the sand. Furthermore,

the percolation of 'swash water' into the relatively dry beach encourages sand to settle out at the beach face'.

No further details were available (for this study) of the bespoke design considerations or workings.

A grading curve of a beach sample taken from the site (Figure C4.3) shows it to comprise predominantly sand with some gravel.

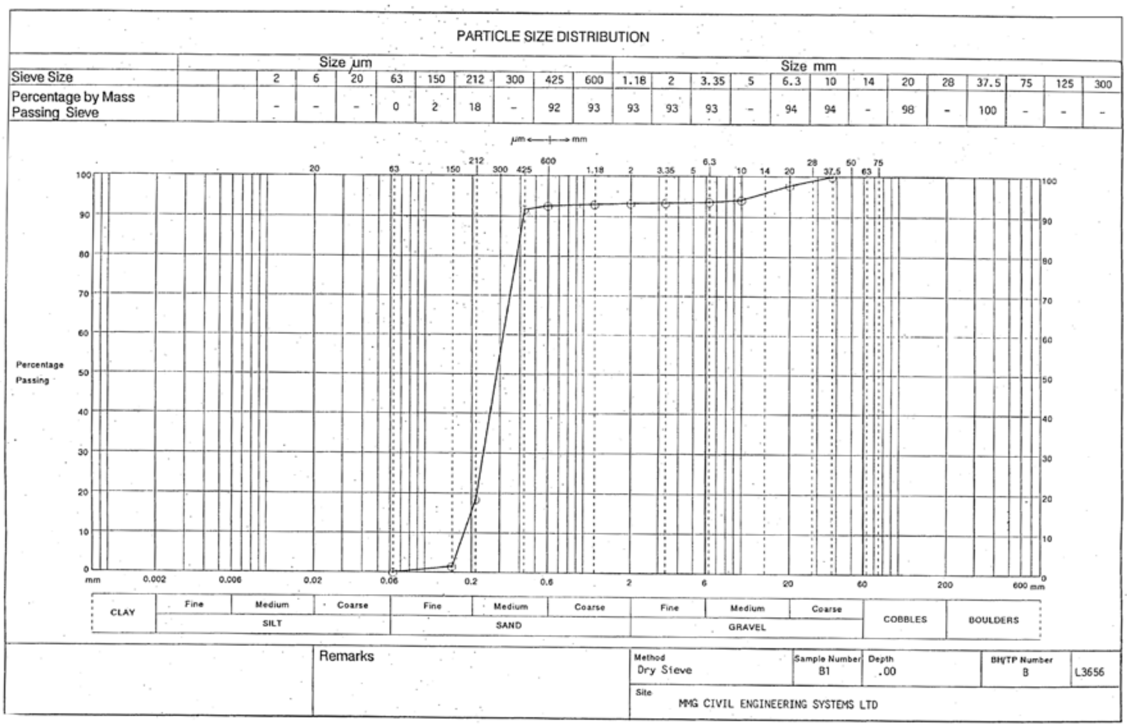


Figure C4.3 Example grading curve

Basically, the scheme comprised a 200 m shore parallel drainage pipe set about 30 m from the toe of the dunes. In fact two drainage pipes were used to yield the overall capture width of 200 m. These two pipes drained into a sump which contained a pump. The pump discharged the collected water back to sea through a higher outfall pipeline. Figure C4.4 shows cross-sections for the west limb together with a schematic section taken through the pump sump.

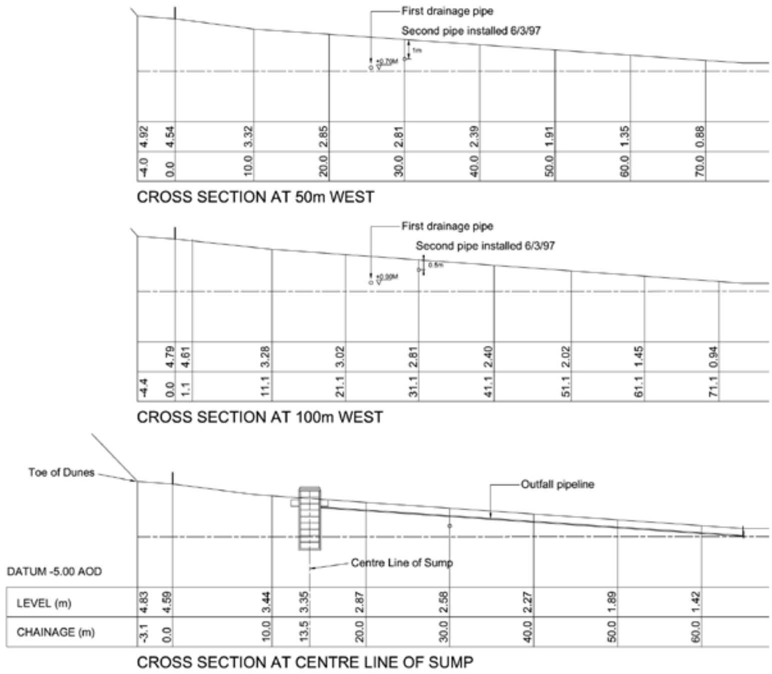


Figure C4.4 **Beach sections**

C4.4 Detailed design

The drainage lines consisted of 200 mm high density polyethylene (HDPE) perforated pipes, 100 m long in each direction running broadly parallel to the beach surface, as indicated in Figure C4.4.

For the initial trial, the shore parallel parts of the pipes ranged in level from +0.9 mODN at their seaward ends (100 m east and 100 m west) to +0.7 mODN at 50 m east and 50 m west. These levels resulted in a depth of sand cover of about 1.5–2.0 m depending on beach level and pipe gradient.

In an attempt to make the beach drainage more effective, an additional pipe was installed after a few weeks of operating the system with a reduced depth of cover of between 0.5 and 1.0 m as illustrated in Figure C4.4. The levels shown in Figure C4.4 can be compared with the tide levels for Hunstanton (posted in 1996) of MHWS +3.7 mODN, MHWN +1.9 mODN, MLWN -1.2 mODN and MLWS -2.8 mODN. Hence, the drainage pipe inverts were installed at a level between mean tide level and MHWN, while the beach (at the time of illustrated survey) was at a level between MHWN and MHWS.

The drainage pipes fed seawater back to the 4.5 m deep sump (Figure C4.5). A pump located in the bottom of the sump extracted the collected seawater and returned it to sea via the outfall pipe. The pump was rated at 55 l/s (against a 5 m head) and was powered by a generator located on the dunes.

C4.5 Construction issues

The most significant construction issue was thought to be a problem with floatation of the sump chamber. This was corrected by ballasting the structure using concrete rings attached near the top (Figure C4.5).

C4.6 Post construction issues

A beach monitoring programme was set up and maintained for about three months (February to April 1997) – the period of the trial. This was based around a series of beach profiles located at the centre line and at ±50, ±100 and ±200 m from the centre line. The outer profiles were taken as a reference check on the ambient beach level to distinguish changes that might be linked to the beach drainage scheme from natural changes happening along the whole frontage. Visual observations were also made.

Initial observations, following installation of the first drainage pipes at 0.7–0.9 mODN, showed little or no effect in terms of drying of the beach. A second pipe was installed at a higher level (see Figure C4.4). After installation of the second pipe, the beach surface in the vicinity of the pipe appeared to be drier but, over the trial period, there was no change to beach levels that could be confidently attributed to the drainage system. During construction of the sump and operating the system, the water levels in the beach were noted to be very high at all times and states of the tide, so much so that the beach drainage system would be working by siphon effect even at low tide.

By the end of the formal trial a decision was needed whether to adopt a permanent system, which would have required payment of the licence royalty or curtail the project. Given the results up to that point, the latter was opted for, although it is understood that MMG funded the system’s continued operation for a short further period.

C4.7 Lessons learnt

Based on the formal trial, the system did not appear to afford an improvement in the beach level. It should also be noted that three months is a very short period over which to conduct such a trial, as perceived performance depends not just on efficacy of the system but also on the pressures applied to it in terms of wave and tidal factors. Nevertheless, upon completion of the trial, as there was no noticeable effect on the frontage and given that a permanent scheme would have attracted royalty payments, the scheme was abandoned.

Before embarking on any new permanent scheme, a considerable amount of technical data need to be gathered and evaluated before proceeding to detailed design

C4.8 Acknowledgments

We would like to acknowledge the invaluable advice and assistance provided by Paul Miller of the Environment Agency in the preparation of this case study.

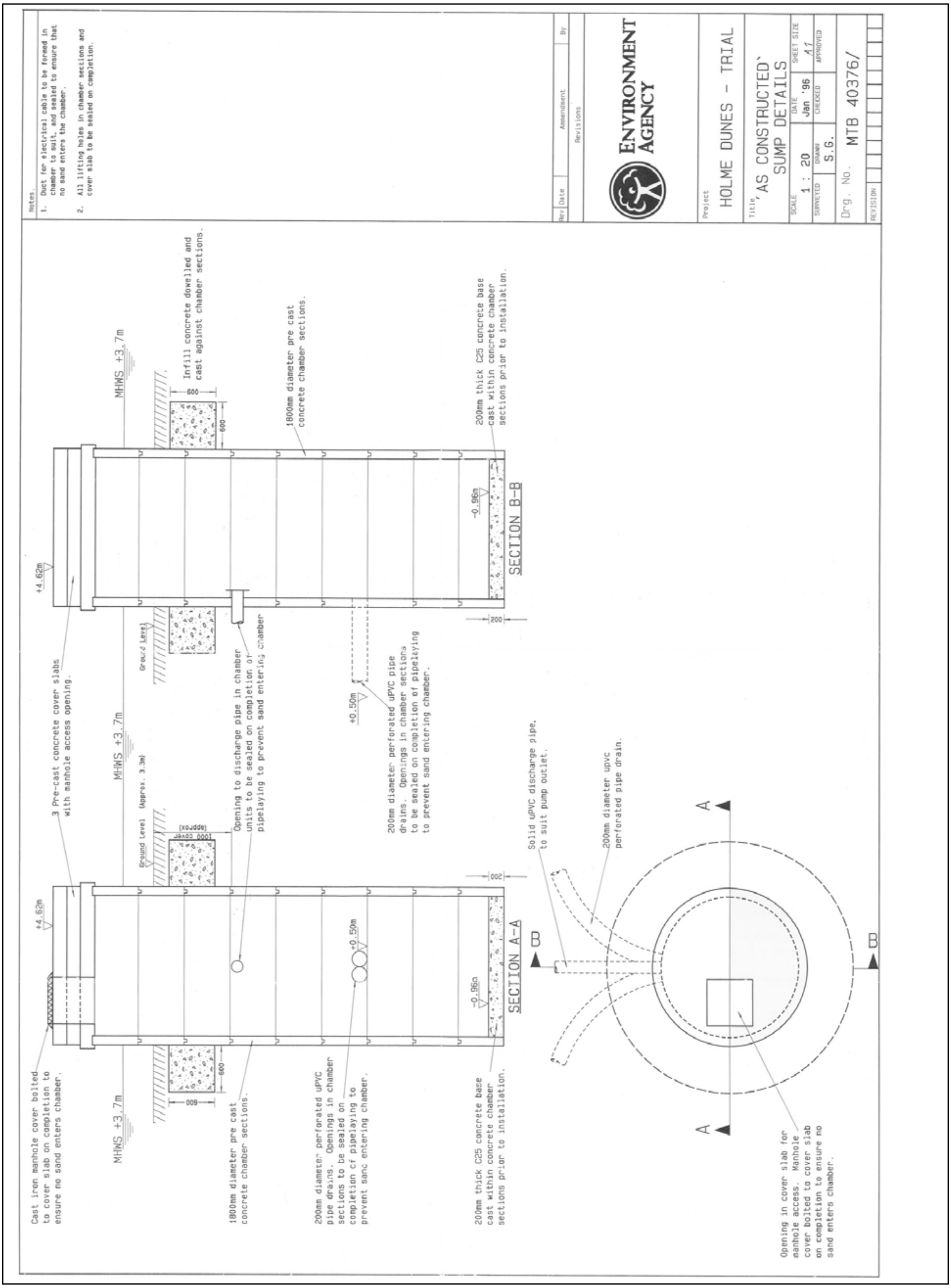


Figure C4.5 Sump detail

C5 CASE STUDY: Overstrand, north Norfolk

Courtesy of North Norfolk District Council

C5.1 Identification of the problem

Overstrand is located 3 km to the east of Cromer on the north Norfolk coast (Figure C5.1). The site in question is located at E3 on Figure C5.2, at a mild turning point in the frontage which is defended both to east and west. The long-term average rate of undefended cliff retreat here is about one metre per year. The sandy foreshore falls away at a mild slope of about 1 in 50 and covers chalk some 2 m below the sand level at the seawall.

Figure C5.1 Location of Overstrand, north Norfolk

Figure C5.2 Site location

Prior to construction of the present structure, the coastal defence consisted of a sheet pile toe behind which there was a lower concrete apron, concrete retaining walls and a higher concrete apron, all containing the coastal cliff to a height of about 25 mODN. The old sheet piles were badly corroded and there were holes in some of the pile pans. The whole structure was believed to date from c.1908. Figure C5.3 shows part of the wall that did not fail.

Figure C5.3 The original wall

During December 1997, a 25 m length of the lower apron collapsed leading to part failure of the main seawall and promenade. This initial failure put the whole of the west promenade in jeopardy. It was believed that the old piled wall failed through corrosion and apron failure leading to loss of backfill.

Further failure was expected to follow, which would progress to erosion of the cliff. In order to make the structure safe in the short term, 100 m^3 of concrete were placed in the void behind the wall. This provided a short-term solution while a permanent scheme was planned and constructed.

C5.2 Appraisal

The engineers report, *Overstrand Coast Protection Scheme 973* (1999), served the purpose of the Project Appraisal. It was recommended that the project to restore the Overstrand seawall be undertaken as an emergency scheme under section 5(6) of the Coast Protection Act 1949. The scheme was to comply with a number of limiting factors, that is:

- works were to be contained within the financial constraints and have a minimum benefit/cost ratio of 1:5;
- works were to be designed to take account of the degree of difficulty in working below the high water mark in a hostile marine environment;
- the scheme was to comply in all respects with the criteria required by the then Ministry of Agriculture, Fisheries and Food (MAFF) including that of being environmentally acceptable.

To expedite the process of option identification, discussions were held with several experienced contractors to elicit their views and recommendations on design and construct schemes. The various options considered included:

- do nothing (baseline);
- reconstruct the existing structure – this was rejected due to the inherent residual shortcomings of the original structure (for example, inadequate foundations) together with the increased pressures due to beach lowering making reconstruction major and costly endeavour;
- construct the lower apron at a lower level than before – this was rejected on the grounds of practicality because of the need for substantial excavation below the existing wall together with the downwards extension of the wall and foundation, thus requiring a lot of intertidal working;
- construct a rock revetment – this was examined but considered to be uneconomic in this case because of the short length of the works (about 100 m) and hence the relatively high mobilisation cost;
- sloping or stepped concrete apron – this was a preferred option, the choice between in situ concrete or pre-cast units being one of practicality.

Consent was required from The Crown Estate. English Nature was notified as the western 20 m of the site fell within the Overstrand Cliffs SSSI and the land beyond Overstrand village was an Area of Outstanding Natural Beauty (AONB). There were no significant environmental issues to consider other than rectification of the failed frontage and public access.

The chosen scheme delivered a benefit/cost ratio of 2.2 and thus qualified for MAFF grant aid.

C5.3 Outline and detailed design

North Norfolk District Council was keen to use pre-cast concrete in this location to overcome the problems associated with the placing and curing of concrete in this highly exposed location. Four contractors were invited to discuss the Council's design proposals before the final design was prepared and the contract was put out to tender. McKinnon Construction won the tender.

The final design (Figure C5.4) comprised a sheet piled toe with pre-cast concrete superstructure. The components were:

- steel sheet piles driven into the chalk – these cantilevered piles were considered to be secure, even in the case that the beach lowers to the chalk level as their stability did not rely on having a depth of beach to provide support;
- a concrete beam located behind the piles;
- pre-cast concrete step bridges;
- pre-cast concrete steps.

