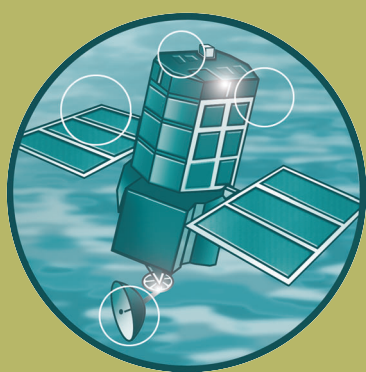


Beach lowering in front of coastal structures

Research Scoping Study

R&D Technical Report FD1916/TR



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

Beach lowering in front of coastal structures

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R&D Technical Report FD1916/TR

This also constitutes HR Wallingford Report SR 633

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Executive Summary

Toe scour is blamed for the failure of many coastal structures (CIRIA, 1986) but toe scour holes are infrequently observed in the field (e.g. Griggs *et al*, 1994). This leads some, such as Weigel (2002a, b, c) to believe that a beach will go through the same cycle of erosion and accretion, whether it has beach control structures or not. Many studies of toe scour have been carried out, but the results have been highly varied. Therefore, this scoping study was commissioned to improve understanding of beach lowering in front of coastal defence structures. The objectives of the report were:

- To identify the generic elements and processes involved and the research and development needs;
- To provide preliminary guidance on the mitigation of scour.

Coastal defence structures are commonly constructed because of coastal erosion. This erosion will continue, despite the presence of the seawall. The seawall neither adds nor removes sand, although it does impound or imprison it, preventing it from entering the coastal sediment transport system. Seawalls can cause local toe scour during storms, but there is no evidence that coastal defence structures delay the recovery of beaches. Beach lowering is a process that takes place on a number of different timescales (years, seasons, storms) and which combines cross-shore and longshore sediment transport. This report concentrates mainly on toe scour, which is the short term lowering of beach level close in front of a coastal defence structure. The overall lowering of beach levels, which occurs at longer timescales and over larger spatial scales, is referred to as general beach lowering. Most beach lowering studies have considered toe scour only and have treated it as a purely cross-shore transport phenomenon.

There are indications that toe scour may be a short-lived phenomenon, with scour holes generated during storms filling in within a few hours as the storm subsides. This would explain why few scour holes are observed or surveyed at low tide. Toe scour has been reproduced in several small-scale laboratory experiments (e.g. Fowler, 1992) which have treated it as a wave-driven, cross-shore, often bedload transport dominated phenomenon. There have been few laboratory toe scour tests that have generated suspended sediment transport despite the fact that bedload and suspended load scour occur by different mechanisms and occur in different places (e.g. Irie and Nadaoke, 1984). It is therefore questionable whether small-scale bedload transport experiments provide reliable design guidance on toe scour depths at full scale (Tørum *et al*, 2003).

Design relationships for scour depths in sand beaches (unless otherwise stated) at vertical seawall, subject to normal incidence waves include those from Xie (1981, 1985) Fowler (1992) Powell and Lowe (1994) for shingle, Powell and Whitehouse (1998) O'Donoghue (2001) for bedload and CEM (2002). Even if the empirical equations derived from laboratory tests are taken as reliable, there is still no design equation for the following cases:

- Oblique incidence waves at vertical walls;
- Oblique incidence waves at sloping impermeable walls;
- Oblique incidence waves at permeable sloping walls (such as rubble mound breakwaters).

In most of these cases toe scour can be estimated from normal-incidence vertical wall cases, either by taking this as the likely worst case or by adjusting it according to rules-of-thumb. In other cases, such as breaking waves at normal incidence on a sloping impermeable wall the only design guidance comes in the form of rules-of-thumb. There is little evidence of these formulae being used in the design of coastal structures or in the design of mitigation measures, indicating a lack of belief in these methods amongst designers.

The above (physical and numerical) modelling has regarded toe scour as a short-term wave-driven phenomenon caused by cross-shore transport of sand (or shingle). Case studies in the UK (Appendix A) have often indicated that longshore transport plays an important and sometimes even dominant role in beach lowering in front of coastal structures. Variations in beach level due to changes in longshore transport tend to have longer timescales (up to the centuries required for coastal realignment) than toe scour.

Many of the mitigation schemes implemented around the UK have involved protective aprons at the toe of a pre-existing structure. Consequently, much of the design guidance is for this type of mitigation measure and examples of guidance on thickness, width and stone weight for revetment toes are provided. Alternative mitigation measures include the following:

- **Rock dumping for bed stabilisation** - This can be a crude form of apron or toe berm that fills the scour hole, but has no filter layer or geotextile so is subject to winnowing;
- **Mattresses** - These can be of two main types: flat gabions and linked precast units. They absorb energy, are flexible enough to fit an irregular seabed, are cheap to fill and relatively easy to lay. Neither has a filter layer or geotextile so both forms are subject to the winnowing of bed material;
- **Soil improvement to increase bearing capacity and reduce scour potential** - An example of this, which is in use at a few sites, is beach drainage (Shaw, 2003). In the UK there have been three beach drainage installations: a full scale trial at Holme-Next-The-Sea (Norfolk), a commercial system at Towan (Cornwall) and an experimental system at Branksome Chine (Dorset);
- **Beach renourishment** - In many cases the long-term development of beach levels depends more on longshore transport than on cross-shore transport. In some such cases, beach nourishment has been the solution to scour problems.

The approach to mitigation schemes is often practical and empirical. Further information on the performance of mitigation schemes is needed in order to assess how successful the approaches are. The performance of a selection of

mitigation schemes should be looked at in some detail. It would be useful if this work could also be tied into an ongoing survey programme to maximise the benefits from funding.

Designers/coastal managers are encouraged to state the assumptions made about beach lowering, and to define a minimum beach level for triggering intervention. Beach lowering happens on such a wide range of time scales and space scales that the entire process cannot reasonably be modelled in a single numerical or conceptual model. Therefore a variety of approaches should be taken to address these issues and the first steps should be taken towards the development of a probabilistic risk-based method of assessing the safety of coastal defence structures. Such an approach would be developed to give information on the range of possible beach levels to be expected in different scenarios. It will require further development of fragility curves for generic structure types.

There is a dynamic interaction between a beach and a coastal defence structure. Relatively few studies have considered the interactions between beaches and structures (preferring to study one or the other). Areas to consider using physical and numerical modelling are:

- Flow through structures and beaches;
- Suffusion and material retention;
- Foundation support, including settlement and liquefaction.

There is a shortage of large-scale laboratory and field data on toe scour. Both should be collected to fill gaps in existing knowledge. In particular, the development of the depth of scour during storms is of interest. Any field experiment devised should be tied in with existing survey programmes to maximise the use of resources.

There should also be some consolidation of recent research to determine the relative importance of hydraulic induced scour and pressure induced liquefaction on bed levels/properties at the toe of coastal defences. Most studies have concentrated on sand beaches and impermeable structures. More work is needed to determine the relationship between scour depth and sediment type.

Key strands from the above have been presented in a section of the report outlining recommendations for research. Research into scour and the performance of structures could potentially lead to cost savings should it show that less conservative designs for coastal structures would be appropriate. Burgess (2003) compiled the following approximate potential cost savings for each kilometre of coastal defence:

- Reducing toe depth by 0.5m could save £50,000;
- Reducing crest level by 0.2m could save £50,000;
- Reducing armour thickness by 0.2m could save £100,000;
- Steepening structure slopes from 1 in 2.0 to 1 in 1.5 could save £150,000.

There are therefore considerable potential long-term returns from investment in beach lowering research, and the dissemination of knowledge and guidance. In order to safely minimise the cost of coastal defence structures, the minimum safe size of toe protection and the minimum safe depth of toe excavation should be investigated.

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Appendix 3	Details of tell-tail scour monitors

1. Background and objectives of study

Beach lowering, including the effects of toe scour, in front of coastal defence structures is recognised as one of the principal causes of their failure. This document summarises the outputs from a scoping study to identify the generic elements and processes involved and the Research and Development needs in this area. It also outlines the research and development needs to improve our understanding of the lowering of beaches in front of coastal defence structures caused by toe scour and provide preliminary design guidance to best practice for the mitigation of the scour.

This scoping study includes a review of beach lowering in front of structures and preliminary guidance on its mitigation. Both of these review aspects will be useful to coastal engineers and stakeholders. However, the main use of the results will be to guide Stage 2 – which will be the implementation of the research identified in Stage 1 and the production of an improved set of guidance notes. Hence it is not intended that this document forms design guidance.

1.1 Statement of problem

Lowering of the “ground levels” in front of seawalls, revetments or other coastal structures is a common phenomenon not only in the U.K. but also around the world.

In some circumstances, the beach becomes flatter and lower over a wider area in front of the structure, sometimes with the sand or gravel being largely removed to reveal the underlying rock of the shore platform (Plates 1 and 2). In other cases, a further and more localised effect can occur, leading to formation of a “trough” along the seaward toe of the structure (see Plates 3 and 4).



Plate 1 Beach and shore-platform lowering, Shakespeare Cliff, Thanet



Plate 2 Removal of beach sediments from shore platform, Colwyn Bay



Plate 3 Localised scour runnel at toe of solid seawall, Wyre, Lancashire



Plate 4 Localised scour runnel at toe of permeable seawall, Lincolnshire

Both localised scour and the more widespread beach lowering can lead to problems, the greatest of which is the undermining of the foundations along the seaward toe of the structure, which can lead to its partial or total collapse (see for example Plates 5 and 6). For example, a comprehensive survey published by CIRIA (1986) concluded that toe scour represented the most prevalent and serious form of damage to seawalls in the UK.



Plate 5 Beach lowering and undermining of seawall, Lee-on Solent

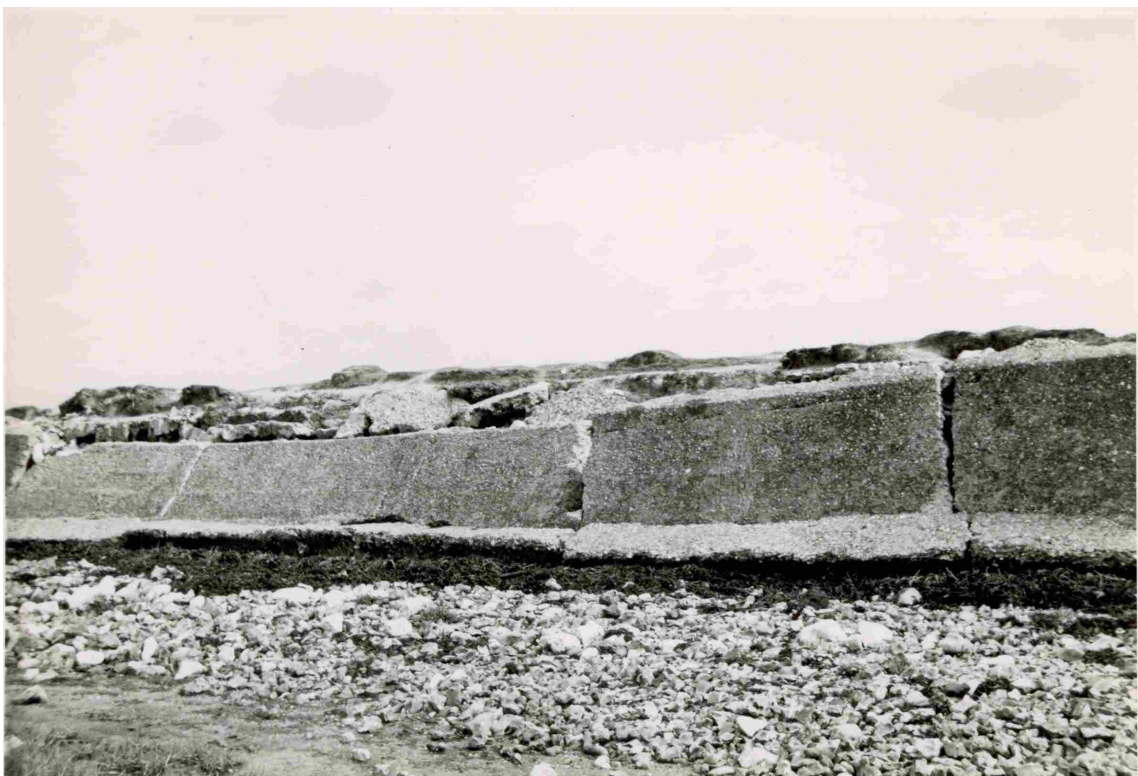


Plate 6 Scour, undermining and collapse of seawall, Farlington Marshes, Hants

Coastal managers are often faced with the consequences of this problem, and have to undertake expensive remedial works to restore an adequate standard of defence. “Traditionally” this involved encasement, underpinning or adding an

apron at the toe of a wall. Many walls show that such remedial works have had to be undertaken on several occasions, as the problem of lowering continues. In Plate 1 for example, the original steps finish well above the base of the seawall that has been extended downwards. The move away from the use of near-vertical concrete seawalls in areas experiencing more widespread beach lowering to “softer” engineering has had some advantages, at least in reducing localised problems of toe scour. In addition, similar types of engineering structures have been used to improve the performance of older structures and extend their life (e.g. the placing of a rock fillet against the toe of a concrete seawall).

The interaction between seawalls and beaches is extremely complex. There has been much debate about whether the presence of seawalls initiates or accelerates beach erosion (for example Kraus, 1988; Tait and Griggs, 1990; Pilkey and Wright, 1988, Wiegel 2002a, b, c). Erosion is clearly occurring on many beaches truncated by seawalls, but it is difficult to differentiate between erosion caused by the seawall and that due to 'regional' or widespread erosion. Most seawalls, after all, are built because the shoreline is eroding.

While seawalls have not been proven to actively cause wide scale beach erosion (Kraus and McDougal 1996), there is indisputable evidence that localised scour may occur at the base of, or close to, the seawall. This type of erosion is termed toe scour and it is often evident as a trough running parallel to the wall (see Plate 3 and 4).

It is evident that failure of a coastal structure due to toe scour will most likely be initiated when the ground level in front of it extends below the toe of the foundations. Localised and short-term scour effects can add to this problem.

However, the development of toe scour is a dynamic process, highly dependent on the water level at the wall as well as the incident wave conditions. In areas of varying tidal range and wave climate, the development of a scour hole will be an erratic process with periods of erosion followed by infilling, and perhaps even general accretion of bed levels (Powell and Lowe, 1994). The scour hole itself may therefore be a short-lived feature with no obvious evidence of its seriousness, or perhaps even its existence, after a storm has declined and infilling has taken place as the tide recedes. Hence, there is a need to be able to predict the maximum vertical excursion of the scour hole during storms, as well as the more widespread and longer-term processes that cause the lowering of beach/shore-platforms. This is important both in the design stage of a coastal structure, and in its subsequent monitoring if the risk to the future integrity of the wall is to be fully understood and timely remedial action undertaken.

Sea level rise and climate change may also affect the likelihood and severity of toe scour. Rising sea levels will result in greater water depth at the structure toe, which will allow higher waves to reach the toe. Rising sea levels will also affect tidal range and the propagation of storm surges in ways that are yet unclear. Sutherland and Wolf (2002) and Hulme *et al*, (2002) have looked at future wind speeds and directions. In both cases the changes were small and

uncertain. Sutherland and Wolf (2002) calculated changes in wave heights and concluded that these would generally be less than 5%. Similarly, longshore drift rates were generally within 20% of present day rates, within the uncertainty range of present day sensitivity studies.

Climate change will effect the wave / structure interaction and the velocities and turbulence levels in front of the structure. They may increase or decrease the scour depth in front of a structure, in ways that will be described later in the report. As the effects of climate change on surge height, wave height and direction are unclear and the predicted increase in sea level rise depends on the scenario modelled, designers of structures are encouraged to look at the potential scour problem associated with a number of sea level rise scenarios. They are also encouraged to review the literature on this subject and adjust recommendations periodically as advice becomes more certain.

1.2 Objectives of this research project

The objectives were set out in the Terms of Reference in the invitation to tender for this research project (to be carried out as a desk study). They are repeated below:

“The project comprises of two stages. The overall objectives of Stage 1 (this scoping study) are to:

1. Undertake a scoping study to identify the generic elements and processes involved and the research and development needs;
2. Define a research project to improve our understanding of the lowering of beaches in front of coastal defence structures caused by toe scour and provide design guidance to best practice for the mitigation of the scour;
3. Provide preliminary advice regarding the mitigation of the scour.”

In addition, existing practice for the mitigation of beach lowering has been reviewed.

1.3 Method statement

Steps towards the completion of the report have included:

- Consultation with academic researchers;
- Consultation with stakeholders who have a scour problem;
- Review of the main processes involved in beach lowering;
- Review of methods for predicting the scour depth at both sand and shingle beaches;

- Review of the effect of different structure types on the depth and extent of scour;
- Review of methods of assessing the long-term variations in beach levels;
- Review of scaling issues regarding the sediment transport processes in laboratory models;
- Re-examination of existing data to identify the scale and dominant processes;
- Identification of shortcomings in the available data sets;
- Some new analysis of existing data;
- Internal expert panel discussions;
- Discussion of draft report at COZONE meeting (4 June 2003);
- Review of report by Peer Review Panel.

1.4 Layout of report

This report has six chapters:

1. **Background and objectives of study** (this chapter);
2. **Review of existing knowledge.** This covers mechanisms and parameters, wave kinematics, sediment transport regimes (bedload and suspended load), prediction methodology, a review of scale effects in laboratory models and a summary of existing scour experiments;
3. **Review of methods for predicting beach lowering;**
4. **Review of mitigation methods.** This includes a description of precautionary methods that can be incorporated at the design stage, monitoring methods and localised intervention methods;
5. **Research and development needs.** A summary of research and development needs, aimed at identifying areas where research into processes could be of most use and how guidance on the prevention and mitigation of scour could be improved;
6. **References and bibliography.** This includes more papers than are referenced in the report.

In addition there are three appendices. The first contains nine case studies where beach lowering has led to remedial action being taken. The second contains details of published methodologies for determining scour depths at coastal structures. The third contains information on the HRW Tell-Tail scour monitors referred to in the report.

2. Review of existing knowledge

2.1 Introduction to beach lowering around the UK

Lowering of “ground levels” in front of seawalls, revetments or other coastal structures is a common phenomenon not only in the UK but also around the world.

In some circumstances, the beach becomes flatter and lower over a wider area in front of the structure, sometimes with the sand or gravel being largely removed to reveal the underlying rock of the shore platform (Plates 1 and 2). In other cases, a further and more localised effect can occur, leading to formation of a “trough” along the seaward toe of the structure (see Plates 3 and 4).

Both localised scour and the more widespread beach lowering can lead to problems, for example:

- Making access to the beach, e.g. from steps and ramps, more difficult;
- Increasing the water depth in front of the structure, hence allowing larger waves to reach it. This in turn can lead to increased wave forces and wave run-up on the structure, and hence greater damage or overtopping;
- Increasing the height of the waves reflected from the face of the structure;
- Allowing faster tidal current speeds along the face of the structure;
- As a consequence of the increased wave action, and perhaps faster tidal currents, there will be greater water turbulence and increased agitation of the beach sediments in front of the wall.

The last four of these problems may, in some circumstances, lead to a further lowering of the beach levels, and thus an intensification of the problems listed above. Non-specialist texts have been written suggesting that this “vicious circle” of processes and effect continues indefinitely and thus that seawalls and revetments are the cause of the low beach levels that are commonly observed in front of them. While an attractive explanation for lay readers, it is often untrue. Many seawalls were built in response to pre-existing coastal erosion and beach lowering. As will be explained in section 2.3, although they may not have remedied this natural regime of long-term erosion, they certainly are not in many cases to blame for its continuation.

If the levels of the beach or shore-platform fall to below that of the toe of the foundations of the structure, then further problems will usually follow. Flows under the front edge of the structure, and approximately perpendicular to it, will not only remove granular “fill” from behind the face of that structure but could also further add to the lowering of ground levels in front of the toe (see Plate 3). Many seawalls have collapsed as a result of this “undermining” process (see Plate 4), and indeed along with outflanking, this is the main cause of such failures (see CIRIA, 1986). (A seawall is outflanked when erosion occurs at the end of a seawall, allowing the removal of material from behind the structure.)

2.2 Review of literature and UK case histories

During this project, an extensive literature review has been carried out, in order to obtain information on a variety of aspects of the lowering of beach levels namely:

- Past and present views on the mechanisms that cause the lowering;
- Prototype experiments on, or measurements of lowering;
- Laboratory experiments of scour in front of coastal structures;
- The advice given to those designing coastal structures on predicting such lowering;
- Advice given on reducing such lowering or mitigating its effects;
- Examples of problems experienced around the UK coastline and the remedial measures taken.

The information obtained regarding the first of these aspects is summarised in the following section of this report, which sets out a “conceptual framework” explaining the various processes involved. This review, briefly, has resulted in the separation of the mechanisms into long-term and widespread lowering, and more localised and shorter-term scouring.

There seems to be little written on the measurement of beach lowering in front of seawalls or other structures around the UK coastline, and hence the second aspect of the literature review is strongly reliant on studies carried out in the United States. Here the findings have generally been that lowering of sandy beaches in front of seawalls is almost entirely due to the longer-term and widespread processes, which do not depend on the seawall characteristics. Evidence for localised scour in front of walls in the US, however, is remarkable by its absence in reports.

A comprehensive survey providing details on the causes of failures of seawalls in the UK was published by CIRIA in 1986. In this report it was concluded that toe scour represented the most prevalent and serious form of damage to seawalls in the UK. It directly accounted for over 12% of the case histories studied and indirectly responsible for up to a further 5% of cases, including collapsing/breaching of seawalls and washing out of fill materials. Similar conclusions were drawn by Markle (1986) in the US for rubble-mound structures.

There have been few experiments into scour in front of seawalls around the UK coastline, although some studies have been carried out by HRW at Blackpool, Shoreham and Teignmouth. These demonstrated the complexity of the hydraulic processes, and measurements from the former site show the lowering and recovery of beach levels at the base of the wall in the short-term, i.e. over a tidal cycle. Therefore, measurements of beach profiles made at successive low tides would not reveal the changes in level that occurred when the beach was submerged.

Measurements made at a shingle beach at Shoreham gave a noisy signal that was difficult to interpret. This may have been due to the greater permeability of a shingle beach allowing stronger flows within the beach. The active layer of sediment may also have been deeper, making it more difficult to determine where the surface of the beach was. Vertical movements up to 800mm down and back up were nevertheless recorded during a single high tide period.

This leads on to the third aspect, namely the review of laboratory experiments of scour in front of coastal structures (i.e. seawalls and breakwaters). This part of the literature review forms the basis for chapter 3 of this report, in particular the review of the scour mechanisms and the predictions of the depth of scour in front of the toe of the structures.

Despite the undoubted importance of beach lowering in front of coastal structures, there is little in the current literature that enables the design engineer to anticipate this phenomenon.

