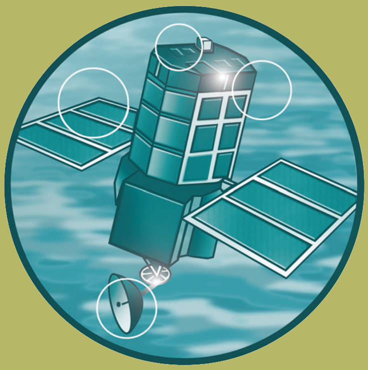
Joint Defra/EA Flood and Coastal Erosion
Risk Management R&D Programme

Understanding the lowering of beaches in front of coastal defence structures, Stage 2
Technical Note 8

R&D Project Record FD1927/PR8



Environment Agency

defra
Department for Environment
Food and Rural Affairs

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

Mitigation methods

Technical Note CBS0726/08 Release 4.0

Address and Registered Office: HR Wallingford Ltd**. Howbery Park, Wallingford, OXON OX10 8BA**
Tel: +44 (0) 1491 835381 Fax: +44 (0) 1491 832233

Registered in England No. 2562099. HR Wallingford is a wholly owned subsidiary of HR Wallingford Group Ltd.

Document Information

Project	Understanding the lowering of beaches in front of coastal defence structures, Phase 2
Technical subject	Mitigation methods
Client	Department for Environment, Food and Rural Affairs
Client Representative	Stephen Jenkinson
Project No.	CBS 0726
Technical Note No.	TN CBS0726/08
Filename.	CBS0726-TN08_mitigation_methods_4_0
Project Manager	Dr J Sutherland
Project Sponsor	Dr RJS Whitehouse

Document History

Date	Revision	Prepared	Approved	Authorised	Notes
14/02/08	4.0	Brampton	Sutherland	Whitehouse	Re-issued to client following review.
21/08/06	3.0	Brampton	Sutherland	Whitehouse	Issued to client
28/07/06	2.0	Brampton	Sutherland	Whitehouse	Revised and extended for internal review
27/04/06	1.0	Brampton	Sutherland	Whitehouse	Initial draft

Contents

1. Introduction 1

2. An approach to choosing mitigation methods 1

3. Monitoring and accommodating the effects of beach lowering 2
   - 3.1 Introduction 2
   - 3.2 Assessing the condition and performance of existing defences 3
   - 3.3 Accommodating the effects of lower beach levels 4

4. Ancillary works 5
   - 4.1 Fagotting and wave breakers 5
   - 4.2 Scour mattresses 9
     - 4.2.1 Gabions 9
     - 4.2.2 Concrete mattresses 10
     - 4.2.3 Geotextile containers 10
   - 4.3 Rock blankets, toe berms and fillets 11
   - 4.4 Breakwaters, reefs and sills 16
     - 4.4.1 Detached breakwaters 16
     - 4.4.2 Low-crested breakwaters and reefs 18
     - 4.4.3 Sills 19
   - 4.5 Groynes 23

5. Adjusting defence structures to reduce toe-scour 26
   - 5.1 Underpinning and encasement 26
   - 5.2 Addition of apron or steps 27
   - 5.3 Reconstruction of seawalls 30

6. Major beach improvement methods 33
   - 6.1 Introduction 33
   - 6.2 Planning 34
   - 6.3 Beach recharge methods 36
   - 6.4 Reducing sediment losses 36

7. Mitigation measures: conclusions and recommendations 37
   - 7.1 Conclusions 37
     - 7.1.1 Monitoring and accommodating beach lowering 37
     - 7.1.2 Ancillary works 38
     - 7.1.3 Underpinning and encasement 39
     - 7.1.4 Adding steps or aprons 39
     - 7.1.5 Rebuilding defences to reduce toe scour 40
     - 7.1.6 Major beach improvement schemes 40
   - 7.2 Recommendations 40

8. References 42

9. Acknowledgements 43

Figures

Figure 1a Simple infill of scour trench (see Plate 7) 12
Figure 1b Rock scour blanket 12
Figure 1c Sloping rock toe (see Plate 8) 13
Figure 1d Rock fillet in front of wall (see Plate 10) 13

Plates

Plate 1 Fagotting at Cooden shingle beach, Pevensey Bay, 1983 6
Plate 2 Fagotting at Medmerry, West Sussex, 2006 7
Plate 3 Wave breaker at Cooden Beach, Pevensey Bay, 1986 8
Plate 4 Wave breaker remnants, Pett Levels, Rye Bay, 2005 8
Plate 5 Concrete mattress revetment, West Huntspill, Somerset, 1993 10
Plate 6 Rock fillet and groynes, Blue Anchor Bay, Somerset, 1988 14
Plate 7 Rock infill of scour trough, Le Dicq, Jersey, 2005 15
Plate 8 Installing sloping rock toe, Stonehaven, Aberdeenshire, 2004 15
Plate 9 Concrete unit and rock fillet, Selsey Bill, West Sussex, 2006 16
Plate 10 Detached breakwater, Rhos-on-Sea, Clwyd, 1986 17
Plate 11 Detached breakwaters, Elmer, West Sussex, 2005 18
Plate 12 Concrete sill, East Wear Bay, Folkestone, 1986 20
Plate 13 Rock sill, California, Norfolk 21
Plate 14 Concrete sill and groynes, Bawdsey, Suffolk, 1996 21
Plate 15 Concrete sill, Bawdsey Manor, Suffolk, 1986 22
Plate 16 Old groyne field, Sandgate, Kent, 1986 24
Plate 17 Variations in beach width caused by rock groynes at Jaywick 25
Plate 18 Saw toothed beach plan shape, Brighton 25
Plate 19 Extension of seawall footings, Lee on Solent, Hampshire 27
Plate 20 Vertical faced apron, Overstrand, Norfolk, 1991 28
Plate 21 Sloping apron to seawall, Dymchurch, Kent, 28
Plate 22 Asphaltic revetment under construction, Porthcawl, Glamorgan, 1984 29
Plate 23 Asphaltic apron at high tide, Porthcawl, Glamorgan, 1995 30
Plate 24 Stepped sloping seawall, Heacham, Norfolk, 2003 31
Plate 25 Cellular seawall, Folkestone, Kent, 1986 32
Plate 26 Seathorne, near Skegness, Lincolnshire, 2001 32
Plate 27 Energy dissipating concrete block wall, Shoreham, West Sussex, 2005 33

1. Introduction

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

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

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

A summary of these methods is presented in Sections 3 to 6, together with some general and very preliminary comments about their applicability, advantages and disadvantages. For any particular location, however, the choice of a suitable mitigation method will depend not only on its technical feasibility, but also on a number of other factors, such as impacts on amenity, access, ecology, aesthetics, the length of frontage affected, the required lifetime of the works, the initial and maintenance costs etc. It is beyond the scope of this study to discuss these issues in detail.

2. An approach to choosing mitigation methods

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

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

It is clear from this that the choice of an appropriate mitigation method will depend considerably on local conditions. Consequently, different methods might be appropriate for two locations where the waves, tides and beaches are very similar. In the following sections, we have reviewed the most common schemes used to cope with beach lowering, starting with those that are likely to be least expensive.

- Section 3 - Monitoring and accommodating the effects of beach lowering;

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

In each section, a brief summary of the likely applicability, strengths and weakness of each of these is provided. Section 7 draws some preliminary conclusions and then sets out some recommendations for improving and disseminating good practice on mitigation methods.

3. Monitoring and accommodating the effects of beach lowering

3.1 INTRODUCTION

Many coastal defence structures, such as seawalls, have survived and performed adequately over long periods, despite gradual lowering of beach levels in front of them. Long-term beach lowering in front of coastal structures is commonplace in the UK, although often at no greater a rate than would have been expected if the structure had not been built (see Sutherland et al., 2003).

In most cases, a lower beach in front of a structure will allow larger waves to reach it. This often leads to increased wave overtopping and thus a reduced standard of defence (i.e. a decrease in its “functional performance”). Perhaps more critical is that the larger waves and increased water depth may also affect the integrity of the structure itself. Both wave impact forces and overturning moments will be increased, but the greatest danger is usually that the seaward toe of the structure becomes undermined, typically leading to a loss of “fill” from behind it. This is the most common cause of seawall failures in the UK (see Thomas and Hall, 1986).

However, mitigation measures are not necessarily needed immediately. In the short-term, beach lowering and its consequences, such as wave overtopping can be accepted or accommodated without risking structural failure. This allows the coastal manager sufficient time to establish the need, best technique and timing for any future intervention. However, such situations require regular monitoring, to ensure that such lowering does not lead to unacceptable risks as time passes, for example more frequent and severe overtopping. Methods of monitoring beach levels in front of coastal structures, and for predicting how the levels may change in the future, have been described by HR Wallingford (2006).

In situations where beach lowering appears critical/rapidly worsening (i.e. where the minimum acceptable beach level in front of a structure might possibly occur within one or two years) it would be reasonable to increase the frequency of surveys and thus to decide more quickly on the appropriate mitigation measures, whether temporary or more permanent. A suggested procedure in such cases is described below.

However, it should be noted that if it cannot be easily determined how quickly the situation is likely to deteriorate, installing relatively low-cost mitigation works sooner rather than later may be advisable, instead of waiting for all the necessary data to become available, to decide on an appropriate long-term management scheme.

3.2 ASSESSING THE CONDITION AND PERFORMANCE OF EXISTING DEFENCES

Ideally, the likelihood of beach level becoming lower in front of any coastal defence structure should have been anticipated during its design, and the consequences evaluated. In practice, however, there is often no such information available, because either low beach levels were never explicitly considered during the original design study, or the details of the "as built" structure were not kept, or have been lost.