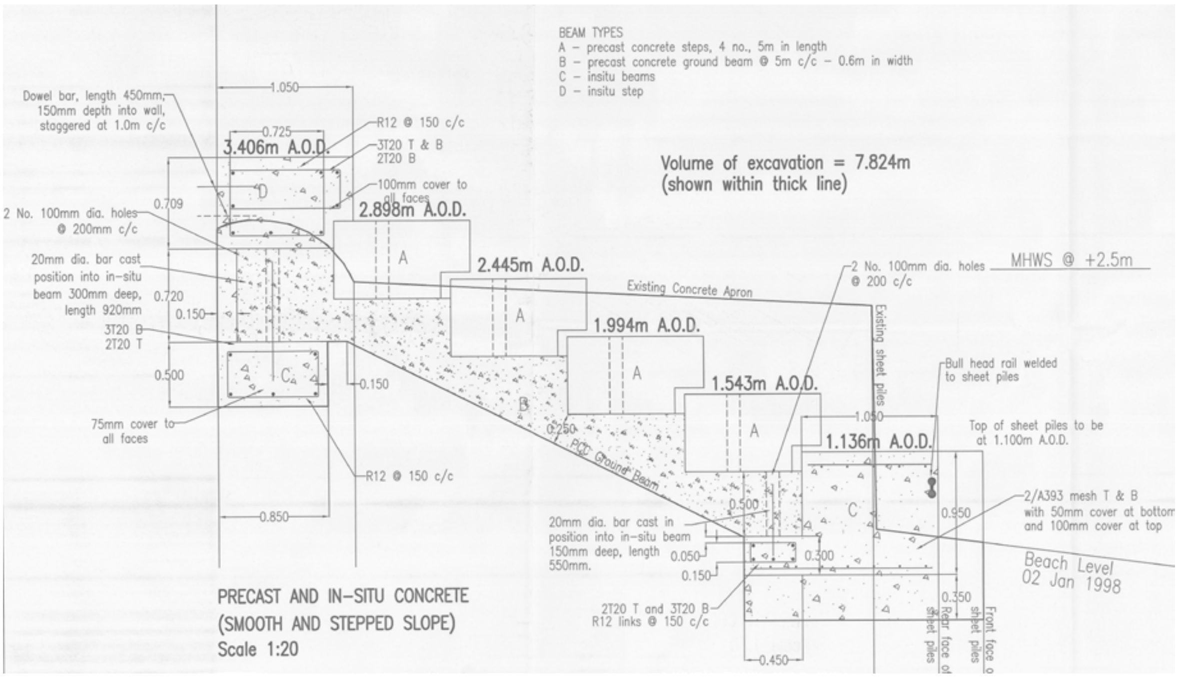


Figure C5.4 **Design cross section drawing of the Overstrand defence (extract)**

C5.4 Construction Issues

The use of substantially pre-cast construction minimised in situ works and hence exposure to action of the sea during installation.

The end details, where the section had to tie into existing remaining details (Figure C5.5), were most difficult both in terms of design and construction.

C5.5 Post construction issues

Figures C5.5 and C5.6 show the finished construction.

The front edges of the steps have become rounded (being sharp edged when cast). In retrospect, it would have been better to allow for corner chamfers in the shuttering.

The new construction has extended the life considerably of the defence. The Overstrand Foreshore Study ET 4252(2008), carried out for the Council by St La Haye Consulting Ltd, assigns the new section a residual life of 35–40 years allowing for beach degradation. However, the adjacent wall sections have considerably shorter estimated residual lives.

Figure C5.5 End detail

Figure C5.6 Finished toe and wall

C5.6 Lessons learnt

The scheme provides amenity benefits and has been successful. The use of pre-cast concrete provided a quick economic solution in this case. Better attention to the detailing of the steps (edges) in future should be noted.

C5.7 Acknowledgements

We would like to acknowledge the invaluable advice and assistance provided by Brian Farrow, North Norfolk District Council, in the preparation of this case study.

C6 CASE STUDY: West End Dovercourt, Essex

Courtesy of Tendring District Council

C6.1 Identification of the problem

Figure C6.1 shows the location of Dovercourt, just to the south of Harwich on the Essex coast. The site had been used as a landfill before Tendring District Council (TDC) took possession. The waste material was isolated from the sea only by a clay covering and embankment. Shoreline retreat presented a growing risk of erosion of the embankment containing the fill. Had this been allowed to continue then penetration of the embankment would have resulted in the release of landfill waste into the marine environment.

Figure C6.1 Dovercourt site location

The site fronts a conservation area for ground nesting birds and is immediately adjacent to Hamford Water SSSI. It was essential therefore to prevent contamination of the shore.

Earlier attempts were made to arrest erosion of the spoil site by placing interlocking mattresses on the embankment (Figure C6.2). However, these were inadequate for the given exposure and were soon destroyed by storm wave action (Figure C6.3). An alternative low-cost revetment was sought.

C6.2 Appraisal

No formal appraisal was carried out but a number of mitigation options were considered. Although a comparatively new technique to the UK, an asphaltic revetment appeared to offer good value. It would be sufficiently robust for the comparatively benign wave climate and, for the Dovercourt location, its (sometimes undesirable) appearance was not a significant issue.

Figure C6.2 Flexible armoured revetment (earlier defence structure)

Figure C6.3 Failed flexible armoured revetment

C6.3 Outline design

The first phase of the protection works were carried out in the early 1990s, being extended in 2001. Figure C6.4 shows a plan layout of the revetment. The revetment extended to the edge of the SSSI designated area.

Specialist contractor, Hesselberg Hydro, was commissioned to design and construct the works.

The toe was to be buried and backfilled with disturbed clay. Given the nature of the site, there was no sand covering to the clay. The exposed clay (Figure C6.2) was susceptible to continued lowering. The toe structure was therefore designed to rotate once the underlying ground level was reached.

C6.4 Detailed design

A design drawing is reproduced in Figure C6.5. The construction consisted of:

- regrading of the shore to adopt the revetment slope and deeper excavation for the toe;
- a 200 mm layer of open stone asphalt laid over a 150 mm layer of lean sand asphalt on a 1:2.5 slope (150 mm layer of open stone asphalt used in the more sheltered sections);
- a 350 mm thick grouted stone slab placed over a geotextile on a 1:15 slope to form the buried toe (the main revetment section extended into the excavation to form buried toe in the more sheltered areas).

C6.5 Construction issues

Beach excavation and toe construction were undertaken using plant on the foreshore. Plant operated from the crest of the embankment for placing asphaltic materials on the revetment. Construction of the works was straightforward except for some damage to the top landscaped area due to plant movement; see Figures C6.6 and C6.7.

C6.6 Post construction issues

The revetment is subjected to wave attack which, together with shingle and debris, results in abrasion of the asphalt. The interface between the shore and the open stone asphalt revetment suffers most wear. Erosion tends to happen in pockets and seems to return to the same places.

Monthly monitoring of the beach is carried out to identify problems and repairs are usually carried out once per year. Repairs are made either by appointed contractors or by TDC's in-house resources – typically one repair is needed per year. Maintenance is an important management consideration but is not a problem providing that a regular monitoring and repair regime is adhered to.

Recently, TDC experimented with the use of a polyurethane binder (instead of bitumen) for repairs. The advantages of this are:

- it is basically clear;
- there is no need for a hot pot;
- the material is not affected by water except for some discolouration if the binder gets wet too soon.

To date, the buried toe has not been exposed and so its ability to articulate with falling beach levels has not yet been tested.

C6.7 Lessons learnt

The use of asphaltic construction provided a good economical solution in this case. It is accepted that, due to its appearance, the use of asphalt could be location sensitive but this was not a problem at the Dovercourt site.

A good inspection and repair regime is essential.

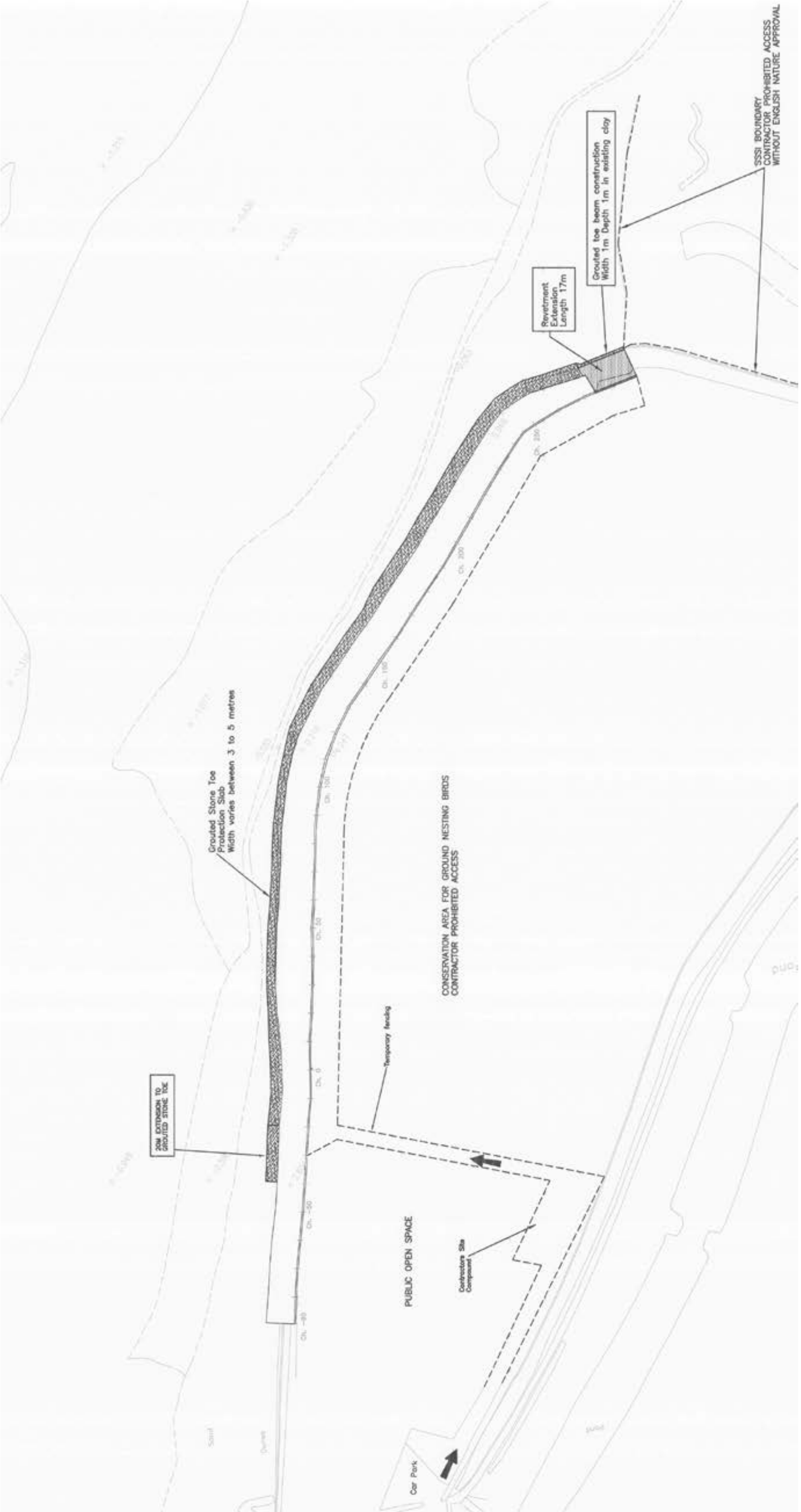


Figure C6.4 Plan extent of revetment

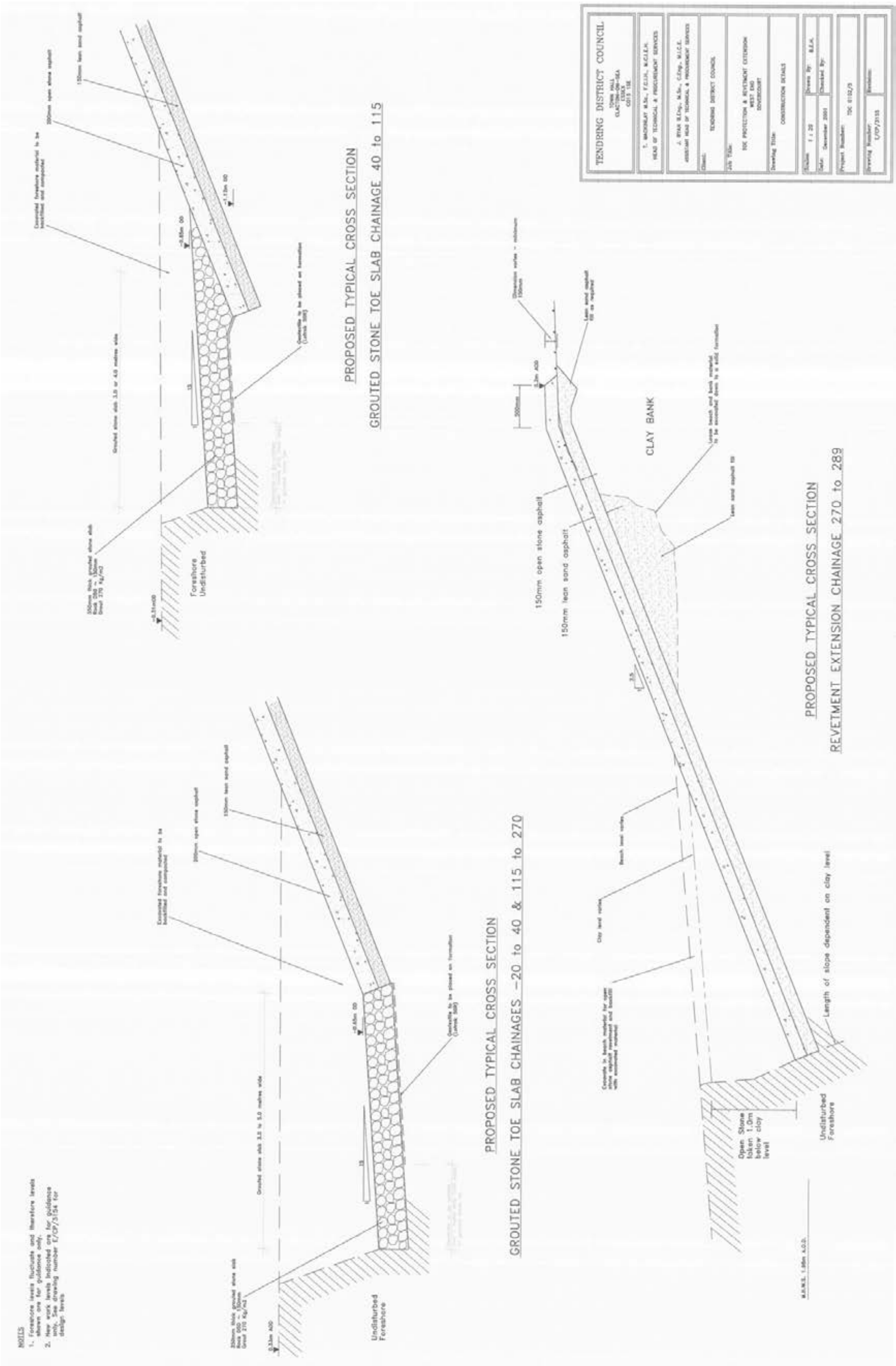


Figure C6.5 Design drawing of asphalt revetment

Figure C6.6 Revetment construction

Figure C6.7 Toe construction

C6.8 Acknowledgements

We would like to acknowledge the invaluable advice and assistance provided by Bob Howell, Tendring District Council, in the preparation of this case study.

C7 CASE STUDY: Teignmouth to Dawlish sea wall

Courtesy of Network Rail

C7.1 Identification of the problem

During the early months of 1986, the sand level at a section of the sea wall supporting the main Paddington–Penzance railway disappeared completely between Smugglers Cove and Sprey Point on the line between Dawlish and Teignmouth (Figure C7.1). Although fluctuation in sand levels can be quite significant throughout the length of the sea wall, total loss at this location had not been experienced within the memory of those involved with the wall at the time.

Figure C7.1 Location of Dawlish and Teignmouth

The sea wall at this location is a vertical masonry wall, with a small toe section of approximately 1.5 m^2 in cross-section probably constructed at some time after the main wall. The bedrock onto which the wall is founded is comprised of soft red sandstone interlaced with bands of breccia. This sea wall was originally constructed in about 1840 by Isambard Kingdom Brunel as part of the South Devon Railway and has undergone improvements to various sections throughout its history.

With the loss of the beach material in 1986, erosion quickly occurred in three areas to the extent that the wall became undermined for the full cross-section, resulting in the fill material which supported the railway line being ‘sucked out’ by successive tides. Consequently, the line was closed for a period of approximately two weeks, and it was touch and go as to whether these three sections of wall would be lost.

C7.2 Appraisal

The first objective facing British Rail was to secure the wall and restore rail services. Block stone from Meldon Quarry was delivered to site by rail on the ‘Up’, London-bound line, which had not become undermined. This was placed into the voided areas in concrete and the breached wall face repaired with concrete filled sand bags, allowing the supporting ballast to be replaced, and the service restored.