There is some guidance in the US Shore Protection Manual on the maximum depth of scour (see section 3.4), and other authors have reproduced this guidance in later texts. However, the empirical formula given may be very conservative, in the light of practical experience and observations. Little or nothing was found on predicting the long-term changes in beach level, save for that presented in the US report TR4 produced by the Beach Erosion Board in 1954.

More advice is available on reducing the extent of beach lowering, and/ or preventing this effect causing damage. Some of the older books, e.g. Owens and Case (1908), Stanton (1909) and Matthews (1934) suggest various methods for preventing the undermining of wall by scour at their toe, e.g. aprons, groynes and cut-off walls. These texts also advise on reducing such scouring effects, e.g. the use of sloping or stepped seawalls rather than those with a near vertical face. In more recent times, the advantages of introducing a permeable face to a coastal structure, e.g. a sloping rock revetment, have also been realised and this is now a frequently-used alternative to concrete walls.

To complete this review of existing knowledge, a number of case histories have been assembled for various locations around the coastline of the UK. These have been chosen to cover a range of different beach sediments, causes of coastal erosion/ beach lowering and alternative remedial works. These case histories are presented in Appendix 1 to this report.

2.3 Conceptual framework

The lowering of beaches and/or shore-platforms, i.e. ground levels, in front of coastal structures is caused by a number of mechanisms. Some of these mechanisms are largely independent of the type of structure, i.e. they occur whether the structure is permeable or impermeable, whether it is steep-faced and reflects waves or whether it is more gently sloping and dissipates wave energy. Instead, these mechanisms reflect the characteristics and

geomorphological processes of the coast where the structure has been installed.

Other effects that lead to ground lowering, however, are dependent on the characteristics of the structure, but for the most part, these seem to have only localised effects, and are short-lived and reversible, at least on sandy beaches.

Previous investigations have separated out these two classes of effect into “long-term” (i.e. over years/ decades) and “short-term” (i.e. over a tidal cycle, a few days or perhaps seasonally). This is a useful basis for arranging an initial “conceptual framework” of the processes described in this report. There does not seem, however, to be any consistent nomenclature for the various processes involved, and therefore some provisional process “names” have been proposed in the following sub-sections.

Continued shoreline recession

Many coastlines around the world are eroding, as a consequence of the continual and damaging effects of winds, rainfall, waves and currents. This tendency is further strengthened by the gradual rise in sea level, allowing larger waves to travel further inshore, hence increasing their damaging effects on the seabed and beaches.

Where, for example, a seawall is built to protect an eroding shoreline (as sketched in Figure 1A below), it will not directly prevent erosion of the adjacent sections of that coastline. In this hypothetical situation, it is assumed that there is no longshore sediment transport and that the material eroded from the coastline and the shore platform is so fine-grained that it travels out into deep water. (This situation is similar to that found on the coastline of the Great Lakes in North America).

The principal cause of coastal recession in this situation is the continual erosion of the shore-platform where the wave-induced water velocities are high, particularly in the breaker zone and just outside it.

As the erosion of the coast either side of the seawall proceeds, then the ground level either side of the toe of the wall will fall. Now assume that the seawall affects the hydrodynamic/ geomorphological processes in front of it “beneficially” in the short-term, e.g. by reducing the height of the reflected waves compared to those in front of, say, adjacent cliffs. It might then be hoped that the ground levels in front of the wall would not lower as quickly as those in front of the cliffs.

However, this would require an increasingly steep lateral slope to the ground levels (in this case the shore-platform levels) as time passes, i.e. at the two ends of the seawall. Experience indicates that this does not occur. It is a reasonable approximation, therefore, to assume that the ground level at the toe of the wall will be (at best) equal to that of the ground level on either side of it. This was the advice provided by the U.S. Beach Erosion Board (1954). Griggs *et al*, (1991) refers to this process as “passive erosion”. In this simple situation,

therefore, the ground levels in front of the coastal structure depend on the ground levels on either side of that structure. If these continue to fall, beach lowering in front of the structure will occur.

Where groynes are built out from the shoreline, along the seawall and perhaps along the adjacent unprotected coastline as well, they can reduce the tendency for levels immediately in front of the wall to match those on either side of it, at least in the short-term. Even so, the ground level at the seaward end of the groynes will be similar to that on either side of the groyne field. To maintain higher levels than would be expected without the groynes would therefore require a steeper bed/beach gradient within the groyne bays than outside.

The simple example sketched in Figure 1A for a short length of seawall may not apply for situations, such as at Blackpool or Bournemouth, where seawalls stretch along many kilometres of coastline. A further complication arises when different types of seawall are present along a stretch of coastline, since the lowering of the beach in front of one section of an energy-dissipating seawall may be altered by the effects of adjacent, more reflective structures.

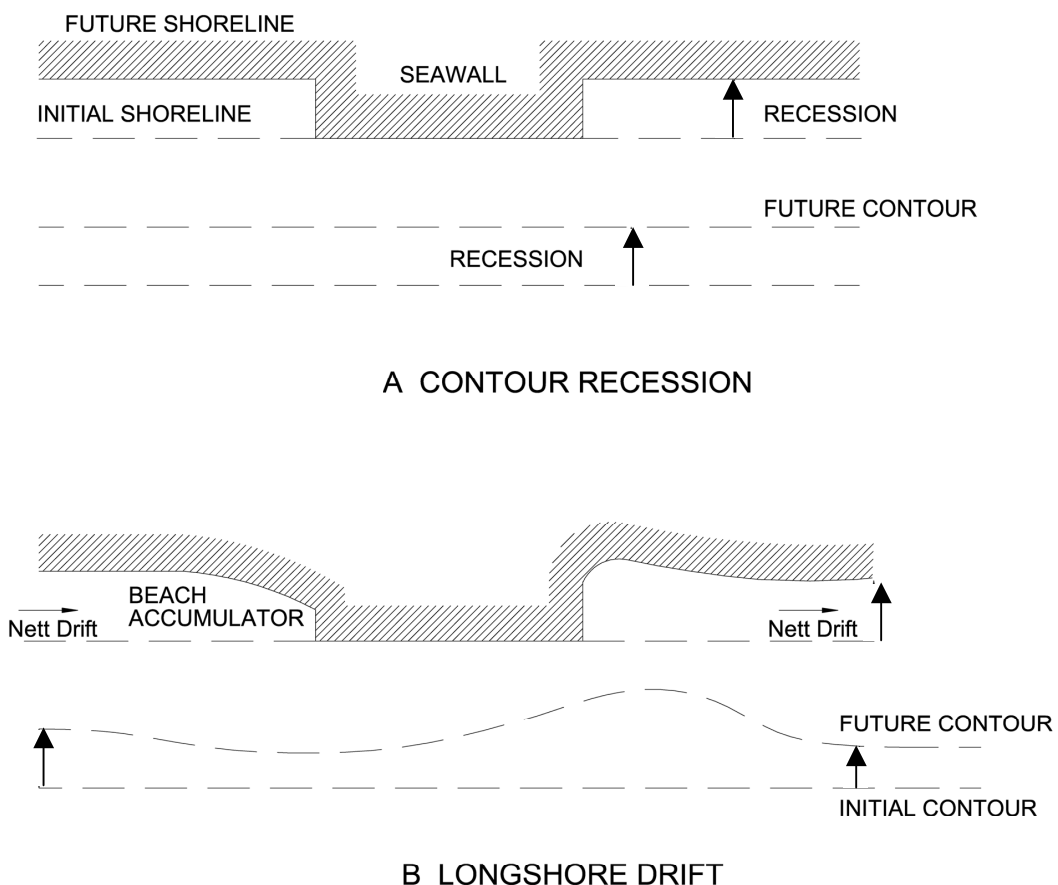


Figure 1 Conceptual sketches of the effect of a seawall on coastal erosion

While the continuing effects of erosion of the shore platform, assisted by sea-level rise, will continue in front of such structures, it is unclear whether this will be faster or slower than would have occurred without the seawall. This is an area where further investigation may be warranted.

In parallel, a Defra funded study on 'Understanding and predicting beach morphological change associated with the erosion of cohesive foreshores' (FD1915) was led by Posford Haskoning and this has also produced a scoping report on relevant topics.

Effects of longshore drift

Many coastal erosion problems are a result of the interruption or alteration of the rates of sediment transport along the coastline, i.e. littoral drift. If we consider the simple situation above, but now assume that there is a beach and a nett longshore drift rate, then the seawall will interfere with that sediment transport. Experience has shown that the normal effect of a seawall on longshore drift is to reduce its rate in front of the wall. As a consequence of the differences between the drift rates in front of an either side of the wall, there then tends to be an accumulation of beach sediments "updrift" of the wall and corresponding erosion downdrift. On the updrift side, the accumulation of beach sediments will tend to compensate a trend long-term recession of the shoreline, and indeed may prevent this from occurring (e.g. the situation at the western end of the promenade at Sheringham – see Plate 7).



Plate 7 Accumulation of shingle at updrift end of seawall, Sheringham, Norfolk

Conversely, the interruption of the drift by the seawall results in greater downdrift erosion problems, at least locally, than would have occurred otherwise (e.g. the situation at the end of the seawall shown in Plate 8 - Zanzibar). This localised erosion problem will be often reflected in the beach/ shore/platform levels just downdrift of the wall, and the ground level contours in front of the wall can be expected to be lower at its downdrift than at its updrift end (see Figure 1B). Notice that where a seawall prevents the erosion of cliffs or dunes that would otherwise, by receding, have provided sediment to the beaches, then there will be a further deleterious effect on the downdrift coast. Many “promenade seawalls” built over the last 200 years around the UK coastline have not only caused this problem (e.g. at Bournemouth – see Plate 9) but have “impounded” (Griggs *et al*, 1991) or “imprisoned” (Case and Owens, 1908), a considerable amount of beach sediment. A photograph taken during the construction of the promenade at North Beach, Llandudno, for example, (from Case and Owens) seems to show that the shingle ridge that ran along the beach there was being used as fill material for the new wall. It seems likely that this practice was commonplace. In some cases, structures may have also have been filled with sand from the beach in front of them, hence leading to the likelihood of an “instant” lowering of the beach in front of and downdrift of the wall.



Plate 8 Erosion at downdrift end of seawall, Zanzibar



Plate 9 Seawall preventing sand supply from cliffs, Bournemouth

It should be remarked, however, that the processes by which a seawall (or other shore-parallel coastal structure) affects the longshore drift rate are not fully understood. A vertical wall situated in moderately deep water might totally reflect small waves, before they can break. As a consequence, while the agitation of the seabed in front of the wall may increase (because the wave energy passing over it is doubled), the waves will not break and the only longshore current will be due to streaming in the wave boundary layer. Therefore, for example, while the amount of beach sediment in suspension will increase, there may be a decrease in the local longshore transport rate compared to that experienced on a natural beach because of the lack of breaking wave induced currents to carry that sediment along the shoreline.

This very simple analysis, however, conflicts with observations made at Teignmouth (Miles *et al*, 1997). Here, both increased wave activity and increased longshore currents were measured in a situation very similar to that described above. For more complex situations, where waves have started to break over a beach before reaching the coastal structure, and where that structure dissipates some of the incident wave energy that reaches it, then the effects on longshore sediment transport are correspondingly more difficult to anticipate. This is an area where fundamental research could be usefully undertaken, for example to improve the representation of such structures in numerical models of shoreline evolution. Such modelling could then provide better predictions of the evolution of the coastline either side of the seawall, hence providing an initial prediction of the lowering of the ground levels in front of it (see Figure 1B).

As with the discussion in Section 2.3., this description of the effects of longshore drift is valid for a short section of seawall, or a similar coastal structure, on a coastline without groynes. Where there are long lengths of seawall, possibly of different character, and/ or where the longshore drift is being altered by the presence of groynes, the situation inevitably becomes more complex.

There is, therefore, a need to combine the representation of the mechanisms of shore-platform lowering, and the effects of seawalls or revetments on longshore drift rates, into a modelling framework that predict the long-term changes of long stretches of coast with various types of structures.

Localised scour problems

As discussed in the introductory chapter to this report, there is a commonly-held view that ground lowering in front of coastal structures such as seawalls is a direct consequence of the installation of that structure, leading to scour of the beaches or shore platform. As can be seen from Section 2.3. (p 12. Continued shoreline recession) above, this is certainly not always the case. Indeed, it might well be that the structure has no long-term influence on the lowering of ground levels. The inverse, however, is often not true as the undermining of so many seawalls following lowering of the ground levels in front of them has demonstrated.

However, this does not rule out short-term (and typically more localised) effects of scour, particularly at the seaward toe of such structures. There are apparently a number of hydrodynamic processes that can contribute to this scour, as follows:

Wave reflections Either partial or total, that increase water pressures and orbital velocities and the agitation of beach/ seabed sediments and in some circumstances produce a strong vertical water motion that impacts on the bed in front of the structure.

Currents Currents flowing along the coastline can become “trained” against it, hence becoming stronger close to the face of the wall. This is particularly a concern near the mouths of tidal inlets or river mouths.

Groundwater flows Current flowing perpendicular to the coastline, e.g. down the face or underneath the base of the structure (e.g. springs) can also add to the mobility of sediments just in front of the wall.

Permeability The construction of an impermeable barrier in a permeable beach will alter the water flows within the beach/ seabed sediments and hence tend to destabilise them (e.g. Muir-Wood, 1969). This probably has a greater effect on shingle beaches than on those of sand.

The consequences of these effects on the hydrodynamic regime in front of a coastal structure may lead to the redistribution or removal of beach/ seabed sediments either along or perpendicular to the contours, and hence to lower levels in front of the structure. This process was called “active erosion” by Griggs *et al*, (1991). The expected consequence of this process is the formation of a scour “trough” directly in front of the structure, perhaps of sufficient depth to allow the foundations of the wall to be exposed and undermined.

This type of effect, unlike the longer-term and less localised effects described previously, can be expected to depend on the type of structure, i.e. on the slope, roughness and permeability of its face, and on its plan-shape. This is the process of principal interest in this study, since it is possible that such scour problems could be reduced, or even avoided entirely, by altering the types of structures that are built.

Such localised scour effects are often seen in laboratory experiments (see Section 3.5) but are less easy to find in prototype situations. There is a concern, however, that these effects may occur during storms, and at the time of high tidal levels when it is not possible to observe the ground levels in front of the structure. It is known that beach levels can recover rapidly after such events, i.e. within a few days after a severe storm, or possibly be significantly higher by the time of the next low tide. Clearly, it is the level of the ground in front of a coastal structure at the time of high tide, during a storm, that affects the wave heights and hence the wave-induced forces, run-up, overtopping etc. at that crucial time. Even such short-lived changes in bed level due to scour can therefore be important. Figure 2 below is a sketch of the hypothetical changes in ground levels in front of a coastal structure over a few days, indicating this potential concern. This sketch indicates the possibility of changes in level during each tidal cycle, with lower bed levels occurring at times of high water. Notice also that this sketch suggests that bed levels might well have been completely restored only 1-2 days after the peak of the storm. This is in accord with observation made along coastlines in this country and the US, which suggest that even in front of seawalls, the recovery of beach levels is rapid. Actual data on scour depths in front of seawalls during storms is lacking. Therefore it is impossible to say how realistic the conceptual model is.

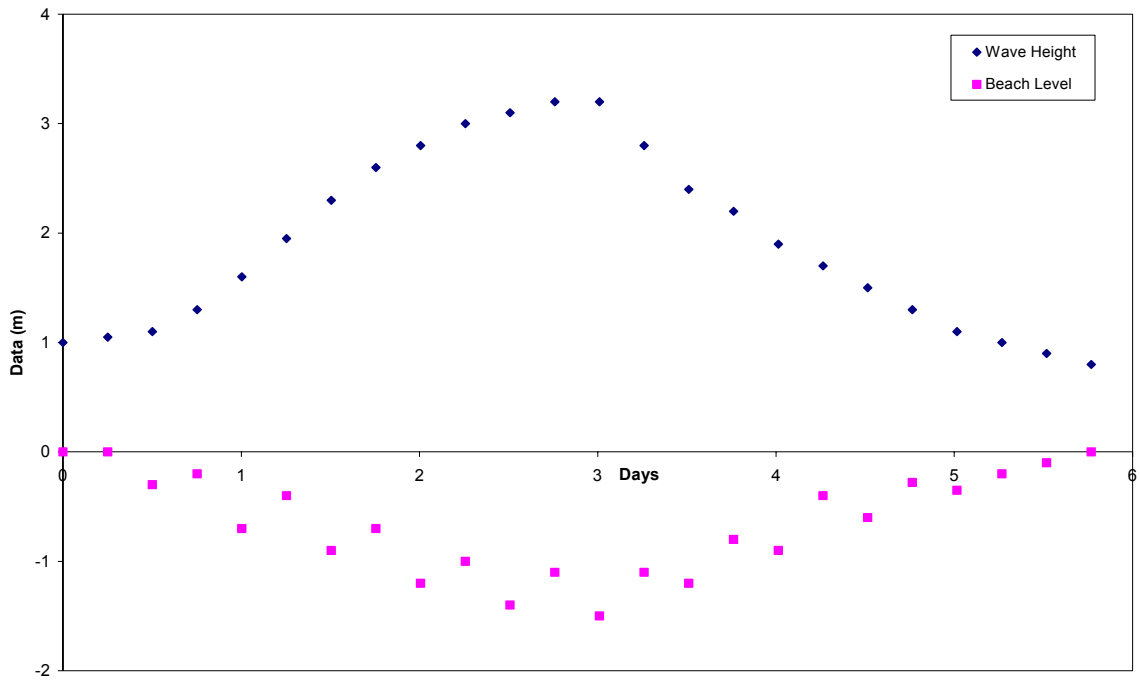


Figure 2 Conceptual sketch of toe scour during a storm

3. Literature review of methods for predicting beach lowering

3.1 Mechanisms and parameters controlling toe scour

This section outlines the scour mechanisms and lists the environmental parameters that control toe scour. Some of these parameters are further investigated in other sections of the report.

Seawall toe scour occurs when the base of the wall can be acted upon by waves, either directly, when the sea level is higher than the bottom of the wall, or through wave run-up. The presence of a structure in relatively shallow water, for example, abruptly breaks the wave and the energy is dissipated within a much smaller zone than on a natural, unimpeded beach profile. This sudden release in energy is converted into turbulence and wave reflection. The extra kinetic energy released around the toe of the seawall induces lowering of the beach at the bottom of the wall by:

- increasing local shear stress on the bed to levels exceeding the threshold for sediment motion;
- generating shock waves through the impact of waves breaking on the seawall (the pressure waves set up in the water column are transmitted to the bed, and away from the wall, these high pressure gradients disturb the sediment and make it more vulnerable to erosion). Wave-induced liquefaction of bed sediments may become a contributing process – as discussed in Section 3.4;
- increasing removal of the suspended sediment by longshore currents (the extra turbulence sustains sediment motion and allows it to be transported by currents);
- reducing sedimentation (the greater water velocity close to the seawall reduces settlement of sediment brought into the area from longshore drift).

The process of toe scour can be self-sustaining. For example, consider the situation where the beach level at the base of the seawall is above the mean high water spring tide level and therefore not vulnerable to scour under normal conditions. Once an exceptional storm (surge water level plus storm waves) produces initial scour, a greater range of wave/water level conditions can reach the seawall and the beach level in front of the seawall lowers progressively.

As the beach lowers further the water table is closer to the surface, pore pressures increase and the sand can be fluidised, thus the degree of sediment removal through backwash increases (Powell and Lowe, 1994). Periodically, conditions may allow a recovery of the beach level if there is a sufficient sediment supply, but for narrow beaches with a sediment deficit, it may never accrete to the pre-scour level. Further discussion about the processes of scour can be found Whitehouse (1998), Kraus and McDougal (1996) and Sumer and Fredsøe (2002) amongst others.

The extent and type of scour process is dependent on: (1) the wave climate and water level (2) the beach and (3) the design and position of the seawall on the shore profile. The remainder of this section discusses the influences of the first two categories on toe scour; the influence of seawall design will be discussed in Section 4.1.2.

Wave Climate and Water Level

The following wave/water level characteristics dictate the extent of accretion or erosion at the toe of the seawall:

- Wave height
- Wave period
- Water depth at the toe of the seawall
- Storm duration
- Angle of wave approach
- Overtopping

(a) Wave height and period

Wave height and period determine offshore wave length (L_m). Evidence suggests that low steepness waves (i.e. a low H_s/L_m ratio) cause greater toe scour than steeper waves (see iso-parametric plots in Appendix 2). This conclusion is supported by the work of Herbich *et al*, (1965), Sato *et al*, (1969) and Yokoyama *et al*, (2003), although the influence of depth should not be ignored. Powell and Lowe's results (1994, see Appendix 2) also show significant scour depths for steep waves ($H_s/L_m \approx 0.06$) at a toe wall depth around zero.

(b) Water depth at the toe of the structure

The depth of water at the toe of the structure relative to wave height governs the wave orbital velocities at the bed. Physical and numerical modelling conducted previously at HR Wallingford (Powell and Lowe, 1994, Powell and Whitehouse, 1998) indicates that the most severe toe scour occurs for relatively low steepness swell waves ($H_s/L_m = 0.005$) when the initial water depth at the toe is approximately twice the offshore wave height ($d_w/H_s = 2$). This relationship was found to hold true for both the coarse sediments tested in a laboratory wave flume ($5\text{mm} < d_{50} < 30\text{mm}$) and numerical model results for sand (0.2mm). Yokoyama *et al*, (2003) noted that the largest scour depths occurred around $d_w/H_s = 2.5$. This is approximately at the initiation of breaking and agrees with Tanaka (1974).

(c) Storm duration

The duration of the wave/water level conditions is also an important control on toe scour development. Scour is not an instantaneous process - the trough deepens over a number of waves. Figure 3 (Powell and Lowe, 1994), derived from physical wave flume models of a coarse grained beach, demonstrates how scour develops until a quasi-equilibrium is obtained within about 3000 waves. It illustrates the typical pattern of rapid initial scour that declines exponentially towards the equilibrium depth. Similar trends are also apparent for sand beaches, though results from model studies (McDougal *et*

al, 1986) suggest that the scour hole is slower to develop, with equilibrium unlikely to be achieved within a realistic storm/water level duration. This is supported by the result contained in Powell and Whitehouse (1998), (also see Figure 3). See Section 3.5.4.

The studies mentioned above were all conducted at a constant water depth. Few, if any, studies have looked at the effect of varying tidal levels on scour depths even for constant wave conditions. As water levels vary through the tidal cycle, potential scour depths, and even the position of scouring will vary. In some cases this may be expected to smooth out the profile of wave-induced scour.