It is therefore logical to start with an assessment of the existing situation, first defining the **condition** of the beach and existing structure(s), as joint components of the coastal defence. This should be followed by an assessment of the expected **performance** of this defence during storm events (i.e. a combination of high tidal levels and large waves), before deciding whether or not to intervene.

In considering the **condition** of the defence, the first priority is to establish the level of the seaward toe of the structure, and to ascertain whether there is a risk of it being undermined if the beach falls below this level. If drawings of the structure "as built" are not available, then localised excavations may be needed to establish the level and condition of its toe, including any supporting piling.

Note that in some cases the structure may be founded into a hard rock platform (substrate) underlying a beach. It may therefore not necessarily be at danger of immediate undermining, even if the beach was to disappear entirely. It is much more common, unfortunately, for coastal managers to only become aware of the threat of undermining of a particular structure after beach levels have fallen beneath its toe.

In addition, the overall structural soundness of the defence structure will need to be assessed, e.g. to form a view regarding the risks to it from larger wave-induced forces and overturning moments as water depths in front of it increase.

Consideration must then be given to the standard of defence provided by the structure as beach levels fall. This assessment of the **performance** of the defence needs to consider both present-day and future combinations of wave conditions and high tide levels for a range of return periods. It is worth noting in this respect that the expected values of a high tide level with a specified return period may have increased significantly since the structure was built, in part due to increased mean sea levels relative to the land.

By considering a range of high water level and wave conditions, and undertaking sensitivity tests on the effects of lower beach levels on the frequency of occurrence and volumetric overtopping rates, it will be possible to gauge how quickly beach lowering will lead to unacceptably high risks of overtopping at any location. Guidance on tolerable mean overtopping discharge limits are given by Besley (1999) and more recently by Allsop et al. (2005).

Note that it should not be assumed that the beach level at which overtopping becomes unacceptable is higher than that at which the structure may be undermined. Indeed, the situation where a coastal defence may suffer undermining before overtopping becomes a serious problem is particularly dangerous, since it may give a false impression of the protection it offers.

Given these dangers, the design of any coastal structure should include a warning regarding the "minimum acceptable beach level" in front of it. This threshold level could then be used as a basis for analysing the results from a beach monitoring exercise, and deciding when it would be wise to intervene.

When dealing with existing structures, this same approach is recommended, i.e. establishing a critical “lowest beach level”. Surveyed beach levels can then be compared with this threshold values and hence used as an indication of the condition and hence expected performance of a coastal defence

3.3 ACCOMMODATING THE EFFECTS OF LOWER BEACH LEVELS

If continued beach lowering is likely to lead to undermining or damage to a coastal defence structure, then intervention works will be needed to reduce this threat. Various possible remedial options are described later in this chapter.

However, if the result of future beach lowering will first be to decrease the performance of the structure (for example, causing an increase in the frequency and rates of wave overtopping) then it may be possible to “accommodate” these effects without altering either the structure, or improving beach levels. The options available can be grouped into short-term and “long term” categories.

In “immediate” or “emergency” category, the options include

- Storm warning systems to anticipate overtopping events and evacuate areas at risk (Sayers et al., 1999); and
- Preventing access to areas immediately to landward, e.g. closing roads.

In many areas, the greatest threats from overtopping at high tide are to people or vehicles attracted to the sea-front to “wave watch”. Deploying operational staff in good time to close flood gates and ensure that risks to people and properties are minimised during an event can be effective, but can also be expensive. It is likely that such “emergency responses” will also involve some “clean up” operations after an event, for example removing debris and beach sediment carried over the sea defences. Recording the times and dates of such events, and evaluating the costs of responding, would provide valuable quantitative information on this approach to managing coastal flood risks, to be used in deciding on and designing more permanent mitigation measures.

In the “short-term” category, the mitigation options include:

- Installing secondary flood defences to limit the extent of flooding; and
- Improving the drainage or storage of overtopped water.

Where wave overtopping is both frequent and substantial, it may be worthwhile making provision for managing the resulting flooding, for example by installing secondary flood defences and/or making arrangements for the safe detention or drainage of the seawater that overtops the main defences. Any secondary flood defences might be demountable and temporary or installed seasonally, perhaps in the same fashion as for fluvial flood defences.

In the longer-term category, the options available include:

- Increasing the flood resilience of structures, surfaces, and properties behind the structure to withstand greater flows; and
- Relocating major assets at risk from flooding and restricting future development.

In some cases, particularly where a defence structure or what is landward of them is easily erodible, e.g. a clay embankment, glacial till cliffs or dunes, then the damage caused by overtopping waves could be reduced by strengthening them.

All of the above methods of accommodating the effects of beach lowering, however, can only be implemented and sustained in some situations. Where these cannot provide sufficient protection against erosion of flooding risks, alternative methods may need to be taken to mitigate beach lowering, and explained in the following sections. However, it is likely that for at least some of the lower-cost mitigation methods described, there will be advantages in also using some of the "accommodation" methods described immediately above.

4. Ancillary works

Where beach levels have fallen substantially, or are likely to drop to below a critical level in front of a coastal structure, threatening its structural integrity or resulting in unacceptably high flood risks, then more direct methods of mitigating the beach lowering or scour problems will be needed.

The first approach to be considered is to install "ancillary works" in front of a coastal defence structure. These works are aimed at to improving beach levels, or at least preventing them from falling further. Ancillary works, as described here, do not involve altering or adding to the structure itself, although some techniques might involve building against or on the face of the structure, e.g. placing a "fillet" of rock to cover the toe of the seawall. In the remainder of this section, we describe some of the ancillary works that we have found during this study.

4.1 FAGOTTING AND WAVE BREAKERS

Low-cost beach stabilisation, using fagotting, i.e. installing lines of brushwood stakes "dug" into the beach, was in wide usage on the south-east coast of the UK some forty to fifty years ago. Relatively low cost stabilisation has also been carried out, in the past, by means of driving timber posts mechanically into a beach to provide partial shelter to the upper beach from waves. Both methods have now largely fallen out of favour, due to the need for periodic repair or replacement, which can be labour-intensive.

There appears to be no available information on the success of such techniques, either in terms of their performance, or costs over an extended period. This is because such techniques have fallen out of usage and are therefore not reported on. Their impact on aesthetics, and perhaps on public safety, means that they are unlikely to be suitable for heavily used beaches. These techniques, however, may still be useful on sheltered coastlines (e.g. in estuaries and inlets) in some circumstances (e.g. Porlock Weir harbour). It is thus possible that they might provide a method that can be employed by voluntary organisations where labour costs are not a major concern, carrying out low cost works on lightly protected, partly sheltered shorelines.

Examples of fagotting are shown in Plates 1 and 2. Plate 1, taken in 1983, shows how fagotting was used to stabilise a shingle "storm beach" at Cooden, East Sussex, on a semi-urban coastline that was vulnerable to flooding, but where major coast protection works could not be justified. Here, the fagotting had been installed to prevent a breach forming at an erosion "hot spot" in the centre of an embayment. Here, the fagotting consisted of bundles of timber palings driven into the beach substratum in shore parallel and shore perpendicular rows. The palings were of a restricted height above general beach level, and designed to prevent beach levels from falling, by preventing shingle from washing out seawards. It has been observed that while this type of construction did indeed trap shingle, the increased turbulence tended to prevent sandy sediments from settling out.

Fagotting was also used by the local Water Authority at Medmerry, West Sussex, on a low-lying coastline that was totally dependent on a narrow shingle ridge to prevent flooding (see Plate 2). Here, it was installed to retain a shingle cover over the underlying substratum of Bracklesham Clays. In the 1970s, the shingle ridge was nourished with shingle and has subsequently been managed by recycling, with periodic re-nourishment also being carried out. Falling beach levels have recently exposed the fagotting that has been buried for so long (see Plate 2). It can be seen that, despite all expectations, the fagotting is relatively intact, and with few signs of decay or marine borer infestation. However, the fagotting does appear to be ineffectual in stabilising beach levels at this location.

Plate 1 Fagotting at Cooden shingle beach, Pevensey Bay, 1983

Plate 2 Fagotting at Medmerry, West Sussex, 2006

Wave breakers are normally fairly substantial structures, which intercept and hence reduce wave action further up the beach. Like fagotting they too, have been used primarily to protect relatively undeveloped frontages, where the costs of more substantial protection could not be economically justified. They were often intended to reduce the tendency for beach lowering at the toe of the coastal structure that they were protecting. These structures are thus the forerunners of present-day methods that involve the installation of rock structures, such as detached breakwaters, reefs and sills (see Section 4.4).

Examples of wave breakers are shown in the Plates 3 and 4. Plate 3 was taken in 1986 at Cooden Beach, Pevensey Bay. This is one of several such structures employed along this frontage to reduce shingle beach erosion and minimise the threat of wave overtopping (timber breastworks have also been used in this area). The structure shown is now too permeable to be effective in reducing wave action, or for retaining material to landward.

Plate 4 shows the remnants of a timber wave breaker at Pett Levels, in Rye Bay, that was once used to reinforce a shingle ridge that protects low lying land from flooding. It was constructed in response to the formation of a breach in the shingle ridge that occurred in 1930 (Minikin, 1952). The wave breaker formed a permeable screen in front of a relatively impermeable timber breastwork (since replaced by a sloping concrete panel revetment). The gaps between slats in the wave breaker allowed shingle to be transported landwards towards the breastwork. Following the construction of the scheme the shingle beach accreted substantially. However, Minikin (1952) noted that this was only a temporary improvement; the beach eroded eventually to more or less the same condition that it had been earlier. The wave breaker may well be thirty years old, if not more. Such wave breakers are be prone to wave damage, similar to that experienced by high, timber groynes.

