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

Beach lowering in front of coastal structures

Appendices 1-3

R&D Project Record FD1916/PR



Environment Agency

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This also constitutes HR Wallingford Report SR 633

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Appendix 1:Case Histories

CASE HISTORY 1- Prestatyn, North Wales

Background history
The coastline between Rhyl and Prestatyn has been eroding for several decades, through a combination of reduced sediment supply and coastal squeeze. At the end of the nineteenth century some two million tonnes of gravel were removed from the beaches in the Rhyl area (updrift and to the west of Prestatyn) to provide ballast for building Liverpool Docks. As a result, the pebble storm beaches that once extended along much of this frontage have largely disappeared. Tourist pressures have also led to the reclamation of large stretches of marshland for the construction of holiday camps, golf courses etc. The holiday camp and housing at Prestatyn are situated close to the shoreline and are protected by a seawall. To the east of this wall there is a line of dunes that have been eroding.

Prestatyn was first protected by a stepped concrete seawall in 1960. At the same time the foreshore was protected by a series of long timber groynes. Already by the 1970’s the beach in front of this wall had fallen significantly. The flatter gradient allowed the ridge and runnel systems, common on this wide foreshore, to migrate shoreward. The increased water depth at the toe of the wall then caused strong overtopping. The wall itself was at risk of foundation failure. The sand transport was then concentrated at some distance seawards of the wall, effectively starving the dunes immediately downdrift of the sand supply.

The photograph below, taken in early 1990, shows the tidal runnel very close to the wall. Note that the runnel extends a considerable distance alongshore, cutting across several timber groynes. All that remains at the toe of the wall is a narrow strip of pebbles. Some emergency works in the form of a fillet of rock armour-stone can be seen at the toe of the wall. The growth of algae on the concrete steps indicates frequent wave overtopping.

Toe scour in front of stepped concrete seawall, Prestatyn

Mitigation measures
This frontage has required regular maintenance. In the 1980's, field investigations were carried out into the problems of wave and tidal induced scour and the strength of tidal currents over the foreshore. Rock groynes were constructed to reduce inshore tidal currents. This improved beach levels over the foreshore but the wall toe remained vulnerable to wave attack. Following substantial damage in 1990/1 a major scheme was implemented, including massive sand recharge coupled with the construction/ upgrading of the rock groynes. In addition, the vertical face of the seawall below the concrete stepped face was replaced by an asphaltic sloping apron.

Performance of mitigation measures
The increased height and width of the upper foreshore has resulted in the disappearance of the ridge and runnel features from in front of the seawall. The high foreshore levels have also removed the problems of wave overtopping. The photograph below shows the swash limit along the line of the new sloping revetment. There is some sand build up above the revetment and on the seawall steps. The amenity value of the promenade, at the crest of the wall, has also been greatly improved.

Sloping asphalt apron and rock groynes in front of steeped concrete seawall, Prestatyn

Other comments
A scheme of this type needs to be monitored carefully. Not only do beach levels in front of the seawall need to be checked regularly, but the evolution of the downdrift beaches must also be assessed.

CASE HISTORY 2 – Colwyn Bay, North Wales

Background history
Colwyn Bay is a popular tourist resort on the North Wales coast. The sand beaches in this area have been eroding as a result of coastal squeeze and a lack of sediment supply from the west (updrift). The reasons for this are also described in the case histories for Penrhyn Bay and Rhos-on-Sea. Due to the fallen beach levels, the promenade and the road immediately to the landward, have been affected by heavy wave overtopping.

The construction of sea defences within Colwyn Bay dates back to nineteenth century. The masonry seawall has, over the years, suffered considerable damage, requiring extensive repairs and reconstruction. In the 1970's there was a groyned upper beach of shingle, with an almost continuous sand cover over the flatter, lower part of the beach. Subsequently the groyne system fell into disrepair, allowing beach levels at the wall toe to fall. Thus, by the 1980's the shingle beach had largely disappeared from the wall toe, causing foundation problems (see photograph below). In addition, the falling sand levels had exposed the underlying pebbles over much of the lower foreshore.


Beach lowering in front of vertical seawall, Colwyn Bay

Mitigation measures
In 1987 a rock berm was constructed along the most severely affected stretch of wall. This encouraged some beach build up in the immediate vicinity of the wall toe, assisting seawall stability. More recently, a number of low rock groynes were constructed from the wall out to the low water line. These project no more than 1m above the beach surface, hence are not visually intrusive. Other than that, little other palliative action has been required.

Performance of mitigation measures
The Rhos-on-Sea breakwater may be responsible for trapping what little shingle drift there is, in its lee. Therefore, the construction of the rock berm has encouraged sand rather than shingle accretion. Further seaward, the sand cover has increased significantly, so that only small areas have the underlying pebbles exposed.

This scheme is a good example of how relatively modest defences can be used effectively to improve beach levels. The photograph below, taken in 2002, shows the sand build up at the foot of the seawall. The long rock groynes can (just) be seen in the background, while a redundant timber groyne can be seen in the middle distance.