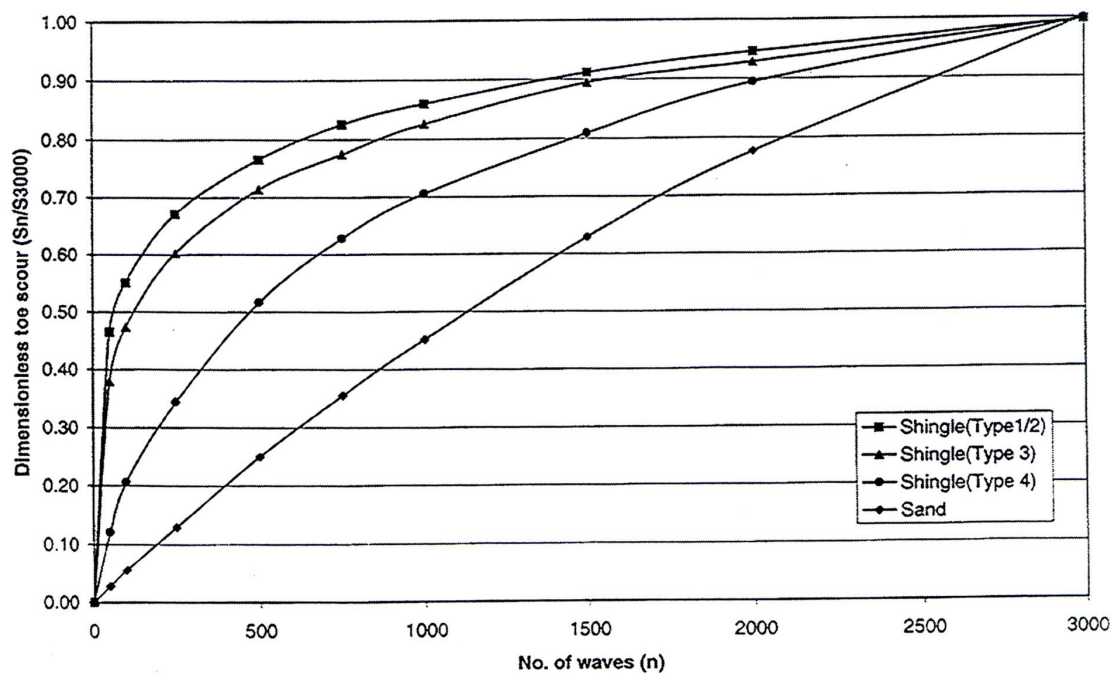


Figure 3 Time development of scour (after Powell and Lowe, 1994; Powell and Whitehouse (1998)). Types given in Table 3

(d) *Angle of wave approach*

The angle at which the wave front hits the seawall has also been proposed as a factor affecting toe scour (Hsu and Silvester, 1989). The depth of toe scour is expected to be greater if waves hit the wall obliquely, because the incident and reflected wave trains interfere with each other *constructively*, producing an interference pattern of short crested waves. Consequently, the wave height and hence scour potential at the base of the wall should be larger if the angle of incidence is oblique rather than perpendicular to the seawall. In addition, oblique waves may induce local currents parallel to the seawall, (Lin *et al*, 1987 and Oumeraci, 1994), which enhance sediment removal at the toe of the structure.

(e) *Overtopping*

A further factor of importance may be the extent of any overtopping of the seawall. It is reasonable to expect that seawalls that experience heavy wave overtopping will offer less scour because the proportion of energy reflected

or dissipated as turbulence at the wall will be reduced. This effect has probably not been taken into account in previous studies of toe scour, for which the majority of walls appear to have been of sufficient size to limit the extent of any wave overtopping. Thus most empirically based methods for the prediction of toe scour may be conservative if applied to low crest structures, which experience regular overtopping. Similarly, to date, most numerical models can only simulate overtopping by reducing the reflection coefficient for a given seawall profile. Recent developments in phase-resolved modelling of non-linear shallow-water waves (e.g. Dodd, 1998) have allowed wave-by-wave overtopping events to be modelled. Such models could be coupled with sediment transport and bed updating models to investigate the effect of overtopping on scour, although such work is in its infancy. Few, if any, models are able to simulate accurately the turbulent dissipation occurring at the wall.

There are no design relationships to take into account the overtopping influence on scour depth. Nishimura *et al*, (1978) studied the scour at seawalls caused by an incident tsunami. In this case the overtopping water returned down the face of the structure and much of the scour was caused by the flow return.

They noted that:

- Scour depth decreases with decreasing wave height and increasing crown elevation (as there is less return flow) however, the area of serious scouring is displaced towards the seawall in this case;
- Scour increases (and it occurs at the toe precisely) when the face slope is mild;
- Scour decreases markedly when the water depth at the seawall increases;
- When waves are applied repeatedly, much less scouring is induced by each successive wave.

Beach Characteristics

Assuming that the volume of material within a beach, and its crest level, are insufficient to prevent wave action from reaching a seawall, then the following beach characteristics will influence the depth of any scour hole:

(i) *Sediment supply*

The amount of sediment supply to a beach greatly influences the impact of seawalls. The larger the supply the quicker storm scour holes are filled, and thus the risk of seawall damage reduced. Most field and laboratory studies, have shown that beach levels at the seawall do recover after storms as long as there is an adequate sediment supply. However, recovery may take longer than for neighbouring beaches without a seawall (Kraus, 1988).

(ii) *Grain size (and erodability of the bed)*

Surprisingly, there is little information in the literature regarding even the qualitative effect of sediment size on toe scour depth. An equation by McDougal *et al*, 1996, presented in Appendix 2 suggests that the smaller the

grain size the larger the scour depth. This is partly supported by the review undertaken in preparation of this report which established that, although similar trends exist for toe scour on sand and shingle beaches, the actual depth of scour for any given sediment is dependent on the precise combination of water depth at the seawall and the incident wave conditions. Thus for relatively moderate wave conditions ($H_s/L_m \leq 0.03$) scour depths are generally greater on sand beaches than on shingle beaches, as suggested by McDougal *et al*, (1996). However for more severe waves ($H_s/L_m \geq 0.03$) greater scour depths may be observed on shingle beaches. A review of laboratory and field data by Yokoyama *et al*, (2003) showed maximum scour depths decreasing as grain diameter increased for laboratory tests. However, limited results from field studies indicated increasing scour depths with increasing grain size. The authors suggested that scour holes in small grain beaches were more likely to fill in and not be observed, as the smaller sand would be more mobile in lower wave condition than larger grains sizes. Alternatively, this may be due to larger grain size beaches having a more volatile wave climate.

(iii) *Depth of beach material*

Toe scour depth may be limited if material with a higher tensile strength, such as a clay platform, underlays the mobile beach sediment, preventing further erosion.

(iv) *Beach slope*

The effect of beach slope on toe scour is yet to be determined conclusively and has long been a matter of debate. Some researchers have found that varying the initial slope has little or no effect on the final beach profiles, while others suggest that shallower beaches are less vulnerable to toe scour than steeper ones under the same set of wave/water level conditions. Previous tests at HRW indicate that toe scour depth decreases with decrease in beach slope angle. This relationship also agrees with the results from other numerical and laboratory studies (McDougal *et al*, 1996 and Ichikawa, 1967). It is likely that bed slope affects scour processes because it determines the critical wave steepness (which effectively divides breaking and non-breaking sea states) and therefore the way in which the wave breaks. For example, for a given offshore wave height period and water level, waves may break by collapsing and plunging on a steep beach profile; whereas on a shallow shore under the same conditions, the mode of breaking could be spilling. Thus there is likely to be more energy available for scour on steep rather than shallower beaches.

3.2 Kinematics in front of a reflective structure

Waves incident upon a coastal structure are reflected from it to some extent. The interaction of incident and reflected waves sets up a partial standing wave pattern in front of the breakwater. It is common in coastal engineering studies involving random waves to consider the random sea as the linear sum of a large number of incident component waves, plus the reflected components (O'Donoghue and Sutherland, 1999). This approach ignores wave-wave

interaction (such as clapotis) but allows solutions to random wave problems to be formulated relatively easily. Each component has a reflection coefficient, given by the ratio of reflected over incident component amplitude (Sutherland and O'Donoghue, 1998b) and a phase shift, which relates the phase of the incident and reflected waves at the toe of the structure (Hughes and Fowler, 1995, Sutherland and O'Donoghue, 1998a).

For example, when a single obliquely-incident wave is reflected from a seawall, the short crested wave field is characterised by a diamond-shaped pattern of 'island crests and troughs' (Hsu and Silvester, 1989). Lines of island crests and troughs occur at regular intervals in front of the structure. The reflection coefficient determines the magnitude of the island crests and troughs from the given incident wave height while the phase shift on reflection gives the distance from the structure to the first line of crests and troughs.

When a regular wave is reflected from a vertical wall (with a reflection coefficient K_r of 1 and zero phase shift) the incident and reflected components are in phase at the wall, so an anti-node is formed. This is an area with a relatively high root-mean-square (rms) surface elevation but zero horizontal velocity. On moving away from the wall the incident and reflected components move out of phase until they are completely out of phase (when the incident and reflected surface elevations cancel each other out so there is no surface movement but a maximum in horizontal velocity). This occurs at a distance of a quarter wavelength in front of the wall. On moving further out, the incident and reflected components move back into phase and another anti-node occurs at a distance of half a wavelength in front of the wall.

If there is a random sea state, instead of a regular wave, all reflected components will be in phase with the incident component of the same frequency at the wall, hence an anti-node is formed at the wall. On moving further out from the wall, each component will move out of phase at a different rate, as each has a separate wavelength. Therefore the next anti-node out from the wall will have a lower rms surface elevation than the one at the wall, and the one out from that will be lower again. Hughes (1992) showed how the spatial variation in rms surface elevation depends on relative depth (kh , with $k=2\pi/\lambda$ the wavenumber, λ the wavelength and h the water depth) and the narrowness of the wave spectrum. The narrower the wave spectrum, the larger the cross-shore distance over which the partial standing wave pattern will be apparent. Moreover, the shallower the water, the greater the cross-shore distance over which this phase-locking will be apparent. The variation of root-mean-square orbital velocity with distance from the toe of a vertical wall is shown as a function of relative depth in Figure 4 (from Hughes and Fowler, 1991). For a sloping seawall, the incident and reflected components are already out of phase at the structure toe and reflection coefficients are lower than for a vertical wall. Both factors mean that the partial standing wave pattern generated in front of a sloping seawall is less obvious than that in front of a vertical wall (Hughes and Fowler, 1995).

O'Donoghue and Goldsworthy (1995), Hughes and Fowler (1995) and Sutherland and O'Donoghue (1997, 1998) measured the phase shift, γ , in front

of reflective structures. Hughes and Fowler (1995) related wave phase shift to the non-dimensional parameter, χ , given by:

$$\chi = \frac{l}{\tan \alpha} \sqrt{\frac{d_i}{gT^2}} \quad (1)$$

with α the structure slope above horizontal. Sutherland and O'Donoghue (1998) used a number of datasets to derive the empirical equation

$$\gamma = -8.64\pi \chi^{1.22} \quad (2)$$

The Sutherland and O'Donoghue (1998) data is shown in Figure 5. The above equation is Equation 17 in Figure 5. Equation 3 is the simplistic shallow water linear theory phase shift $\gamma = -8\pi\chi$, while Equation 11 is derived by matching standing waves at the toe of the structure and is:

$$\gamma = -2 \arctan \frac{J_1(4\pi\chi)}{J_0(4\pi\chi)} \quad (3)$$

where J_1 is first order Bessel function of the first kind and J_0 is a zeroth order Bessel function of the first kind.

Hughes and Fowler (1995) and O'Donoghue and Sutherland (1999) have showed how to combine linear theory expressions for the kinematics with reflection and phase shift spectra to determine the velocities and surface elevations in front of reflecting coastal structures. These are analytical models that do not include wave breaking due to shoaling.

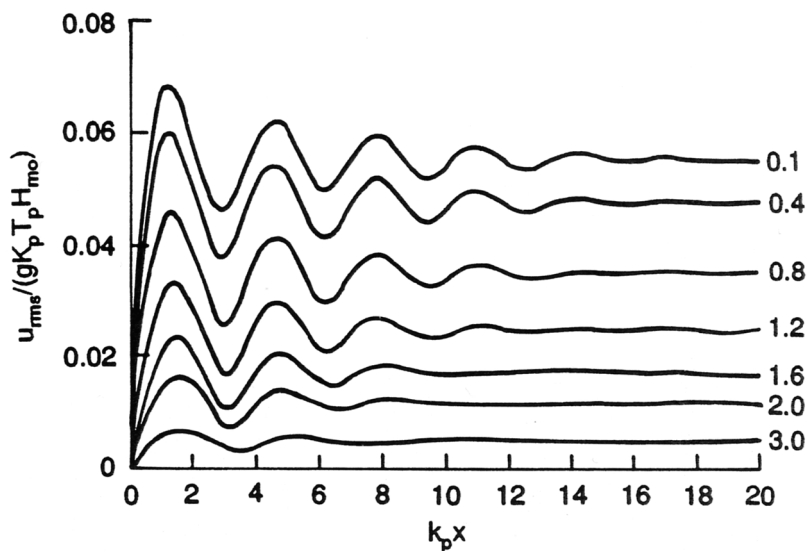


Figure 4 Variation of rms wave orbital velocity on the bed with $k_p x$ as a function of $k_p h$ (Hughes and Fowler, 1991)

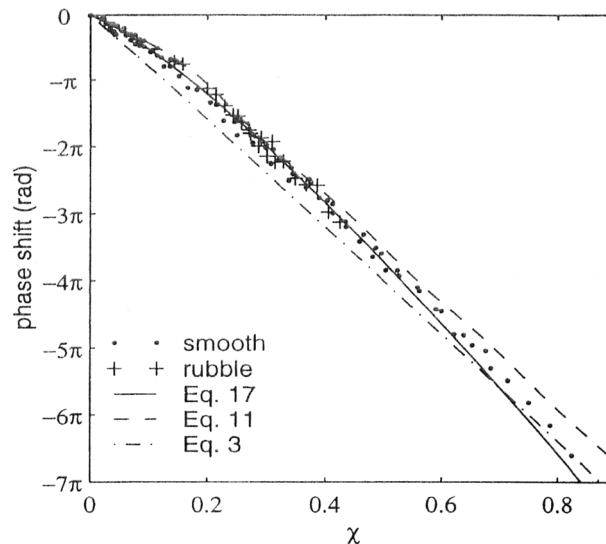


Figure 5 Phase shift on reflection (Sutherland and O'Donaghue, 1998)

3.3 P-type and N-type sediment transport

A regular wave reflecting off a vertical wall generates a standing wave, which in turn generates steady streaming in the thin bottom boundary layer (Longuet-Higgins, 1953, 1957). This streaming is manifested as a slow recirculating current from anti-node to node at the bottom of the bottom boundary layer and from node to antinode at the top of the bottom boundary layer as shown in Figure 6 (from Sumer and Fredsøe, 2000). The current at the top of the boundary layer drives a counter-rotating re-circulating cell in the (much thicker) body of water above the boundary layer. This work was extended to oblique-incidence by Carter *et al*, (1973).

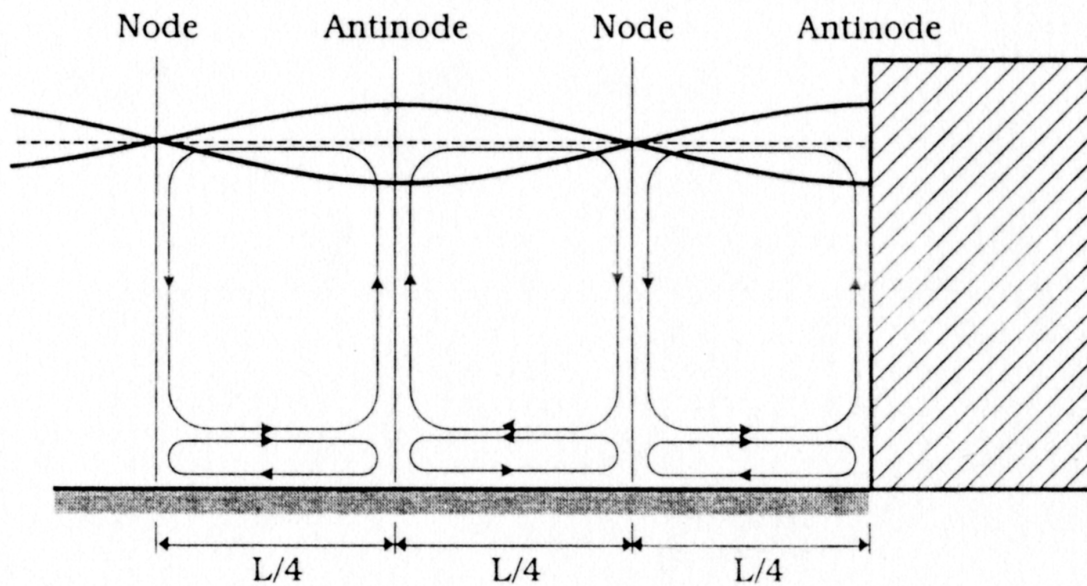


Figure 6 Re-circulating currents due to streaming under standing waves (Sumer and Fredsøe, 2000)

If the sediment in the bed is coarse and travels close to the bottom, it will be most influenced by the horizontal movements in the bottom boundary layer, which are towards the node (N-type). The result is scouring midway between anti-node and node and deposition under the node. If the sediment is small and is maintained in suspension, it will be most influenced by the current above the bottom boundary layer, so the net movement is away from the nodes towards the antinodes (L-type). Thus the pattern of sediment erosion and accretion varies with the mode of sediment transport – bedload transport gives a different pattern from suspended load transport (Sumer and Fredsøe, 2002) as shown in Figure 7 (also from Sumer and Fredsøe, 1997).

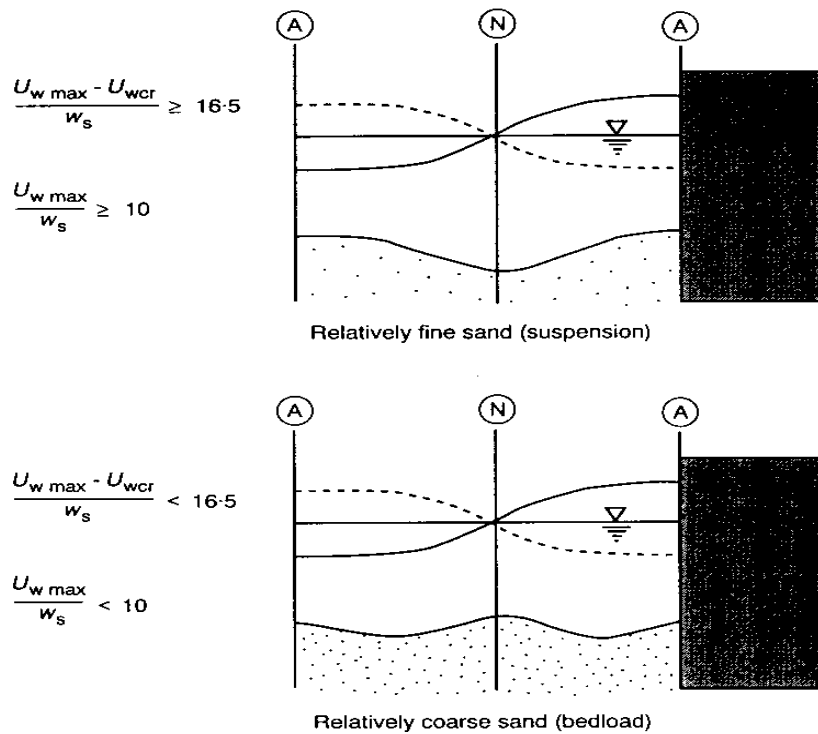


Figure 7 Scour and deposition patterns in front of a vertical seawall (a) Suspended sediment transport (L-type) (b) Bedload sediment transport (N-type)

Various formulae have been derived that distinguish between suspended and bedload hydrodynamic regimes. Sumer (1986) gives a criterion for currents: suspension will be initiated when the Shields parameter is greater than the threshold value, expressed as a function of dU_{fm}/ν , where d is the grain diameter, U_{fm} is the maximum value of the friction velocity and ν is the kinematic viscosity of the water. Sediment will not remain in suspension unless $U_{fm}/w > 1$, where w is the fall velocity of the sediment.

Xie (1991) proposed the following criterion for the initiation of suspension under waves:

$$\frac{U_m - U_{cr}}{w} \geq 16.5 \quad (4)$$

Where U_m is the maximum value of orbital velocity at the bed and U_{cr} is the critical velocity for incipient sediment transport. Irie and Nadaoka (1984) argued that in most practical cases U_m is much greater than U_{cr} , so Xie's criterion could be simplified to:

$$\frac{U_m}{w} \geq 10 \quad (5)$$

3.4 Wave-induced liquefaction near seawalls

In 2001 an EU funded research programme began that was entirely dedicated to the problem of liquefaction around marine structures, LIMAS (Liquefaction Around Marine Structures). The project involved numerical, physical and field modelling of the processes that lead to liquefaction and the subsequent response of various structures (e.g. caisson breakwaters and pipelines). At the conclusion of the project, April 2004, the findings of the project will be made available and a special issue in a leading ASCE journal will be published. A review of the potential role of seabed liquefaction as a mechanism operating at the toe of a seawall has been included based on this ongoing research.

Definition of liquefaction

In common usage, the term *liquefaction* refers to the loss of strength in saturated, cohesionless soils due to the build-up of pore water pressures during dynamic loading. A more precise definition of liquefaction is given by Sladen *et al.*, (1985):

“Liquefaction is a phenomenon wherein a mass of soil loses a large percentage of its shear resistance, when monotonic, cyclic or shock loading is applied, and flows in a manner resembling a liquid until the shear stresses acting on the mass are as low as the reduced shear resistance.”

As waves propagate over the seabed they cause the pore pressures in the bed to vary inducing flow and hence variations in the stresses between the grains. In some cases, the excess pore pressure, p^* , may increase to such a degree that contact between the grains (effective stress) is lost and the bed liquefies. If this takes place near a structure (e.g. breakwater) then failure may occur. In terms of the submerged unit weight of the soil, γ'_s , this criterion can be written as:

$$p^*(z) \geq \gamma'_s z \quad (6)$$

Case studies of liquefaction in the vicinity of coastal structures

A case study of a damaged breakwater was demonstrated by Zen *et al.* (1988). It was suggested that the cause of a slip circle failure of the foundations could have been wave-induced liquefaction. Their analysis showed that the presence of a thin layer of clay near the surface could have a very large influence on the liquefaction potential.

Silvester and Hsu (1989) investigated the large-scale failure of the Sines breakwater in 1978. The damage consisted of the complete loss of some two-thirds of the armour layer of 42t dolosse units. A number of possible mechanisms for the collapse were suggested but due to the suddenness and completeness of the failure, Silvester and Hsu favoured the hypothesis that the bed near the toe of the structure may have liquefied, causing the armour units

to become unstable. They also pointed out that scour at the toe of the structure can also be significant. In this case it was not easy to determine the relative contribution to failure of liquefaction and scour.

Maeno and Tsubota (2001) carried out scale model experiments investigating the flow out of back-filling sand behind revetments due to wave loading. They showed that the cyclic seepage force which occurs around the revetment plays an important role in the flow out of the sand. They also showed that a limiting criterion could be derived below which the sand would not move. This criterion depended on the wave height, wave period, permeability and depth of penetration of the revetment into the sand. An illustration of the mechanism is shown in Figure 8.