Plate 3 Wave breaker at Cooden Beach, Pevensey Bay, 1986

Plate 4 Wave breaker remnants, Pett Levels, Rye Bay, 2005

Both the faggotting and wave breakers discussed above are clearly detrimental to the aesthetic and amenity attributes of beaches. While their efficacy at reducing rates of beach lowering is perhaps be implied by their continued usage over long periods and at different locations, it is likely that they have fallen out of favour as alternative construction materials such as rock have become cheaper and more widely available. The anticipated structural life of faggoting is generally low, possibly as little as 5 to 10 years.

While faggotting may perhaps still be useful in remote rural locations, particularly where coastal management is carried out by volunteer labour e.g. in nature reserves, it is less likely to be an option seriously considered by public bodies such as local authorities and the Environment Agency, at least for beaches on the open coast. It may, however, still have a role to play in tidal inlets and estuaries where abrasion by beach sediments and the potential adverse impacts on amenity and recreation are less of a concern.

Wave breakers are perhaps even more unlikely to be considered in the future given the alternatives of rock breakwaters or sills and the difficulties of sourcing and repairing unusual hardwood timber structures.

4.2 SCOUR MATTRESSES

The use of scour mattresses close to structures such as weirs and bridge piers in rivers, at quays and berths in harbours, and over pipelines and around the legs of oil rigs in offshore waters, is a well-established technique (Whitehouse, 1998). Scour mattresses are typically deployed to prevent the undermining of structures, as bed levels near them are lowered by scour caused by the presence of the structures themselves. These mattresses, which are normally prefabricated, provide an interface between the normally solid and impermeable structures and the mobile, permeable sediments surrounding them.

One of the main advantages of using prefabricated mattresses is that they can be laid in a controlled manner. Another advantage is that they will, to a degree, adjust to the bed contours, because of their inherent flexibility; this also allows them to adjust to falling beach/bed levels, without necessarily suffering damage. In the very turbulent conditions in front of a seawall, for example, a mattress would probably require rigidly attaching to the wall itself.

Mattresses come in a variety of types (CIRIA, 1991), including:

- Interlinked gabion baskets that are filled with pebbles or stone;
- Precast units (normally concrete blocks) linked together by cables to form a flexible mattress, which can be linked in-situ to other mattresses;
- Fascine mattresses, which are laid on the seabed and then overlain by stone;
- Stone asphalt mattresses, that are similar to road surfaces, usually laid in the dry; and
- Geotextiles formed into containers that usually filled with sand in situ.

Surprisingly, in the light of their successful uses in other situations, there appears to be little or no information on such mattresses being used in front of coastal structures to mitigate problems of beach lowering in the UK. Indeed there are few instances of this type of protection being used within the inter-tidal beach zone anywhere in the UK (their usage on the open coastline is generally restricted to backshore protection although there are a few examples of these being used as lightweight revetments at or above high water). A number of types of scour mattresses are described below.

4.2.1 Gabions

Perhaps the most widely used of such structures are gabions. These were used as a short-term measure to maintain levels in front of a seawall in Blue Anchor Bay, Somerset, prior to the placement of a more substantial rock revetment. As in many other applications, however, conventional gabions would be prone to damage and partial collapse especially where beach sediments contained gravel. Heavyweight gabions have also been used successfully to protect a low cliff of sands and gravels at Hengistbury Head, Dorset and to protect the crest of Chesil Beach at Chiswell, Portland. However, it is much more common for toe scour to be mitigated by rock armour where the armour stone is "free standing", rather than being contained within a gabion box (see later).

4.2.2 Concrete mattresses

Precast articulated concrete mattresses have been used successfully, in some sheltered locations, to protect earth embankments along the coastline and have the potential to be used as scour mattresses in front of seawalls. These would probably now be preferred in most situations to the older technique of using a fascine mattress overlain by stone. In 1983, a scheme consisting of interlinked concrete mattresses linked by cables, to form a continuous revetment, was installed at West Huntspill, Somerset, near the mouth of the river Parrett (see Plate 5). In this relatively sheltered environment the revetment has been relatively successful in reducing erosion of the salt marsh edge. However, even here, where the wave action is weak and infrequent, the system has been subject to some damage. Such damage would obviously be much more serious had the revetment been laid down within the inter-tidal zone, rather than on the backshore. Repairs by concrete patching, however, are fairly simple and cheap. We have found no other examples of these types of concrete mattresses around the UK coastline in the present research project. Further details on the use of such mattresses to form revetments, can be found in McConnell (1998).

Plate 5 Concrete mattress revetment, West Huntspill, Somerset, 1993

4.2.3 Geotextile containers

Sand-filled geotextile containers ("geotubes") may provide a cheaper alternative than placing stone-filled gabions in front of a coastal structure to restore and retain satisfactory bed levels.

Such containers have been successfully used in less harsh conditions recently, for example as the core of a flood embankment, to form an offshore reef, or as a simple way of forming an extra "wave-wall" atop an existing defence to reduce overtopping. Because of the minimal amount of construction material needed, they may be worth consideration as scour mattresses, for example instead of using a fascine mattress, but would probably need to be covered with a protective "armour" layer to prevent premature failure as a result of abrasion damage. Experience on the Adriatic coast of Italy indicates that rapid deterioration is always likely to occur with this type of material.

4.3 ROCK BLANKETS, TOE BERMS AND FILLETS

The availability of suitable rock, and greater awareness of the consequences of scour, has led to its increasing use in mitigating problems caused by beach lowering. A good starting point for the design of rock structures in coastal engineering is the "Rock Manual" (CIRIA, 1991, CIRIA / CUR / CETMEF, 2007).

The range of applications is sketched in Figure 1, ranging from the filling of a local scour trough, to the construction of a substantial "fillet" of rock against the face of a structure. In the latter case, the rock is designed not only to prevent undermining of the original structure, but also to absorb some of the incident wave energy, with the intention of reducing one or more of wave run-up, overtopping, impact pressures, reflections and scour. (It is noted that such armouring will not reduce pulsating wave loads as they are simply transferred to the wall through the armour.) Such a fillet will also directly protect the lower face of the seawall against abrasion by beach sediments. Care must be taken, however, to ensure that such a fillet is designed carefully, since in some situations its effect may be to cause *greater* wave run-up and overtopping, see Besley (1999) and Allsop et al (2003, 2005) and/or increased wave impact pressures, see Allsop et al (1996a, b). The critical factors in the design relate to the level and width of the horizontal berm at the crest of the fillet relative to local wave lengths and depths, and to a lesser extent the slope of its front face.

More modest uses of rock as sketched in Figures 1(a) - (c) are intended to prevent undermining of the structure, allowing it to continue to perform as originally intended. The simple infilling of a scour trough, even using relatively small rock, e.g. rubble, is reported as being beneficial, although there is little published information on this topic. Wider and more substantial rock "blankets" as sketched in Figure 1(b) are less common in the UK, but have been used with some success to cover trenched cables and pipelines, and to reduce scour around breakwaters. These two options, together with the sloping rock toe shown in Figure 1(c) are normally designed to be at or just below lowest beach levels, and may well be covered by sediment for much of the time, only "emerging" in severe conditions. The permeable rough surface of these rock structures allows sand and gravel to remain within the interstices even in storms, and the rock is readily re-covered by beach sediments once wave conditions reduce. The sand beach at Southbourne (HR Wallingford, 2006, Plate 1) has a rock fillet placed at a low beach level, at the toe of the sloping revetment shown in that photograph.

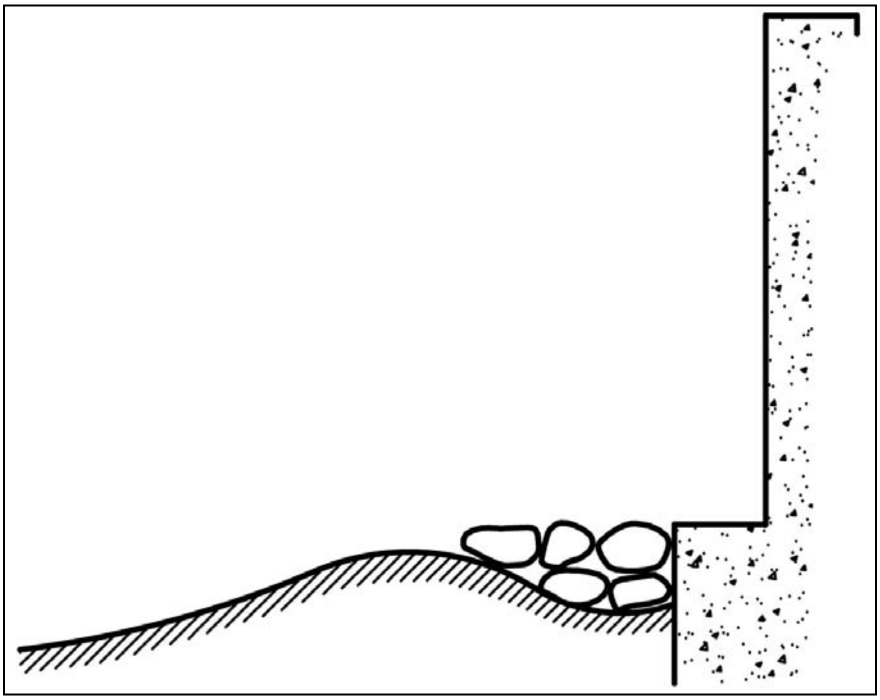


Figure 1a   Simple infill of scour trench (see Plate 7)