It then became clear that the interface between the soft bedrock and the wall toe required underpinning and sealing to secure the wall and to mitigate against future sand loss. This was achieved by providing an extension to the toe in wet sprayed concrete (shotcrete) of approximately 1.5 m^2 in cross section. The principle behind this approach was that should total sand loss occur in the future, the bedrock would have to erode beneath the sprayed concrete, generally 1.5 m wide, before the main wall was affected, by which time the sand level would hopefully restore naturally – affording protection to the toe once more.

A length of wall of approximately 850 m was treated and the solution provided protection for a period of about 10 years. During this period, the sand level continued to fluctuate, with total loss occurring in some areas for a short period. For the majority of the length of wall, the erosion of bedrock was approximately 100–200 mm, but in one particular area, the loss was over 800 mm. By this time, ownership of the network had passed to Railtrack under privatisation of the rail industry, and a more lasting solution was being considered, with the principal objective of securing a solution ensuring a safe and reliable rail service. The options under review are summarised in Table C7.1.

Table C7.1 Possible solutions

Option	Advantages and disadvantages
Rock armour	• Would not have found favour with the local authority due to loss of beach amenity. • Costly due to poor access. • Concern about loss of fine material through holes opening in main wall, which could not be properly accessed after the placement of rock armour. This could lead to difficulty in maintaining track geometry.
Sheet piled toe	• Difficulty in driving in bedrock. • Limited longevity. • Poor access for size of plant required.
Concrete toe construction	• Preferred solution.
Groyne field	• Already in place and would be maintained.

The area where rock erosion had been greatest was selected for a trial construction. The term sea wall maintenance contractor was commissioned to design and construct a concrete toe solution, utilising a consultant experienced in coastal engineering.

C7.3 Outline design

The principal design comprised a mass concrete toe, cut 1.5–2 m into the bedrock, approximately 1 m wide, with three mass concrete steps each approximately 1 m × 1 m, as shown in the cross-section (Figure C7.2). The principal considerations were:

- to provide long-lasting protection against the effects of bedrock erosion;
- to improve passive resistance against sliding and overturning;
- to minimise sand loss;
- to allow continued access to the masonry wall for inspection and repair.

Consultations were undertaken with organisations including the Environment Agency, The Crown Estate and Defra, and planning consent was obtained.

C7.4 Detailed design

After successful completion of the trial, the project was extended to cover the majority of the sea wall between Dawlish Warren and Teignmouth. The more vulnerable areas, namely between Smugglers Cove and Teignmouth (850 m), and a section at Rockstone between Dawlish Warren and Dawlish (28 m) received the full strengthening profile as shown in Figure C7.2, while most of the remaining sections, totalling 1,690 m received the partial strengthening profile as shown in Figure C7.3. The decision to adopt the lesser profile was based on the relative stability of the sand level and exposure of the length concerned. Other areas of the sea wall either did not require a strengthened toe, had been strengthened by other means, or the existing arrangement or position did not lend itself to the design.

Locally, provision in the detailed design was made to extend existing steps to the beach downward, to provide ramped sections where necessary, to negotiate features, and to make provision for watercourse outlets. Figures C7.4 and C7.5 show the full foundation arrangement at Smugglers Cove and Rockstone respectively. Figure C7.6 shows partial foundation strengthening.

C7.5 Construction issues

The main issues at the construction stage were:

- de-watering during the outgoing tide period;
- avoiding undermining the existing wall during excavation;
- trimming the bedrock to a suitable constancy and achieving a clean cut into the bedrock for the deep foundation;
- transporting large volumes of concrete from delivery point to site;
- constructing formwork able to withstand the incoming tide.

C7.6 Post construction issues

No significant issues were identified.

C7.7 Lessons learnt

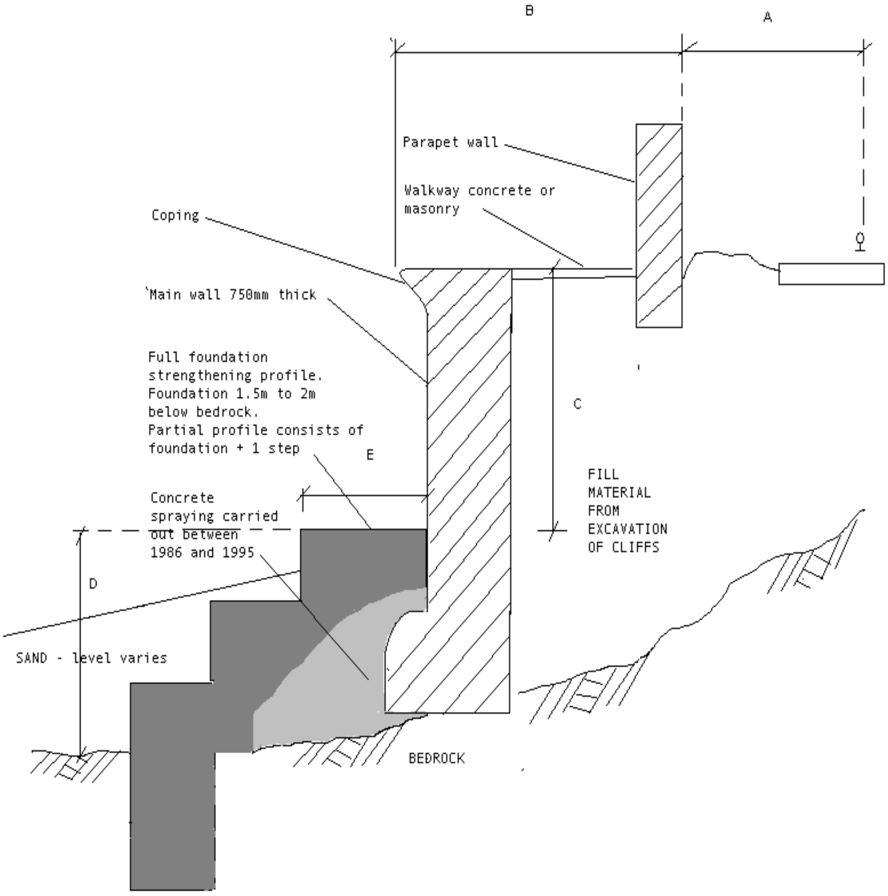
The most important lesson learnt was the vulnerability of vertical sea walls to sand loss and toe erosion. This prompted an ongoing need for frequent inspection, monitoring of sand levels, an adverse weather mitigation plan and consideration of suitable methods of keeping beach sand in place.

C7.8 Acknowledgements

We would like to acknowledge the invaluable assistance of Peter Haigh, Network Rail, for the provision of this case study.

Full Foundation Strengthening

Typical cross section with full foundation strengthening to be found at :
205m 36c to 205m 51c
207m 46c t0 208m 09c (Smugglers Cove to Spray Point)



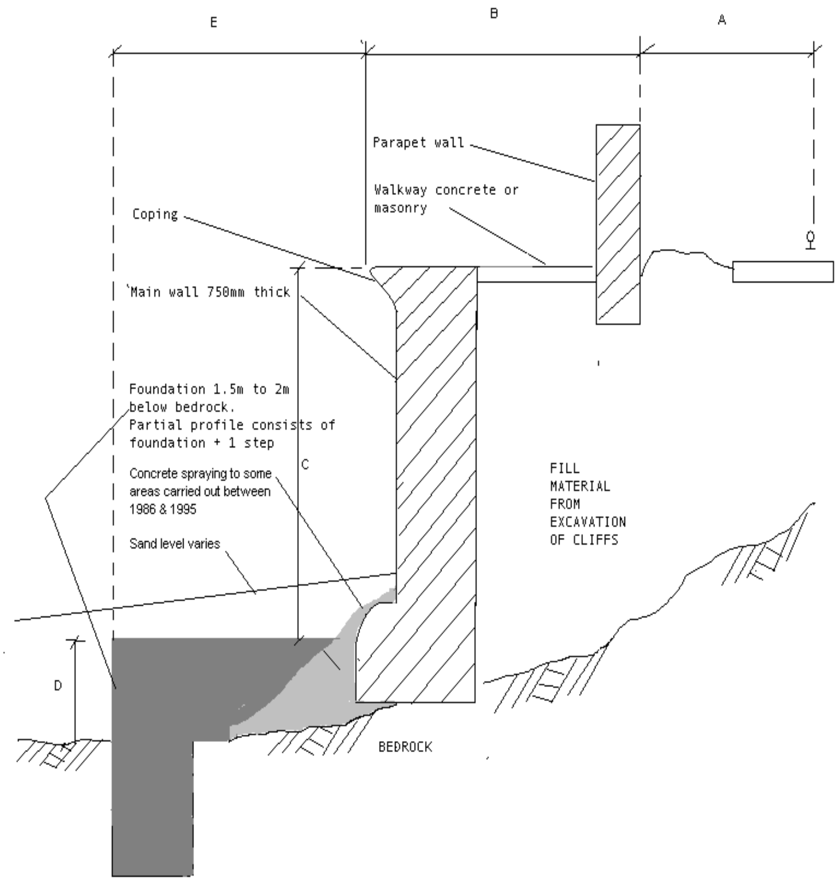
Mileage	A	B	C	D	E	Bedrock AOD	Walkway AOD	Track AOD
205m 48c	2.56	3.56	3.08	4.00*	2.41	N/A	5.82	5.75
208m 04c	2.19	3.31	2.97	4.00*	2.14	N/A	5.46	N/A

* Approximate
Mileages are from Paddington Station

Figure C7.2 Typical cross-section: full foundation strengthening (not to scale)

Partial Foundation Strengthening

Typical cross section with partial foundation strengthening to be found at :
205m 00c to 205m 36c
205m 70c to 205m 74c
208m 31c to 208m 55c



Mileage	A	B	C	D	E	Bedrock AOD	Walkway AOD	Track AOD
205m 25c	2.40	3.31	5.0*	1.00*	N/A	N/A	N/A	N/A
205m 71c	2.06	3.16	5.70	1.20*	4.00*	N/A	5.90	6.08
208m 39c	2.40	2.72	4.86	1.00*	N/A	N/A	5.93	N/A

* Approximate
Mileages are from Paddington Station

Figure C7.3 Typical cross-section: partial foundation strengthening (not to scale)

Figure C7.4 Full foundation strengthening, Smugglers Cove to Sprey Point

Figure C7.5 Full foundation strengthening – Rockstone footbridge

Figure C7.6 Partial foundation strengthening – Sprey Point to Teignmouth (note only outer edge of single step visible at the time photo was taken)

C8 CASE STUDY: Southern Felixstowe sea defences, Suffolk

Courtesy of Black & Veatch Ltd

C8.1 Identification of the problem

The town of Felixstowe is located on the Suffolk coast between the estuaries of the River Deben and River Orwell (Figure C8.1). The area is low lying and there is a risk of flooding due to high surges and tides. During the 1953 floods, 39 people died and 700 homes were damaged at Felixstowe.

The area that extends from the war memorial, just north of the pier, to Landguard Common is referred to as Felixstowe South, forming part of the 11 km coastal defences protecting the flood risk area of southern Felixstowe. This 2.7 km section is the most exposed part of the Felixstowe coastline; failure of the defences here could result in inundation of the entire southern Felixstowe flood risk area. The locations of the key points featuring in this case study are shown in the Black & Veatch 'Figure 1 Site Area' map provided as an annex to the case study.

Figure C8.1 Site location

The Felixstowe South frontage can be further considered in two parts, which are demarked by their respective jurisdictions. The northern part of the frontage is principally the responsibility of Suffolk Coastal District Council (SCDC) under the provisions of the Coastal Protection Act 1949, while the southern part is the responsibility of the Environment Agency under the Land Drainage Act 1991.

SCDC frontage

The earlier coastal defences consisted of a mass concrete seawall and promenade, constructed in 1903. The seawall was fronted by a mixed shingle–sand beach, with timber groynes encased in concrete, spaced at approximately 30 m centres (Figure C8.2). The beach was in a poor condition, sediment loss having been accelerated by wave reflection from the exposed vertical wall. Severe overtopping occurred regularly, damaging the promenade and causing localised flooding.

Figure C8.2 **Seawall and groynes at the SCDC frontage, 2005**

Approximately 20 m landward of the promenade, there is a 1983 floodwall of a sheet piled construction; this is clad in concrete with facing bricks (Figure C8.3). This floodwall was in good condition in 2006 and provided a standard of protection of 1 in 200 years. However, its performance as a flood defence depended on the closure of floodgates and, in any case, was conditioned by the frontline seawall which was calculated to have a standard of protection of less than 1 in 1 year based on critical overtopping for damage. Furthermore, the seawall was at imminent risk of collapse due to undermining. It was estimated that following failure of this seawall, failure of the crucial floodwall would occur within five years.

Figure C8.3 **Floodwall and gate**

The seawall, promenade and groynes are the responsibility of SCDC, while the floodwall and gates are the responsibility of the Environment Agency.

Environment Agency frontage

The Environment Agency frontage consists of a sheet pile toed concrete seawall. There were two variants on the type of structural defence:

- the Manor Terrace (northern) section of the wall was constructed in 1981 and has a concrete block revetment (Figure C8.4);
- the Landguard Common (southern) section of the wall was constructed in 1985 and has a stepped concrete apron.

The beach was partially controlled by a series of old timber and concrete groynes. The standard of protection of the Environment Agency floodwall was estimated to be 1 in 200 years, though this was calculated to reduce to 1 in 10 years in 100 years' time. Moreover, because of regular exposure above the beach level, the residual life of the steel piled toe was considered to be shortened through deterioration. Were the toe to fail then this would initiate failure of the whole flood defence structure.

Figure C8.4 Environment Agency seawall and flood defence, 2005

Recent problems

Failure of some groynes in 2003 on the Environment Agency frontage resulted in a rapid fall in beach levels (Figure C8.5), requiring urgent repair works to the groynes and blockwork.

Again, due to very low beach levels, the SCDC section of the frontage was undermined in May 2006 resulting in collapse of 150 m of the seawall (Figure C8.6). SCDC undertook emergency works in response to the damage, costing around £900,000 in total (Figure C8.7). Had action not been taken, the loose backfill behind the seawall would have washed away leading to rapid retreat, thus putting the floodwall at risk and indeed the low lying land behind. This temporary restoration was replaced with a permanent structural repair as part of the later main works.

Figure C8.5 Exposed toe piling at Environment Agency frontage, 2003

Figure C8.6 Rotating seawall, 2006

Figure C8.7 Emergency repairs to the seawall, 2007

C8.2 Appraisal

Chronology

The Lowestoft to Harwich Shoreline Management Plan (1998) recommended a policy of ‘Hold the Line’ for the southern Felixstowe frontage. Subsequent to the SMP, SCDC partnership with the Environment Agency, commissioned a Strategy Study (Halcrow 2003), together with a Strategic Environmental Assessment. The Strategy Study confirmed the Hold the Line policy, identifying the highest priority as the 2.7 km long frontage at South Felixstowe.

In 2004, Black & Veatch Ltd was appointed by the Environment Agency–SCDC partnership to further investigate options for the Southern Felixstowe frontage and to produce a Project Appraisal Report (PAR) (Black & Veatch 2007).

Scheme construction had been scheduled to begin in May 2006, however the PAR could not be approved as the earlier Strategy had not been approved by Defra. Consequently, the Strategy needed to be reviewed and updated in line with the then latest best practice guidance (Black & Veatch 2008). The coastal defence strategy review began in January in 2007 and approval from the Environment Agency was obtained in February 2008.

Technical considerations

Coastal studies were carried out as part of the PAR preparation (Black & Veatch 2005). The main conclusions of the studies were as follows:

- the shoreline (mean sea level) had been retreating landward by as much as 1.8 m per year (based on data from 1991 to 2003), equating to a rapid rate of beach lowering;
- the frontage is exposed to two significant wave sectors – from the south and from the north-east;
- between Cobbolds Point and the war memorial (part of the central Felixtowe frontage), the net annual longshore drift is from north to south;
- between the war memorial and Landguard Common (southern Felixstowe) longshore drift prevails in both directions – these drifts are highly variable, but generally the net drift is from south to north;
- day-to-day cross-shore sediment movement is relatively small except close to the pier, being at the focus of drifts from both central and southern Felixstowe – however, the beach is susceptible to significant drawdown during storm events;
- with limited sediment input, the sediment volume is reducing due to drift towards the pier, coupled with offshore loss due to cross-shore movement – the net result is an ongoing trend of beach retreat across the southern Felixstowe frontage.

While the overall standard of protection against coastal flooding in 2006 was high (1 in 200 years), the main concern was that the defences would fail due to lowering beach levels.

From a longer list of options, the following were taken forward for further consideration:

- timber groynes and beach recharge;
- rock groynes and beach recharge;
- fishtail rock groynes with visible timber root and beach recharge;
- fishtail rock groynes with buried root and beach recharge.