Further references are provided by Jeng (2001). Loveless *et al*, (1996) also demonstrated the effect that groundwater flows under a seawall could have on the scouring of a model gravel beach, even without inducing liquefaction. In the most extreme case, an onshore flow (beach de-watering) produced accretion near the seawall toe and an offshore flow (under the seawall from the landwards side) produced more erosion than occurred during the no flow test.

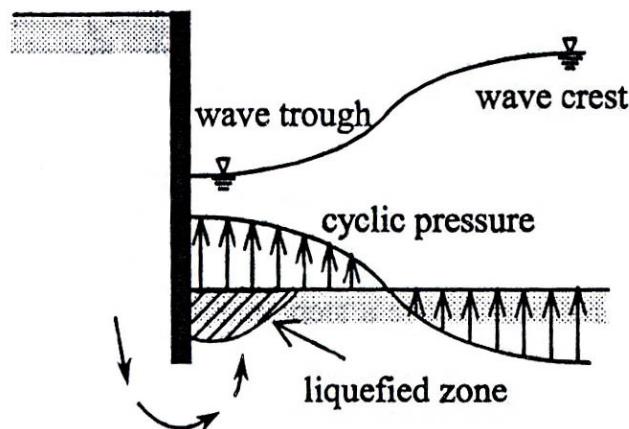


Figure 8 Mechanism for flow out of sand from behind revetment (from Maeno and Tsubota, 2001)

Liquefaction in cohesionless soils

For sands, two different types of liquefaction have been identified in the literature, momentary and residual. The primary difference is the relative time-scales of the two processes. The first, momentary liquefaction, tends to only last for a small fraction of a wave period and only affects a limited area of the seabed. On the other hand, residual liquefaction, takes place over a time scale of tens to hundreds of wave periods and has the potential to affect a larger area of seabed.

Momentary liquefaction was dealt with in detail by Madsen (1978) and Yamamoto *et al*, (1978). In this analysis the seabed is assumed to behave elastically and the pore fluid is allowed to be compressible. The pore pressure

fluctuates at the wave frequency and induces reversing flow in the bed. It was shown that under the wave trough, the flow is directed upwards and out of the bed, causing the effective stresses to reduce. If the bed liquefied it only remained so for a small proportion of the wave period as it regained strength as the crest approached and the flow reversed. The extent of the liquefied zone depended on the compressibility of the pore fluid and seabed material, the wave characteristics (wave height and period) and the water depth. The second cause of liquefaction in a sand bed is more analogous with earthquake-induced liquefaction, and is termed residual liquefaction. Due to the oscillating bottom pressure under a wave, significant shear stresses are created which can be as large as the cyclic shear stresses induced by large earthquakes. Consequently, if the seabed is in a relatively loose initial state, excess pore pressures will be generated as the bed attempts to consolidate, as in the case of an earthquake. The rate of pore pressure build-up will depend on the magnitude of the forcing (i.e. wave conditions and water depth), the relative density of the sand and the coefficient of consolidation (i.e. drainage characteristics). This has been dealt with in detail by Rahman and Jaber (1986) and McDougal *et al*, (1989) and more recently by Cheng *et al*, (2001). This type of liquefaction affects the whole seabed, irrespective of the position of the wave crest, and occurs over a longer period (typically in the order of minutes).

Liquefaction in cohesive soils

Various concepts have been proposed for the mechanism that causes the liquefaction (fluidisation) of very soft clay beds. Foda *et al*, (1991) suggested a “soil piping” concept where resonance is initiated within vulnerable cavities in the fine-grained bed, followed by progressive failure of the soil matrix around such cavities. Feng (1992) and de Wit and Kranenburg (1993) carried out wave flume experiments on soft muds where the initiation and subsequent growth of the fluid mud layer was measured.

From later work by de Wit (1995) it was found that the potential for fluidisation of a soft clay bed was linked to the magnitude of the wave induced stresses and the undrained shear strength of the bed. Typically if the wave-induced stresses exceeded the shear strength of the bed, the bed will fluidise. The depth of fluidisation will be controlled by the variation in depth of the forcing and the strength. For given wave conditions and soil strength profile there will be an equilibrium fluidisation depth.

Predicting Liquefaction

There are numerous methods for predicting the occurrence of both mechanisms of liquefaction, but in the following paragraphs one method for each will be presented and the important parameters highlighted.

Momentary liquefaction can be reasonably well predicted using linear-elastic theory and accounting for the compressibility of the pore fluid and the drainage characteristics of the soil. Analytical solutions to the problem of progressive waves over a poro-elastic seabed were first presented by Madsen (1978) and Yamamoto *et al*, (1978). These solutions highlighted the importance of the

relative compressibility of the pore fluid. If the pore fluid had even a small trace of air bubbles (i.e. < 1%), the liquefaction potential was significantly increased. Likewise, the potential also increased for decreasing values of the permeability. Figure 9 shows how the depth of liquefaction varies against two key non-dimensional parameters, $KG/(\rho_w g^2 Th)$ and G/β , where G (Pa) is a measure of the soil stiffness, β (Pa) is a measure of the pore fluid stiffness (including effect of gas bubbles) and K (ms^{-1}) is the permeability, ρ_w (kgm^{-3}) is the fluid density, g (ms^{-2}) the acceleration due to gravity, T (s) the wave period and h (m) the water depth. Analytical solutions for standing, or partially standing waves in front of seawalls have been presented by Tsai (1995). A review of liquefaction in front of seawalls was recently presented by Jeng (2001).

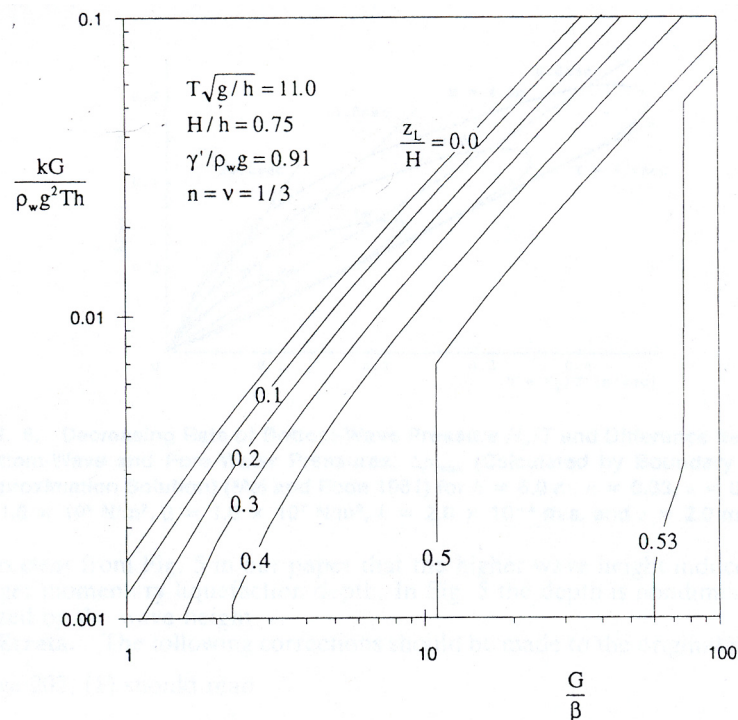


Figure 9 Depth of momentary liquefaction for typical wave conditions (from Law, 1993)

Unlike momentary liquefaction, residual liquefaction cannot be predicted using elastic soil models. The reason is that the mechanism for building up excess pore pressure comes from the plastic behaviour of the soil. The continuous loading and unloading of the soil as the wave passes overhead tries to compact the seabed. For the seabed to densify, it must 'squeeze' the water out of the pores. If the drainage capabilities of the soil are poor (e.g. fine sand or silt), the pore pressure within the bed will rise faster than it can dissipate and liquefaction may occur. Therefore this type of liquefaction is unlikely to occur in coarse grained sands.

In the past there have been two types of solutions to this problem, one based on empirical equations relating the cyclic shear stress ratio, $|\tau_{xz}|/\sigma'_{v,0}$, to the number of cycles to liquefaction determined from laboratory (triaxial) testing and another based on more sophisticated constitutive models for the sand.

$|\tau_{xz}|$ (units) is kPa and σ'_{v0} (units) is kPa. Most of the work has been based on research done in the earthquake field. A review of the fully empirical models can be found in Cheng *et al*, (2001). Sassa *et al*, (2001) gives an in depth description of a sophisticated time-domain, finite element model used for predicting residual liquefaction.

3.5 Prediction methodology

Available approaches for toe scour prediction

At present, there are four categories of methods for predicting toe scour:

- 2-dimensional physical modelling;
- 3-dimensional physical modelling;
- Empirical formulae and parametric plots derived to fit experimental results;
- Process-based numerical models of near-shore sediment transport.

Because the physics of scour are not fully understood, most of the methods used are based on experiments rather than theoretical models; but this is changing slowly as research on sediment transport processes continues. Table 1 summarises the advantages and disadvantages of each technique.

Rules of thumb and semi-empirical equations are useful for determining whether scour is likely to be a significant problem. These methods, however, may not be adequate if the site conditions are complex, and in these cases, physical modelling is recommended. Three-dimensional models are preferable to two dimensional models because cross-shore as well as long-shore processes can be simulated. The model scale should be as large as possible to reduce scaling limitations and it is recommended that random rather than regular waves are used as the latter artificially constrict the zone of sediment transport on the beach profile. In the last few years, attempts have been made to incorporate seawall effects into near-shore sediment transport computer models that predict changes in beach profile (McDougal *et al*, 1996; Southgate, 1989, Steetzel, 1987, Rakha and Kamphuis, 1997, Baquerizo and Losada, 1998, Lawrence *et al*, 2003). Note should also be taken of the analytical model of Ruggiero and McDougal (2001). These models all incorporate reflected waves. At present, results from these models should be treated with caution as not all the effects that a seawall has on hydrodynamics are modelled, including for example, the turbulence that is produced by the impact of a wave on the seawall. (For a somewhat harsh critique of numerical models of sediment transport, see Thieler *et al*, 2000).

Table 1 Comparison of methodologies for toe scour prediction

Scour prediction category	Outline of Method	Advantages	Disadvantages
Physical modelling (2-dimensional)	Experiments are conducted in a flume (rectangular wave channel) - typical	Relatively easy method The model can be	Does not simulate longshore currents or sediment

	<p>dimensions: length 15m to 50m, widths 0.3m to 3m, water depths 0.2 to 2m</p> <p>A scaled-down model of the seawall, the beach profile and sediment are put into the flume and subjected to appropriately scaled unidirectional wave conditions. Change in the beach profile adjacent to the wall is then measured.</p>	<p>re-run with various seawall design options and positions relative to the surf zone, to compare the impact of alternative design options</p> <p>Actual site profiles and wave conditions can be modelled</p> <p>In addition to predicting toe scour, the stability and or overtopping performance of the various seawall designs can also be investigated</p>	<p>transport</p> <p>Limited to orthogonal wave attack</p> <p>Scaling considerations may limit the confidence with which model results can be transferred to the field. Conclusions are thus at best qualitative rather than quantitative, and where scaling problems are severe, toe scour prediction may be totally misleading</p> <p>Requires specialist facilities and expertise</p>
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Physical modelling (3-dimensional)	<p>Experiments are conducted in rectangular wave basins - typical dimensions: length of sides 15m to 30m, water depths 0.2m to 1m</p> <p>A scaled-down model of the seawall and beach, in plan and section, is constructed in the basin</p> <p>Waves are generated by paddles and currents may be included by pumping</p>	<p>Wave paddles can be positioned to provide oblique as well as shore-normal wave trains and it is normally possible to alter the direction and position of currents</p> <p>Models both cross-shore and long-shore sediment transport.</p> <p>Complex 3-dimensional site specific features can be modelled. This is not possible using empirical</p>	<p>Scaling problems, particularly for finer sediments, reduce the applicability of the results to field situations and it may be that only qualitative answers are possible</p> <p>Appropriate cost and timescale needs to be allowed for</p> <p>Requires specialist facilities and expertise</p>
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	the water across the basin	mathematical formulae In addition to predicting toe scour, the stability of the various seawall designs can also be investigated, as can the overtopping performance	
Semi-empirical mathematical formulae	These are mathematical relationships derived from the results of physical model tests and/or field measurements	Inexpensive and easy to use	Validity is restricted to conditions within the range used to deduce the empirical relationship Inadequate for complex situations (for example irregular beach profile or where waves and currents interact in a complex way). For these cases 3-dimensional physical modelling is recommended
n-line numerical models	Commonly 1-line models such as Beachplan or Genesis are used to model the planshape evolution of beach contours	Flexible in terms of combinations of parameters that can be tested	Have crude representation of the effect of seawalls on longshore transport. Do not explicitly model scour processes
2D coastal profile models	Numerical modelling of the cross-shore transport on a straight beach with shore-parallel contours. Influence of structure typically	Many combinations of beach profile, water level, wave height and period can be tested quickly and at less cost than physical	Not all physical processes are parameterised well in the models. For example, wave reflection from

	included through its reflection coefficient	modelling	seawall is crudely represented, interaction of incident and reflected waves is commonly ignored, as is turbulence generated by wave breaking
3D coastal area models	Commonly 2DH models with quasi-3D components, that represent currents, waves, sediment transport and morphodynamics	Can model irregular bathymetries and include the effect of currents	Slow to run compared to 2D coastal profile models. Similar deficiencies to 2D coastal profile models, but with less (often no) resolution in the vertical. Important surf zone processes, such as undertow are omitted

Empirical formulae for toe scour prediction on shore parallel structures

This section presents the available design formulations to predict scour depth on shore parallel structures (namely, seawalls and the trunk of breakwaters). This information has been summarised in Table 2. Because of the scarcity of design formulations, widely-used rules of thumb and useful remarks have also been included in the table. The formulations and rules of thumb in the table are for sandy beaches unless otherwise stated. Further information is given in Section 3.7 and Appendix 2.

This table can be treated as the first reference point when looking for a formulation to predict scour. The formulations are presented in separate columns for vertical and sloping structures. The entries in the table are split into rows for breaking waves (which includes conditions when the waves break before reaching the structure or at the structure itself) and non-breaking waves (they reach the structure without breaking and are reflected). Both these categories are further subdivided into normally incident waves and obliquely incident waves. Once the appropriate category has been identified, detailed information on the design formulations, their applicability, method of derivation and any limitations can be found in Appendix 2.

For worked examples of maximum scour depth calculations using the Jones' equation, the Song and Schiller equation, and the Fowler equation refer to Fowler (1993). Scour prediction methodology and formulae are also discussed in Powell (1987) and Fowler (1992) and (1993).

Table 2a Design Relationships for non-breaking waves

		DESIGN RELATIONSHIPS	COMMENTS
Normal Incidence	Vertical Walls	Hughes and Fowler (1991)	Suspended transport, irregular wave tests
		O'Donoghue (2001)	Bedload and regular waves
		Xie (1981, 1985)	Bedload and suspended, regular waves
	Sloping walls	No design relationship for suspended load transport.	Use formulae for vertical walls taking into account CEM, (2002). <i>“A standing wave field similar to that at a vertical structure will be created, except that the variation between the sea surface elevation nodes and antinodes is less pronounced and the location of the node nearest to the structure toe varies with wave condition and structure reflection conditions.”</i>
		Sumer and Fredsøe (2000) for bedload transport at rubble mound breakwaters.	Apply to seawalls with caution as reflection coefficient of seawall is higher, Scour depth at a rubble mound breakwater is smaller than at a vertical wall breakwater Scour depth decreases as slope of rubble mound breakwater decreases
Oblique Incidence	Vertical Walls	No design relationship.	Use formulae for normal incidence, taking Silvester (1991) into account: Obliquely incident waves tend to scour more than equivalent normally incident waves Mode of transport may change, resulting in overall scouring in front of structure
	Sloping Walls	No design relationship	Use formulae for normal incidence waves and a vertical wall, taking into account advice of CEM (2002), Sumer and Fredsøe (2000) and Silvester (1991) as appropriate.

Table 2b Design relationships for normal incidence waves breaking at or before the structure

		DESIGN RELATIONSHIPS	COMMENTS
Normal Incidence	Vertical Walls	CEM (2002)	$S_m = H_{max}$ or $S_m \approx h$ Max scour occurs when vertical wall is located around the plunge point of the breaking wave Reducing wall reflection reduces the amount of scour
		Powell and Whitehouse (1998, sand)	Iso-parametric plot shown in Appendix 2.
		McDougal, Kraus and Ajiwibowo (1996)	Formula shown in Appendix 2.
		Powell and Lowe (1994, shingle)	Iso-parametric plot shown in Appendix 2.
		Van Rijn (1993)	S_{max}/h_{toe} for depths 1.5 to 1 <2m 1 to 0.7 2 to 4m 0.7 to 0.5 4 to 10m 0.5 to 0.3 10 to 20m
		Fowler (1992)	Formula given in Appendix 2.
		Powell (1987) for shingle	max scour occurs when $h_{toe}/H_s \approx 1.5$
		Powell (1987) for sand	$S_{max} \approx H$ for $0.02 < H_s/L_m < 0.04$
		Dean (1986)	Approximate principle
		SPM (1984)	$S \leq H_o$
		Jones (1975)	Equation in Appendix 2 from analytical model.
		Song and Schiller (1973)	Formula shown in Appendix 2.
		Chesnut and Schiller (1971)	Maximum scour occurs when distance from seawall to point of wave breaking is between $\frac{1}{2}$ and $\frac{2}{3}$ of distance from point of wave breaking to pre-seawall position of mean water line.
		Herbich and Ko (1968)	Formula shown in Appendix 2
	Sloping Wall	CEM (2002)	$S_m \leq H_{max}$ Maximum scour occurs for vertical wall Scour decreases with decreasing structure reflection coefficient (i.e. with decreasing slope and increasing porosity.)
	Van Rijn (1993)	S_{max}/h_{toe} for depths 1 to 0.5 <4m 0.5 to 0.3 4 to 10m 0.3 10 to 20m	

			<p>Powell (1989): For impermeable sloping structures of 1:1.5 to 1:2 there is no significant reduction in scour depth compared to that at a vertical wall Reducing the slope of an impermeable structure to 1:3 can reduce local scour typically by 25% but up to a maximum of 50% compared to a vertical wall. Rock armoured revetments generally show less susceptibility to local scour and may even allow accretion.</p>
			<p>Herbich (1984): scour depth is only significantly reduced for seawall slopes of 15° or less.</p>

Table 2c Design relationships for scour depths due to oblique incidence breaking waves

		DESIGN RELATIONSHIPS	COMMENTS
Oblique	Vertical walls	No design relationship	Design as for normal incidence vertical walls, taking into account the following, as appropriate: Rakha and Kamphuis (1997): $S \sim H_b$ for angle of incidence of 10° <i>CEM (2001):</i> Short-crested waves increase in size along the structure, which may cause greater scour than normally incident waves <i>S significantly increased when along-structure currents exist</i> <i>Oblique waves generate flow parallel to the structure</i>
	Sloping walls	No design relationships	Design as for normal incidence sloping walls, taking into account the following, as appropriate: Baquerizo and Losada (1998): Depending on the angle of incidence, scour or deposition may occur at the toe of the structure

Similarities and differences between response of shingle and sand beaches to toe scour processes

This section discusses the similarities and differences between coarse and fine grain toe scour as obtained when comparing the isoparametric plots for shingle (Powell and Lowe, 1994) and for sand (Powell and Whitehouse, 1998). These parametric plots are reproduced in Appendix 2.

Similarities between response of shingle and sand beaches to toe scour processes:

- Depending on wave steepness, maximum scour for both coarse grain sediment (d_{50} between 5 and 30mm) and sand ($d_{50} = 0.2\text{mm}$) occurs when the ratio initial water depth at the seawall toe h_{toe} to significant wave height H_s is around 2.0;
- The maximum toe scour after 3000 waves is caused by the wave condition with the lowest wave steepness at the relative water depth of (h_{toe}/H_s) of 2.0. For a wave steepness of 0.01, S/H_s was 1.5 for both tested grain sizes. Moreover, for a given time duration, low steepness (i.e. swell waves) are more damaging than steeper waves. Scour is most severe between $0.005 < H_s/L_m < 0.02$, and there is no significant difference in scour within this range (peak S/H_s is about 1.0 after 1 hour of storm conditions). For waves steeper than this, the dimensionless peak scour value after one hour of

storm conditions decreases from 0.3 for $H_s/L_m = 0.03$, to 0.12 for $H_s/L_m = 0.08$;

- For a given number of waves, or a given storm duration, low steepness waves (i.e. a low H_s/L_m ratio) cause more toe scour than steeper waves (this holds true for both grain sizes tested).

Differences between response of shingle and sand beaches to toe scour processes:

- Scour appears to occur over a larger range of relative water depth on sand beaches than gravel for low steepness waves but over a smaller range for steeper waves. For example, for the wave steepness $H_s/L_m = 0.01$ scour occurs between $H_{toe}/H_s = 0$ and at least 6 for sand, but 1.25 to 3.1 for gravel - but for the steeper wave of $H_s/L_m = 0.06$ the range in relative water depth over which significant scour ($S/H_s = 0.25$) occurs on sand is between $h_{toe}/H_s = 0.6$ to 1.8 whereas for coarse grains this increases to between approximately -0.07 to 2.2;
- Maximum scour depth for the sand beach was approximately equivalent to the offshore wave height for wave steepness up to around 0.025 (a lower limit than predicted for coarse sediment H_s/L_m around 0.04);
- For wave steepnesses, $H_s/L_m > 0.02$ the scour depth after 3000 waves is greater for gravel rather than sand beaches;
- Sand beaches take longer to come into equilibrium with the wave climate than coarse sediments. This may be related to the reflection coefficient of the beach (as coarse beaches tend to be steeper and more reflective). Sá-Pires *et al*, (2003) showed that, after an erosional storm, the recovery time of a reflective beach was faster than that of intermediate and dissipative beaches nearby. (See also Section 3.5.4);
- Unlike coarse sediment, no accretion at the base of the seawall was predicted for sand, however, this could well be due to the limitations of the numerical model (COSMOS) rather than a real effect. SBEACH, a similar type of near-shore sediment transport model also reported the same problem (MacDougall *et al*, 1996).

Time development issues

Powell and Lowe (1994) derived iso-parametric plots of the scour and accretion of a shingle beach after 3000 waves (Appendix 2) but they also measured the time development of the scour and plotted curves to allow the scour depth after different numbers of waves to be plotted. The time development was found to depend on the type of scour/reflection behaviour (Section 3.5.5). Powell and Whitehouse (1998) added a curve for sand. Their data for the time-development of scour is shown in Figure 3.

McDougal *et al*, (1996) used numerical model results to investigate the time evolution of the scour and derived an expression similar to the one used for piles and pipelines:

$$S(t) = S_e \left(1 - \exp^{-\frac{\mu t}{T}} \right) \quad (7)$$

$S(t)$ is toe scour depth at time t

S_e is equilibrium toe scour depth

T is wave period (as opposed to the characteristic timescale used in pile and pipeline formulations; Whitehouse, 1998)

A best fit line to their data gave $\mu=0.000321$.