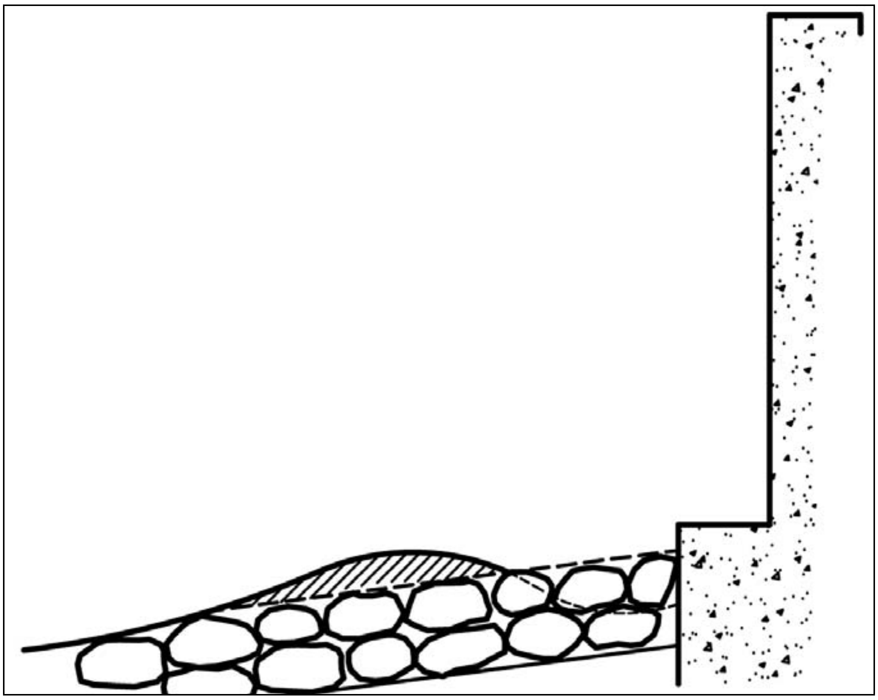


Figure 1b   Rock scour blanket

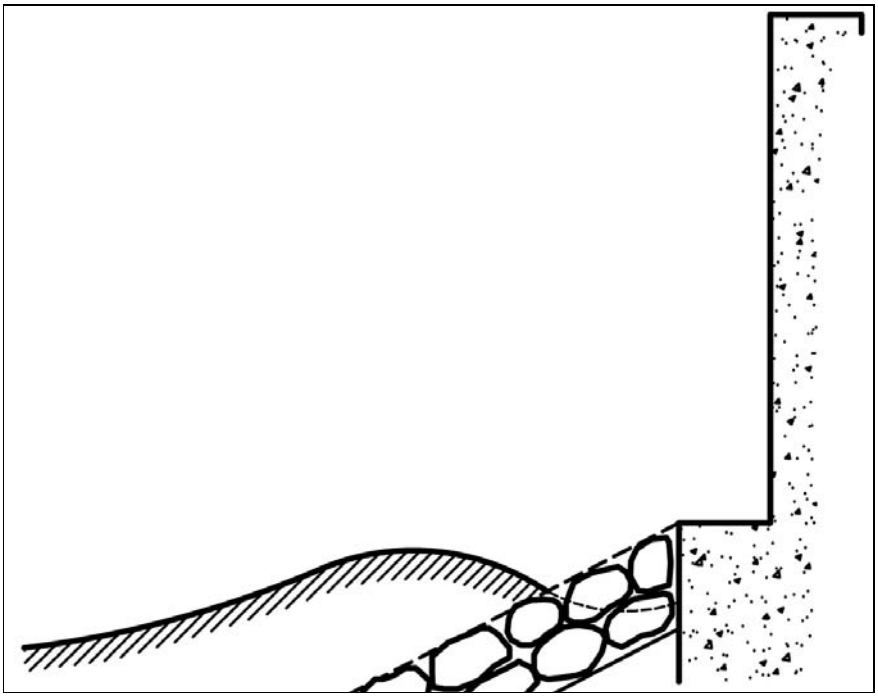


Figure 1c Sloping rock toe (see Plate 8)

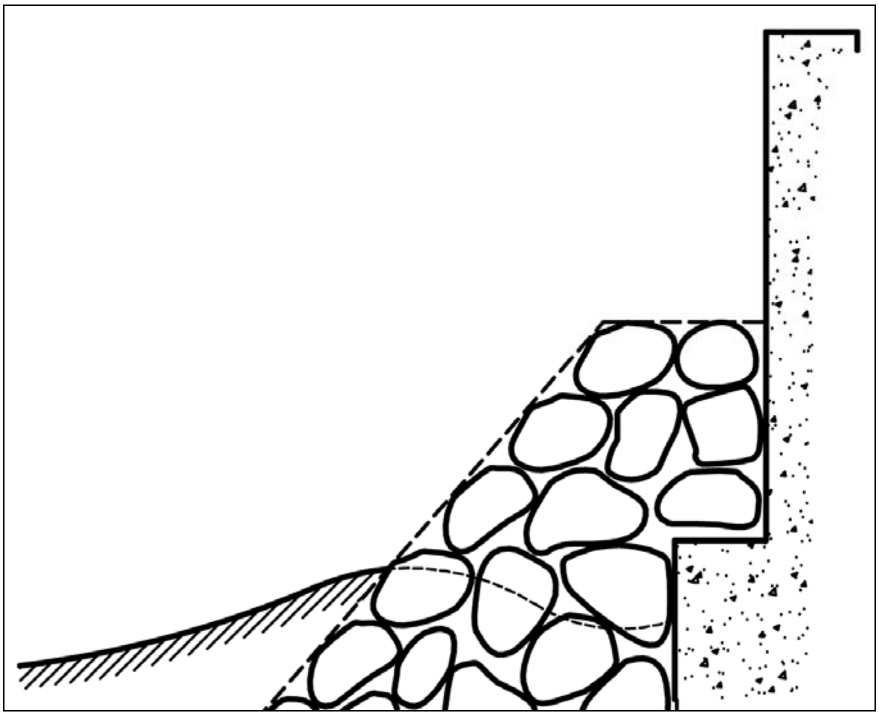


Figure 1d Rock fillet in front of wall (see Plate 6)

However, all the options shown in Figures1 are likely to experience further lowering of beach levels along their seaward edge if the processes of scour/beach erosion continue. This probability must be considered during their design. For the more substantial rock structures, it is now normal to consider the use of “bedding layers” and/ or geotextiles to form an interface between the rock and the beach sediments. These are intended to separate the large rock from the underlying sand such that fine material is restrained from moving upwards through the armour and vice versa. This requirement is generally met by obeying Terzaghi's filter laws, see the Rock Manual (CIRIA, 1991 or CIRIA / CUR / CETMEF, 2007) or Allsop and Muir-Wood (1987), Allsop and Williams (1991), although some simplifications appear to have performed reasonably in practice, see Crossman et al. (2002 and 2003). Obeying these rules will inhibit mixing of the main body of the scour blanket and underlying sand, but will still allow the rock to settle at the outer edge of the protection as beach levels become lower; this aspect of the design is sometimes referred to as a "falling toe" or "falling apron".

Plates 6 to 9 provide illustrations of the use of rock to protect seawalls that were at risk of undermining, and were suffering from increased overtopping and/ or abrasion. Notice that Plate 8 shows a fillet built using a mixture of rock and concrete armour units. All of the mitigation measures described in this section could potentially be built using concrete armour units if this was a cheaper option, although this would probably make the filter requirements more onerous.

Plate 6 Rock fillet and groynes, Blue Anchor Bay, Somerset, 1988

Plate 7 Rock infill of scour trough, Le Dicq, Jersey, 2005

Plate 8 Installing sloping rock toe, Stonehaven, Aberdeenshire, 2004

Plate 9 Concrete unit and rock fillet, Selsey Bill, West Sussex, 2006

4.4 BREAKWATERS, REEFS AND SILLS

Breakwaters, reefs and sills are all structures that are designed to reduce the wave climate, thus modifying the evolution of the beach close inshore. They are normally built parallel to the shoreline and designed to increase the beach width, and have been widely used in tourist beaches in Mediterranean countries. However, judicious placement will allow them to control beach levels in front of coastal structures in the UK as well.

Sills are normally constructed close to the shore, reefs are normally constructed just seawards of the low water line, while detached breakwaters can be considerable further offshore. Because of the smaller volumes of construction materials needed, sills can be expected to provide a cheaper method of mitigating scour than the other two structure types.

A description of each of these types of structure follows.

4.4.1 Detached breakwaters

Detached breakwaters may be built singly, or as a group placed a similar distance offshore, to reduce wave heights at the shoreline in their lee, and thereby the effects of wave impacts on and potential overtopping of a coastal structure. A secondary effect is the accretion of beach sediment in the lee resulting in higher beach levels and therefore reducing the risks of undermining or abrasion of the face of a structure, as well as increasing/ safeguarding the amenity value of the beach at least locally. However, this effect can also cause a reduction in each levels and widths along the adjacent coastline.