Rock berm in front of vertical seawall, Colwyn Bay

CASE HISTORY 3 – Rhos-on-Sea, North Wales

Background history
Rhos-on-Sea is situated on a small headland at the eastern end of Penrhyn Bay. The headland is a focus point for wave action and the residential development on low-lying land to the landward was at risk from heavy wave overtopping in the recent past.

As described in the Penrhyn Bay case history, there is a shortfall in the supply of beach sediments in Penrhyn Bay and to the east. The headland of the Little Orme acts as an efficient groyne, cutting off any potential supply of shingle from the west, as well as seriously reducing the amount of sand transport. By contrast, the small promontory of Rhos Point has not prevented beach material from being transported eastwards (downdrift) into Colwyn Bay.

The construction of sea defences within Penrhyn Bay in the late nineteenth and twentieth centuries had cut off the supply of sediments from the erosion of boulder clay cliffs at the western end of the bay. This left only a small supply of sand from the nearshore zone, feeding around the Little Orme headland in suspension. Beach levels had therefore gradually deteriorated both within Penrhyn Bay and around Rhos Point itself.

The seawall around Rhos Point was built in the 1860's and, prior to the breakwater protection scheme described here, had been breached and repaired in the recent past. Further falls in beach levels were anticipated, similar to the progressive deterioration of the beaches that had taken place earlier in Penrhyn Bay. In view of the falling beach levels, upgrading the existing sea defences, for example, was considered to be insufficient as a long-term solution to the problems that had developed around Rhos Point.

View to the east showing usage by small boats

Mitigation measures
Following wave overtopping studies at HR Wallingford a rock armour breakwater was constructed off Rhos Point in 1983. This was located opposite low-lying land to the south of the Point. Rock left over from the breakwater construction was used to construct a short groyne on the coast immediately to the north, to encourage material to collect around the Point itself.

View to the west showing beach recharge

Performance of mitigation measures
The breakwater has eliminated the problems of wave overtopping that were becoming increasingly more serious to the south of Rhos Point. The sheltered conditions in the lee of the breakwater have allowed small boats to anchor there (see the first photograph below). This has been possible because the breakwater is sited some distance away from the wall, but not so far offshore that its sheltering effect would be significantly reduced. The low groyne has been overtopped by beach material and shingle and sand have tended to collect in the lee of the breakwater, further reducing any potential risk of wave overtopping (see the second photograph below).

In view of the 7m tidal range on spring tides the offshore breakwater is a large structure. As well as trapping the small volume of littoral drift from Penrhyn Bay, the breakwater has also attracted a reverse westerly drift of material from Colwyn Bay to the east. In addition, the high degree of shelter has attracted a small amount of mud from offshore. The accumulation has not affected the development of sailing leisure facilities in the lee of the structure.

CASE HISTORY 4 – Penrhyn Bay, North Wales

Background history
Penrhyn Bay is situated to the east (downdrift) of the headland of the Little Orme. The problems of beach erosion in Penrhyn Bay are primarily due to a lack of contemporary sediment supply. The main source of beach material for the bay has been the erosion of boulder clay cliffs on the east side of the Little Orme and a (potential) feed from the nearshore seabed.

Since the 19th century the construction of sea defences has progressively cut off the supply of beach material derived from the cliff erosion. The seawalls themselves, which date back to the 1860's, have contributed to coastal squeeze, causing further deterioration of beach levels. By the 1970's the conditions had deteriorated to such an extent that the beach material had largely disappeared from the eastern (downdrift) part of Penrhyn Bay. Even in the more sheltered western part of the bay there was little beach material remaining. As a result, the groynes were no longer effective and were allowed to deteriorate, as shown in the photograph below. The lack of beach cover and the serious overtopping in the exposed central frontage resulted in the construction of timber breastwork to the seaward of the existing wall, to reduce wave overtopping. These measures did not deal with the root cause of the problem, which was the lack of sediment supply.

In the last twenty years, more effective forms of construction have been implemented, including offshore breakwaters at Rhos-on-Sea, rock groynes and artificial beach nourishment and rock armour groynes and sills in Colwyn Bay (see other case histories).

Lowered beach exposing shore platform, Penrhyn Bay

Mitigation measures
Although partly sheltered from the west the refraction/diffraction around the headland of the Little Orme and the edge-wave effects, as waves propagate alongshore, along the line of the seawalls, produces a significant eastward littoral drift within Penrhyn Bay. Since Pernrhyn Bay is not fully enclosed at its eastern end, this has produced a gradual emptying of sediments out of the bay.

The seawall in Penrhyn Bay, which was badly overtopped during storms, is now protected by a wide cobble beach maintained in place by two fishtail-type rock groynes. On the other hand, the downdrift frontage to the east, where beach lowering might have occurred, is protected by a rock revetment. Use has been made of locally available quarried rock and marine-won sand and gravel in the artificial nourishment.

Performance of mitigation measures
This innovative scheme has been very successful. The combination of the artificial beach recharge and the fishtail groynes has effectively reduced the erosion of the upper beach and contained the beach sediments within Penrhyn Bay. The artificially formed beach has a sufficiently high crest to prevent waves reaching the seawall (see photograph below).