Environmental considerations

Consultation was undertaken throughout the development of the scheme with Environment Agency specialists, statutory consultees, interest groups and the public.

The scheme would reinstate the beach at Felixstowe and remove the existing health and safety hazards associated with the existing structures.

Landguard Common including the foreshore is designated as a SSSI, mainly for its vegetated shingle interest. It was therefore important to demonstrate to Natural England that the coastal processes would not be adversely affected by the works. There is also a local nature reserve immediately behind the promenade, close to the Martello Tower, designated for its vegetated shingle. Natural England provided a comfort letter giving its support to this scheme. An Appropriate Assessment was not required. However, under the Town and Country Planning (Environmental Impact Assessment) Regulations 1999, the proposed scheme was required to have a statutory EIA. An Environmental Statement (including an Environmental Action Plan) was submitted with the planning application. Planning permission was received in September 2005.

The Martello Tower located behind the frontage approximate mid-way along the frontage is a Scheduled Monument. The Landguard Peninsular is also a designated Scheduled Monument due to the presence of Landguard Fort and the associated field works. Access across this area was therefore prohibited during construction. A pre-construction walkover archaeological survey was also undertaken to record any features – none were found.

The identified environmental impacts of the scheme were mostly limited to the construction phase. Disruption to the local community and tourism industry of the town were to be mitigated as much as possible through careful planning and liaison with the local community. The promenade (a public right of way) was to be kept open during the construction phase with manned plant crossings.

Preferred option

The preferred option was a scheme designed to raise beach levels and to reduce beach lowering. It would provide a minimum standard of defence of 1 in 100 years in 100 years' time allowing for sea level rise. This was raised to a standard of 1 in 150 years, being justified by the reduction in the risk to life. This scheme would comprise beach recharge and a series of rock fishtail groynes.

Consultees were asked to respond to the question of whether the fishtail groynes should have exposed or buried roots. The exercise clearly showed that consultee preference was for groynes with buried roots. The feedback to the preferred option was positive. The configuration of the groyne field would lead to a more open beach, allowing continued access along the beach for both pedestrians and maintenance plant.

Economic considerations

Economic considerations examined the benefits of providing defence. The benefits consisted of the protection to a range of assets at risk which included 960 residential properties, 428 non-residential properties and the Port of Felixstowe (the largest container port in the UK). Although the scheme also provides protection to other assets (such as tourism, Landguard Common SSSI, Landguard Fort and associated Field Works Scheduled Monument and the Martello Tower Scheduled Monument), the economic benefit was not considered as there was ample justification from the port and properties.

The final economic analysis yielded a benefit/cost ratio of 29 as summarised in Table C8.1.

Table C8.1 Results of economic analysis

	Do Nothing	Rock fishtail, buried root and beach recharge
PV costs		£33,017,000
PV damage	£967,044,000	£5,422,000
PV benefits		£961,622,000
Net present value		£928,515,000
Benefit/cost ratio		29.05

C8.3 Outline design and consents

Outline design

The preferred option was to construct a new groyne field comprising fishtail rock groynes together with beach recharge. This option was found to be technically the most effective in terms of retaining beach material. It was also the option most favoured during consultation and was the most economic.

The proposed scheme involved the construction of 21 rock fishtail groynes, 200,000 m^3 of beach recharge material and 150 m of sheet piling to reinstate the failed section of seawall. Figure C8.8 (taken from the PAR) illustrates the basic form of the groyne cross-section.

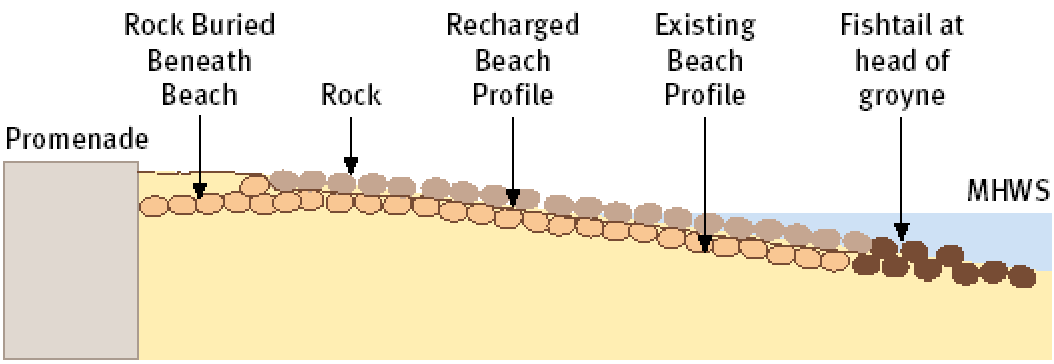


Figure C8.8 Fishtail rock groyne with buried rock root

Consents

The following consents were required:

- Agreement to the project was secured with Natural England. Section 28 consent under the Countryside and Rights of Way Act 2000 was granted by Natural England for working within Landguard Common SSSI.

- Scheduled Monument consent was obtained for the emergency access route within the Landguard Fort and Associated Fieldworks Scheduled Monument site.
- A FEPA license and consent under the Coast Protection Act 1949 for works below MHWS were required for elements of the scheme below MHWS.

C8.4 Detailed design

Figures C8.9, C8.10 and C8.11 show extracts from the detailed designs drawings of the layout and groyne structures. Figure C8.12 shows a design cross-section for the permanent repair over the 150 m failed seawall.

C8.5 Construction issues

The works were procured through the Environment Agency's framework agreement with Team Van Oord – identified as the preferred contractor following a mini-competition.

Undermining of the seawall

The scheme was designed through discussions with the contractor so that the beach recharge works would be undertaken before construction of the control structures. This would improve the buildability of the groynes and maximise access along the frontage under the tidal conditions.

This meant that the existing groynes and temporary rock protection needed to be removed ahead of beach recharge and without any beach control structures in place, thus exposing the seawall to increased risk of undermining. To manage this risk, the project team agreed a methodology that involved this work being undertaken in a staged approach so that at no time was the seawall in this vulnerable state for more than 12 hours (that is, one tide). The construction team also monitored the weather forecast carefully and delayed unnecessary exposure of the seawall if adverse conditions were likely. If a storm was expected or a section of wall was to be exposed for longer than one high tide, temporary protection was placed in front of the seawall at the toe.

There was also the risk that the wall would fail if the temporary rock armour protection was removed. Although this risk was assessed as low (the wall had been standing up before the rock protection was put in place), the works to remove the first section of rock protection were carried out under close control to minimise any collapse and to enable the team to react quickly. This work was all undertaken from the shore to minimise any loading of the promenade area behind, which had to be closed off to public access during these operations. The operation to remove the rock protection proved successful and no movement of the seawall structure was encountered.

Rock protection that had been placed further south around Manor End several years previously was left in situ as little was known about these works or the condition of the seawall behind. The risk of failure during the rock protection removal operation was therefore assessed as high.

New seawall

For the seawall reinstatement works, it was required that sheet piles were driven in front of the existing seawall. It was known that the seawall toe 'kicked out' and that over the failed section it was leaning slightly seawards. These details therefore needed to be determined on-site and the design refined to suit. When the seawall was fully exposed, it was surveyed to determine its actual cross-section and any deviation from the vertical so that the required line of the sheet piling could be determined.

Rock groynes

At the landward end, the groyne section extended down to the foundation of the seawall. Excavation below the foundation level was therefore minimised in design (Figure C8.11). Although the form of construction of the seawall beneath the promenade and behind the seawall face was not known, it was known from the behaviour of the seawall that it did not rely purely on material on the seaward side for support. The team were confident that excavation at the seawall would not lead to instability providing that the wall was not undermined. No issues were encountered in the groyne construction.

C8.6 Post construction issues – performance

The frontage has now been subjected to sufficient wave action to develop a dynamically stable beach profile and plan shape. The scheme performed well over its first winter.

Due to the very dynamic nature of this part of the coast, beach nourishment will be required every ten years to maintain a healthy beach and protection to the seawall.

Figure C8.13 shows the completed scheme.

C8.7 Lessons learnt

The PAR benefits assessment included consideration to the avoidance of risk to life using recently published guidance (HR Wallingford et al. 2006; Defra 2008). This consideration should be included in future PARs, in particular where flood risks are prevalent and the risks to people are significant.

The decision as to whether to recharge the beach first or install control structures always presents a logistical problem. Installation of the control structures first can create a draw on the existing depleted beach, thus aggravating erosion in some areas. However, the placing of recharge first can result in sediment loss until such time as the control structures are installed.

Having considered all the relevant factors at Felixstowe, it was decided to recharge the beach first. Contrary to the arguably more popular approach (control structures first), this method of working had a number of advantages which were realised in practice and which are worth noting here:

- This method allowed the works to be constructed essentially as a land-based operation, thus avoiding the use of marine plant.
- Health and safety risks were alleviated due to the substantially reduced requirements for wet working and tidal working.

- The improved scope for access across the new beach avoided having to provide an emergency access route across the SSSI vegetated shingle and the Landguard Fort scheduled monument.
- The improved access increased the working window, reducing the programme time (and hence less impact on community) and the cost.

Measures were taken to ‘over recharge’ the beach to take into account sediment losses incurred before the control structures were put in place.

On a more general note, the project highlighted the need to manage the risks carefully during both planning and construction. This is a continuous process that requires both forethought and the flexibility to deal with rapidly changing conditions, for instance the wall collapse that occurred in 2006.

C8.8 Acknowledgements

We would like to acknowledge the invaluable advice and assistance provided by Alexandra Schofield, Black & Veatch Ltd, in the preparation of this case study.

C8.9 References

Black & Veatch, 2005. *War Memorial to Landguard Common, Felixstowe Strategy Implementation Plan, Coastal Processes Report*, June 2005, for the Environment Agency and Suffolk Coastal District Authority.

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Black & Veatch, 2008. *Southern Felixstowe Coastal Strategy (update on Halcrow Strategy of 2003).*

Defra, 2008. *Defra Flood and Coastal Defence Appraisal Guidance. Social appraisal. Supplementary notes to operating authorities: assessing and valuing the risks to life from flooding for use in appraisal of risk management measures*. London: Defra.

HT Wallingford, Flood Hazard Research Centre and Risk & Policy Analysts Ltd, 2006. *R&D outputs: flood risk to people, Phase 2*. Joint Defra/Environment Agency Flood and Coastal Defence R&D Programme, FD 2321/TR2 Guidance Document. London: Defra.

Figure C8.9 **Extract from detailed design drawings – layout**

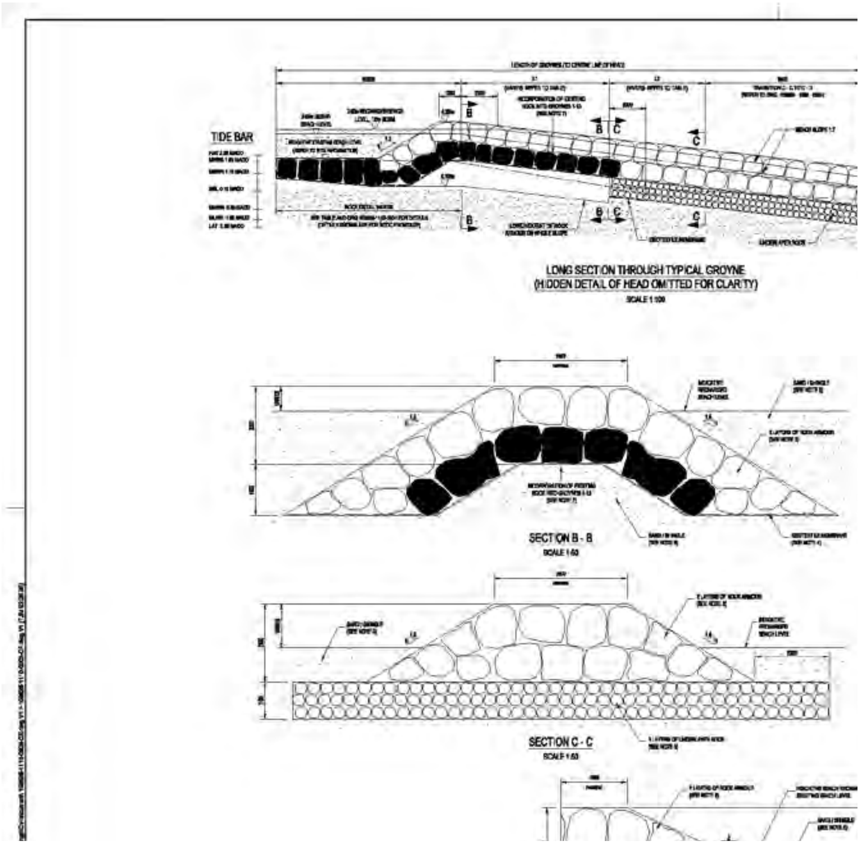


Figure C8.10 **Extract from detailed design drawings – sections through rock groynes**

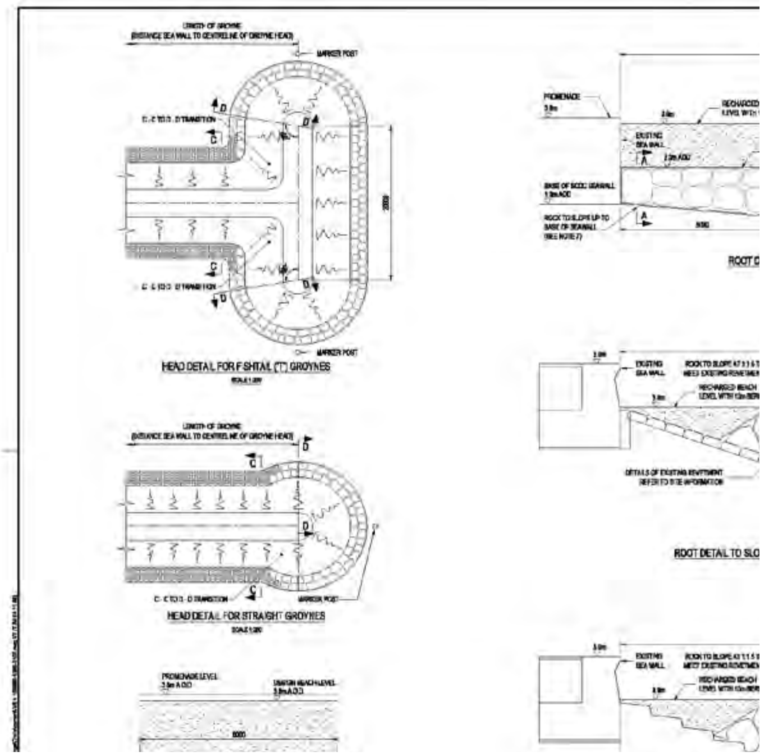


Figure C8.11 **Extract from detailed design drawings – root connections with existing seawall/toe**

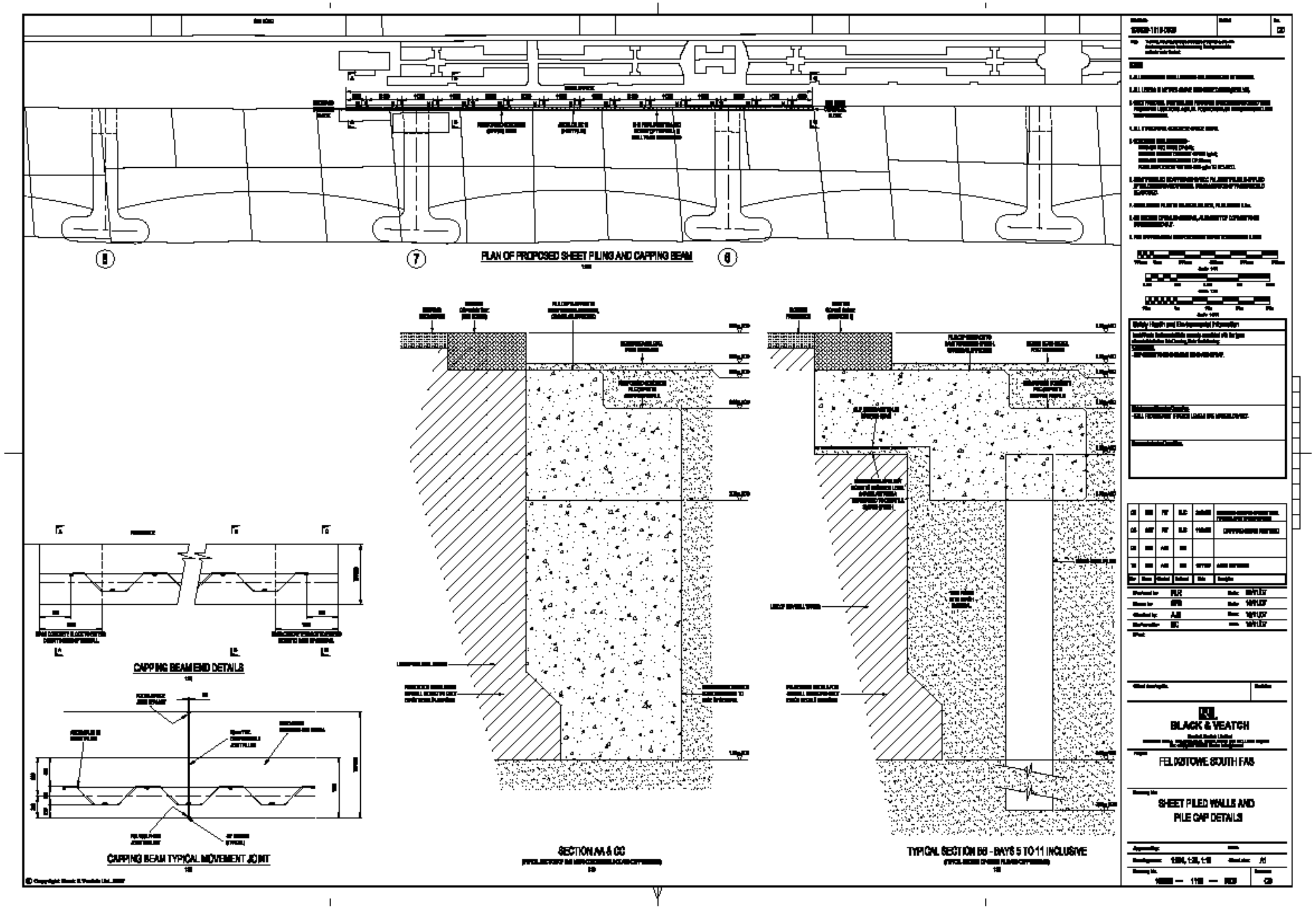


Figure C8.12 **Extract from detailed design drawings – detail of permanent wall restoration**