Detached breakwaters cause breaking, energy dissipation and reflection of waves even during extreme conditions. Some incident wave energy will still reach the shoreline immediately behind such a breakwater, but the shelter it provides in its lee decreases the longshore transport capacity, potentially to zero, with beach sediment therefore deposited and retained, forming a bulge or "salient" in the beach contours. If there is a sufficient supply of beach material and the structures are very efficient at reducing wave energy, typically when built close inshore, then a "tombolo" may be formed, i.e. the salient extends all the way to the breakwater, forming a narrow "neck" of sediment connecting it to the land, although often only at low tide.

Where detached breakwaters affect tidal currents there is also the danger of longshore drift being diverted offshore into deep water to the seaward of the breakwater, with the danger that sediment is permanently lost from beaches. Scour in front of the outer face of the breakwater is also more likely where there are strong tidal flows. Detached breakwaters are typically built from rock with crests at or above the highest tidal level and aligned shore-parallel. They are most often built in moderate wave energy/micro-tidal conditions, where such structures can be smaller and thus less costly to build.

Where beach lowering is affecting a substantial length of a frontage, detached breakwaters can be built in groups, typically with equal lengths and gaps between structures. If the individual breakwaters are too widely spaced, the beach within the gaps will erode, with the sediment moving laterally into the shelter of each structure. However, if they are built too close together, longshore transport may cease altogether. Unwanted deposition of fine sediment together with flotsam and jetsam onto the beach may occur of a breakwater is built too close inshore.

The first detached breakwater in the UK was built at Rhos-on-Sea, Clwyd, in 1983 to reduce overtopping of the promenade and the coast road behind. This problem was particularly serious over a short frontage where the seawall formed a slight promontory jutting out from the general alignment of the frontage. The breakwater immediately reduced the overtopping problems and built up a sand and shingle beach in its lee (see Plate 10). The beach has now widened so that it now extends along the seaward face of the breakwater, so that the longshore sediment transport past this point is not interrupted, restoring the supply of sediment to the downdrift beaches, further east.

Plate 10 Detached breakwater, Rhos-on-Sea, Clwyd, 1986

More recently, a series of detached breakwaters were constructed over the sandy lower foreshore at Elmer, West Sussex (Plate 11) immediately updrift of a "terminal groyne" with associated nourishment to prevent downdrift erosion. They have eliminated the overtopping that was taking place at the shorefront properties, although some erosion has occurred in one of the gaps between the breakwaters, and along the downdrift coastline.

Plate 11 Detached breakwaters, Elmer, West Sussex, 2005

Overseas, especially in Japan and Italy, the use of offshore breakwaters has been a much more common practice than in the UK. Until the 1980/90s wide use was made of emergent breakwaters in countries bordering the Mediterranean Sea, particularly Italy. In many instances they were primarily constructed with the intention of safeguarding amenity beaches and halting erosion caused by a deficit of beach materials. While an increased beach width in the structure lee was produced, this improvement was almost always at the expense of downdrift erosion. Thus, the deficit of beach sediments became markedly more severe with time, causing further breakwaters to be built along the coast in the downdrift direction. To mitigate such negative effects, many detached breakwaters are now being built in conjunction with beach nourishment.

4.4.2 Low-crested breakwaters and reefs

In many cases, the role of breakwaters in widening a beach in its lee is the major benefit required, while the reduction of wave heights and overtopping at high tide is less crucial. In these situations, it may be better to build a low-crested breakwater or an offshore reef instead, reducing or eliminating the aesthetic impacts of a detached breakwater as well as reducing the amounts of construction materials needed. There is no clear distinction between the terms "low-crested", or "submerged" breakwaters or "reefs", although "reef" may be best reserved for a structure submerged at all states of the tide. The recent DELOS project (2005) used "low-crested" as a descriptor for a wide variety of such structures.

Their primary function is to create and maintain high beach levels, increased beach widths and hence levels to landward by reducing wave heights in their lee, either by partially breaking incoming waves or by altering their refraction patterns. As with detached breakwaters, these structures may be used singly, or in a series extending along the coastline with roughly uniform lengths and gaps between them. In countries with low tidal ranges, this type of structure is regarded as less visually intrusive than detached (and surface piercing) breakwaters, and has therefore been used where aesthetics is particularly important.

Patented concrete reefs have been widely used in the USA, however, mainly in relatively sheltered conditions. The performance of some proprietary reefs is described by Woodruff and Dean (2000). At Palm Beach it was found that the reduced wave conditions landward of one such installation were not enough to offset the losses due to the increased currents on the landward side of the structure, leading to scour. Similar problems have been reported in Italy. While encouraging the development of innovative technologies Woodruff and Dean (2000) stress that such installations must be given an “appropriate engineering analysis independent of the manufacturer or their consultants”.

While we do not have any direct experience with such structures in this country, based on the observations made elsewhere, it is considered that the applicability of submerged reefs for mitigating problems of beach lowering or scour in front of coastal structures remains uncertain. It seems possible that a low-crested breakwater or a reef could be used to widen a beach locally, for example in front of a promontory along a seawall, but this is not a routine technique that could be employed “off the shelf”. Extensive studies would need to be made to assess their applicability in any particular situation, especially in the light of the macro-tidal conditions along most coastlines of the UK.

Advantages of such structures include a smaller effect on the longshore drift after the beach plan-shape has adjusted to the changed wave climate, and at least the possibility of combining the beach improvement function of the structure with other uses, for example recreation such as surfing and habitat creation for marine plants and animals. As with detached breakwaters, research is continuing into the effects of such structures on shorelines, see for example Ranasinghe and Turner (2006).

4.4.3 Sills

Sills are structures that are normally built within the inter-tidal zone, often on the upper beach. Typically they run continuously along a stretch of coastline and may be submerged for a short part of the tidal cycle, around high water. (When used as cliff toe protection works they are sometimes constructed above the high water line).

Material retained shoreward of a sill forms a “perched beach”, and there is likely to be a distinct variation in beach level from the seaward to the landward of a sill. This should be borne in mind when considering their use on amenity beaches.

Sills are used to alter cross-shore sediment transport processes on a beach, retaining higher beach levels to the landward, sometimes at the expense of lower beach levels to the seaward of the sill. Wave heights to the landward of the sill are principally reduced by the higher beach levels, rather than by dissipation of energy over the narrow sill itself.

Experience of sills in the UK is rather limited, in part because of the problems associated with building them where the tidal range is large. This makes it difficult to control their efficiency. Such a structure may be effective at low water but much less effective at high water if the tidal range is large. Under these conditions material may well collect on the lower foreshore rather than the upper foreshore, where an increase in beach levels is usually needed most.

One example of a concrete sill is found in East Wear Bay, Kent (see Plate 12). In this application the sill was constructed at the same time as the wall itself and thus forms an integral part of the seawall toe. The concrete sill is exposed at mid tide and serves to reduce the wave impact on the seawall, as well as preventing scour at the wall toe itself. Sills of this type are clearly not ideally suited for use on amenity beaches, but are very good at prolonging the life of the wall itself.

Plate 12 Concrete sill, East Wear Bay, Folkestone, 1986

It is more usual for sills to be constructed of rock. An example of this can be found at California, Norfolk, where the sill has been constructed to maintain high beach levels at the foot of the sandy cliffs. With appropriate access over the sill, this appears to have been beneficial for tourism. Despite a large footprint, the rock sill provides a dry beach at all states of the tide (see Plate 13).

At Bawsdey in Suffolk, the erosion of a low cliff downdrift of a deteriorating seawall led to the experimentation with interconnected concrete units, rows of which were used to form low-crested groynes and sills (see Plate 14). In this instance the structures appear to be too modest in size too affect beach processes substantially. A similar structure constructed at Bawdsey Manor appears to have been more successful, being situated in more sheltered environment, having some protection by banks at the mouth of the river Deben (see Plate 15).

Experience in other parts of the world also indicates that submerged sills tend to be more successful in low to moderate wave energy, micro-tidal environments. Although the technique could be applied to shingle beaches, its use in the UK has been restricted to sand beaches with shallow inter-tidal and sub-tidal beach/seabed profiles.

Plate 13 Rock sill, California, Norfolk

Plate 14 Concrete sill and groynes, Bawdsey, Suffolk, 1996

Plate 15 Concrete sill, Bawdsey Manor, Suffolk, 1986

In Italy, sills are often designed to retain artificially nourished beaches. At one stage these sills were made of sand-filled geotextile tubes, but structural weakness of the bags was observed and no structures of that type have been built since the 1990s. The functional effectiveness of submerged structures is open to question and it would appear that the design has not yet been perfected. At Lido di Ostia in Italy, for example, a beach nourishment scheme was carried out in 1990, where the sand fill was retained by a rock sill (Franco *et al*, 2004). Losses of fill have been experienced, due to the action of littoral currents. The sill crest was also damaged by wave action. The sill was subsequently armoured with larger stone and its height increased, so it is now better able to cope with the incident wave conditions. In addition, a series of groynes have been constructed to decrease alongshore sediment transport. This appears to have greatly improved the overall effectiveness of the scheme. Rock groynes in combination with sills have also been used successfully at Pellestrina, near Venice, where a barrier beach has been affected by erosion for several centuries. Only very small sand losses have occurred since nourishment.

In order to provide toe protection in macro-tidal and high-energy environments, sills would have to be constructed with their crest close to high water (sills with lower crest elevations are unlikely to retain beach sediments during storm events at high tide, when protection is most needed). In these situations the sills may experience a nett loss of material, due to the significant beach gradient from landward to seaward, encouraging rip currents to form. Small craft users and swimmers would also be put at greater risk than when the structures are of modest size (as would be the case in a smaller tidal range).