Comparing this formulation and Figure 3, Whitehouse (1998) proposed an as yet undefined dependency of μ on the initial bed condition/water depth at the wall and the sediment transport rate, modifying McDougal *et al*, (1996) expression to:

$$S(t) = S_e \left(1 - \exp\left(-\frac{\mu t}{T}\right)^p \right) \quad (8)$$

with p taking values other than 1.

Xie (1981) stated that the number of waves to reach the final scouring depth (at time t_{\max}) was dependant on the wave steepness, H/L , so that (for regular waves period T):

- for $H/L > 0.02$: $t_{\max}/T = 6500-7500$ for “fine” sand (in suspension) and $t_{\max}/T = 7000$ for “coarse” sand (not in suspension)
- for $H/L < 0.02$: $t_{\max}/T = 7500-10000$ for “fine” sand (in suspension) and $t_{\max}/T = 6500-10000$ for “coarse” sand (not in suspension).

For irregular waves he found values of t_{\max} more than double that for regular waves. Xie (1981) also derived an expression to derive the scour S_t at any time t , once the maximum scour, S_{\max} , has been calculated:

$$\frac{S_{\max}}{S_t} = \left(\frac{t}{t_{\max}} \right)^a \quad (9)$$

with $a=0.3$ for “fine” sand (S) and $a=0.4$ for “coarse” sand (NS)

Previous bed level

Powell and Lowe (1994) found that the initial scour response at the wall could be very different for different initial conditions, as these define the type of scour / wave reflection behaviour. The four types identified are described in Table 3. (Although these data are specifically for shingle, they are considered

reasonably typical for scour at seawalls in general). The time development of scour for the different types is shown in Figure 3.

Table 3 Influence of initial beach level on scour response in shingle (Powell and Lowe, 1994)

Type	Initial beach	Reflected energy	K_r	Onshore / offshore movement	Beach response and scour
Type I	Initial beach level is high relative to the wave height	Low	Low and constant through time	Onshore movement high; offshore movement low	The beach responds in much the same way as a natural beach. The seawall plays little part in beach processes as it is not under direct wave attack
Type II	Combination of beach levels and wave heights is such that there is marginal scour	Low	K_r increases through time	Predominant onshore transport; some offshore transport	Initially there is scour at the toe, however, over time the beach can build up due to the predominant onshore movement of material. As the beach level increases so the amount of reflected energy, and hence the proportion of offshore to onshore transport, decreases. Given sufficient time the beach level can build up to the Type 1 condition
Type III	Initial beach level is low relative to the wave height	High	K_r increases through time	Low onshore movement; high offshore movement	The toe depth increases with time and there is a general lowering of beach levels. As the beach level decreases so does the amount of reflected energy, and hence the proportion of offshore to onshore transport, increases. Given sufficient time the beach levels can reduce to the Type IV condition
Type IV	Initial beach level is very low relative to the wave height	High	K_r large and constant through time	Predominantly offshore movement	Scour is small compared to water depth. The movement of material is low in both the onshore and offshore directions due to the large depth of water relative to the wave height. Eventually a stable situation is attained from which, whatever the incident wave conditions, the beach cannot recover its former levels

Xie (1981) found (when starting a test with a deformed bed level) that once a greater scouring depth was created by storm waves, it was difficult for the following waves to fill in the scouring trough. The influence of the bottom geometry on the orbital velocities was the cause suggested for this behaviour.

3.6 Scale effects

A very thorough discussion of scale factors in various types of hydrodynamic models, both with and without sediment can be found in Hughes (1993). The discussion below is taken from Sutherland (1999), which was derived from Sutherland and Whitehouse (1998) which relied on the work of Kamphuis (1985, etc). A brief summary of some of the main parameters is given below.

It is assumed throughout that waves are scaled by the Froude number:

$$F_r = \frac{U^2}{gL} \quad (10)$$

where U is a velocity, g is gravitational acceleration and L is a length. To maintain similitude, this number must be the same in the model and prototype. The 5 main non-dimensional numbers for sediment transport similitude between prototype and model are listed below.

The Grain Reynolds Number: $R_{e*} = \frac{u_*d}{\nu}$ (11)

where u_* is the shear velocity, d the (median) grain diameter and ν the fluid viscosity. The shear velocity is derived from the grain shear stress, τ , and the water density ρ , as $u_* = (\tau/\rho)^{0.5}$. The grain shear stress is that applied to a grain, not the total stress applied to a ripple and is used to represent the mobilising force due to the waves.

The Shields Parameter: $\theta = \frac{\tau}{g(\rho_s - \rho)d}$ (12)

where ρ_s is the density of the sediment. The Shields parameter relates the mobilising forcing applied to a grain on the bed to the restoring force due to gravity and is related to the Densimetric Froude number. The Shields parameter can be used to determine sediment mobility:

- if $\theta < \theta_{cr}$ (where θ_{cr} is the critical Shields parameter) then the bed is immobile;
- if $\theta_{cr} < \theta < 0.8$ (roughly) then the bed is mobile and rippled;
- if $\theta > 0.8$ then the bed is mobile and flat with sheet flow.

Relative density (or specific gravity): $s = \frac{\rho_s}{\rho}$ (13)

Relative length: $l_s = \frac{l}{d}$ (14)

where l is a characteristic length of the system. Kamphuis (1985) gives some guidance as to which length should be used in different experiments.

Relative fall speed.

The relative fall speed can be chosen in a number of different ways. If the model is to be bedload dominant (so sediment mobility is dominated by shear stress applied at the bed) then the relative fall speed can be defined as:

$$\frac{u_*}{w_s} \quad (15)$$

as commonly used in river sediment transport studies. Here w_s is the sediment fall speed, which can be determined from common equations in Soulsby (1997). If the ratio is greater than 1 then the sediment is often taken to be in suspension. In this case sediment mobility may be dominated by turbulence and the Dean fall speed parameter, D_{ws} , is commonly used:

$$D_{ws} = \frac{H}{w_s T} \quad (16)$$

where H is wave height, so H/w_s represents the time taken for a particle to fall a wave height.

All five of the above numbers should be the same in model and prototype if similitude is to be guaranteed. Unfortunately that is impossible to achieve except at a scale of 1:1 so compromises must be made. The most important numbers must be preserved, and the similitude between the values of others must be relaxed. In order to make an informed choice about which scale factors to preserve and which to relax, the hydrodynamics of a situation and the sediment response to the hydrodynamics must both be known and the relative dominance of different mechanisms estimated.

Bedload transport scale models

Kamphuis (1985) came up with 4 scaling models for the bedload transport case. They are discussed in Hughes (1993) and short comments outlining the disadvantages of each may be found in Oumeraci (1994). The scaling models are:

1. Best model, which preserves the Shields parameter, the relative density and the relative length scale;
2. Lightweight model, which preserves the grain Reynolds number and the Shields parameter and uses lightweight sediment in the model, thereby preserving the relative density within a factor of 2 or so;
3. Densimetric Froude model, which preserves the Shields parameter and uses a lightweight sediment, thereby preserving the relative density within a factor of 2 or so;
4. Sand model, which preserves the relative density.

All the models have their advantages and disadvantages and the inevitable scale effects. Alternative scaling criteria will apply if the test concerns a permeable beach or structure. Yalin (1963) proposed a method for the selection of model sediment to predict the scouring of a shingle beach in front of a vertical wall using a physical model. The method used the following criteria for model scaling:

1. The relative magnitudes of the onshore and offshore motion should be the same in model and prototype;
2. The threshold of motion should be correctly scaled (by maintaining the same ratio of drag forces and submerged weight in model and prototype);
3. The permeability of the beach should be correctly reproduced (by ensuring that the percolation slope of the model and prototype are the same).

These similitude criteria formed the basis of the physical modelling of shingle beaches using coal, carried out by Powell and Lowe (1994). Loveless (1994, 1995, 1996) has challenged Yalin's approach, both in terms of the values of hydraulic gradient and percolation velocity assumed, and also in terms of the processes represented. A modified version of Yalin's scaling was proposed instead. If Loveless is correct, then many of the scour predictions for gravel beaches, derived from lightweight sediment models, will be overpredictions of the actual scour. However, to test the scaling ideas out thoroughly would require a series of tests to be performed at a range of scales from almost full scale to the more typical 1:20 laboratory scale. No such test series has ever been run and so there is still uncertainty over how to scale the sediment for model tests and whether small scale model test results can be extrapolated to full scale.

Suspended load scale models

Dean (1985) argued that in cases where suspended sediment is predominant the Shields parameter does not have to be preserved as the wave breaking and turbulence were more dominant mechanisms in determining sediment mobility than the wave shear stress. His recommendations were for an undistorted model with Froude scaling using the same value of the Dean fall speed parameter as in the prototype. A lightweight sediment could be used if necessary and the model should be large enough to prevent there being any effects from viscosity, surface tension or sediment particle cohesiveness. Oumeraci (1994c) recommended a similar set of scaling requirements for scour tests in standing waves or under breaking waves. Hughes and Fowler (1991) also concluded that prototype conditions could be modelled if the Froude number and the Dean fall speed were preserved.

The scaling of fall speed can be done by an iterative process. Simple Froude scaling rules exist for both small and large particles (if the same density material is used in model and prototype). The fall speed of small diameter particles follows a Stokes law of viscous drag so $n_{ws} = n_L^{0.25}$ (where n_{ws} is the scale ratio of fall speeds and n_L the geometric scale ratio) as shown in Oumeraci (1994c). However large particles fall by a quadratic bluff-body law (see Soulsby 1997) with fall speed proportional to the square root of the diameter. It follows that the scaling here is $n_{ws} = n_L$. As the scaling rule changes with the size of the sediment, and a slightly different density of water is often used in model and prototype, it is wisest to iterate to a model sediment diameter from its fall speed.

According to Xie (1991) similitude between model and prototype is achieved if the parameter $\frac{(U_{max} - U_*)}{w}$ remains the same for both, where U_{max} is the maximum horizontal velocity at the bed, U_* is the critical velocity for incipient motion of sediment and w is the sediment fall velocity. This parameter is also used to separate suspension mode and non-suspension mode (Section 3.3).

Kraus and McDougal (1996) list the consequences that incorrect scaling may have:

1. Dominance of threshold of motion in the laboratory, which could alter the direction and magnitude of bed load sediment transport;
2. Presence of ripples in laboratory surf zones, which do not exist in the field and which can obscure trends in profile change;
3. Differences in sediment transport mode as suspended load or bedload between the laboratory and field;
4. Inability to scale simultaneously both bedload and suspended load, which may be particularly troublesome for experiments involving both cross-shore and long-shore transport, and different Reynolds numbers and turbulence intensity which in turn affect sediment transport mode and magnitude.

Van der Meer and Veldman (1992) conducted a berm breakwater physical model study at a scale of 1:35 and at the Deltaflume at a scale of 1:7. The depth of the scour hole produced when comparing average profiles (before the highest waves hit the structure) in both cases was the same, its shape being completely different in the seaward direction. However, when comparing final profiles, the scour hole for the 1:35 test was deeper, indicating that scale effects were present in the development of the scour hole. Moreover, the authors acknowledged that these scale effects might have caused the difference in behaviour of the breakwater crest and rear face damage.

This supports the contention that the results from small-scale physical model results are likely to be misleading unless the most important scaling parameters are satisfied and even then should be considered as providing qualitative information. Kraus and McDougal (1996) recommended future laboratory studies to be done with justification of the scale used and with awareness of the ambiguities that have arisen in previous experiments done at small-scale. As a minimum it is recommended that the dominant transport mode (bedload or suspended) be reproduced in the laboratory. In many cases this will require suspended sediment transport to be produced in the laboratory for a significant proportion of the time. This will require experiments to be performed at a relatively large scale.

3.7 Summary of scour experiments

Field experiments

Long-term field observations of the seawall and beach interaction have been conducted at two locations in the USA, one on the East Coast (Virginia Beach and Sandbridge, Virginia) by Basco and colleagues (Basco, 1990 and Basco *et al*, 1992, 1997) and the other along Monterey Bay, California, by Griggs and colleagues (Griggs *et al*, 1991, 1994, 1996).

Sandbridge. Basco *et al*, (1992) found the rate of berm lowering in front of seawalls to be slightly higher at unwallled sections, as compared to neighbouring beach and dune sections not backed by walls. Basco *et al*, (1997) analysed the fifteen years of survey data (seven or eight of which were taken

prior to wall construction) at three timescales (historic, seasonal, storms) and provided an answer to three concerns often expressed:

- It was determined that volume erosion rates were not higher in front of seawalls. However, seasonal variability of the sand volume in front of walls was found to be greater than at non-walled locations;
- Walled beaches were found to recover in about the same time as non-walled beaches for both seasonal transitions and following erosional storm events;
- At a few non-walled locations, the sand volume landward of adjacent walls was found to be eroding at a faster rate after wall construction. At some non-walled locations, the sand volume remained constant or increased in time after nearby wall construction. These results are somewhat inconclusive.

Monterey Bay. Griggs *et al*, (1991) found there was no consistent difference in the beach profile at vertical impermeable walls and at permeable sloping walls, a result that contradicts conventional paradigms. Plant and Griggs (1992) studied the beach groundwater elevations, concluding that the reduced permeability and porosity below beach level due to a rock revetment inhibits groundwater flow and increases watertable elevation at the wall-natural beach boundary during times of elevated sea level. In the seven years of surveying (including one year of surveys made before and after major storms), Griggs *et al*, (1994) found that a scour trough was never observed in front of any of the seawalls studied. This was justified by the fact that wave and storm conditions were considerably milder as compared to more severe storms that occurred off California in the 1980's. Griggs *et al*, (1996) described the effects that the storms of 1995, the most severe during eight years of monitoring, had on the beaches monitored. Although beach elevations were significantly lowered, the beach response was not nearly as severe as that which occurred in 1983 (by el Niño storms). The authors justified this difference by the lack of coincident high tides and elevated sea levels with storm waves during the winter of 1995. Similar responses of the control and seawall beaches to the storm waves of 1995 are observed, consistent with the long-term observations presented above. In addition, the beach in front of the seawall quickly lost the imprint of accelerated scour and a general alongshore homogeneity began evolving within months of the 1995 storms. They did not find any evidence of impaired recovery and, if anything, initial recovery was more rapid on the seawall-backed beach, in contrast to what was seen by Basco *et al*, (1992).

Field investigations on failures of wave-dissipating concrete blocks were carried out by Gomyoh *et al*, (1996), reporting that toe scour is a common failure at the breakwater trunk (five of the twenty-seven cases analysed showed failure due to toe scour). They also acknowledged liquefaction as playing a key role on the settlement failure, concluding that further research is needed in both toe scour and liquefaction.

Field observations of scour

There have been a number of cases where toe scour has been reported in the literature. In many cases there were no measurements of conditions and no surveys of the site, but scour, or damage attributed to scour was observed. Silvester and Hsu (1997, §7.2.2) have provided a variety of examples from around the world, in more detail than is reproduced here.

Silvester and Hsu (1997, §7.2.2) contend that Sines may have been due to scour and/or liquefaction. “When the seabed is deepened adjacent to a breakwater, the floor slope up to the structure is increased and can almost reach the angle of repose. The consequence when storm waves arrive include a buildup of pore pressure in the soil with possible collapse of the face, or certain reduction of its ability to withstand the load imposed on it.” They also suggest (p 414-) that the scour at Hirtshalls (DK), Rotterdam (NL), and Europort (NL) could all have been exacerbated by partial standing wave patterns. The wave stirring would have increased the potential for sediment to be suspended and transported away by currents. Silvester and Hsu (1997) also consider that the Thorshave (Faroes) breakwater may have failed well short of design conditions due to swell causing scour in front of the structure (see also Sorensen, 1985). Several of the following cases were also reported by Silvester and Hsu (1997).

De Rouck and de Meyer (1987) and Silvester and Hsu discussed the scour erosion in front of large breakwaters at Hirtshalls, attributing to an interrupted sand supply, oblique reflecting waves and induced currents.

Markle (1986) surveyed field experience of scour in USA. “It is generally thought that toe scour is the significant problem after major storms. Bedding layers slough off into the scour holes and this damage migrates back to the toe of the primary armour. The resulting instability of the armour stone leads to downslope migration of the onslope armour and eventual deterioration of the structures”. Examples include Grays Harbor, Washington and Newburyport, Massachusetts. This process of unravelling, due to beach lowering at the toe can apply to scour protection layers as well as the armour layer.

At least four cases of toe scour have been reported in Africa. Arzewel Djedid (Algeria) had a 1:1.33 tetrapod breakwater until it slumped (Sorensen, 1985, Silvester and Hsu, 1997). The cross-sectional area of collapsed debris was smaller than the cross-sectional area of the breakwater as built. Therefore compaction (including possibly due to breaking of tetrapods) and slumping due to scour and/or liquefaction may have caused the failure. Bartels *et al*, (2000) reported on the failure and repair of the toe of an accropode breakwater at a small craft harbour near Cape Town, South Africa. In five years after the construction of breakwater, beach levels dropped by almost 4m. The ‘scour’ (reduction in beach level) extended for about 200m in front of the breakwater toe. Toe erosion has also been observed at several places along the main breakwater in Tripoli harbour, Libya (Lindo and Stive, 1985, Silvester and Hsu, 1997). Oumeraci (1994a) reported that seabed scour may well have contributed to the failure of Mustapha Breakwater (Algiers).

Migniot *et al*, (1983) reported on scour during the construction of Ashdod (Israel) breakwater, where the local scouring of the seabed caused the toe to slide into the scour trench. The tetrapod armour was left unsupported and subsided several metres.

The Japanese have recorded a large number of cases of breakwater damage in the last 50 years. Takeyama and Nakayama (1975, in Japanese) reported 900 cases of breakwater failure, 259 of which were explained by bed scouring (summarised by Silvester and Hsu, 1997). Also see Hattori *et al*, (1984, in Japanese) as quoted by Oumeraci (1994a). Examples of breakwaters at angles to the coast include Naoetsu (Tsuchiya *et al*, 1976) Fukui (modelled by Irie and Nadaoka, 1984) Niigata, Kashimi and Tagonoura (Ichikawa, 1967). Yokoyama *et al*, (2003) pulled together a large database of laboratory experiments, field observations and hazard evaluation data. The latter are based on the damage and subsidence of coastal structures during storms (determined after the storms were over) and are not highly accurate. Yokoyama *et al*, (2003) used field data from Kihara *et al*, (1988) Arai *et al*, (1987) Uda *et al*, (1989), Tokuda *et al*, (1989) Syouji *et al*, (1990, 1991) and Kuroiwa *et al*, (1990, 1993). Yokoyama *et al*, concluded that:

1. Scour depth is related to wave height;
2. Scour depths are largest if the structure is in the outer surf zone (where waves start to break);
3. Scour depth near porous structures are about half those at impermeable structures;
4. A Boussinesq wave model combined with a sediment transport model gave promising results when modelling scour depths;
5. Numerical models of scour should be calibrated using field data.

Toyoshima (1978) reported how some Japanese sea dykes were upgraded following the Ise Bay Typhoon in 1959. Several of the new seawalls were destroyed by scour within one year of their construction. The seawalls were observed to have high reflection coefficient and the reflected waves were blamed for washing out the sand at the toe of the seawalls.

Suzaki and Shimmura (2002) reported an interesting case of embankment collapse at a Japanese railway line. In this case the beach lowered over many years. This allowed scouring and greater overtopping to occur. The foundation became exposed by the scour and the soil relaxed due to greater saturation by overtopping. The seawall / embankment collapsed due to scouring during a storm, with the slope failure affected by the increased hydraulic pressure in the soil behind the embankment.

Laboratory experiments

A review of the most relevant experiments has been included in this section. However, further references to scour experiments can be found in Section 6 (References and Bibliography) and the reader is referred to Kraus (1988) and Kraus and McDougal (1996) for more extensive reviews.

This section can be divided into small and large-scale experiments, depending on the wave characteristics and sediment used. A further division could be made into two and three-dimensional experiments. However, because of the scarcity of 3D studies the reader should assume that the experiments quoted are 2D unless otherwise stated.

Small-scale experiments

Small-scale refers to experiments conducted with waves of height less than 0.15m on models composed of fine to very fine sand (Kraus and McDougal, 1996). One has to be careful when using relations or results derived from small-scale experiments, as they may not maintain similitude and the resulting observations will not be indicative of field conditions.

One of the earliest studies, by Russell and Inglis (1953) is worth mentioning, as it is the only one to have used varying water levels to reproduce tidal conditions. These tests confirmed that a vertical wall constructed at the top of the beach (in the run-up zone) increased turbulence and hence led to the depletion of the beach immediately in front of it. It was concluded that scouring would probably cease at a level about one wave height below low water, although the ultimate scour depth was not determined.

Sawaragi (1966) investigated the influence of the void ratio of permeable structures on the coefficient of reflection, concluding that although the coefficient of reflection was fairly constant for void ratios greater than 20%, it increased rapidly for smaller values. Also, for a particular void ratio, the reflection coefficient increased with increasing sea wall slope. The effect of reflections on the scour depth was then examined, concluding that for breaking waves (in line with Sato *et al*, 1968) the scour depth does not always increase in time and that the scour process is often interspersed by periods of accretion or infilling of the scour hole. It was also observed that the relative scour depth increased with the reflection coefficient, suggesting a boundary value at 0.25, so that rapid increases in scour are seen for coefficient of reflections up to 0.25 and a more gentle increase for values greater than 0.25. No scouring was observed at the toe of permeable walls for coefficient of reflection lower than 0.10.

Ichikawa (1967) investigated the scouring of sloping beds under breaking waves, observing that scour at the foot of vertical walls was always a maximum under "curling" breakers. Although in general these scour depths were equivalent to the offshore wave heights, there was some indication of a bed slope effect, with steeper slopes suffering slightly greater scour than shallow slopes under the same wave conditions. Further analysis of the results (Powell, 1987) suggested that maximum scour occurs when the initial water depth at the seawall is approximately 1.5 times the offshore regular wave height; for water depths either side of this value, scour depths appear to reduce quite rapidly.

Herbich and Ko (1968) investigated the scouring of sloping beds under breaking waves. Although an empirical formulation for scour prediction was developed (Section 3.5.2) the fact that the predicted scour was defined as the constant

average depth towards which scour tended, over a distance (of 4.5m normal to the physical model) makes it an inappropriate measure of toe scour. Another important conclusion was that the distance-averaged scour was not a function of the reflection coefficient. This was reiterated in a review of earlier work in Herbich *et al*, (1984), where the seawall slope influence was also explained. Seawall slopes between 45° and 90° had no effect on the distance-averaged scour, whereas for seawalls flatter than 45° the scour depth was observed to decrease with decreasing slope.