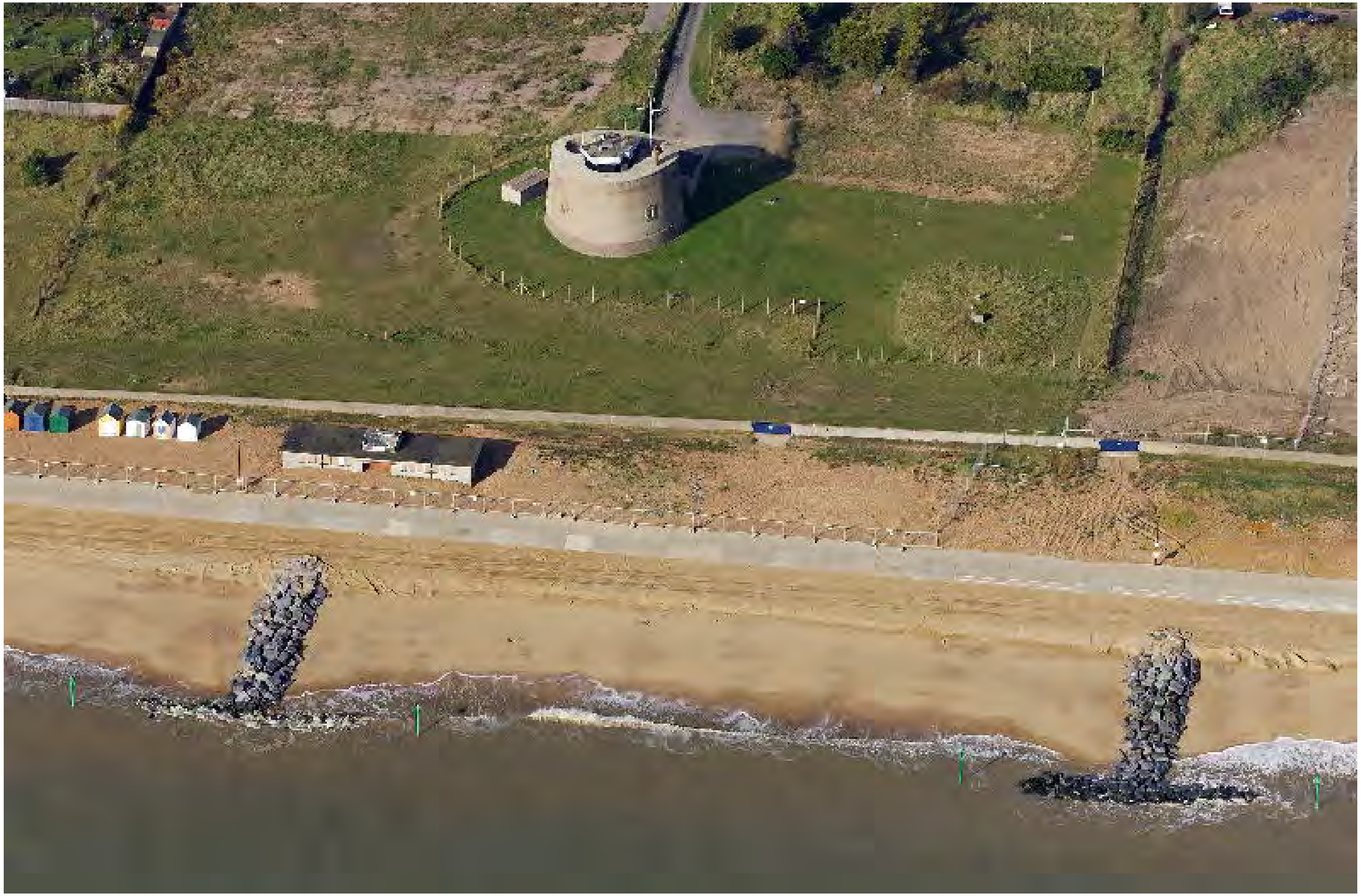
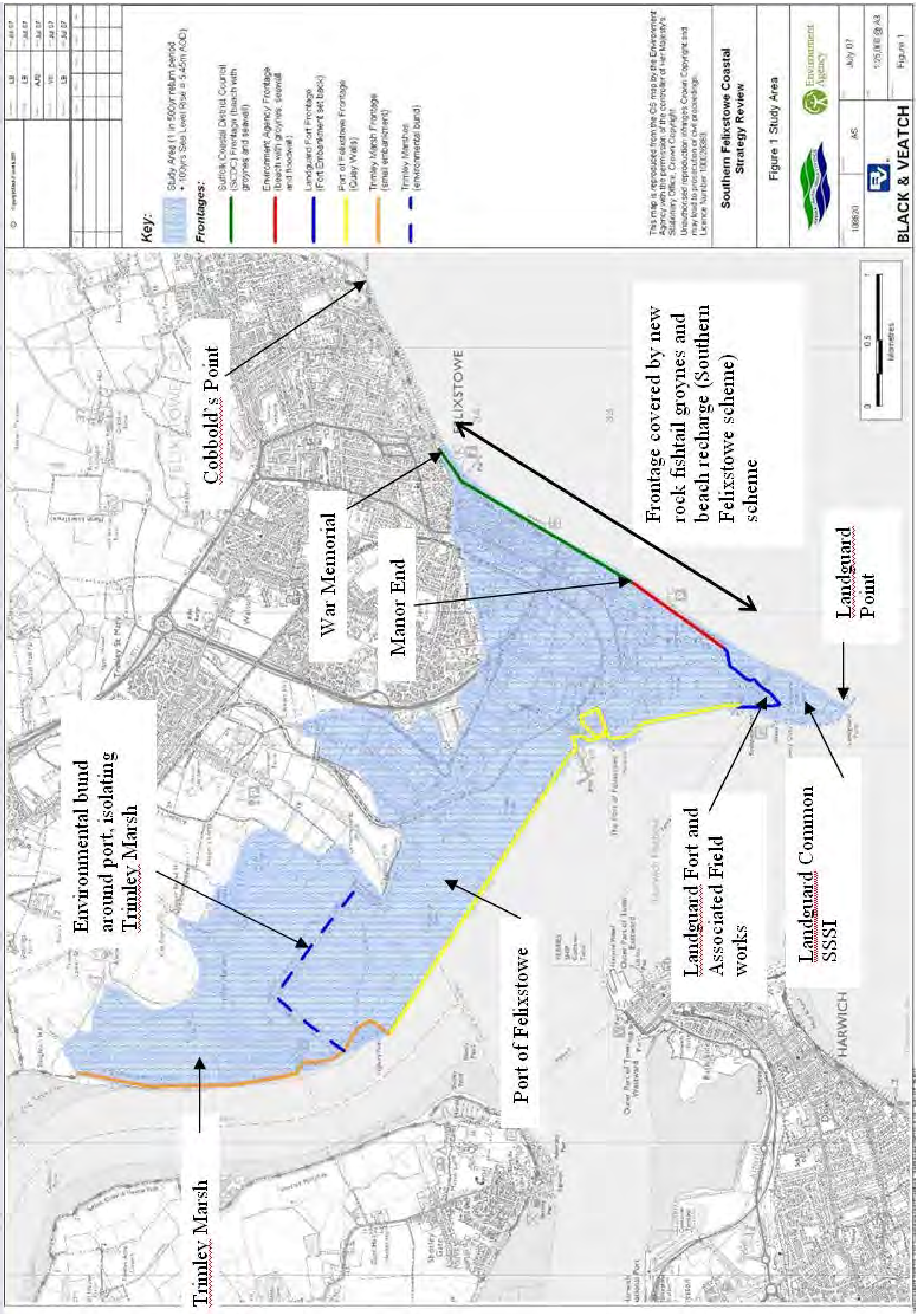


Figure C8.13 Southern Felixstowe – the completed scheme



Annex: Black & Veatch Figure 1 Study area

C9 CASE STUDY: Clayton Road – Selsey

Courtesy of Royal Haskoning

C9.1 Identification of the problem

Selsey Peninsula is a low=lying headland situated in the lee of the Isle of Wight, just to the east of Chichester Harbour (Figure C9.1). Selsey is popular with holidaymakers who are mainly attracted to the caravan parks in the area. At West Beach, the subject of in this case study, the frontage is occupied by private properties that extend to the waters' edge.

Figure C9.1 Location map

Selsey Peninsula is exposed to waves from the English Channel together with waves originating in the Atlantic Ocean. The sheltering effect of the Isle of Wight means that severe wave attack due to oceanic waves is very sensitive to wave direction. In addition to a complex wave regime, there are strong currents that sweep around the end of the peninsula. Sediment is driven onto Selsey Bill from an offshore bank. However, the orientation of the shoreline in relation to the complex and sensitive hydraulic regime is such that most of the sediment is driven up the east side of the peninsula, with West Beach receiving only intermittent feed. The natural consequence has been for the west shore, where the private properties are located, to gradually retreat.

There has been a sea defence at West Beach for some considerable time. In response to falling beaches, the seawall was reconstructed during the 1950s and 1960s. The defences originating from this time are made up of a steel sheet pile toe surmounted by concrete steps and/or a battered concrete apron with a wave return wall.

It was concluded at the time of these earlier works that it would be necessary to realign the wall to produce a more sustainable longer term solution (Lewis and Duvivier 1950).

However, there were a number of lengths of substantial private defences; for the sake of expediency these were left in place, that is, to seaward of the then newly adopted defence line. These discreet protrusions subsequently formed local promontories (Figure C9.2).

The predicted continued beach lowering, aggravated by wave reflection from the vertical seawall, resulted in scour at the toe of the protuberant wall sections. There were no failures of the wall as such but the promontories required a high level of maintenance. Moreover, the risk of the seawall being undermined was increasing with time.

The principal threat to the seawall was toe scour. Failure of the toe would have led to failure of the whole seawall and ensuing land erosion. In view of the impending threats to infrastructure and consequent economic loss, it was concluded in the late 1980s to carry out coastal defence studies and prepare a case for improved coast protection at Clayton Road.

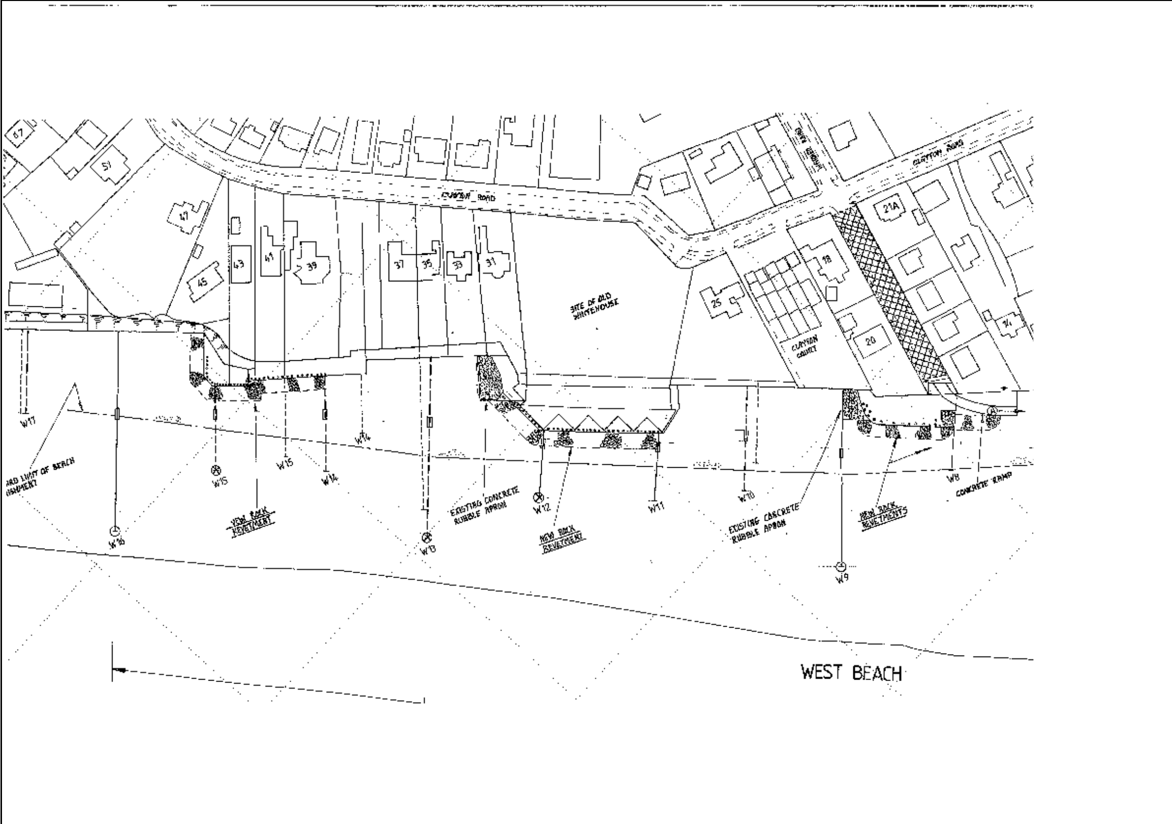


Figure C9.2 **Site plan showing the promontories created by selectively holding the line**

C9.2 Appraisal

During the appraisal process, a number of options were considered for continuing protection to the Clayton Road frontages. On the basis that retreat of the defence line would not be acceptable, the options tended to focus on alternative means of securing a hard defence line along the existing, albeit convoluted, alignment. These options therefore included extending the apron to a new lower level steel sheet piling and the (then) comparatively novel use of a rock revetment applied to the toe.

Technically, a major consideration was how to fit a given type of construction to suit the complicated seawall toe alignment. This called for a design that was suitable in its application and detail.

From an environmental perspective, a rock revetment was preferred as it reduced wave reflections from the wall, thus providing an improved environment for the natural build up of beach levels (or more precisely, reduced scour). Despite common concerns regarding the placing of rocks on the beach, the public generally accepted the scheme.

In economic terms the principles of best value applied. In essence this meant targeting low cost works that would be funded by under the Coast Protection Act 1949. Alongside this, there were opportunities to combine the works as part of a larger groyne refurbishment scheme, thus yielding economies of scale. A more formalised economic appraisal, project prioritisation and procedures that are nowadays required for grant qualification did not apply.

Consultation on the appraisal was principally with/through Chichester District Council.

The preferred solution was arrived at on the basis of:

- relatively low cost;
- flexibility;
- environmental acceptability (or improvement).

A rock revetment was found to offer the best overall solution in respect of the above objectives.

C9.3 Outline design

Input parameters to the outline design process comprised:

- the existing seawall cross-sections;
- newly surveyed beach levels;
- tide levels and wave heights.

Royal Haskoning's predecessor in the UK, Posford Duvivier (and previously Lewis and Duvivier), had worked on the frontage for some 40 years and so was well acquainted with the ground conditions and previous engineering at the site.