In summary, shore parallel sills have been used with some degree of success in Mediterranean countries, but only in micro-tidal/moderate wave energy conditions. Their usefulness in high energy/macro-tidal conditions is yet to be proven. However, they have been successful as backshore protection, i.e. in conditions where the tidal range is not a critical factor.

4.5 GROYNES

Groynes are the most widely used method of controlling beach levels in the United Kingdom, and are typically built in shore-normally aligned groups along the length of the shoreline, creating a series of artificial "bays" as a means of reducing beach lowering. They are built to modify sediment movement, trapping a portion of the sediment moving alongshore thus stabilising and widening eroding beaches.

Groynes modify the movement of sediment along a shoreline in two main ways, namely by:

- Modifying currents running along and close to the shoreline; and
- Directly intercepting coarse sediment particles (e.g. of gravel) moving along the shoreline.

Most commonly, shore-parallel currents are created by waves which break obliquely to the coastline. In this situation groynes will intercept and retain part of the longshore sediment transport, widening the beach locally but reducing sediment supply to the downdrift coast. However, UK beaches can be affected by strong tidal flows, particularly if they are close to the entrance to an estuary of tidal inlet. In this situation, it is not uncommon for such currents to preferentially run along the face of a coastal defence structure e.g. a seawall, causing localise beach lowering problems. In either case, groynes reduce current speeds within the bays between them, thus producing calmer conditions for the deposition of sediment, and hence an increase in upper beach levels. This however, is achieved at the expense of increased flows past the end of the groyne tips, which can lead to problems of scour there.

Groynes have traditionally been constructed as impermeable structures, mainly of timber, but also occasionally using masonry, concrete or sheet piles. There is now a move towards the use of permeable groynes, constructed of rock, or more rarely, of concrete armour units. This has allowed the building of more complex groynes, for example with T- or Y-heads that allow some of the attributes of "low-crested" breakwaters to be combined with those of conventional "linear" groynes. In this case the groynes have been constructed to stabilise the beach near the toe of the seawall, rather than attempting to build up the beach over the entire inter-tidal foreshore. It is evident that in this application the short rock groynes are more effective in raising beach levels than the nearby timber groynes.

Groynes have been used in a wide variety of situations around the UK coastline. On shingle beaches, groynes can be used in any tidal range and under most wave conditions. On sand beaches, however, groynes are most effective in low to medium tidal ranges, because their cost can be prohibitive in areas with large tidal ranges. The spacing of groynes is related to their length. Shorter, higher and closer spaced groynes are used on shingle beaches reflecting the steeper gradients that occur on such beaches, both perpendicular and parallel to the shoreline. In contrast, groynes on sandy beaches are longer, lower and more widely spaced, typically at twice their length or more. Details of the design of groyne systems, with or without beach recharge, can be found in the CIRIA Beach Management Manual (1995). It is worth noting, however, that the design of a groyne system should not be based purely on such guidelines alone; their design always needs to be matched to local conditions.

High impermeable groynes may cause the beach plan shape to become saw-toothed, concentrating scour on one side of the groynes and increasing beach levels on the other, rather than a general improvement in beach levels along the whole shoreline (see Plate 16). This is a particular danger if groynes are built without an accompanying recharge as is now recommended (in this instance the frontage was subsequently nourished, long widely spaced rock groynes constructed, and beach material recycled as and when required). In front of coastal structures, the result of uneven beach plan shape may be the creation of localised “weak points” that could even place the structure at greater risk than previously.

Plate 16 Old groyne field, Sandgate, Kent, 1986

Plate 17 shows the wide variations in sand beach width caused by the rock groynes at Jaywick. However, in most instances the differential build up of beach levels is more acute on shingle than on sand beaches. Plate 18 shows an aerial view of the beach in the vicinity of Brighton’s easternmost pier. The acute change in beach alignment in this instance is affected by the changes in alignment of the backshore. This makes the efficiency of adjacent groynes of equal length vary markedly. In this photograph large quantities of sand being carried in suspension are apparent, by strong littoral currents skirting the groyne ends. Designing groyne systems under these rapidly varying conditions is greatly enhanced through the use of laboratory models.

Installing groynes without recharge will normally lead to problems of erosion further along the coast in the direction to which the sediment is moving, (i.e. “downdrift”) potentially extending over many kilometres. The greatest problems of erosion, however, tend to occur just downdrift of the last groyne, and may lead to the “outflanking” of a coastal structure if care is not taken to avoid this possibility. Design of effective groyne systems thus requires field experience, numerical and physical modelling, coupled with environmental studies. Groynes are often used in combination with beach recharge, and under such conditions model testing becomes almost mandatory.

Plate 17 Variations in beach width caused by rock groynes at Jaywick

Plate 18 Saw toothed beach plan shape, Brighton

5. *Adjusting defence structures to reduce toe-scour*

The third category of mitigation schemes involves alterations to the coastal structure itself. Most commonly, such alterations are simple and inexpensive additions, designed to extend the life of the structure, to prevent it being undermined and/or damaged by abrasion or impact forces from the larger waves reaching it.

However, where the performance of a structure such as a seawall has deteriorated as a consequence of beach lowering, it may also be necessary to carry out further alterations to reduce wave run-up or overtopping. In either case, consideration should be given to altering the existing structure so that the rate of further lowering of beach levels in front of it by scour is reduced or perhaps reversed.

5.1 UNDERPINNING AND ENCASEMENT

Probably the most common adjustments to a coastal structure are remedial works such as underpinning, encasement, or the addition of an apron. Such works are rarely a permanent solution to the problem and have to be repeated later, either extending the protection downwards or further along the coastline. However, they do make maximum use of an existing structure which is often still sound, or can be repaired at less expense than rebuilding the structure.

Underpinning typically requires excavation beneath, and often behind, the face of the structure, the construction of a new and deeper “toe” and backfill of the area behind. Encasement also involves the covering of the front face of the structure, and sometimes building above its existing crest and even over an existing back-slope, i.e. covering some or all of the original structure with a new, normally concrete, layer.

This type of mitigation of the effects of beach lowering is widely used in the United Kingdom, although there are few guidelines for such schemes. Plate 19 shows a typical operation to extend the toe of a seawall downwards. The base of the original wall, on the left-hand side of the photograph, can be seen to be well above the level of the gravel beach. This resulted in the loss of the “fill material” behind the seawall, followed by a partial collapse of the promenade surface into the void beneath. After the forming of a new deeper concrete toe, the void behind the seawall face was filled and the promenade re-instated. Apart from the small seaward extension at the toe of the existing wall, there is no change in the seawall profile.

The depth below beach level at which such extended footings should be founded will depend on several factors, including the extent of available funding, the depth of the beach sediments and the practicalities of safely excavating and keeping open a trench on an inter-tidal beach. However, such works should certainly be designed keeping some degree of further beach lowering in mind. This rate should be quantified on the basis of the past history of the site, and suitable information provided on the minimum beach levels assumed during the design process. As mentioned previously, the assumed minimum beach levels in the design of such works should be clearly stated and used in subsequent monitoring to allow early warning of future undermining occurring.

Plate 19 Extension of seawall footings, Lee on Solent, Hampshire

5.2 ADDITION OF APRON OR STEPS

Where a more substantial solution than simple encasement or underpinning is required, it may be possible to mitigate the effects of localised scour by adding a stepped or a sloping apron in front of an existing structure. The addition of an apron or steps may also reduce wave overtopping and/ or increase the stability of the structure itself. It is often thought that extending a seawall downwards will increase the wave reflection characteristics, thereby leading to further problems in the future. Whilst it was once very common to provide concrete steps as an extension to a seawall toe, a rock fillet at the wall toe is now more typical (e.g. Figure1(d)).

Laboratory testing is very useful for the design of seawalls, particularly where wave overtopping being a problem. Such modelling has shown that scour depth generally tends to decrease with the slope of a seawall, although the differences are small for slopes steeper than 60 degrees (and this is very much a simplification of the complex interaction between the beach and the structure). Nevertheless it is generally recommended that the seaward slope of an apron be no steeper than 45 degrees. For flatter gradients, the depth of scour at the toe of a seawall is observed to decrease with the slope.

Plate 20 shows the seawall at Overstrand, Norfolk. Here the extended footings form a substantial vertical-faced apron, modifying the profile of the wall. It can be seen that beach levels in front of the wall are lower close to the wall than just to the seaward, where a “bar” has formed. This may be an effect of the profile of the wall. Tidal flows and wave-driven longshore currents are strong along this frontage, and may also affect the beach profile.

A very large apron extension can be found at Dymchurch seawall (see Plate 21). The latter has a sufficiently shallow slope that deters scour. However, it was actually constructed in this manner to dissipate wave action on the toe of the structure, by making waves break in the shallow water there.

Plate 20 Vertical faced apron, Overstrand, Norfolk, 1991

Plate 21 Sloping apron to seawall, Dymchurch, Kent,

While adding steps or a gently sloping apron may reduce scour locally and hence both protect against undermining at the toe, and reduce forces on or overtopping of an existing structure, these measures will not prevent more widespread beach lowering.

An alternative to extending the toe of a seawall in concrete is to provide an asphaltic revetment, or similar structure. Asphaltic aprons have now been used at several UK sites, to prevent beach lowering and to reduce wave overtopping and/or to safeguard the integrity of the walls themselves. One of their advantages is the relatively simple construction. An example of this is the asphaltic apron built in front of the seawall at Porthcawl, South Wales in the 1980s. This apron was designed to prevent further abrasion of the seawall and to reduce the wave overtopping that was affecting the roadway immediately behind the wall. The asphaltic apron was designed to cover the lower part of the seawall, thus not only increasing its structural life, but also making it more effective in dissipating wave energy. Plates 22 and 23 show the wall during construction and a decade afterward. The second photograph shows some minor damage to the asphaltic apron that is very easily repairable.