Scouring at the foot of vertical and inclined seawalls as well as at the base of composite breakwaters at different positions on the beach was examined by Sato *et al*, (1968). Depending on the location of the seawall to the initial breaking point, five types of scour were identified in terms of the scour time development. Occurrence of maximum scour depth at the seawall toe in relation to wall location is given in Table 4. For all wall locations offshore of the original breakpoint, the scour depth was found to decrease with sea wall slope, the difference being small for slopes in the range of 60° and 90°. It was also concluded that the relative maximum scour depth was inversely proportional to the offshore sea steepness, although such a trend may only be applicable for walls located seaward of the shoreline. An important conclusion from their study was that under normal storm conditions ($0.02 < H/L < 0.04$), the maximum scour depth could be expected to be less than or equal to the deep water wave height. However, in calmer conditions the scour depths could be very much greater than the corresponding wave heights.

Table 4 Occurrence of maximum scour depth at the seawall toe in relation to wall location according to Sato, Tanaka and Irie (1968)

Seawall Type	Wave Conditions	Beach Profile	Location of seawall for maximum scour depth at the toe of the seawall
Vertical	Low steepness	Steep	When wall located at or about the wave plunge point
Vertical	Steeper waves	Storm or bar type	When wall sited at either the shoreline or just landwards of breaking point
Inclined	Storm conditions	Storm or bar type	Only at one position – just landward of the break point

A similar study was carried out by Song and Schiller (1973) focusing on the effects of seawall location on wave reflection. It was concluded that low reflection coefficients generally corresponded to greater scour depths, despite the scatter in the results. It was explained as the scouring cause being the turbulent nature of wave breaking, rather than any reflection of energy (low reflection coefficients coincided with waves breaking at the seawall). However, this explanation does not explain the differences between Song and Schiller (1973) and Sato *et al*, (1968) regarding the effect of the seawall location on the scour depths. For all wave conditions Song and Schiller (1973) found that maximum toe scour

occurred with the wall at or about the wave plunge point, while minimum scour always occurred when the wall was at the break point. They derived an empirical expression to predict relative scour depth (Section 3.5.2) from which they concluded that, for small values of the standing wave steepness (lower than 0.02), the relative scour depth was virtually independent of the position of the seawall on the beach, which seems an over-simplification.

Hotta and Marui (1976). Although their results with permeable walls did not fit the Sato *et al* (1968) classification, some similarities between the scouring characteristics of permeable and solid walls were found. Scouring depths were observed to be at their minimum at or about the plunge point, as suggested by Song and Schiller (1973) and were independent of the bed materials or the hydraulic properties of the walls tested. Maximum scour depths were found to be of the order of the deep-water wave height. Further analysis of their data (Powell, 1987) suggested that for walls located seawards of the plunge point, the maximum depth of scouring occurs when the water depth is in the region of 1.6 times the deep-water wave height. They also highlighted one of the major problems in any study of toe scour, which is that localised toe scour is always likely to be superimposed on much larger natural beach movements.

Nishimura *et al*, (1978) tried to simulate the scouring at the toe of seawalls caused by tsunamis with the simulation of single waves breaking on a mobile slope, colliding against the inclined seawall, overtopping and running up on the gentle slope behind the wall. Although small overtopping was observed during this process, when the overtopping water returned back over the wall, a pronounced scouring of the beach material was evident. This scour depth was found to depend on the slope of the seawall, the depth of water in front of it, the velocity of the returning flow and the specific gravity and size of the bed material. In general, it was found that lighter and finer materials suffered greater scour.

Irie and Nadaoka (1984) carried out experiments on composite breakwater models in both 2 and 3 dimensions, using regular waves, in order to determine whether prototype scour behaviour could be reproduced in laboratory models, and if so, the requirements that need to be fulfilled. They found two types of sediment movement (as described in Section 3.3). L-type scour under suspended load conditions during storm events is most critical for the stability of the structure. This type of scour was found to be dominant for ratios of near-bed peak orbital velocity to fall velocity of sediment over 10, regardless of the Ursell number. The 3D tests with irregular waves at 30° (to a line normal to the breakwater) showed near-bed drift velocities parallel to the breakwater toward the shoreline and scour at the nodal locations close to the toe in case of dominant suspended load transport. Scour was found to be largest near the tip of the breakwater.

Xie (1985) investigated scour at vertical breakwaters by using two tanks and mainly regular incident wave conditions. He classified the two major scour patterns found according to a velocity ratio similar to that proposed by Irie and Nadaoka (1985). An empirical relation for the prediction of the scour depth was developed (Section 3.5.2) in which the maximum scour depth at the toe is proportional to the incident wave height and a hyperbolic function of the ratio of

depth to wavelength. Also, a scaling law was proposed for comparing laboratory and prototype scour depths.

Ling *et al*, (1986) investigated qualitatively the influence of short-crested waves on the scouring around a breakwater in a physical model and effected a comparison with field data. Their main conclusion was that oblique incident waves and their reflected waves are formed into a system of short-crested waves in front of the breakwater that progresses along it and gradually swells up. The breaking short-crested wave height is larger than that of the progressive wave and belongs to the plunging type, resulting in serious scouring around the breakwater. They suggested the consideration of short-crested waves in the design/ building of breakwaters.

Barnett and Wang (1988) performed a series of physical experiments to investigate the effect of a vertical seawall on profile response. For all the cases tested, profile configurations with and without a seawall were remarkably similar in overall planform; the authors suggesting that the major transport process is not significantly influenced by the presence of the seawall. The presence of the seawall accentuated the trough formed into a scour hole instead of spanning over the swash zone, as in a natural beach. Although local scour was noted to be severe in many of the seawalled profiles, the volume of sand retained inshore of the structure (that would otherwise have been eroded if no seawall was present) was experimentally found to be approximately 60% greater than the additional volume eroded at the toe of the structure. Wave reflection influence on scour in front of a seawall did not appear to play a significant role. An interesting finding was the fact that the seawalled beach exhibited a more substantial recovery volume in the vicinity of the structure toe than that observed for the natural profile. However, the authors noted that this was not sufficient evidence to conclude that placement of a seawall on an eroded beach will promote recovery.

Toue and Wang (1991) examined the effects of a seawall on the adjacent beach by a three-dimensional physical model test. It was found that, under normal incidence waves, the rate of volumetric erosion as well as the total eroded volume in front of the seawall was smaller than that of the natural beach. Under oblique incidence waves, due to groyne effects, down-drift erosion was severe, but the effects were found to be localised within a region about three to four times the seawall length, centred on the seawall.

Silvester (1991) presented a review of scour by reflecting non-breaking waves in three-dimensional movable-bed experiments, observing that oblique waves at a vertical wall are reflected with almost 100% efficiency. Scour trenches were generated by waves and wave-induced drift velocities parallel to the reflecting wall, depending on the obliquity of the incoming waves.

Kamphuis *et al*, (1992) carried out a set of three-dimensional tests on a longshore uniform beach backed by a seawall. The equilibrium profile developed in front of the seawall was a complex function of the initial profile, the storm surge level and the wave climate; the longshore sediment transport rate decreasing as the beach eroded in front of the seawall. The local depth was found to be closely related to the local wave height, the ratio H/h approaching a constant value as the beach

approached an equilibrium condition. However, they concluded that average scour depth in front of a seawall could not be simply related to offshore wave height.

Seaman and O'Donogue (1996) conducted an experimental study of beach response in front of reflecting structures under N-type (bedload) beach response, presenting an equation for the equilibrium beach profile acquired. From this equation, the maximum scour depth can be obtained in terms of the amplitude of the profile, which is a function of the bed profile wavelength (which is a function of the water wavelength and the lengths of bed mobilised). However, further work at a larger scale with different sand sizes was acknowledged to be needed to investigate the general application of such a formulation. The authors also acknowledged the ripples superimposed on the bed profile to play a crucial role in the suspended sand transport process.

Sumer and Fredsøe (1997) carried out a three-dimensional experimental programme on the scour at the round head of a rubble-mound breakwater. They concluded that there were two mechanisms that cause scour: the steady streaming occurring above the bed, around the breakwater head, and the plunging breaker, which occurs at the head of the breakwater. The first mechanism causes a scour hole in front of the breakwater and adjacent to it and is governed by the Keulegan-Carpenter number. The second mechanism produces a scour hole at the lee side of the breakwater head and is governed by a parameter involving the wave period, wave height and water depth. The use of a stone protection layer was also investigated and an empirical formulae for its width was proposed (as stated in Section 4.3).

Gao and Inouchi (1998) conducted an experimental exercise to prove the importance of the scouring in front of a vertical breakwater by broken clapotis to be of greater importance than the scouring produced by any of the other two wave motions (standing wave and breaking clapotis). They also identified three typical patterns of scour and deposition within one wavelength from the breakwater under the action of broken clapotis, depending on a criterion based on a relationship between wave conditions and grain sizes. These patterns were denoted as DSD, SDSD and DSDS types, where D and S represent a deposited ridge and a scouring trough respectively.

Sutherland *et al*, (1999, 2000) carried out three dimensional experiments on the scour and deposition around a single detached offshore rubble-mound breakwater, investigating how oblique incidence and longshore currents affected the distribution of scour and deposition. The 3D tests complemented the 3D head and 2D trunk scour tests of Fredsøe and Sumer (1997) and Sumer and Fredsøe (2000). The results clearly showed the influence of three-dimensional effects – even for the normal incidence case, as the central section of the model breakwater was only 4m long. The sediment transport was mainly by bedload, with some near-bed suspended load transport. Deposition dominated at the trunk section of the breakwater for the normal and oblique incidence cases, due to longshore gradients in the sediment transport. When the oblique wave case was run with a tidal longshore current as well, the deposition was pushed further downstream and a deeper scour trough formed

along the toe of the breakwater. The scour depths were compared with the 2D results of Fredsøe and Sumer (1997) by Sumer *et al*, (2001).

Sumer and Fredsøe (2000) studied the scour at the trunk section of a rubble-mound breakwater in an experimental study in which they used two breakwater models with different slopes and a vertical wall breakwater as a reference structure. The scour/deposition pattern found in front of the rubble-mound breakwater was in the form of alternate areas of scour and deposition parallel to the breakwater, similar to the case of the vertical-wall breakwater (Xie, 1981). The maximum scour depth was found to be smaller than that of the vertical-wall breakwater and comparing results between the regular and irregular wave conditions led to the conclusion that the scour is greater in the case of regular waves (as obtained by Hughes and Fowler, 1991 and Xie, 1985). Toe protection comprising an apron with one or several layers of stones was also investigated.

Neelamani and Sandhya (2003) investigated the hydrodynamic performance of vertical and sloping plane, indented and serrated seawalls in a physical model study. Wave reflection from the different seawalls (at five different inclinations) under regular and random waves was measured to assess the dissipation character of the seawalls. Predictive equations for the coefficient of reflection for the different walls are given. Further investigation is apparently being conducted at the moment on the scour development under the different seawalls.

Medium and large-scale experiments

The main advantage of large-scale experiments is that they are not affected by similitude problems to the same extent as smaller laboratory experiments. They are generally carried out in large-scale wave flumes where suspended sediment transport can be generated. The difference between large scale and small scale experiments is taken to be that small scale experiments are determined by bedload sediment transport, while large scale experiments have a significant percentage of suspended sediment transport.

The equations below can be used as a first approximation to obtain the percentage of waves that will generate suspended sediment transport. A more detailed approach can be found in Tørum *et al*, (2003).

Xie (1981) formulated the following criteria for suspended sediment transport to occur:

$$\frac{U_m - u_{cr}}{w_s} \geq 16.5$$

where U_m is the maximum wave orbital velocity, u_{cr} the critical wave orbital velocity for start of motion and w_s = sediment fall velocity. Linear theory can be used to calculate the wave height at the threshold of motion, H_t , from which u_{cr} can be calculated (given depth, h and period, T). The minimum bed orbital velocity for suspension to occur is then $U_s = 16.5w_s + u_{cr}$. The minimum wave

height for suspension, H_{sus} then follows. Assuming a Rayleigh distribution for wave heights gives the probability that any wave is lower than or equal to a height H as: $P(H) = 1 - \exp\{-H^2/H_{rms}^2\}$ with $H_{rms} = 0.71H_s$ = the root mean square wave height and H_s = significant wave height. The probability that a wave will not cause sediment motion, $P(H_t)$ and the probability that a wave will not cause suspended sediment transport, $P(H_{sus})$ can then be estimated. It is easy to calculate the percentage of waves that will cause suspended sediment transport (%sus) from $P(H_{sus})$.

Some example calculations are given below for a sediment diameter, $d = 0.11mm$ (fine sand). Fresh water at $15^\circ C$ was assumed. The sediment fall speed, $w_s = 0.0084m/s$ for a particle density of $\rho_s = 2650kg/m^3$. Calculations based on possible experimental conditions are given in Table 5. To obtain cases where more than about 50% of the waves should cause suspended sediment transport will require wave periods of around 2s and inshore significant wave heights of at least 0.11m in 0.2m depth or 0.15m in 0.3m depth or 0.20m in 0.5m depth. These wave heights in 0.2m and 0.3m depth are close to saturation and such wave heights are only generated in deeper wave flumes. The limited number of scour tests where suspended sediment transport will have played a large role are listed below.

Table 5 Example calculations of the percentage of waves causing suspended transport

h	T_p	H_s	H_{rms}	H_t	$P(H_t)$	H_{sus}	$P(H_{sus})$	%sus
0.3	2	0.15	0.105	0.028	0.07	0.065	0.44	56
0.3	2	0.10	0.071	0.028	0.15	0.065	0.73	27
0.3	1	0.10	0.071	0.030	0.16	0.110	0.91	9
0.3	1	0.15	0.105	0.030	0.16	0.110	0.67	33
0.2	2	0.11	0.078	0.021	0.08	0.064	0.49	51
0.5	2	0.20	0.141	0.039	0.07	0.115	0.48	50

Hughes and Fowler (1991) carried out model tests to validate their theoretical description for predicting scour at a vertical wall produced by normally incident non-breaking irregular waves. Maximum scour was substantially less under irregular waves than under regular waves. The authors concluded that the phenomenon may not be of significance for design and that prediction methods for the majority of the scour problems experienced at coastal structures is still lacking. They also mentioned that scour might be enhanced by lateral currents in situations involving both cross-shore and longshore water motions.

Fowler (1992) performed mid-scale (wave heights between 0.2 and 0.3m) laboratory tests of scour in front of a vertical wall using a scaling law to preserve the similitude of the dimensionless fall speed number between model and prototype. Results from the tests were compared with those from several previous laboratory studies and an empirical equation for scour prediction was developed (Section 3.5.2), in which the ratio of the depth of water at the wall to the deep-water wavelength was the important parameter. Because of the

relatively mild initial beach slope (1 in 5), Kraus and McDougal (1996) considered that the planar initial slope, which was not in equilibrium under surf zone waves, may have exaggerated the scour produced.

The large-scale SUPERTANK Laboratory Data Collection results involving seawalls are discussed in Kraus, Smith and Sollitt (1992), Kraus and Smith (1994) and McDougal *et al*, (1996). The programme involved three seawall tests and wave heights and periods were selected to correspond to destructive and constructive wave conditions. A remarkable result was that the profiles in front of the walls did not develop a large scour trench (nor did they erode or accrete). A small scour trench was created at the toe of the wall, but the influence was highly localised in the immediate vicinity of the wall. The limited scour found suggests that the scour trench sometimes observed in the field after storms may be a result of longshore transport or combined cross-shore and longshore transport occurring at the time of the storm. Measured results were used to verify the modified (including wave reflection at vertical walls) profile response model SBEACH, in the three cases obtaining good comparisons. Comparison between the original and modified profile response model SBEACH showed numerically that the beach profiles developed with and without a seawall were similar, in agreement with Hughes and Fowler (1990) results. The magnitude and time dependency of scour in front of vertical seawalls were numerically investigated with the enhanced SBEACH model, developing predictive equations for scour (Sections 3.5.2 and 3.5.4).

Experiments on dune stability with dune protection structures (consisting of sandbags) were carried out in the GWK large wave flume as part of the 96/97 experiments for the SAFE project, as described in Dette *et al*, (1998a, 1998b, 2002). Two different heights of such protection structures were tested (no-overtopping and partial overtopping allowed) and compared to the profile development when unprotected. As the sand container barrier interrupted the seaward-directed sediment transport, no dune erosion occurred and the profile in front of the barrier was flattened, mainly due to reflections. The initial 1:20 beach slope in front of the barrier between the 4m and 5m contour line disappeared completely, the material having been moved seaward to form a bar. The main difference between the two barrier height tests was that the profile change for the partial overtopping was less pronounced as the reduced barrier allowed partial overtopping of waves by which sand from the dune behind the barrier was transported into the foreshore profile.

Sakakiyama and Kajima (2002) investigated toe scouring in front of a seawall covered with armour units using large- and small-scale physical model tests. Comparison of the profile changes between both tests showed that there was no scour at the toe of the armour layer. Scour was only found under regular waves for the small-scale tests. In the large-scale tests some tetrapods settled through a gravel mat into the sand bed.

A summary of the hydrodynamic conditions of the large scale tests is provided in Table 6.

Table 6 Summary of large scale experiments

	H ₀ offshore wave height (m)	T _p Period(s)	h _t water depth at toe (m)	Structure type	d ₅₀ Grain size (mm)	m Bed slope	Grain fall speed (m/s)
Fowler (1992)	0.2-0.29	1.95-2.49	0.06, 0, - 0.06	vertical wall	0.13	1 in 15	0.019
Supertank	0.4 to 1.0	3.0 to 8.0	≈ 0.35, 0.4, 0.6	vertical	0.22	Initially at $x^{2/3}$ but for the rest of the tests using end conditions from previous tests	0.033
Sakakiyima and Kajima (2002) Scale of 1/22.7	0.45- 0.56	3.36	≈1.05	Tetrapod units in front of caisson	0.2	1 in 40	
GWK (Dette <i>et al</i> , 1998)	0.65-1.2	5.5	≈0	vertical dune barriers (sandbags)	0.3	End position from previous test	0.042

4. Review of mitigation measures

This section is subdivided into three main parts that reflect the steps that can be taken to mitigate against scour at different stages of a structure's life. Section 4.1 looks at the precautionary measures that can be used at the design stage to minimise future problems. Section 4.2 considers the monitoring that can be done during the lifetime of a structure that could be used to trigger intervention or a more detailed study, before the structure fails. Section 4.3 considers the methods used to prevent a scour hole from deepening any further (so it considers the types of intervention that might be triggered by monitoring). As stated in Section 1.2 the aim is to provide preliminary advice on the mitigation of the scour rather than comprehensive guidance.

4.1 Precautionary measures (at design/construction stage)

Anticipating of general changes in ground level

Beach lowering happens over a range of time-scales and spatial scales, illustrated by Table 7. The changes are due to longshore and cross-shore sediment transport processes. At the larger scales, coastal evolution is generally governed by longshore transport. At the shorter scales, cross-shore transport becomes more important.

The importance of inter-annual variations in forcing can be illustrated by looking at the variability in mean annual nett potential longshore drift rates calculated by numerical models (e.g. Sutherland and Wolf, 2002). Timeseries of wind data covering typically 15 to 30 years can be used to generate nearshore wave time series for the same period. A CERC-type formula for longshore transport can then be used to estimate time series of longshore sediment transport. An annual nett longshore drift rate may then be calculated for each year. A mean annual nett longshore drift rate and the standard deviation in the net annual longshore drift rate may then be calculated from the years in the timeseries. In many cases the standard deviation is of the same order of magnitude as the mean annual drift rate. In some years the direction of net drift may reverse (see, for example, HR Wallingford, 2002). Such significant changes in longshore drift rate may be expected to influence the beach level in front of a structure, particularly on coasts where the longshore drift is impeded at some point.

Many beach profiles show a pronounced seasonal variation, characterised by beach draw-down (and the consequent flattening of beach profiles) during winter storms and a building up of the beach in the summer months. The seasonal changes in beach level in front of a coastal structure should be an important consideration in the design of the structure. It will influence the toe depth required, the forces on the structure and the overtopping rate that may be expected.

This study concentrates on local toe scour, with timescales of hours and a spatial scale of up to about 100m (away from the seawall) where cross-shore

sediment transport may be expected to be important (but will not necessarily dominate). Indeed, UK case studies (Appendix 1) indicate a preponderance of longshore-dominated cases, in the longterm. This does not obviate the need for research into short term changes, however, as these may trigger the failure of a seawall.

Table 7 Time and spatial scales for beach lowering

Phenomena	Time scale	Spatial scale
Long-term coastal evolution	Decades to centuries	10s to 100s of kilometres
Inter-annual variations in forcing	Years	Kilometres
Seasonal	Months	Metres to kilometres
Storm	Hours	Metres
Tides	Hours	Metres to kilometres

Minimising short-term localised scour by changing the seawall profile

Actual scour at the base of a seawall depends not only on the environmental variables described in Section 3.1 but also on the characteristics of the structure and its position on the beach profile relative to the swash zone.

Toe scour may be reduced by:

1. Positioning the seawall above the surf zone
2. Reducing the reflection coefficient of the structure
3. Resisting toe scour

Positioning the seawall above the surf zone

The position of the seawall relative to the surf zone is a critical parameter controlling toe scour (see Figure 10) and beach recovery (Kraus, 1988). Laboratory studies have shown that seawalls constructed in the surf zone are at the greatest risk of toe scour. The position of the seawall relative to the zone on the profile in which waves break, changes with variations in water level, wave height and period. The general rule, however, is that the further seaward the seawall is constructed the greater its influence as the impact on wave hydrodynamics is greater. If it is high up on the beach, above spring tide high water (MHWS), it will still provide storm protection but the opportunity for toe scour will be lessened (Tait and Griggs, 1990).

The position on the profile at which waves break, influences the position and depth of the scour hole. Waves breaking at the toe of the structure usually cause greater toe scour than waves breaking on the beach in front of the seawall or on the wall of the structure itself.

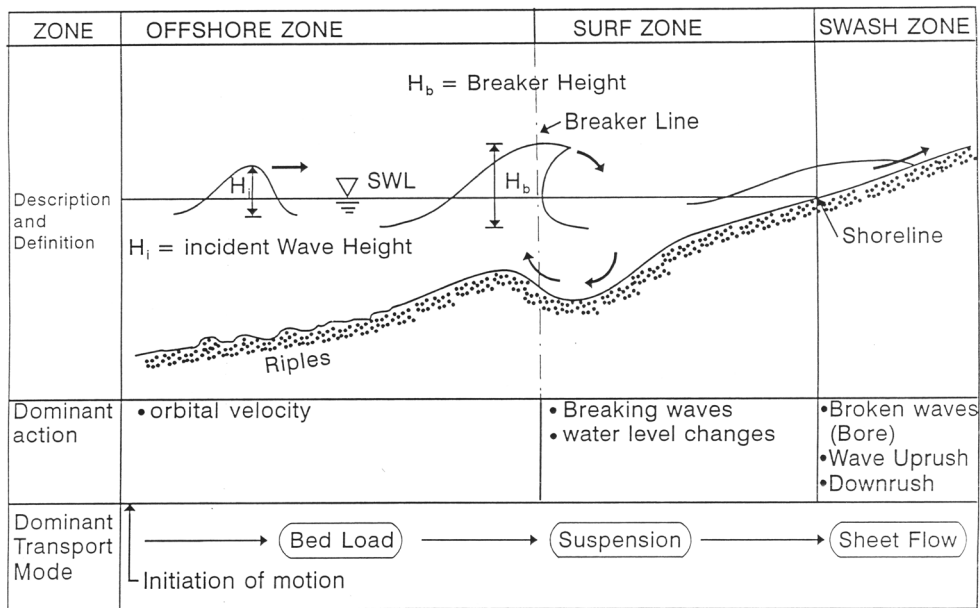


Figure 10 Dominant processes and modes of sediment transport (after Oumeraci, 1994c)

Reducing the wave reflection coefficient

The wave reflection coefficient is the ratio of the reflected wave height to the incident wave height. Reducing the amount of wave reflection theoretically reduces toe scour. This opinion is upheld by the majority of laboratory and field data. However, there does not seem to be a simple relationship between scour and the reflection coefficient. This is illustrated by Figure 11, produced by Powell and Lowe (1994) by comparing values of dimensionless scour, S/H_s , and reflection coefficient, K_r for different characteristic sea steepnesses on a shingle beach. Regardless of wave steepness, reflection coefficient can increase or decrease with the nondimensional scour depth.