Further to the basic parameters needed for technical design, other important considerations included site access and the availability of suitable rock. There were few access points to the beach within reasonable distance of the works and vehicles would have to negotiate groynes to get to the site. Rock would be obtained from Frome in Somerset, the individual limestone boulders being delivered on flat top transporters by road. Cost considerations, due to the small scale of the project, ruled out the prospect of procuring more durable rock from more distant sources (for example, granite from Norway).

Figure C9.3 shows a typical cross-section through the revetment. The essential principles of the design concept were as follows:

- the revetment had to have low wave reflectivity in order to reduce bed scour which had previously been aggravated by the presence of the vertical seawall – this was achieved by using a double layer of armourstone;

- the rock mound was required to provide passive support to seawall toe – this was achieved by setting the rock profile into the beach at the face of the toe pile;
- the revetment armour was to be very stable in order to prevent movements against the existing seawall steps;
- the structure cross-section had to be sufficiently compact to fit into the limited space available.

Consents for the scheme were obtained from:

- The Crown Estate as the land owner;
- FEPA licence administrators;
- planning authority.

C9.4 Detailed design

The scheme was designed by Posford Duvivier (now Royal Haskoning). The design was developed and fully detailed for construction tender purposes. In addition to the rock structure, the detailed design included intermittent concrete blocks ('Dragon's Teeth') cast onto the upper existing concrete apron (certain sections only). These provided a back-stop to the boulders at the crown of the rock mound.

Four different cross-sections were prepared which encompassed the variations in the existing wall profile and beach variations. The designs were of sufficient detail that there was effectively no requirement for the contractor to adapt or develop the design either before or during construction.

The first construction contract was awarded to Z Peskett & Sons Ltd of Littlehampton in 1990. Since then there have several additions to the toe defence works such that now some 50 per cent of the whole defence length has toe protection using rock.

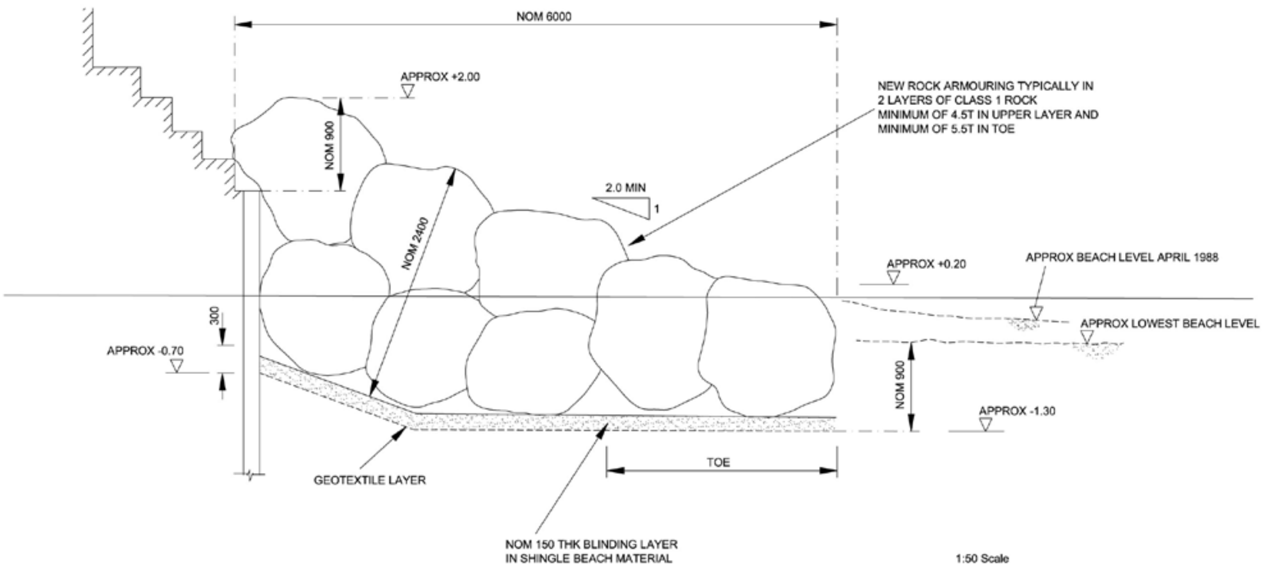


Figure C9.3 **Typical cross-section through the revetment**

C9.5 Construction issues

The construction was entirely land-based including the use of plant from the beach. Some of the limestone rocks split during handling and placing. Usually, damaged rocks were discarded. Rocks with hairline cracks that were well embedded within the structure were left in place where it was felt that this did not compromise the security of the structure.

Despite the complexity of the site geometry, no changes from the tendered design were found to be necessary.

Careful placing of the boulders together with protection provided by the Dragon's Teeth meant there was no damage to the existing concrete steps or apron.

Figure C9.4 The completed armour revetment

C9.6 Post construction issues

The scheme is included in an annual asset survey. This entails a qualitative inspection of the general condition of the rock mound including observations on settlement, wear and tear.

Since its construction in 1990, the protection measures appear to have performed well in stalling scour at the toe.

The main problem has been attrition of the limestone rocks through abrasion by the hard flint fraction of the beach sediments. As a result, the exposed edges of the rock armour units are becoming rounded. There also appears to have been some settlement in some sections. The rock interlocks have, however, remained reasonably sound.

In places, there appears has been some localised increased wear of the steps and apron where overtopping water jets have penetrated through gaps between the rocks and apron. However, generally the rock revetments have protected the steps and apron from excessive wear.

C9.7 Lessons learnt

The simple design comprising largely single size rocks with no filter layers has worked very well.

The durability of the limestone may eventually limit the lifespan of the revetment. However, whether the use of more expensive granite would have represented better value in the long term is open to question.

C9.8 Acknowledgements

We would like to acknowledge the invaluable advice and assistance provided by Simon Howard, Royal Haskoning, and David Lowsley, Chichester District Council, in the preparation of this case study.

C9.9 References

Lewis and Duvivier, 1950. *Report on coastal protection.* Report to Chichester Rural District Council.

C10 CASE STUDY: Fort Wall, Canterbury

Courtesy of Canterbury City Council

C10.1 Identification of the problem

The site is located just to the east of Herne Bay on the north Kent coast (Figure C10.1) and is landmarked by St Mary's Church immediately to the east. The subject of the case study is known as Fort Wall; a short defence of 80 m in length.

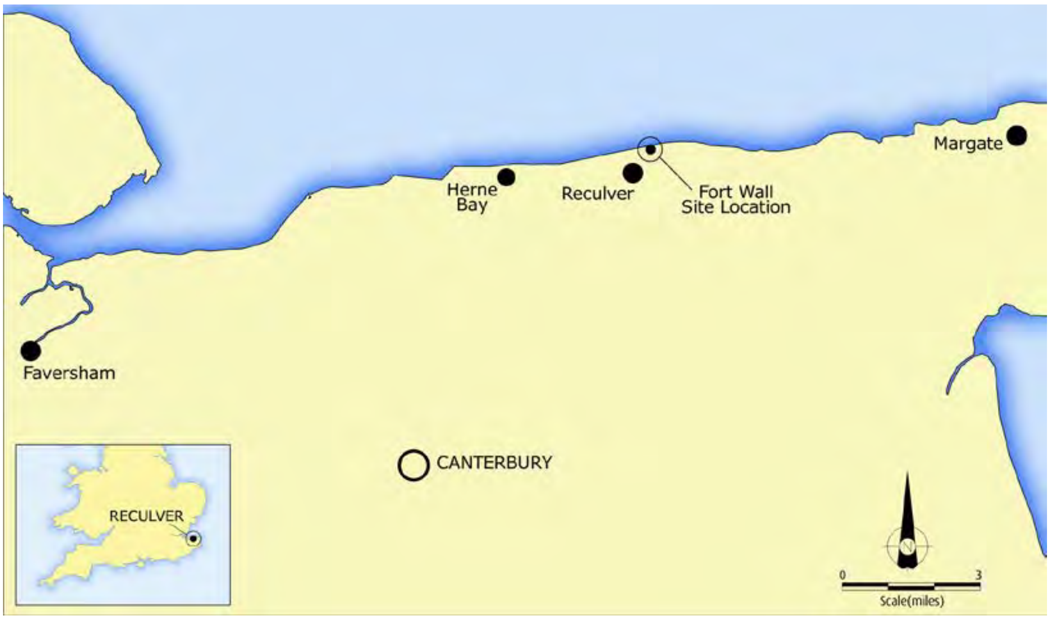


Figure C10.1 Location of Fort Wall in north Kent

Fort Wall, which is owned by English Heritage, protects the archaeological remains of a Roman fort. The historic site is located on a slight hill but the land behind falls away to about high tide level. Hence, in addition to securing the archaeological remains, the Fort Wall defence together with the immediate land strip provide flood protection to the lower area behind.

The site surface geology comprises a mixed sand and shingle beach overlying Thanet sandstone (at about 1 m below MHWS). The sandstone is easily eroded once exposed, though there is no quantitative evidence of sandstone downcutting. However, Roman ruins have been discovered 2 km out to sea implying, in very approximate terms, a horizontal rate of coastal retreat of about 1 m per year.

The history of the seawall that preceded the present structure is relevant to the case study and is outlined below. Built in 1965 by the then Ministry of Works, the earlier wall (Figure C10.2) comprised:

- a mass concrete block (formed in 6 m lengths) founded on the sandstone;
- bearing onto this from above was a revetment (variable slope) which contained the mixed fill embankment.

The revetment consisted of a concrete bed over which were laid Ragstone blocks.

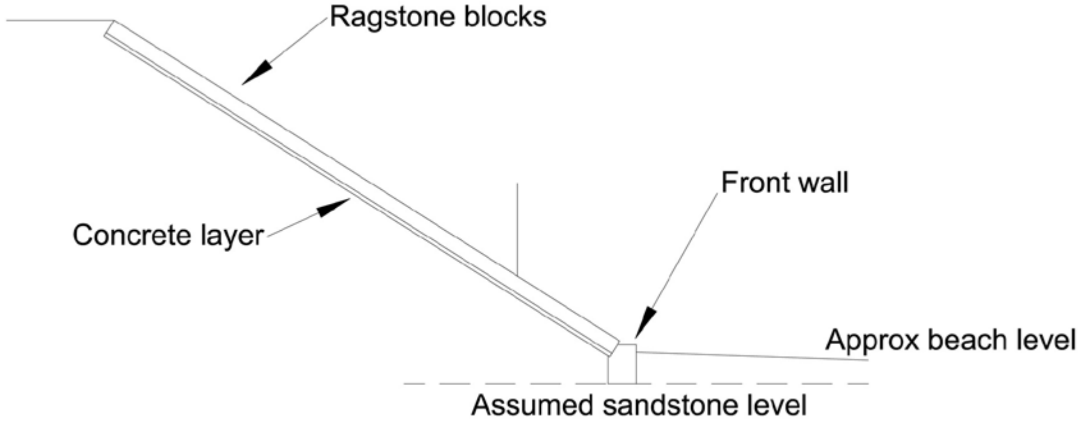


Figure C10.2 Sketch of former wall section (1965 construction)

Initially, beach levels at the site had been healthy. In the early 1990s, however, a major scheme (not by Canterbury City Council) including a large terminal groyne was put into place coming to within 100 m east of the old defence. Though not well recorded, beach lowering at the Fort Wall section ensued.

A storm with an estimated return period of 10 years occurred in February 1996, resulting in substantial destruction of the old wall. About half the Ragstone blocks came off the revetment. It appeared that the large toe blocks had just turned over rather than being undermined. An estimated 4 m of erosion occurred within four days as the mixed fill became exposed to subsequent wave action (Figure C10.3).

Figure C10.3 Failure damage, 1996

An immediate fix was needed to prevent further retreat, which might have put the archaeology at risk. This was achieved by backfilling the remaining toe blocks using pumped concrete and then carefully shoring up the exposed face (Figure C10.4). These emergency works were carried out in February 1996 and consisted of about 100 m^3 of mass concrete to the failed toe and 90 m^2 of sprayed concrete with mesh reinforcement to the exposed slope face. The cost of emergency repairs was about £27,000 in 1996. Further works to the slopes were again necessary in May 1996 with further concrete infilling (costing about £10,000). These emergency works then remained intact until the permanent works commenced in August 1998.

Figure C10.4 Emergency repair works, 1996

C10.2 Appraisal

The discussion below relates to the long-term repair works carried out from 1998 onwards.

Application was made to Defra in March 1996 to undertake a coast protection study for this and the adjacent Reculver frontage. However, Defra required a full coastal defence strategy plan to be carried out to also include the Environment Agency frontage to the east. This Strategy, covering a total frontage of 5 km was carried out in-house by Canterbury City Council (CCC); it began in June 1996 and was completed in October 1997.

The Strategy covered a full range of options for the full frontage. Options for the Fort Wall section included the use of timber groynes with nourishment, this being required due to lack of any natural feed. This would have to be a pocket beach including a terminal groyne which would, therefore, have had its own downdrift effect. Difficult access would have made the scheme very expensive; moreover, the fact that it was a small scheme would have made the necessary use of large marine plant and operations uneconomic. As beach amenity was not an important issue at this site, a hard defence was the preferred option.

Other significant issues to consider included:

- the scheme was required to protect archaeology and so this introduced important site access considerations;
- construction had to be during summer to avoid disruption to overwintering birds.

Application was made to Defra in November 1997 for implementation of a coast protection scheme, as recommended in the Strategy, apportioned between CCC and English Heritage (385 m) and the Environment Agency (95 m). Defra scheme approval for both the CCC and Environment Agency lengths was received in March 1998.

C10.3 Outline design

The design took into account the (then) lowered beach profile and wave attack which was depth limited at the new structure. The design of the Fort Wall section was to comprise:

- steel piling driven into the sandstone;
- concrete capping that would encompass the old concrete wall;
- a rock armoured revetment in front of the piled wall;
- a replacement concrete and Ragstone composite revetment.

C10.4 Detailed design

CCC undertook the site investigation to determine sandstone levels and the strength of the material, which turned out to be very varied. Over the length of the works the sandstone base varied by only about 0.6 m.

Detail design and contract preparation for the whole project (including the Environment Agency and English Heritage lengths) was carried out in-house by CCC. Figure C10.5 shows details of the new Fort Wall.

C10.5 Construction issues

The tender for construction was issued in May 1998 and works commenced on-site in August 1998. Harbour & General were the appointed contractor. The contract period was 32 weeks but the works were actually completed within 20 weeks.

As access overland was difficult, the armour rock had to be delivered be sea (Figures C10.6 and C10.7). Limestone rock armour was sourced from Boulogne.

Pile driving into the sandstone went well (Figure C10.7). Where possible the original Ragstone blocks were salvaged and reused.

Figure C10.8 shows the final product.

The contract value for the whole project was £1.18 million, of which the value of Fort Wall works was £220,000. The main quantities included:

- 1,510 m^3 of 3–6 tonne armour rock;
- 190 m^3 of reinforced concrete;
- 430 m^2 of sheet piles driven to a depth of 3 m.