Plate 22 Asphaltic revetment under construction, Porthcawl, Glamorgan, 1984

Plate 23 Asphaltic apron at high tide, Porthcawl, Glamorgan, 1995

A similar philosophy has been employed in extending the life of the coast protection works at Prestatyn, Denbighshire, where coastal erosion was leading to falling beach levels over a wide frontage. This was caused by reduced littoral supply of shingle by the presence of coastal defences further to the west (updrift). The tidal scour at the toe of the seawall at Prestatyn was not only threatening its stability, but was also very unwelcome because of its impact on beach usage (the tidal gully formed at the seawall toe was hindering access to the beach). The toe of the wall has now been extended by means of an asphaltic apron, while the beach itself has been renourished.

5.3 RECONSTRUCTION OF SEAWALLS

In some cases it may become more cost effective to reconstruct/replace an old seawall, rather than mitigate scour in front of it. Such a situation might arise when the wall has been in place for many years, during which time, beach conditions have progressively worsened as well as tidal levels having increased.

An example is provided by the defences at Heacham North Beach, Norfolk. Here a new stepped concrete seawall had to be built, see Plate 24, because of falling beach levels at the toe of the previous articulated concrete-block revetment. Its seaward face is sufficiently shallow to reduce wave reflections, and hence reduce the associated problems of localised scour at its toe. However, building a wall with this shallow slope to its front face is not always practicable. On a steeply sloping beach, such a wall would result in the occupation of a substantial part of the inter-tidal area, reducing its amenity value and interfering with alongshore sediment transport processes.

Plate 24 Stepped sloping seawall, Heacham, Norfolk, 2003

It is theoretically possible to construct a wall that minimises wave reflection while still maintaining a near vertical profile. One such way is to provide a wave screen in front of the wall. The reflection performance of a wall and screen combination would primarily be dependent on the screen porosity and the distance of the screen from the wall (McBride, Smallman and Allsop, 1995) but may result in reflection coefficients as low as 0.11 to 0.15 compared to 0.9 to 1.0 for a vertical seawall. A rare example of this “cellular” type of seawall construction is found east of Folkestone Harbour (see Plate 25). This type of construction has the advantage of reducing wave energy by dissipation under the arches. However, this has been at the expense of structural integrity. Due to the high impact forces the wall has suffered significant damage over the years.

Examples of energy absorbing structures are the interlocking concrete block revetments that have been constructed at Shoreham, West Sussex and Seathorne, near Skegness, Lincolnshire (see Plates 26 and 27). In both cases the armour blocks are designed to absorb wave energy. The structures require site specific modelling, since if one unit were to be dislodged then the whole revetment would tend to “unravel”. Also, it is evident that the dissipative characteristics will be strongly dependent on the size of the incident waves. As a general observation it would appear that such blocks are most effective when the wave height is about the size of the individual blocks or smaller (when the waves are much larger the blocks would be swamped during wave uprush).

Plate 25 *Cellular seawall, Folkestone, Kent, 1986*

Plate 26 *Seathorne, near Skegness, Lincolnshire, 2001*

Plate 27 Energy dissipating concrete block wall, Shoreham, West Sussex, 2005

6. Major beach improvement methods

6.1 INTRODUCTION

Where the problems of beach lowering are sufficiently severe and widespread, i.e. affecting more than a few hundred metres of coastline, it may be worth considering a major recharge scheme to improve those beaches. This direct remedy to such problems involves importing substantial quantities of beach sediment, i.e. sand or gravel, to replace that gradually lost previously. This approach will immediately cover over the toe of coastal structures and decrease water depths in front of them, and in many cases will improve the amenity value and aesthetic appearance of the frontage. Further details on the design and execution of major beach improvement schemes is provided in the Beach Management Manual (CIRIA, 1996a), but a brief review of such schemes is presented below.

As well as an initial "recharge" of the beaches, there will often be a need for ancillary works, such as the building of groynes, for monitoring and analysis of the changes in beach levels and for periodic addition of extra material in later years. These various elements are now generally identified as components of a beach improvement scheme.

Such major recharge schemes are worth considering/ applicable where:

- Beach levels have continued to fall over a wide area (i.e. both along the coastline and across the whole beach profile), as well as locally in front of a coastal structure, over a prolonged period;
- The problems caused by this lowering at the location of the coastal structure, and often elsewhere, are significant and there are substantial assets, or lives, at risk from erosion or flooding of the hinterland;

- There is no realistic prospect, at least in the short-term, of relocating the assets at risk to higher ground or further inland;
- Attempts to accommodate or mitigate wave overtopping and toe scour problems have not provided a satisfactory outcome; and
- Where the stretch of coastline affected is reasonably short and “self-contained” from a beach sediment viewpoint.

Discussing these various points in turn, it is first worth re-iterating that if the structure is a seawall, or another coastal defence, then the long-term and widespread beach loss was often the underlying reason for the presence of that structure in the first place. In such cases, there is often a fallacious reverse of “cause” and “effect” in the minds of many people. Such beach lowering is often entirely natural. It is a response to rising sea levels and continuous wave action on “soft” rocks or glacial sediment deposits, and inevitable in the light of the geological history of the coastline concerned. In other cases, previous human intervention of various kinds may have caused, or added to the process of beach lowering, for example by protecting “soft rock” coastal cliffs that provided a supply of sediment to the coastal zone through natural erosive processes, damming rivers that previously carried sand to the coast, or interrupting longshore sediment transport by building long breakwaters or dredging deep navigation channels into tidal inlets.

Secondly, as will become clear, undertaking large-scale beach improvement works is necessarily expensive initially and likely to involve periodic maintenance expenditure. There are obviously situations, such as coastal resort towns, where it is impracticable to remove assets at risk, and these assets are of sufficient value to require a “Hold the Line” coastal defence policy. These are situations where localised major beach improvement schemes are certainly worth consideration.

It is, however, increasingly economically viable to use beach recharge schemes to protect long stretches of coastline where the assets at risk are less densely clustered, for example along the coast between Waxham and Winterton in Norfolk and between Mablethorpe and Skegness in Lincolnshire. Beach recharge is the main technique used to protect the predominantly rural coastline of The Netherlands against dune erosion and the consequent risk that the industrialised interior may flood (van Koningsveldt and Mulder, 2004).

It is not essential to have tried other methods, and failed to solve problems caused by beach lowering in front of coastal structures as a prerequisite for undertaking major beach improvement schemes. However, it often turns out that this is the normal sequence of events, i.e. that one or more unsuccessful attempts have been made to mitigate the effects of beach lowering in front of coastal structures and a more effective solution is then sought. The prospect and future costs of continuing short-term works, such as repairing and strengthening seawalls, while not reducing the problems caused by wave overtopping, is often the spur to the consideration of a more fundamental solution of the problems experienced.

As major beach improvement schemes in the UK, and their successes, have both grown, so the scope of such schemes has increased. The largest such scheme so far undertaken in the UK was to improve sea defences along the Lincolnshire coastline between Mablethorpe and Skegness, a distance in excess of 20km.

6.2 PLANNING

The planning of a major beach improvement scheme requires knowledge (and understanding) of site-specific parameters including the present-day, and predicted future, wave climates; the existing beach sediment type, size and grading as well as the nearshore sediment budget;

underlying geology; nearshore bathymetry; and the influence of coastal developments and human activities (CIRIA, 1996b). The design must also consider the extreme conditions the beach is intended to withstand, i.e. be large enough to provide protection to coastal structures even in rarely occurring but very severe storm events.

Often the initial design for a recharge scheme is based on comparison of the existing beach with some previous situation or with similar but healthier beaches nearby. However, there are now a number of mathematical methods of designing a beach profile (i.e. the cross-shore shape of the beach) and its sediment volume, e.g. the Powell equilibrium method for a shingle beach. The design crest level of a beach is generally set at or above 2% wave run-up exceedence level (CIRIA, 1996a), while the beach width at this crest level is determined by amenity/ recreational value and the lifetime of the scheme required.

The selection of sediment type and size is one of the most important decisions for a beach recharge scheme. Assessment of the original beach sediment size, type and grading is advised, as sorting will have naturally occurred, with the general rule of thumb being to use material of similar or slightly coarser size and grading that the naturally occurring beach sediment to ensure beach stability. However, this may not be technologically or economically feasible (CIRIA, 1996a). Further, in recent years, controversy has arisen over the use of recharge materials that are coarser than the indigenous since this has, in a number of cases, led to steeper beach profiles and consequently ‘harder’ breaking waves, as well as producing a less pleasant surface to walk over or rest upon; these changes are likely to be particularly unwelcome on a beach extensively used by holidaymakers.

Information on the type and amount of sediment required is used to identify potential sources, and suppliers are approached. Note that the required size and mix of sediments may not be easily attainable and compromises may be necessary, for example using an offshore aggregate which has a higher percentage content of fine-grained sand than initially specified. This would then alter the likely future beach morphology, and hence alter the volume of recharge required. Iteration of the grading, volume and price (delivered) of sediments that might be used is often necessary to reach an acceptable specification for the recharge material.