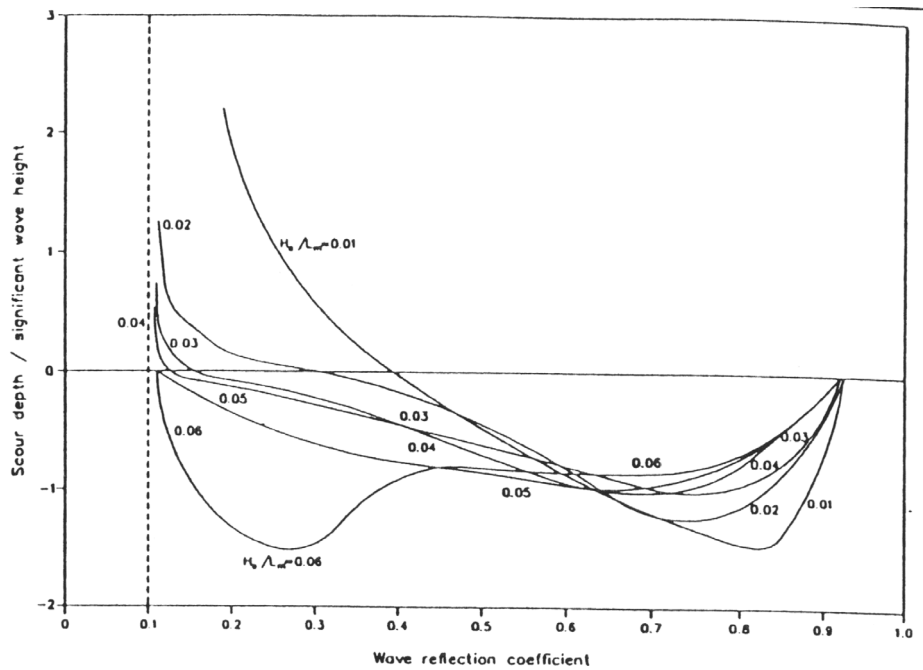


Figure 11 Wave reflection – Scour depth trends: Vertical seawalls (Powell and Lowe, 1994)

Design features to reduce wave reflection include using a sloping rather than vertical wall (Herbich *et al*, 1984, Powell and Lowe, 1994), increasing surface roughness by incorporating rip-rap and increasing the void ratio of the rip rap (Sawaragi, 1967).

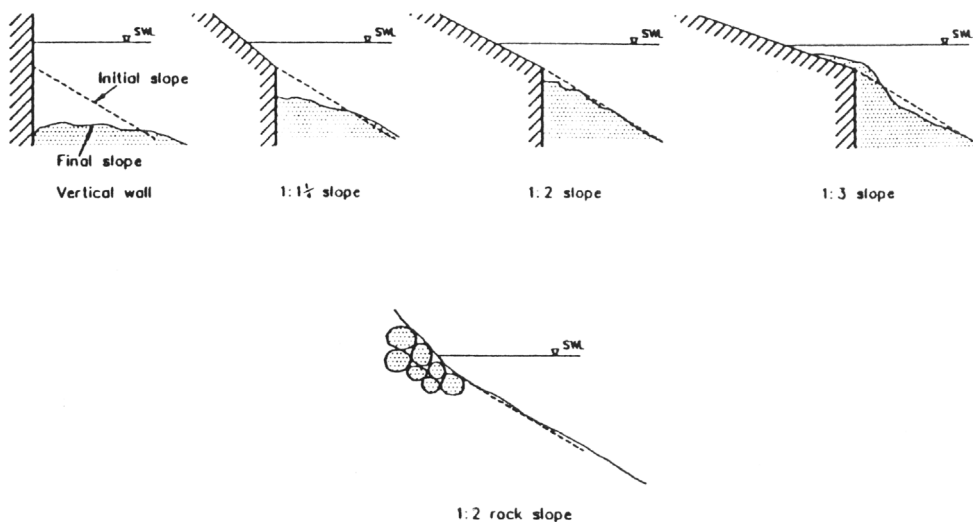


Figure 12 Classifications for scour response types (Powell and Lowe, 1994)

Figure 12, from Powell and Lowe (1994), illustrates the results of a series of laboratory flume tests to qualitatively investigate the influence that seawall type had on scour development of shingle beach. All were subjected to identical

wave and water level conditions. It can be seen that scour is the greatest for the impermeable vertical wall and decreases as the slope increases. A pronounced reduction is also observed when the 1:2 seawall slope remains but is made permeable by use of rock armour.

Sumer and Fredsøe (2000) experimented with the effect that the breakwater slope had on the scour depth using a small scale two-dimensional flume (Figure 13) The milder the slope of the breakwater, the smaller the scour depth, because the reflection and hence strength of the steady streaming at the bed decreases with decreasing slope.

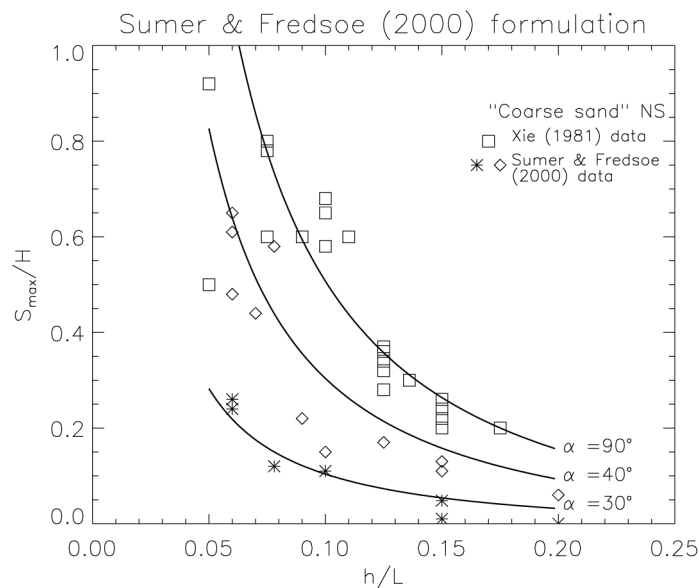


Figure 13 Influence of the slope of the breakwater (Sumer and Fredsøe, 2000)

Herbich *et al*, (1984) also experimented with the effect that seawall angle had on the scour depth using a small scale two-dimensional flume. However, the average scour depth over the area 4.5m in front of the wall rather than toe scour was measured. The tested angles with respect to the horizontal were 90 (vertical), 67.5, 40, 30 and 15 degrees. It was found that a low wall angle was required before there was a significant difference in scour. The results for the 90, 67.5 and 40 degree sloping walls were approximately the same. A cautious conclusion from these tests is that the slope should be less than 45 degrees above horizontal to reduce scouring.

The slope of a breakwater affects the reflection coefficient and phase shift on reflection, and the latter affects the location of nodes and antinodes. Here we recall that, for suspended sediment transport, scour occurs under the nodes of standing waves. It is therefore sensible to ensure that a partial node is unlikely to occur at the toe of a breakwater for typical storm conditions that might induce scour.

Resisting toe scour

This will involve the methods covered in Section 4.3.

4.2 Monitoring methods

The lowering of beaches in front of coastal structures occurs at a range of different timescales. This study concentrates on the short term, relatively localised scour that takes place close to a seawall, rather than the seasonal variation in beach level, or long-term trends in beach volume and level.

As the scour phenomena is frequently short-lived, the twice-yearly beach profile monitoring carried out round the English coastline by, for example, the Environment Agency is unlikely to capture a major scour event. Indeed, the evidence supplied by scour monitor data collected by HR Wallingford at Blackpool suggests that a significant fraction of the scour hole can fill in within a few hours of the peak of the storm. More details of the “Tell-Tail” scour monitors can be found in Appendix 3. Therefore even regular beach profiling with a spacing of a few weeks, supported by profiles collected within a day or two of each large storm may not be enough to capture the transient phenomenon of toe scour in the field.

The deployment of scour monitoring systems that remain on-site, just in front of the breakwater toe operating at all water levels, for periods of weeks at a time may be the only realistic way of assessing the variability of a beach surface with time. For example, HR Wallingford has deployed “Tell-Tail” scour monitors at Teignmouth and Blackpool.

As scour monitors are likely to be deployed for a period of weeks (at most) a monitoring strategy could be implemented that looks for the bed lowering to the point at which short-term fluctuation could de-stabilise the sea defence. For this the likely scour depth for a given storm would have to be estimated, plus the depth of scouring that would create a risk of failure. The monitoring could then take place a few times per year (at least twice) and a more detailed study or remedial action undertaken should the beach level drop below pre-determined values. This approach would rely on the development of a set of fragility curves (representing the probability of failure as a function of scour depth) and on methods for modelling toe scour under a wide range of circumstances. The latter could be obtained using a Monte-Carlo simulation, starting from offshore wind or wave records. The objective would be to obtain a measure of volatility of the beach (how quickly it responds to storms) and probability distribution curves for beach level and structure failure. Threshold beach levels could then be set to trigger further study or intervention.

If a long-term record of beach levels in front of a structure is available, such as the EA’s twice yearly beach surveys carried out in Anglian Region for the last 10 years, then long-term trends in beach level in front of the structure and in intertidal beach volume should be calculated. If these values show a statistically significant decrease in beach level with time then existing trends

should be projected forwards to identify when the structure may become vulnerable to toe scour, should recent trends continue.

4.3 Intervention measures – localised

In Section 4.1.2, a number of recommendations to reduce toe scour in terms of the structure characteristics were given, such as positioning the seawall above the surf zone and reducing the wave reflection coefficient. In this Section, a review of the mitigation measures and advice on its design is given.

Toe details adopted at the phase of designing a structure will be dependent on the assessment of the maximum depth of scour and the depth of mobile beach in front of the structure.

Review of mitigation measures

Although design guidance for mitigation measurements exist, as will be seen in the next section, engineers and practitioners usually find that the design advice apparently leads to conservative values. Having said this the monitoring of in service structures with scour protection is not routinely undertaken and published. The conservatism in apron design usually depends on the accuracy of the methods used to predict the waves and current action and to predict the maximum depth of scour that needs to be accounted for.

Many of the mitigation schemes implemented around the UK have involved placement of protective aprons at the toe of a pre-existing structure. Consequently, much of the design guidance is for this type of mitigation measure and so the advice in this section is heavily weighted to the design of protective aprons. The use of a protective apron is not a guarantee against scouring, however, as shown by the example in Box 1, where a solution was only obtained from the third mitigation measure tried. Alternative mitigation measures include the following:

1. **Rock dumping for bed stabilisation.** This can be a crude form of apron or toe berm that fills the scour hole, but has no filter layer or geotextile so is subject to the winnowing of bed material through it. Nevertheless a rock dump can provide a quick and cheap method of filling in a scour hole. Rock dumps should be monitored to ensure that they continue to provide protection.
2. **Mattresses.** These can be of two main types: flat gabions and linked precast units. The gabions consist of mesh forming low rectangular boxes that are filled with stones. They absorb energy, are flexible enough to fit an irregular seabed, are cheap to fill and relatively easy to lay. The linked precast units (which can be concrete) form a flexible sheet that is easy to lay. Multiple mattresses may be linked to form wider sheets. Neither has a filter layer or geotextile so both forms are subject to the winnowing of bed material.

3. **Soil improvement to increase bearing capacity and reduce scour potential.** An example of this, which is in use at a few sites, is beach drainage (Shaw, 2003). Early studies showed a link between beach water table level and the rate of erosion (Emery and Foster, 1948, Grant, 1948). Later tests showed a link between an artificially lowered water table and increased beach stability and in some cases accretion (Vesterby, 2000, Turner and Leatherman, 1997). In the UK there have been three beach drainage installations: a full scale trial at Holme-Next-The-Sea (Norfolk), a commercial system at Towan (Cornwall) and an experimental system at Branksome Chine (Dorset).
4. **Beach renourishment.** In many cases the long-term development of beach levels depends more on longshore transport than on cross-shore transport. In some such cases beach nourishment has been the solution to scour problems (see Appendix A for case studies). Beach nourishment is often combined with the use of beach control structures, such as groynes, which reduce the rate of longshore drift, thus helping to hold the beach in place. The beach nourishment increases beach levels at the toe of the structure to such an extent that short-term scour holes caused by cross-shore transport during storms are unlikely to be a problem. Hanson *et al*, (2002) provide a review of beach nourishment projects, practices and objectives from a European perspective.

Box 1 Example from Minikin (1952)

Problem: At the toe of a sloping wall of bedded masonry, scour at the toe threatened to undermine the wall.

Solution: Constructing a stone apron 8ft wide with the top surface at then the beach level. All blocks of stone were over 6cwts in weight and concreted in with the long sides vertical. The new toe was further protected with 7in x 2.5in. timber sheeting driven 6ft into the beach.

Result: This did not end depletion of the beach, trenches again scoured out at the toe and the general level of the beach lowered by about 2ft.

Solution: A concrete beam or sill was cast seaward of the timber sheeting.

Result: No further improvement.

Solution: On the length concreted, an apron of long heavy stones placed endways in the beach at the then beach level was then tried. No concrete or binding material was used, so that some degree of porosity was still preserved in the new apron.

Result: Beach has remained static, except for occasional temporary changes, at the level of the new apron.

Mitigation measures design guidance

Toe protection consists of armouring of the beach or bottom surface in front of a structure, which prevents it from scouring and undercutting by waves and currents. It is usually placed in the form of an apron, which must remain intact under wave and current forces, as well as being flexible enough to conform to an initially uneven sea floor.

Toe protection usually consists of a layer/layers of quarystone large enough not to be removed by wave forces and an underlying layer of granular material or geotechnical filter fabric to prevent soil from being washed through voids in

the quarrystone. If the seawall is built on soft or sand soil, a sheet pile cutoff wall, driven deep enough to prevent scour from undermining the wall, may be required at the toe. In severe wave climates, both quarrystone and cutoff wall may be required.

The design of toe protection is related to wave and current intensity, bottom material and structure characteristics (such as slope, porosity and roughness). The design guidance given in the different engineering manuals is general and preliminary; advising on the use of physical modelling for optimising a more detailed design.

In practice, most design calculations or design decisions are based on guidance in the CIRIA/CUR Rock Manual, the British Standard (BS 6349 Parts 1 and 7), the Revetment manual by McConnell (1998) or the 'Shore Protection Manual' and Coastal Engineering Manual (to succeed the SPM) and other USACE Engineering Manuals. This section has compiled the design guidance given in these sources. The practical design guidance from the Defra report "Low cost rock structures for beach control and coast protection" project has been included in Box 2.

There are some rules-of-thumb (Box 3) from Hales (1980) survey on scour protection practices in the USA. It was also found in this survey that the minimum scour protection was typically an extension of the structure-bedding layer and any filter layers.

Box 2 Practical design guidance from “Low cost rock structures for beach control and coast protection”

Sites where the beach is relatively thin and overlies a strata which is not easily eroded
Simple solutions, as the one adopted in the breakwater at West Shore, Llandudno consist of a selected larger rock to provide stability and a good starting point for the structure at the toe. The large toe stone can, however, result in significant reflections of waves leading to increased erosion or scour in-front of the structure. This can be avoided by extending a bedstone apron some distance in front of the structure to protect the beach. The apron has the secondary purpose of providing access beside the structure, thus preventing the construction plant from damaging the beach.

Sites with significant beach above the impermeable strata

The front slope of the structure is often just extended such that the top of the base rock is below the expected depth of scour. Whilst this solution is reliable and robust (providing of course that the depth of scour has been correctly estimated) the toe can require considerable excavation and construction can be a dangerous activity (particularly if geotextile is laid in the foundations). An alternative approach provides a stockpile of armour at the surface of the beach which should settle into any scour-hole protecting the structure from undermining. The latter solution may increase initial scour (due to reflections from the stockpile) and is not as robust as the former, but may provide a useful alternative, particularly where construction of a deep toe cannot be carried out safely.

Sites where the structure is founded directly onto a hard strata

The sites are generally those which need special effort to excavate using standard plant and where it may be necessary to excavate a shallow toe trench to provide additional toe stability against local slipping. If this is the case care should be taken not to unduly weaken or fissure the underlying strata. Other toe details, such as timber piles or concrete units, are sometimes used to enhance toe stability.

Box 3 Rules-of-thumb (from Hales, 1980 survey in the USA)

- Minimum toe apron thickness: 0.6 to 1.0m (1.0m to 1.5m in NW USA)
- Minimum toe apron width: 1.5m (3m to 7.5m in NW USA)
- Material: quarrystone to 0.3m diameter, gabions, mats, etc.

Applicability: Not adequate when the water depth at the toe is less than two times the maximum nonbreaking wave height at the structure or when the structure reflection coefficient is greater than 0.25 (structures with slopes greater than about 1:3).

The next sections deal with mitigation measures for different structures: revetments and seawalls and bulkheads.

Revetments

In the case of revetments, scour protection is usually provided in the form of supplemental armouring. The revetment toe often requires special consideration because it is subjected to both hydraulic forces and the changing profiles of the beach fronting the revetment.

Design procedures are governed by hydraulic criteria (waves and currents-wave induced and tidal). Box 4 gives advice on the weights to use.

Box 4 Stone weight for revetment toes (as recommended by US Army Corp of Engineers, 1995)

For submerged toe stone: weights predicted by Eq 1.

$$\text{Eq.1 } W_{\min} = \frac{\gamma_r H^3}{N_s^3 \left(\frac{\gamma_r}{\gamma_w} - 1 \right)^3}$$

For toe structures exposed to wave action: Weights predicted by Eq 1 for minimum weight or Eq 2 for median weight.

$$\text{Eq. 1 } W_{\min} = \frac{\gamma_r H^3}{N_s^3 \left(\frac{\gamma_r}{\gamma_w} - 1 \right)^3}$$

$$\text{Eq.2 } W = \frac{\gamma_r H^3}{K_D \left(\frac{\gamma_r}{\gamma_w} - 1 \right)^3 \cot \theta}$$

- N_s : Design stability number for rubble toe protection in front of seawall, as indicated
- W : required individual armour unit weight (or W_{50} for graded rip-rap)
- H : monochromatic wave height
- K_D : stability coefficient (Tables 2 to 3 of EM 1110-2-1614, 1995)
- γ : specific weight of water
- γ_r : specific weight of the armour unit
- θ : structure slope (from horizontal)

There are a number of toe configurations that can be adopted (Figure 2-4 USACE manual, EM1110-2-1614, 1995), depending on scour conditions (moderate), construction procedure (dry or underwater). McConnell (1998) also gives an extensive compilations of toe protection, reproduced as Figure 14. Here d_s is the maximum anticipated scour depth. The revetment designs include those with rock armour and are described as 'typical seawall designs' by Burchart and Hughes (2003).

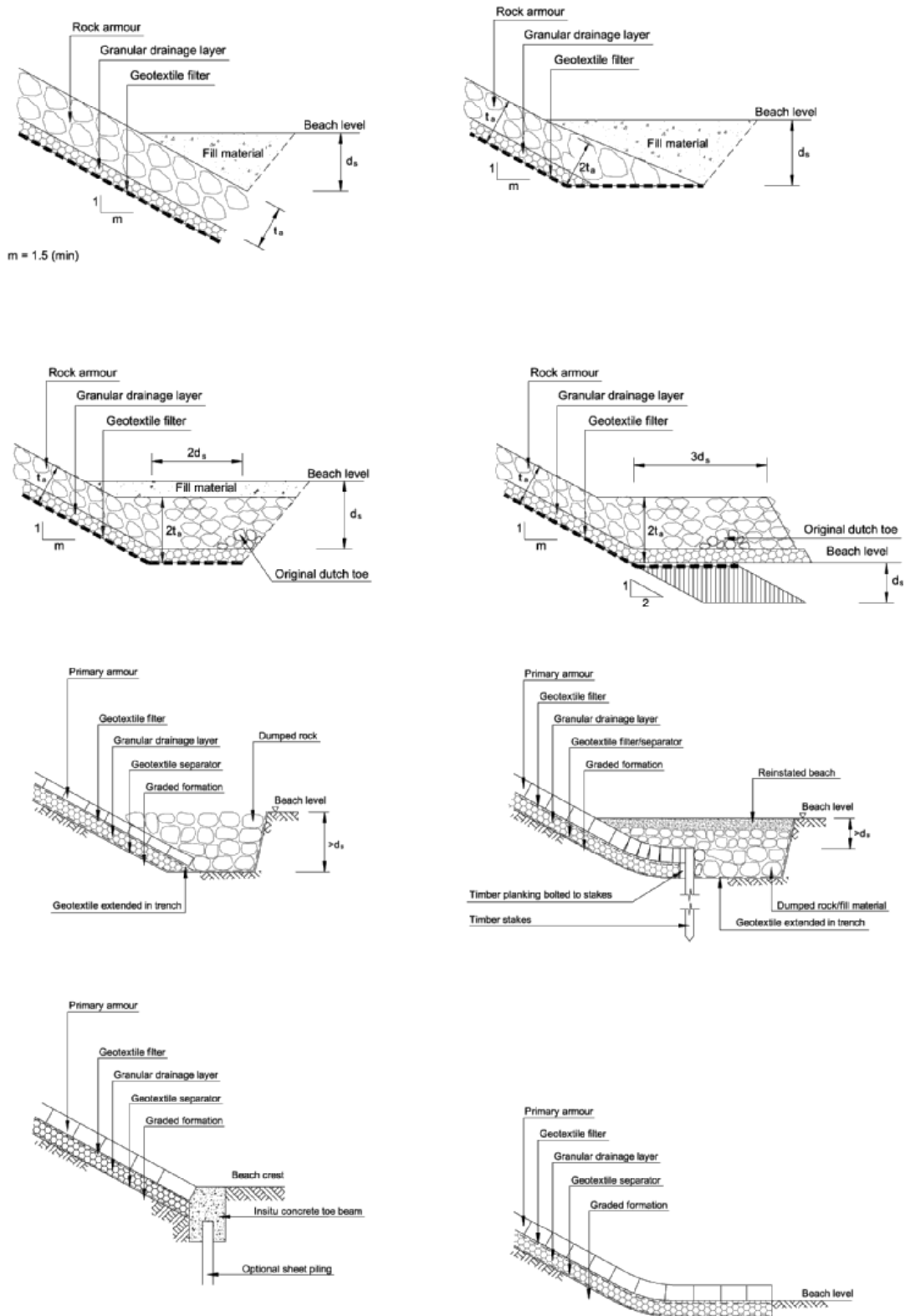


Figure 14 Typical toe details (McConnel, 1998). d_s is anticipated scour depth

Seawalls and bulkheads

Seawalls and bulkheads are usually divided depending on the structure front, which could be either vertical or sloping, as considered in the next sections.