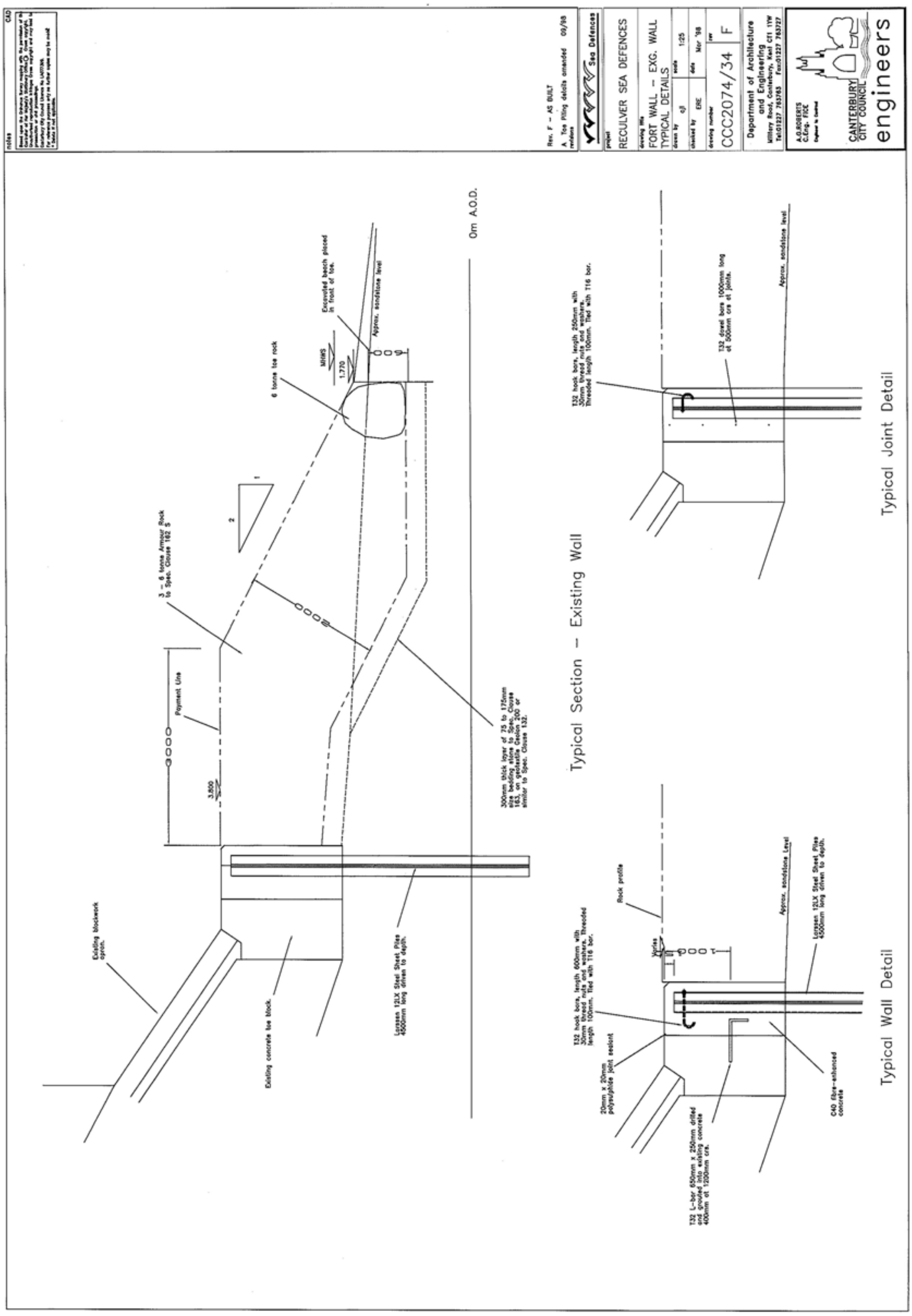


Figure C10.5 **New design (extract)**

Figure C10.6 Rock delivery

Figure C10.7 Pile driving

Figure C10.8 Final construction

C10.6 Post construction issues

A better beach has built up since installation of the rock revetment, most probably due to the reduced reflection (compared with that of the earlier vertical structure). Overall, the scheme has been very successful.

The site is now monitored regularly (three times per year) as part of the south-east regional strategic monitoring programme. Little maintenance has been needed other than repointing of the Ragstone blocks from time to time.

C10.7 Lessons learnt

The 1965 design could not withstand the combined effects of a lowered beach and a 10-year return period storm wave attack. Alongside this, better beach monitoring should have been in place to identify the possibility and imminence of the 1996 collapse but this was not a major consideration at the time.

The new design learnt from the earlier failure and accounted properly for appropriate design conditions.

Because of the very small extent of the works, it was packaged with other schemes to reduce costs due to mobilisation. This made for an economically attractive project and, indeed, a positive lesson for future schemes.

C10.8 Acknowledgements

We would like to acknowledge the invaluable advice and assistance provided by Ted Edwards, Canterbury City Council, in the preparation of this case study.

C11 CASE STUDY: Prestatyn, North Wales

C11.1 Background history

The coastline between Rhyl and Prestatyn has been eroding for several decades through a combination of reduced sediment supply and coastal squeeze. At the end of the 19th century, some two million tonnes of gravel were removed from the beaches in the Rhyl area (updrift and to the west of Prestatyn) to provide ballast for building Liverpool Docks. As a result, the pebble storm beaches that once extended along much of this frontage have largely disappeared. Tourist pressures have also led to the reclamation of large stretches of marshland for the construction of holiday camps, golf courses and so on. The holiday camp and housing at Prestatyn are situated close to the shoreline and are protected by a seawall. To the east of this wall, there is a line of dunes that have been eroding.

Prestatyn was first protected by a stepped concrete seawall in 1960. At the same time the foreshore was protected by a series of long timber groynes. Already by the 1970s, the beach in front of this wall had fallen significantly. The flatter gradient allowed the ridge and runnel systems, common on this wide foreshore, to migrate shoreward. The increased water depth at the toe of the wall then caused strong overtopping. The wall itself was at risk of foundation failure. The sand transport was then concentrated at some distance seawards of the wall, effectively starving the dunes immediately downdrift of the sand supply.

The photograph shown in Figure C11.1, which was taken in early 1990, shows the tidal runnel very close to the wall. Note that the runnel extends a considerable distance alongshore, cutting across several timber groynes. All that remains at the toe of the wall is a narrow strip of pebbles. Some emergency works in the form of a fillet of rock armour-stone can be seen at the toe of the wall. The growth of algae on the concrete steps indicates frequent wave overtopping.

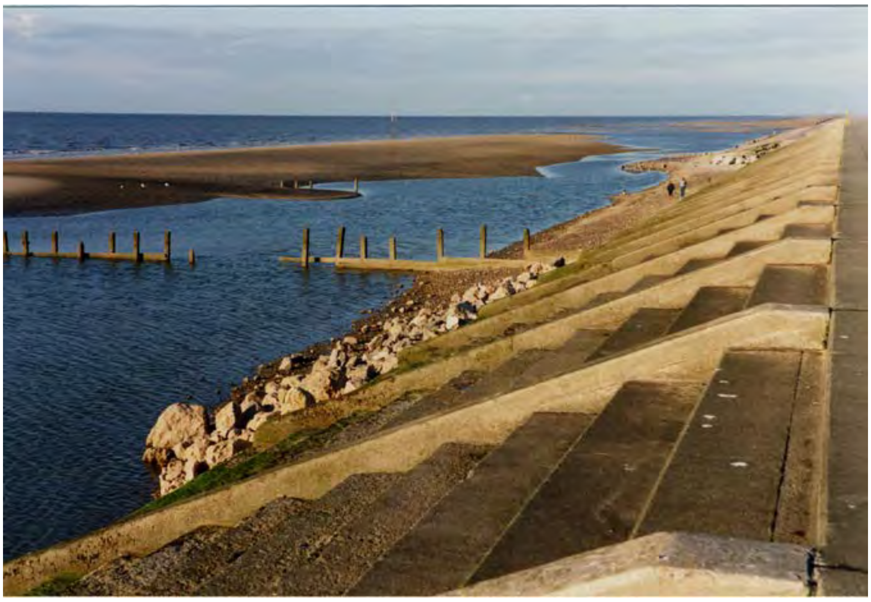


Figure C11.1 Toe scour in front of stepped concrete seawall, 1990

C11.2 Mitigation measures

This frontage has required regular maintenance. In the 1980s, field investigations were carried out into the problems of wave and tidal induced scour and the strength of tidal currents over the foreshore. Rock groynes were constructed to reduce inshore tidal currents. This improved beach levels over the foreshore, but the wall toe remained vulnerable to wave attack.

Following substantial damage in 1990–1991, a major scheme was implemented, including massive sand recharge coupled with the construction/upgrading of the rock groynes. In addition, the vertical face of the seawall below the concrete stepped face was replaced by an asphaltic sloping apron.

C11.3 Performance of mitigation measures

The increased height and width of the upper foreshore has resulted in the disappearance of the ridge and runnel features from in front of the seawall. The high foreshore levels have also removed the problems of wave overtopping. Figure C11.2 shows the swash limit along the line of the new sloping revetment. There is some sand build up above the revetment and on the seawall steps. The amenity value of the promenade, at the crest of the wall, has also been greatly improved.

Figure C11.2 Sloping asphalt apron and rock groynes in front of steeped concrete seawall

C11.4 Other comments

A scheme of this type needs to be monitored carefully. Not only do beach levels in front of the seawall need to be checked regularly, but the evolution of the downdrift beaches must also be assessed.

C12 CASE STUDY: Colwyn Bay, North Wales

C12.1 Background history

Figure C12.1 Site location

Colwyn Bay is a popular tourist resort situated on the North Wales coast (Figure C12.1). The sand beaches in this area have been eroding as a result of coastal squeeze and a lack of sediment supply from the west (updrift). The reasons for this are also described in the case studies for Penrhyn Bay (C14) and Rhos-on-Sea (C13). Due to the fallen beach levels, the promenade and the road immediately to the landward have been affected by heavy wave overtopping.

The construction of sea defences at Colwyn Bay dates back to the late 19th century. The masonry seawall has, over the years, suffered considerable damage, requiring extensive repairs and reconstruction. In the 1970s there was a groyned upper beach of shingle, with an almost continuous sand cover over the flatter, lower part of the beach. Subsequently the groyne system fell into disrepair, allowing beach levels at the wall toe to fall. Thus, by the 1980s the shingle beach had largely disappeared from the wall toe, causing foundation problems (Figure C12.2). In addition, the falling sand levels had exposed the underlying pebbles over much of the lower foreshore.

Figure C12.2 Beach lowering in front of vertical seawall, Colwyn Bay

C12.2 Mitigation measures

In 1987 a rock berm was constructed along the most severely affected stretch of wall. This encouraged some beach build up in the immediate vicinity of the wall toe, assisting seawall stability. In the early 1990s, a number of low rock groynes were constructed from the wall out to the low water line. These project no more than 1 m above the beach surface and hence are not visually intrusive. Since then, other sections of wall have required additional toe protection, usually comprising a rock toe or sheet piling and concrete infills.

C12.3 Performance of mitigation measures

The Rhos-on-Sea breakwater may be responsible for trapping in its lee what little shingle drift there is. Therefore, the construction of the rock berm has encouraged sand rather than shingle accretion. Furthermore, the sand cover has increased significantly seaward, so that only small areas have the underlying pebbles exposed.

This scheme is a good example of how relatively modest defences can be used effectively to improve beach levels. The photograph in Figure C12.3, which was taken in 2002, shows the sand build up at the foot of the seawall. The long rock groynes can (just) be seen in the background, while a redundant timber groyne can be seen in the middle distance.

Figure C12.3 **Rock berm in front of vertical seawall, 2002**

C13 CASE STUDY: Rhos-on-Sea, North Wales

C13.1 Background history

Figure C13.1 Site location

Rhos-on-Sea is situated on a small headland at the eastern end of Penrhyn Bay (Figure C13.1). The headland is a focal point for wave action and the residential development on low-lying land to the landward was at risk from heavy wave overtopping in the recent past.

As described in the Penrhyn Bay case study (C14), there is a shortfall in the supply of beach sediments in Penrhyn Bay and to the east. The headland rock promontory of the Little Orme acts as a groyne, cutting off the majority of supply of shingle from the west, as well as reducing the amount of sand supply. By contrast, the smaller promontory of Rhos Point at the eastern end of Penrhyn Bay had not prevented beach material from being transported eastwards (downdrift) into Colwyn Bay.

The construction of sea defences within Penrhyn Bay in the late 19th and 20th centuries effectively cut off the supply of sediments from the erosion of boulder clay cliffs at the western end of the bay. This left only a small supply of sand from the nearshore zone, feeding around the Little Orme headland in suspension. Beach levels had therefore gradually deteriorated both within Penrhyn Bay and around Rhos Point itself.

The seawall around Rhos Point was built in the 1860s and, prior to the breakwater protection scheme described here, had been breached and repaired in the recent past. Further falls in beach levels were anticipated, similar to the progressive deterioration of the beaches that had taken place earlier in Penrhyn Bay. In view of the falling beach levels, upgrading the existing sea defences, for example, was considered to be insufficient as a long-term solution to the problems that had developed around Rhos Point.

C13.2 Mitigation measures

Following wave overtopping studies by HR Wallingford, a rock armour breakwater was constructed off Rhos Point in 1983. This was located opposite low-lying land to the south of the Point. Rock left over from the breakwater construction was used to construct a short groyne on the coast immediately to the north, to encourage material to collect around the Point itself.

C13.3 Performance of mitigation measures

The breakwater has eliminated the problems of wave overtopping that were becoming increasingly more serious to the south of Rhos Point. The sheltered conditions in the lee of the breakwater have allowed small boats to anchor there (Figure C13.2). This has been possible because the breakwater is sited some distance away from the wall, but not so far offshore that its sheltering effect would be significantly reduced. The low groyne has been overtopped by beach material and shingle and sand have tended to collect in the lee of the breakwater, further reducing any potential risk of wave overtopping (Figure C13.3).

In view of the 7 m tidal range on spring tides, the offshore breakwater is a large structure. As well as trapping the small volume of littoral drift from Penrhyn Bay, the breakwater has also attracted a reverse westerly drift of material from Colwyn Bay to the east. In addition, the high degree of shelter has attracted a small amount of mud from offshore. The accumulation has not affected the development of sailing leisure facilities in the lee of the structure.

Figure C13.2 View to the east showing usage by small boats

Figure C13.3 **View to the west showing beach recharge**

C14 CASE STUDY: Penrhyn Bay, North Wales

C14.1 Background history

Figure C14.1 Site location

Penrhyn Bay is situated to the east (downdrift) side of the headland of the Little Orme (Figure C14.1). The problems of beach erosion in Penrhyn Bay are primarily due to a lack of contemporary sediment supply. Historically, the main source of beach material for the bay was the erosion of boulder clay cliffs on the east side of the Little Orme and a (potential) feed from the nearshore seabed.

Since the beginning of the 20th century, the construction of sea defences has progressively cut off the supply of beach material derived from the cliff erosion. The seawalls themselves, the earliest of which dates back to the 1900s, have contributed to coastal squeeze, causing further deterioration of beach levels. The present seawall structures date from the period 1945–1960.

By the 1970s, however, the beach conditions had deteriorated to such an extent that the beach material had largely disappeared from the eastern (downdrift) part of Penrhyn Bay. Even in the more sheltered western part of the bay there was little beach material remaining. As a result, groynes that had been introduced to try and control beach movement were no longer effective and, having not been maintained, had deteriorated (Figure C14.2). The lack of beach cover and the serious overtopping in the exposed central frontage resulted in the construction of a short length of timber breastwork to the seaward of the existing wall in an attempt to reduce wave overtopping. These measures were only partially successful and did not deal with the root cause of the problem, which was the lack of sediment supply.

Although partly sheltered from the west, the refraction/diffraction around the headland of the Little Orme and the edge-wave effects, as waves propagate alongshore along the line of the seawalls, produces a significant eastward littoral drift within Penrhyn Bay. Since Penrhyn Bay is not fully enclosed at its eastern end, this has produced a gradual emptying of sediments out of the bay.

As a consequence the walls were in danger of being undermined and significant overtopping of the defences occurred during storm events.

In the late 1980s, investigations were carried out to identify a scheme that would remedy the situation.

Figure C14.2 Lowered beach exposing shore platform

C14.2 Mitigation measures

The scheme introduced at Penrhyn Bay in 1989–1990 consisted of the construction of two fishtail-type rock groynes, with the beach between the groynes recharged with a mixture of sand, shingle and cobble sized material. The finer material was used in the more sheltered parts of the bay and the coarser material towards the eastern end of the bay which was more exposed. The groynes provided terminal structures to keep the recharged material within the artificial embayment.

Along the downdrift frontage to the east, where beaches remained low, an alternative form of construction was adopted and a new full height armour stone revetment was constructed in the mid-1990s. Both schemes made use of locally available quarried rock in the structures and locally available sand and quarried rock in the artificial nourishment.

C14,3 Performance of mitigation measures

This innovative scheme has been very successful. The combination of the artificial beach recharge and the fishtail groynes has effectively reduced the erosion of the upper beach and contained the beach sediments within Penrhyn Bay. The artificially formed beach has generally a sufficiently high crest to prevent waves reaching the seawall.

The beach was graded from sand in the west to an (artificial) cobble beach formed of quarried rock in the east. Movement of the material has caused a mixing of sediments to occur with mixed sand and shingle tending to stay in the (sheltered) western corner of the bay (Figure C14.3).

Figure C14.3 **View of beach recharge in western part of Penrhyn Bay**

Further east, material from the centre of the embayment tends to be moved towards the easterly groyne where it can overtop the root of the groyne structure (Figure C14.4). Regular recycling of material back towards the centre of the bay (once or twice a year) is carried out in order to maintain Standards of Protection.

Figure C14.4 **View from easterly groyne (CEUK)**

C14.4 Other comments

A scheme of this type alters the character of the beach significantly. Initially the use of angular quarried limestone was alien but overtime the material has rounded to a more natural appearance. Conversely this has reduced the size of the material and consequently increased its mobility.

Regular (bi-annual and post storm) monitoring is carried out to inform defence performance assessment and the recycling regime. Losses of recharge material are minimal and the structure layout attracts some material from offshore into the bay.