It is important to be aware that a recharged beach will naturally adjust, after placement, to reach a more stable profile, and that some changes in volume over time are always likely. Losses may occur during the placement of the recharge materials, and during the initial beach adjustment phase. Initial losses in volume are typically due to the redistribution of the finest beach sediments as they are exposed to wave action. Some of these migrate downwards from near the surface of the beach deeper into its interior, thus increasing the density of the placed sediment, but not changing its mass. Other fine-grained particles, particularly any silt, are carried offshore within the seawater, eventually settling out in deep water. These initial volumetric losses can be greatly reduced by minimising the percentage of fine-grained sediments used in the recharge, ideally matching that on the existing beach.

Depending on the local circumstances, there may also be a long term loss due to littoral drift or offshore transport (CIRIA, 1996a), particularly if these processes were occurring, and causing beach erosion, before the recharge. In self-contained “pocket beaches” there will be little, if any, long-term loss of sediment due to the processes described above. Losses can also be restricted through use with control structures such as groynes to retain the extra sediment. However the availability of sediment for ‘top-up’ operations in the future needs to be considered during the design of any major beach improvement scheme, to ensure that the improved beach can be maintained or even enhanced, if necessary.

6.3 BEACH RECHARGE METHODS

Most modern beach recharge schemes use sediments obtained from marine sources, i.e. from licensed offshore aggregate dredging areas or from navigation channels. Dredgers are ideally suited for collecting, transporting and delivering large quantities of sand or gravel, and this is often crucial considering the volumes involved in beach recharge schemes. In addition, many UK dredgers have the capacity to sort the sediment they collect from the seabed, as it is loaded, thus allowing a much closer match to that specified for any particular scheme.

Sediment that is acquired from an offshore source is now usually pumped directly ashore from a dredger, as a water/ sediment mixture. The dredger usually discharges through a floating pipeline connected to pipelines running along the length of the beach allowing the sediments to be placed where needed. The initial construction of these pipelines, which are often several hundred metres long, sometimes together with extra pumps, is a major expense. A recent development is to use a small dredger that can approach very close to the beach and pump sediments directly onshore, jetting them through a bow-mounted delivery pipe; this is commonly known as the “rainbow” method. Where neither method is practical, sediment can be transferred at sea to shallow draught split-hull barges, which then deliver to the beach at high tide. Bulldozers are then used at low tide to recover the piles of sediment from the lower part of the beach and redistribute it as required.

Whenever sediment is delivered by sea, there is inevitably some initial dispersion of the fine-grained material, suspended in the water used to pump it ashore or as is it recovered from the lower foreshore.

Recharge materials from inland are transported to site using tipper-lorries and placed on the beach “in the dry”. While this reduces the initial dispersion of fine-grained sediments, such an option is only practicable if the distance to the inland source is short, and the volumes of sediment are modest.

The recharge sediment may either be placed on the upper beach allowing wave action to subsequently redistribute it, hence achieving a natural profile, or redistributed mechanically, e.g. by bulldozers, to the predicted mean profile (ECOPRO, 1996). The latter option is carried out as quickly as possible, to try and avoid losses before a natural profile is achieved, while the former accepts a potentially higher initial percentage loss of sediments but anticipates that this is offset by the reduced costs of placement.

6.4 REDUCING SEDIMENT LOSSES

Once a recharge scheme is carried out, there is an obvious desire to retain the benefits it provides for as long as possible. In particular, there is a natural tendency to attempt to reduce the losses of sediment from the recharged frontage. This can be achieved by installing beach control structures, such as groynes or offshore breakwaters, as discussed in Sections 4.4 and 4.5 above, or by beach recycling. Beach recycling operations, i.e. the collection of sediment from the downdrift end of a frontage, and transporting it back updrift to whence it came, is a well-established technique in the UK, in places having been carried out for over 50 years. It is often a cheaper and more flexible alternative to installing groynes or breakwaters along a recharged beach, and the whole-life costs of recycling should always be compared to those of building and maintaining beach control structures. Such recycling operations are typically undertaken annually, using an excavator and a small fleet of tipper-lorries.

One common advantage of beach recharge schemes is that adverse effects on downdrift coastlines are reduced, because there is usually some “leakage” of sediment to those beaches. It is therefore also worth considering, in some situations, whether it may be better to avoid the costs of recycling or of beach control structures, allowing sediment to travel to the downdrift coast, and making provision for periodic “top up” operations in the future. In practice, no method will guarantee to maintain all the beach recharge within the area where it is initially placed, and so some degree of “topping up” by further, normally much more modest recharge operations, is always likely to be required. The optimum strategy for maintaining adequate beach levels at any location will therefore need to consider all three of these options, i.e. control structures, recycling and periodic “top-up” schemes. This strategy can be re-considered in the light of experience, and altered if necessary.

Partly for this reason, it is also important to carry out regular monitoring, to gauge the success of the recharge scheme, to evaluate any losses that occur, and to decide when and how further operations should be undertaken to maintain appropriate beach levels.

7. *Mitigation measures: conclusions and recommendations*

The previous four sections have briefly reviewed a variety of mitigation measures used to reduce the problems caused to coastal defences by beach lowering and toe scour. This section first briefly summarises the main conclusions of this review, and then sets out some recommendations for future research into and guidance for such techniques.

7.1 CONCLUSIONS

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

7.1.1 Monitoring and accommodating beach lowering

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

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

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

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

7.1.2 Ancillary works

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

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

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

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

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

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

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

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

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

In the UK, groynes have long been the most common method used to improve and retain high beach levels, and have generally been regarded as successful in this regard Groynes can be effective in reducing the problems caused by toe scour in front of coastal defences by diverting longshore currents further seawards. In addition, they can improve and retain higher beach levels along stretches of coastline where this is necessary, although unless supplemented by beach recharge this is often accompanied by increased rates of beach lowering and retreat further along the coast. Traditional vertical-sided timber groynes are now being replaced in some areas by rock structures, which allow the possibility of adding T- or Y-heads to reduce scour along the main stem of the groyne. In addition, some rock groynes have been built with walkways along their crest, and these have proved popular with holiday-makers as well as local residents.

7.1.3 Underpinning and encasement

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

7.1.4 Adding steps or aprons

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

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

7.1.5 Rebuilding defences to reduce toe scour

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

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

7.1.6 Major beach improvement schemes

Virtually all of the mitigation measures described in the earlier parts of this chapter have been restricted to rather modest lengths of coastline, although some groyne schemes may have been extended, over time, along several, exceptionally more than 10km of beach. While early beach recharge schemes in the UK were restricted to modest lengths of the shoreline, typically only a few kilometres, there has been a growing trend to consider and implement such measures over much longer frontages.

Such schemes directly redress the losses of sediment that predominantly cause beach lowering, and immediately reduce the risk of undermining of defence structures. In all but a few exceptional cases, recharge schemes also reduce the flooding risks caused by wave overtopping. Further advantages usually include an improvement in the amenity, recreational and conservation value of the beaches, and an improvement in beach levels along adjacent stretches of the coast.

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

7.2 RECOMMENDATIONS

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

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

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

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

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

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

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

8. References

Allsop NWH, Bruce T, Pearson J & Besley P, 2005. “Wave overtopping at vertical and steep seawall”, Proc. ICE Maritime Engineering Journal, ISSN 1741 7597, September 2005, pp 103-114, Thomas Telford, London.

Allsop NWH, Bruce T, Pearson J, Alderson JS & Pullen T, 2003. “Violent wave overtopping at the coast, when are we safe?”, Int. Conf. on Coastal Management 2003, pp 54-69, ISBN 0 7277 3255 2, Thomas Telford, London.

Allsop NWH, McKenna JE, Vicinanza D & Whittaker TJT, 1996a. “New design formulae for wave loadings on vertical breakwaters and seawalls”, Proceedings of 25th Int. Conf. Coastal Engineering (ASCE), September 1996, Orlando, ASCE, New York.

Allsop NWH, Vicinanza D & McKenna JE, 1996b. “Wave forces on vertical and composite breakwater” Strategic Research Report SR 443, pp 1-94, HR Wallingford, March 1996, Wallingford.

Allsop NWH & Williams AF, 1991. “Hydro-geotechnical performance of rubble mound breakwaters” Report SR183, February 1991, HR Wallingford.

Allsop NWH & Wood L A.1987. “Hydro-geotechnical performance of rubble mound breakwaters” Report SR98, March 1987, Hydraulics Research, Wallingford.

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

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

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

CIRIA (Construction Industry Research and Information Association), 1991. “Manual on the use of rock in coastal and shoreline engineering (SP083)”, pp640, ISBN: 0-86017-326-7.

Crossman M, Bradbury AP & Allsop NWH, 2002. “Innovative rock structures: opportunities and constraints” Proceedings of 28th Int. Conf. Coastal Engineering (ASCE), Cardiff, July 2002, pp 1462-1471, ISBN 981 238 238 0, published by World Scientific Publishing, Singapore.

Crossman M, Segura-Domínguez S & Allsop NWH.2003. “Low cost rock structures for beach control and coast protection - Practical design guidance”, R & D Technical Report (CSA 6020), DEFRA / Environment Agency.

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

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

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

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

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

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

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

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

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

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

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

Sutherland, Brampton, Motyka, Blanco and Whitehouse, 2003. *Beach lowering in front of coastal structures, research scoping study*. Defra report FD1916/TR1.

9. Acknowledgements

This work was funded by Defra as part of the joint Defra/EA Flood and Coastal Erosion Risk Management R&D programme under contract FD1927 "Understanding the lowering of beaches in front of coastal defence structures, Stage 2."

We gratefully acknowledge the assistance given by many individuals for their assistance during the course of this study, in particular Alasdair Smith, Aberdeenshire Council for use of Plate 8.