Vertical-front structures

These structures include large caisson-type gravity structures, gravity retaining walls and cantilevered or anchored sheet-pile retaining walls. Scour protection is usually in the form of a scour apron. Design procedures must consider geotechnical as well as hydraulic factors.

For cantilevered or anchored retaining walls, the passive earth pressure zone must be maintained for stability against overturning. Toe scour results in a loss of embedment length and could threaten the stability of the structure. Gravity walls resist sliding through the frictional resistance developed between the soil and the base of the structure. Seepage forces are also important as the hydraulic gradients of seepage flows beneath vertical walls are thought to significantly increase toe scour and the risk of washout of infill material.

Design advice on apron width is given in Box 5. In all cases, undercutting and unravelling of the edge of the apron must be minimised. There are a number of toe configurations suggested (Figure 2-5 US Army Corp of Engineers, 1995) including vegetated toe (for low scour potential cases), cement bag protection (for low to medium scour potential) gabions or rock mounds (for moderate to severe scour potential).

For the toe stone weight it is recommended to use Brebner and Donnelly (1962). Using this method to determine the median stone weight, W , the allowable gradation should be approximately 0.5 to 1.5 W .

Determining the apron quarrystone size for any type of structure depends on the hydrodynamic conditions:

- **Waves:** Toe quarry stone should be sized using Belmer and Donnelly (1962). The apron thickness should be equal to either two quarrystone diameters or the minimum given in the prior rules-of-thumb (Box 3), whichever is greater.
- **Currents:** Toe quarrystone should be sized using guidance on stone blankets (Box 6).
- **Waves and currents:** Estimate the size of the apron quarrystone for the waves and the currents alone and increase whichever is larger by a factor of 1.5 (Eckert, 1983).

Sloping-front structures

The scour mitigation measure for sloping-front structures depends on the dominating hydrodynamic forcing: waves, currents or both.

Waves

Scour protection of sloping structures exposed to waves consists of a toe berm, whose function is to support the main armour layer and to prevent damage resulting from scour. Armour units displaced from the armour layer may come

to rest on the toe berm, thus increasing toe berm stability. Toe berms are normally constructed of quarry-run, but concrete blocks can be used if quarry-run material is too small or unavailable. Design advice for toe berms is given in Box 5 and typical forms of scour toe protection are illustrated in McConnell (1998) see Figure 14. Preliminary advice on sizing stable rock size is given in CIRIA/CUR (1991).

Additional scour protection is sometimes needed at sloping-front structures to prevent scour by laterally flowing currents.

Box 5 Width of scour apron for vertical-front structures (US Army Corp of Engineers, 1995)

Cantilevered or anchored retaining walls

It is recommended the use of quarystone scour apron, which width will be the greater of Eq 1 (geotechnical considerations) and Eq 2 (hydrodynamic considerations)

$$\text{Geotechnical consideration (Eq.1): } B = \frac{d_e}{\tan(45^\circ - \phi/2)} \approx 2.0d_s$$

$$\text{Hydrodynamic consideration (Eq.2): } B = 2.0H_i \text{ or } B = 0.4d_s \text{ (whichever is greater)}$$

Gravity retaining walls

They do not require the apron to be as wide as that for cantilevered walls.

$$B = H_i$$

Head of a vertical breakwater

Laboratory tests by Sumer and Fredsøe (1996) established a relationship for the width of a scour apron that provides adequate protection against scour caused by wave-generated lee-wake vortices as

$$\frac{B}{D} = 1.75(KC - 1)^{1/2}$$

The authors cautioned that this estimation of apron width may be inadequate in the presence of a current or for head shapes other than circular.

B	:	Width of scour apron
D	:	Diameter of the breakwater circular head (at initial seabed level)
KC	:	Keulegan-Carpenter number
H _i	:	Incident wave height
d _e	:	Depth of the sheet-pile penetration below the seabed
d _s	:	Water depth at structure toe
φ	:	Angle of internal friction of the soil (varies from about 26 to 36 degrees)

Xie (1981) conducted physical model tests using 3 different widths of protective layer (L_p=L/8,L/4,3L/8 where L is the wavelength of the incident waves) which consisted of coarse sand and 10-20mm crushed stones with a 30mm thickness. He concluded that the influence of the protective layer on the sand bed is mainly limited to a distance L/2 from the wall, so that:

- Maximum scour depth decreases with increasing length of protective layer, L_p
- Length of scouring trough also decreases with increasing L_p
- The distance from the wall to first scouring trough increases with increasing L_p
- There is essentially no scouring over a distance of L/2 from the wall when L_p=3L/8.

Box 6 Sloping structures toe protection under waves

Toe Berm (Burchart and Hughes, 2003)

- In very shallow water (with depth-limited design wave heights): support of the armour layer at the toe is ensured by placing one or two extra rows of main armour units at the toe of the slope. (This is a stable solution provided that scour does not undermine the toe causing the armour layer to slide.)
- In shallow water: it is usually possible to use stones or blocks in the toe that are smaller than the main armour.
- In deep water: there is no need for the main armour to cover the slope at greater depths and the toe berm can be constructed at a level above the seabed.

Toe berm stability is affected by wave height, water depth at the toe of the toe berm, width of the toe berm and block density. However, wave steepness does not appear to be a critical parameter.

Practical toe stability formulae for waves, based on small physical model tests, are given by:

Waves	Structure	Toe Stability Formula
Regular, head-on and oblique	Sloping and vertical, trunk and head section	Markle (1989)
Irregular, head-on	Trunk of sloping structure	Van der Meer, d'Agremond and Gerding (1995) Burchart <i>et al.</i> , (1995)
Irregular, head-on	Trunk of vertical structure	Madrigal and Valdés (1995)

Design curves, stability equations and Figures for these cases are provided in Tables I –5-45, VI-5-46 and VI-5-47 of Burchart and Hughes (2003).

Scour apron

Fredsøe and Sumer (1997) laboratory study of wave-induced scour at the round heads of rubble-mound structures included design suggestions for scour protection; the width of the scour apron, B (from the structure toe to outer edge) given by:

$$\frac{B}{D} = A_1 KC$$

Where D is the diameter of the breakwater head at initial bed level and KC, the Keulegan-Carpenter number.

Suggested values of A_1 are:

- $A_1 = 1.5$ for complete scour protection
- $A_1 = 1.1$ for relatively minor scour at the outer edge with a depth equal to about 0.01B

Currents

Toe protection against currents may require smaller protective stone, but wider aprons. Stone blankets constructed of randomly-placed riprap or uniformly sized stone are commonly used to protect areas susceptible to erosion by fast-flowing currents. Blanket applications include lining the bottom and sloping sides of flow channels and armouring regions of tidal inlets where problematic scour has developed. Design of stable stone or riprap blankets is based on selecting stone sizes such that the shear stress required to dislodge the stones is greater than the expected shear stress at the bottom developed by the

current. Preliminary advice in sizing stable rock size is given in CIRI/CUR (1991) and May *et al*, (2002).

Design advice for sloping structures exposed to currents is given in Box 7.

Box 7 Stone blankets (as given by US Army Corp of Engineers, 1994)

Stable stone or rip-rap blankets in current fields should be designed using the formulation from the USACE Engineers manual 1110-2-1601 (1994)

All graded distributions (riprap) used for stone blankets should have distributions conforming to the weight relations given by USACE Engineers Manual 1110-2-1601 (1994). Both sets of information are given in Burchart and Hughes (2003), Part VI-5-3.

Recommended thickness of blanket layer depends on whether the placement is submerged or in dry, with a minimum blanket thickness of 0.5m

Waves and currents

Coastal structures, such as entrance jetties, are exposed to waves combined with currents running parallel to the structure trunk. Toe stability under some circumstances may be decreased due to the vectorial combination of current and maximum wave orbital velocity. For normal wave incidence, the combined wave and current vector magnitude is not greatly increased. However, in the case of jetties where waves approach the jetty trunk at large oblique angles (relative to the normal), the combined velocity magnitude becomes large and toe stability is jeopardised. Preliminary advice in sizing stable rock size is given in CIRI/CUR (1991). Smith (1999) revised Markle's (1989) method to include waves and currents. First use Markle (1989) to calculate the stability number, N_s . Then calculate the current-modified stability number from:

$$(N_s)_c = a \left(\frac{U + u}{\sqrt{gh_s}} \right)$$

where U = current speed

$u = gHT/2L$ = maximum wave orbital velocity in shallow water

$a = 51.0(h_b/h_s) - 26.4$

h_s = water depth in front of toe berm

h_b = water depth over toe berm

H = breaking wave height

T = wave period

L = local wavelength

If $(N_s)_c > N_s$ the stone will be unstable. This method is also given in Burcharth and Hughes (2003).

5. Conclusions and research needs

5.1 Summary of beach lowering report

This report has summarised the present state of knowledge about beach lowering in front of coastal structures (Section 2). The various methods for predicting toe scour have been summarised (Section 3) and the strengths and weaknesses of the methods discussed. Mitigation methods have been introduced and design guidance (where it exists) has been summarised (Section 4). A few general features of beach lowering are summarised below. The rest of this section looks at weaknesses in the methods used and shortfalls in our knowledge, leading onto recommendations for future research where it is expected that the greatest benefits can be obtained.

Coastal defence structures are commonly constructed because of coastal erosion. This erosion will continue, despite the presence of the seawall. The seawall neither adds nor removes sand, although it does impound or imprison it, preventing it from entering the coastal sediment transport system. Seawalls can cause local toe scour during storms, but there is no evidence that coastal defence structures delay the recovery of beaches. Beach lowering is a process that takes place on a number of different timescales (years, seasons, storms) and which combines cross-shore and longshore sediment transport. This report concentrates mainly on toe scour, which is the short term lowering of beach level close in front of a coastal defence structure. The overall lowering of beach levels, which occurs at longer timescales and over larger spatial scales, is referred to as general beach lowering. Most beach lowering studies have considered toe scour only and have treated it as a purely cross-shore transport phenomenon.

Toe scour is blamed for the failure of many coastal structures (Section 3.7.2) but toe scour holes are infrequently observed in the field. There are indications that it may be a short-lived phenomenon, with scour holes generated during storms filling in within a few hours as the storm subsides. This would explain why few scour holes are observed or surveyed at low tide. Toe scour has been reproduced in several small-scale laboratory experiments (Section 3.7.3) which have treated it as a wave-driven, cross-shore, often bedload transport dominated phenomenon. There have been few laboratory toe scour tests that have generated suspended sediment transport despite the fact that (Section 3.3) bedload and suspended load scour occur by different mechanisms and occur in different places. It is therefore questionable whether small-scale bedload transport experiments provide reliable design guidance on toe scour depths at full scale for those situations where suspended load transport is significant (Tørum *et al*, 2003).

Moreover, even if the empirical equations derived from laboratory tests are taken as reliable, there is still no design equation for the following cases (Table 2):

- Non-breaking oblique incidence waves at vertical walls;

- Non-breaking oblique incidence waves at sloping impermeable walls;
- Non-breaking oblique incidence waves at permeable sloping walls (such as rubble mound breakwaters);
- Breaking oblique incidence waves at vertical walls;
- Breaking oblique incidence waves at sloping impermeable walls;
- Breaking oblique incidence waves at permeable sloping walls (such as rubble mound breakwaters).

In most of these cases toe scour can be estimated from normal-incidence vertical wall cases, either by taking this as the likely worst case or by adjusting it according to rules-of-thumb. In other cases, such as breaking waves at normal incidence on a sloping impermeable wall the only design guidance comes in the form of rules-of-thumb. There is little evidence of these formulae being used in the design of coastal structures or in the design of mitigation measures, indicating a lack of belief in these methods amongst designers. For assessment of complex designs it may be more important to simulate 3D hydraulic processes in a physical model and as well to estimate the scour development, even if there are some uncertainties in the scour scaling.

The (physical and numerical) modelling previously carried out has regarded toe scour as a short-term wave-driven phenomenon caused by cross-shore transport of sand (or shingle). Case studies in the UK (Appendix A) have often indicated that longshore transport plays an important and sometimes even dominant role in beach lowering in front of coastal structures. Variations in beach level due to changes in longshore transport tend to have longer timescales (up to the centuries required for coastal realignment) than toe scour.

Various methods have been used to mitigate the effects of beach lowering around the UK coastline (Section 4.3) but there is little guidance on the advantages or disadvantages of such methods. Neither has the performance of such mitigation schemes been well documented.

5.2 Conclusions and recommendations

As a direct outcome of this scoping study a number of key areas for further research into the importance and process of beach lowering in front of coastal structures have been identified under the following headings:

- Policy
- Approaches to different time and space scales
- Development of a probabilistic approach
- New experiments
- Sediment type
- Mitigation measures

The research will benefit from linking to existing monitoring of beaches and structures as well as other Defra/Environment Agency research. Finally, an estimate of the potential cost savings arising from the research is included.

Policy

Designers / coastal managers should be encouraged to state the assumptions made about beach lowering at structures, and to define a minimum beach level for new or existing coastal structures (particularly coastal defences) which should not be reached without mitigation works being undertaken. Monitoring this plays a crucial role in assessing performance, with a proportionate level of attention to analysis, interpretation and reporting.

Further information on the performance of mitigation schemes is needed in order to assess how successful different approaches are. More information could be collected from local authorities and other bodies, including more quantitative information than has been collected for this scoping study. In particular, cross-sections of the structure and the mitigation scheme would be useful, along with beach profiles (ideally before and after) and information on the wave and current climates.

Data collected on a structure should be entered into the National Flood and Coastal Defence Database.

Approaches to different time and space scales

Beach lowering happens on such a wide range of time scales and space scales that the entire process cannot reasonably be modelled in a single numerical, physical or conceptual model. Moreover, not all the processes are understood so a variety of approaches should be taken to address these issues, as discussed below.

Long time-series of data can be used to assess the changes in beach level at and in front of a coastal structure. A good example of a suitable dataset is the EA Anglian Region twice-yearly survey of beach levels that has been collected since 1991. It is now sufficiently long to be used to detect trends in beach level. Such trends, if significant, could reasonably be used to extrapolate forwards a few years to give likely beach levels (if there is no reason to suppose that the inherent processes have changed). Such extrapolations should be updated yearly to include the latest data.

Beach plan shape models can be used to study the effect of changes in longshore transport on the development of a beach contour through time. Although such models do not include scour phenomena they do provide information about the development of a beach level contour in front of the structure. Thus the effects of a seawall or groynes can be represented and the movement of the contour tracked through weeks, seasons and years, and as the wave climate changes over, say, a thirty year simulation. However, the representation of seawalls in these models is very simplistic and the logic behind the representation is open to debate. The representation of seawalls in beach planshape models could be investigated and validated. The natural variability in beach level in front of coastal defence structures could then be modelled. This would be an important step in defining the potential risk to a

structure. The advantages of a planshape model are that they are quick to run so a large number of scenarios can be modelled and they can be used to test what-if scenarios.

A number of different approaches can be taken to estimate how much toe scour may occur. These include:

- Physical modelling, which suffers from scale effects, but can be used to represent complex three-dimensional situations that cannot be addressed using a numerical model or empirical equations;
- Empirical equations, which can be used to calculate scour depths for design cases, or to calculate time series of scour depths based on modelled time series of wave conditions;
- Phase-resolving numerical models of toe scour, which are being developed, but are still at the stage of modelling limited test cases. For example phase-resolving wave models (e.g. Lawrence *et al*, 2003) can model cross-shore bedload transport, while others (e.g. Bacquerizo and Losada, 1998, Mizutani *et al*, 1998) have modelled regular waves reflecting from a porous structure. While the development of such modelling capacity is to be welcomed, these models are not at a sufficiently advanced or validated stage to be used for design guidance. Such models, when more fully developed, may well prove to be valuable design tools and the continued development of such models would be a valuable academic exercise. Due to their computationally intensive nature, the main use of such models in the next few years will be to test a limited number of design cases; and
- Beach profile models can be run to determine cross-shore scour. These models are phase-averaged wave models and do not resolve the phase of the waves. Typically these use a representative wave height, treat wave reflection in a simple manner and do not allow interactions between the incident and reflected waves. They are generally used as storm response models so are tuned to represent offshore transport during storms rather than beach recovery. They should therefore be run for short-term event simulations rather than for long time series of waves.

Development of a probabilistic approach

A number of different approaches are suitable for addressing different aspects of beach lowering, as shown above. They cannot all be reasonably included in a deterministic model, so the first steps should be taken towards the development of a probabilistic risk-based method of assessing the safety of coastal defence structures. This would combine information on:

- the long-term trend in beach levels
- the variability in beach levels due to longshore transport
- the variability in beach levels due to toe scour
- fragility curves for the structure.

Such an approach would be developed to give information on the range of possible beach levels to be expected in different scenarios. This type of

information could be expressed in the form of a warning trigger level (or levels with different degrees of risk). Remedial action would be taken should the beach level drop below the trigger level.

In order to calculate a fragility curve for a coastal defence structure, more work is needed to link the scour process to the risk of structural failure. Scour deepens the water depth immediately in front of a structure. This may allow higher waves to reach the structure, which may in turn cause greater damage to the front face and toe and may cause a greater volume of water to overtop the coastal defence. This increases the risk to pedestrians and property. It may also threaten the stability of the rear face of certain types of coastal defence (such as embankments) and provide an alternative failure mode to the collapse or unravelling of the toe.

In the case of a seawall with sheet piling, it has often been assumed that failure will occur when the scour level drops below a particular point. In practice, there will be a varying probability of failure over a range of beach levels. The case is more complicated for, say, a revetment with a Dutch toe. Here, changes in beach level may cause the collapse of the toe, which may armour the scour hole, thus reducing the rate of scouring. Unfortunately it is not possible to say exactly how the development of scouring will affect the performance of the structure. More work needs to be done in the first instance, to develop fragility curves for generic structure types.

The above cases show that there is a dynamic interaction between a beach and a coastal defence structure. Relatively few studies have considered the interactions between beaches and structures (preferring to study one or the other). More tests need to be done with mobile beaches and structures. Areas to consider using physical and numerical modelling are:

- Flow through structures and beaches;
- Suffusion and material retention;
- Foundation support, including settlement and liquefaction.

New experiments

There is a shortage of large-scale laboratory and field data on toe scour. Moreover, there is some doubt over whether much of the small-scale experimental data can be applied at full scale. The advantages of large-scale laboratory data are that they are collected under controlled conditions where measurements can be made at any time. Moreover the tests are repeatable. The main disadvantages are that the tests suffer from scaling problems and normally only represent cross-shore wave-driven processes. The advantage of field data is the absence of scale effects. The disadvantages are that the conditions are unpredictable, the tests may not therefore be repeatable, and it is difficult to measure *in situ* during a storm, when the worst scour may occur.

Therefore, a combination of medium to large scale laboratory data and field data should be collected to fill gaps in existing knowledge. The purposes of large-scale laboratory tests would be:

- To check the ability to scale small-scale scour tests to a larger scale
- To provide additional data to evaluate empirical equations for scour depth
- To provide validation data for numerical models
- To test the effectiveness of various toe designs and mitigation measures.

The purpose of field measurements of scour immediately in front of a coastal defence structure would be:

- To collect data at full scale (thus avoiding scale effects) to check empirical equations and the ability to scale small-scale and large-scale laboratory scour tests to full scale
- To measure beach levels during storm events to determine the maximum scour depth and the speed of scouring and filling in as the storm rises and falls.

One or two demonstration experiments should be set up to monitor scour in the field. Field monitoring should occur over a prolonged period, of at least 2 spring-neap-spring cycles per deployment and (preferably 6 or more) and there should be deployments in at least 2 years. This will ensure that a range of conditions can be monitored and lessons learned in the first year can be applied in the second. Equipment to monitor scour depths should be employed continuously throughout this period, even during storms. The equipment could comprise HR Wallingford “Tell Tail” scour monitors as deployed at Blackpool and Teignmouth (see Appendix 3).

Any experiment devised should be tied in with existing survey programmes to minimise the cost and to provide the wider picture. In this way, data on waves and water levels could be obtained from the survey programme as could data on wind speeds, and currents (if collected). Moreover, existing monitoring programmes are likely to already have beach surveys of the site and may be able to conduct beach surveys at the field experiment site(s) immediately after a storm. In addition, they will have staff on-site periodically and may be able to download the data or at least check on the state of the experiment. This would bring numerous advantages to a field measurement campaign and would help to give the monitoring programme a greater insight into the processes affecting the coastal defence structures in their area.

There should also be some consolidation of recent EC funded research to determine the relative importance of hydraulic induced scour and pressure induced liquefaction on bed levels/properties at the toe of coastal defences.

Sediment type

Most studies have concentrated on sand beaches and impermeable structures. More work is needed to determine the relationship between scour depth and sediment type. In the UK there is a need to be able to predict the response of mixed and shingle beaches as well as fine sand beaches. Recently funded Defra research on mixed (sand and shingle) beaches (e.g. Blanco *et al*, 2003) has provided much improved understanding of the morphological response of those types of beach. However, determining the behaviour of mixed beaches at

a seawall is a larger area than can be covered by a single project. The greatest benefit would flow from concentrating on sand and shingle beaches while monitoring further improved understanding of mixed beaches and uptake of the recently completed research into mixed beaches.

Mitigation Measures

The approach to mitigation schemes is often practical and empirical. The performance of a selection of mitigation schemes should be examined in more detail. It would be useful if this work could also be tied into an ongoing survey programme to minimise the cost (in the same way as proposed for the monitoring of scour in the field and ideally within the same survey area).

5.3 Potential cost savings

Cost savings could potentially be delivered should research show that less conservative designs for coastal structures would be appropriate. Burgess (2003) compiled the following approximate potential cost savings for each kilometre of coastal defence:

- Reducing toe depth by 0.5m could save £50,000
- Reducing crest level by 0.2m could save £50,000
- Reducing armour thickness by 0.2m could save £100,000
- Steepening structure slopes from 1 in 2.0 to 1 in 1.5 could save £150,000.

There are therefore considerable potential longterm returns from investment in beach lowering research, and the dissemination of new knowledge and guidance. In order to safely minimise the cost of coastal defence structures, the minimum safe size of toe protection and the minimum safe depth of toe excavation should be investigated.

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