Further to the east beach levels remain low but reasonably stable and devoid of fines (Figure C14.5).

Figure C14.5 View along rock revetment protected frontage to east (CEUK)

C15 CASE STUDY: Sandbanks Peninsula, Poole, Dorset

C15.1 Background history

The Sandbanks peninsula is a long, heavily developed sandy peninsula situated immediately to the east of the entrance to Poole Harbour. The flow patterns here are complex. There are rapid tidal currents in and out of the harbour. There is also a subsidiary inshore channel, called the East Looe Channel, which allows fast tidal currents to flow parallel to the peninsula and close inshore. To the seaward of this channel there is a sandbank whose form changes in response to the wave climate, as well as these complex tidal flows. The resulting sediment transport in this area is thus extremely complex. Sand can be transported alongshore by breaking wave action as well as by the tidal currents. As a result of these processes, the direction and magnitude of longshore sediment transport is temporally and spatially very variable. In addition, there is intermittent onshore movement of sand from Hook Sand, by swell wave action.

The sand beaches along the Sandbanks peninsula were originally protected by a system of crib type rock groynes. Historic charts show that these maintained high beach levels over much of the frontage. However, these groynes fell into disrepair, being removed in 1991 for health and safety reasons. Beach lowering was noted subsequently, becoming most serious at the western end of the frontage, where a seawall surrounds the head of the peninsula (Figure C15.1). The offshore transport of sand, due to waves being reflected from the wall and its subsequent removal by the fast tidal currents, caused concerns that the wall would become undermined.

Figure C15.1 Vertical wall and low beach near Haven Hotel, Sandbanks, c.1998

C15.2 Mitigation measures

In 1991 a rock groyne was constructed near the western end of Sandbanks. In 1994 a rock fillet was constructed at the base of the wall. Following the completion of a beach management strategy for the entire Poole frontage in 1994–1995, HR Wallingford was commissioned to produce an outline design for a coast protection scheme for the western end of Sandbanks. This included the modelling of a groyne scheme to reduce flows over the beach in front of the seawall. The optimum plan shape that was derived ensured that the flows of the East Loe Channel were deflected away from the seawall.

The scheme was constructed in 1995–1996. Overall, this was a great success with the earlier beach erosion being reversed and sand dunes forming where there were previously low beach levels (Figure C15.2).

The rock groynes have subsequently been extended southwards in the second phase of the works. These have also been very successful so that virtually the whole of the Sandbanks frontage now has a high level of protection.

Figure C15.2 Widened beach after installation of sill and rock groynes, 2002

C15.3 Performance of mitigation measure

Before the scheme was implemented, there had been an increase in water depths within the East Looe Channel and an onshore movement towards the line of the seawall. When the scheme was built, the tidal currents were deflected away from the wall, enabling sand to settle out of suspension.

This scheme demonstrates how the role of tidal currents on beach lowering should not be overlooked, especially near estuary and inlet mouths.

C16 CASE STUDY: Seaford, East Sussex

C16.1 Background history

Now a resort, Seaford was a port before a great storm in 1579 caused the build-up of shingle to block off the entrance channel and diverted the course of the river Ouse westwards in the direction of Newhaven.

The subsequent development of the port of Newhaven has included the construction of training walls to prevent the littoral from blocking up the new entrance. Successive extensions of the western training wall were necessary as the beach to the west of the entrance accreted. This meant that the shingle beach at Seaford has received a dwindling supply of shingle. This has caused beach levels in front of the seawall to fall at an increasing rate during the last century. By 1980, the beach in the eastern half of the frontage had fallen to such an extent that it was providing very little support to the old mass concrete seawall; this wall was originally a secondary defence behind a then substantial shingle ridge. In places, erosion had exposed the underlying chalk platform, allowing waves up to 6 m high to reach the wall without breaking. In 1981 parts of the wall had become badly damaged and in 1985 undermining had caused local collapse of the promenade. Figure C16.1 shows the situation around 1982.

Figure C16.1 Low beach levels and damaged concrete seawall, c.1982

C16.2 Mitigation measures

The local water authority was aware that reconstructing the seawall would be difficult to justify economically and that such a course of action would be unsustainable given the likely continued fall in beach levels. Following hydraulic and numerical model testing at HR Wallingford, an open beach (ungroyned) nourishment scheme was adopted as the most economical solution to the problem of overtopping and continuing deterioration of the seawall. Due to its south-westerly aspect the shingle beach at Seaford is aligned almost perpendicularly to the predominant direction of approaching south-westerly waves. Because of the relatively low rate of littoral drift generated by these obliquely incident waves at the western end of the nourished frontage, it was determined that a terminal groyne was not necessary there. The large concrete groyne already in place at the eastern end of the frontage was reconstructed to a greater height and length, so as to prevent loss of material to the natural cliffed (and undeveloped) coastline to the east.

In 1987 the central and eastern end of Seaford was nourished with 1.5 million m^3 of shingle won from offshore (Figure C16.2). The material was won by using a trailer-suction dredger, extracting the material from an existing licence area on the Owers Bank, south of Selsey Bill. The material was spread over a 2.5 km frontage. The western frontage was left untouched, as the beach there was already wide.

Figure C16.2 Beach after renourishment

C16.3 Performance of mitigation measure

Following the initial period of adjustment, the beach actually increased in volume within the active beach profile (taken as above –4 mAOD) This is because the wave reflectivity was significantly reduced, causing pebbles to migrate landward from the hard chalk seabed on which material was very mobile.

Monitoring has been critical to the long-term viability of the scheme. In order to maintain sufficient beach width at all points on the frontage, recycling needs to be carried out. Modelling indicated that the average annual recycling volume was likely to between 20,000 and 25,000 m^3 per year. The volume that has had to be recycled has, in fact, varied considerably from year to year and the average value has been considerably higher than anticipated. Nevertheless, the scheme has successfully protected the ageing seawall from wave attack and stopped the heavy wave overtopping that used to take place.

C16.4 Possible improvement measures

It would appear that a nourished fill is considerably more mobile than the native beach material. The reasons for this are not particularly well understood, but it is considered likely that shingle transport on what is a relatively steep beach may be enhanced by tidal current action. Certainly there is evidence in the form of shingle waves, showing enhanced mobility. This type of response has been observed in a number of other beach nourishment schemes involving shingle.

The disadvantages of beach nourishment are that it is difficult to predict the expected life-span of beach nourishment material in view of the unpredictability of the UK wave climate. There may also be difficulty in obtaining the right grade of material, as offshore dredging operations are dependent upon a licence being available. Massive nourishment schemes, particularly where they involve recycling, also have an adverse impact on the usage of the beach. This can be minimised by targeting the recycling operations so as to avoid holiday periods, etc.

C17 CASE STUDY: Selsey Bill, West Sussex

C17.1 Background history

Selsey Bill is situated at the southern tip of a low-lying headland that juts out into the English Channel. The coastline is formed of Bracklesham Clays, which are overlain by gravel deposits at Selsey Bill. These gravels form low cliffs that are easily eroded. Because of its open aspect, Selsey Bill is a focus for wave energy. It is also a drift divide, the cliff erosion having provided material for the development of the beaches east and west of the Bill. In addition marine sediments – principally coarse sands and gravel – are driven onshore during storms from barely submerged banks lying off the Bill. This movement takes place in pulse fashion, so that the thickness of the shingle cover at any location varies greatly with time. The geomorphology of the area is thus very complex and ever changing.

During the early part of the 1900s the coastline west of the Bill had undergone continued long-term recession, which reached an annual rate of the order of 6 m per year. The erosion of the sandy clays and gravel provided large drift along the frontage to the west, providing sediments for the East Head spit at the western side of the entrance to Chichester Harbour, as well as for the ebb bar across the entrance. However, had erosion continued much of the shorefront development would have been lost (some had already been lost before the scheme was implemented)

Sea defences were begun in the 1950s. With an onshore supply near the Bill, the beaches did not deteriorate as rapidly as might have been expected in view of the earlier very rapid rates of retreat. By the late 1980s, however, the walls had deteriorated through continuous wave action and falling beach levels. Figure C17.1 shows the situation in 1988.

Figure C17.1 Low beach levels in front of stepped concrete wall, 1988

C17.2 Mitigation measures

By the early 1990s, the stepped concrete wall at Selsey West Beach was in danger of being undermined. In addition, there was heavy wave overtopping on virtually every high tide, resulting in damage to developments on the immediate backshore.

In 1992 a major scheme was initiated along much of the Selsey frontage. At Selsey West Beach, the seawall was reconstructed at strategic locations where heavy overtopping could not be tolerated. In other areas the wall was strengthened. In places the wall was extended by the addition of concrete armour units and a rock berm, as shown in Figure C17.2. In addition, some shingle was added to the upper beach, to help fill the groyne compartments.

Figure C17.2 Toe berm of rock and concrete armour units, 1994

C17.3 Performance of mitigation measures

This is a very exposed location and it is not possible to maintain a shingle beach in front of the seawall permanently; Figure C17.2 shows the face of a groyne, against which the shingle beach has recently been drawn down. Shingle beach levels continue to fluctuate strongly from season to season. The level of the lower sandy foreshore appears to have been maintained.

While the problems of overtopping along this frontage have not been eliminated, the volume and frequency of overtopping has been significantly reduced. In addition, the stability of the toe of the wall has been secured.

C18 CASE STUDY: Sidmouth, Devon

C18.1 Background history

Sidmouth is situated at the mouth of a river valley that is flanked by cliffs of red marl. The net littoral drift is from west to east, pinching in the river Sid against the cliffs on the eastern end of the valley. (Sidmouth was a port, which became unusable when the river entrance was infilled with shingle.) The shingle beach is formed of material derived from the erosion of the marl cliffs. These cliffs are largely unprotected, so that the supply has been maintained. Over the town frontage, the shingle beach is backed by a seawall, and is therefore vulnerable when waves are able to reach the wall.

Apart from changes due to fluctuations in the rate of west to east littoral drift, the shingle beach in front of the seawall is affected by drawdown during severe storms, leaving the wall exposed to wave attack. In the early 1990s a severe storm caused serious beach lowering and major wave overtopping over the town frontage. After this storm the beach did not recover its former levels.

C18.2 Mitigation measures

Following model testing at HR Wallingford, a beach nourishment scheme was implemented using local beach material, together with the construction of two offshore breakwaters and a groyne at the eastern (downdrift) end (Figure C18.1). The purpose of the two breakwaters at the western end of the frontage is to protect the town frontage against the predominant westerly storms. The groyne is there to prevent material from being transported out of the area by the net west to east drift.

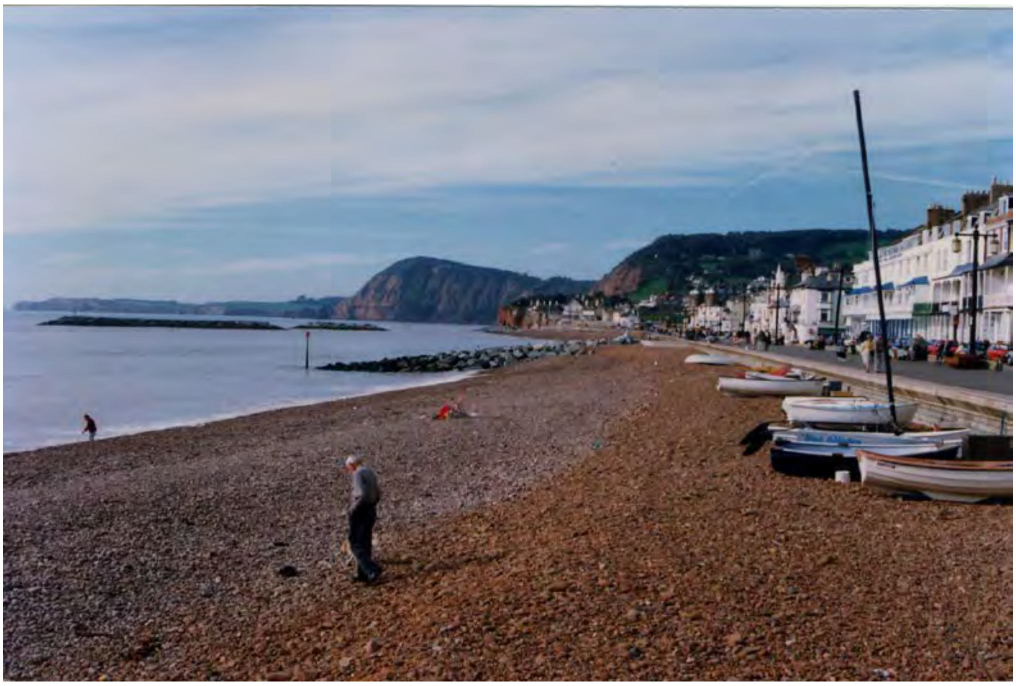


Figure C18.1 Beach recharge, groyne and offshore breakwaters

C18.3 Performance of mitigation measures

The scheme has eliminated beach drawdown and overtopping during westerly storms.

A succession of storms from the east caused shingle to migrate into the lee of the breakwaters, reducing the beach width at the eastern end of the frontage. This was remedied by constructing an additional rock groyne to reduce littoral drift from the east. Since the second groyne was added the beach has maintained an adequate width over the whole frontage.

Figure C18.2 shows the eastern end of the nourished frontage. The improvement in beach width has not only effectively reduced the former problems of beach lowering but has also provided a more attractive beach. In addition, the offset breakwaters are also not visually intrusive, being below the horizon at promenade level.

Figure C18.2 Rock groynes and beach recharge, eastern end of promenade

C19 CASE STUDY: Portobello Beach, Edinburgh

C19.1 Background history

The justification for many beach nourishment schemes in the UK has been that the cost of nourishment is considerably less than the cost of reinstating existing hard defences. One of the first schemes to be justified on this basis was carried out at Portobello Beach, Edinburgh. This beach is situated on the western side of Edinburgh and faces directly into the mouth of the Firth of Forth. It became denuded as a result of sand abstraction for the glass industry, which began in the 19th century and continued up to the 1930s. The promenade seawall at Portobello dates back to 1860. By the late 1950s the lowered beach meant that the wall was under continuous wave attack, with resulting frequent serious overtopping (Figure C19.1). By this time the beach had flattened and the median sand grain size was 0.2 mm.

Figure C19.1 Portobello Beach, c.1970

C19.2 Mitigation measures

Following studies by HR Wallingford, the beach was nourished in 1972. The sand nourishment material had a median size of 0.27 mm, considerably coarser than the beach material. The sand was obtained from a sub-tidal borrow area some 3 km east of Portobello in a sheltered part of the Firth of Forth. This material is so close to Portobello that it may well be the natural sand size for the area. The beach at Portobello had become so eroded that the sand was no longer representative of the beach material under healthier conditions.

Some 180,000 m^3 of coarse sand was extracted from a borrow area by bucket dredger, transported by barge and pumped over a 1.6 km frontage to a foreshore gradient of 1 in 20. The material was placed over a depleted beach whose gradient had fallen as a result of beach lowering to about 1 in 42. The nourished beach was held in place by a number of timber groynes, with an (easily adjustable) gabion-type groyne at the eastern (updrift) end of the frontage (Figure C19.2).

Figure C19.2 Post-nourishment view at Portobello

C19.3 Performance of mitigation measure

From the start, it was recognised that careful monitoring was crucial to the long-term success of the scheme. The beach was monitored in the early year of the scheme by HR Wallingford and then by the local Coast Protection Authority (first Lothian Regional Council, then City of Edinburgh Council)

The beach profile surveys show that, after 18 months, the beach slope had adjusted to 1 in 23, but other than the seaward movement of the toe of the nourished beach, there were no significant offshore losses of beach material. Littoral drift in this area is low so that end losses are small.

Beach volumes remained relatively unchanged until 1978. By 1981 the losses had increased to about 50 per cent of the original nourishment volume as a result of severe storms. Following calmer weather the beach recovered, so that in 1984 there was still some 75 per cent of the renourishment volume remaining above the low water mark! By 1988 the nourishment volume had reduced to 70 per cent of the placed volume. In late 1988 a further 102,000 m^3 of sand were added as a topping up and improvement operation. The trend of beach erosion after storms and subsequent recovery has continued since. Despite a trend of gradually declining beach volume, the beach remains above its pre-1988 renourishment level.[12] (HR Wallingford 2002).

C19.4 Possible improvement measures

This scheme has been so successful that no significant improvements to the mitigation technique employed can be envisaged. The beach has a low littoral drift and the swell waves penetrating through the mouth of the Firth of Forth almost balance the destructive action of locally generated, hence short period and destructive waves. Had

[12] *Southern North Sea Sediment Transport Study, Phase 2*. HR Wallingford Report EX 4526.

finances been available the scheme might have been extended westwards over the partly industrial frontage to Leith Docks.

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