The beach was graded from sand in the west to an (artificial) cobble beach formed of quarried rock in the east. Although the sand has tended to stay in the (sheltered) western corner of the bay some smaller rock fragments have migrated there. In addition, the cobble storm ridge occasionally extends over the trunk of the rock groyne, requiring occasional recycling.

Other comments
A scheme of this type alters the character of the beach significantly, at least until the rock fragments forming the cobbles have become rounded down by wave action. In doing so, the material becomes more mobile. This type of scheme therefore has to be monitored regularly. Material may also have to be recycled to maintain the necessary standard of defence (the large rock groynes are likely to attract material and in so doing, it is possible for beaches away from the groynes to be reduced in size).


View of beach recharge in western part of Penrhyn Bay

CASE HISTORY 5 – Sandbanks Peninsula, Poole, Dorset

Background history
The Sandbanks peninsula is a long, heavily developed sandy peninsula, situated immediately to the east of the entrance to Poole Harbour. The flow patterns here are complex. There are rapid tidal currents in and out of the Harbour. There is also a subsidiary inshore channel, called the East Looe Channel, which allows fast tidal currents to flow parallel to the peninsula and close inshore. To the seaward of this channel there is a sandbank whose form changes in response to the wave climate, as well as these complex tidal flows. The resulting sediment transport in this area is thus extremely complex. Sand can be transported alongshore by breaking wave action as well as by the tidal currents. As a result of these processes, the direction and magnitude of longshore sediment transport is temporally and spatially very variable. In addition, there is intermittent onshore movement of sand from Hook Sand, by swell wave action.

The sand beaches along the Sandbanks peninsula were originally protected by a system of crib type rock groynes. Historic charts show these maintained high beach levels over much of the frontage. However, these groynes fell into disrepair, being removed in 1991 for health and safety reasons. Beach lowering was noted subsequently, becoming most serious at the western end of the frontage, where a seawall surrounds the head of the peninsula. The offshore transport of sand, due to waves being reflected from the wall and its subsequent removal by the fast tidal currents, caused concerns that the wall would become undermined.

Vertical wall and low beach near Haven Hotel, Sandbanks (c1998)

Mitigation measures
In 1991 a rock groyne was constructed near the western end of Sandbanks. In 1994 a rock fillet was constructed at the base of the wall. Following the completion of a beach management strategy for the entire Poole frontage in 1994/5, HR Wallingford was commissioned to produce an outline design for a coast protection scheme for the western end of Sandbanks. This included the modelling of a groyne scheme to reduce flows over the beach in front of the seawall. The optimum plan shape that was derived ensured that the flows of the East Loe Channel were deflected away from the seawall.

The scheme was constructed in 1995/6. Overall, this was a great success, with the earlier beach erosion being reversed and sand dunes forming where there were previously low beach levels.

The rock groynes have subsequently been extended southwards in the second phase of the works. These have also been very successful so that virtually the whole of the Sandbanks frontage now has a high level of protection.

Widened beach after installation of sill and rock groynes, Sandbanks (2002)

Performance of mitigation measure
Before the scheme was implemented there had been an increase in water depths within the East Looe Channel and an onshore movement towards the line of the seawall. When the scheme was built the tidal currents were deflected away from the wall, enabling sand to settle out of suspension.

This scheme demonstrates how the role of tidal currents on beach lowering should not be overlooked, especially near estuary and inlet mouths.

CASE HISTORY 6 - Seaford, East Sussex

Background history
Now a resort, Seaford was a port before a great storm in 1579 caused the build up of shingle to block off the entrance channel, and diverted the course of the river Ouse westwards in the direction of Newhaven.

The subsequent development of the port of Newhaven has included the construction of training walls to prevent the littoral from blocking up the new entrance. Successive extensions of the western training wall were necessary as the beach to the west of the entrance accreted. This meant that the shingle beach at Seaford has received a dwindling supply of shingle. This has caused beach levels in front of the seawall to fall at an increasing rate during the last century. By 1980, the beach in the eastern half of the frontage had fallen to such an extent that it was providing very little support to the old mass concrete seawall. (This wall was originally a secondary defence behind a then substantial shingle ridge.) In places erosion had exposed the underlying chalk platform, allowing waves up to 6m high to reach the wall without breaking. In 1981, parts of the wall had become badly damaged and in 1985, undermining had caused local collapse of the promenade.

Low beach levels and damaged concrete seawall, Seaford (c1982)

Mitigation measures
The local water authority were aware that reconstructing the seawall would be difficult to justify economically, and that such a course of action would be unsustainable, with the likely continued fall in beach levels. Following hydraulic and numerical model testing at HR Wallingford, an open beach (ungroyned) nourishment scheme was adopted as the most economical solution to the problem of overtopping and continuing deterioration of the seawall. Due to its south-westerly aspect the shingle beach at Seaford is aligned almost perpendicularly to the predominant direction of approaching south-westerly waves. Because of the relatively low rate of littoral drift generated by these obliquely incident waves at the western end of the nourished frontage it was determined that a terminal groyne was not necessary there. At the eastern end of the frontage a large concrete groyne was already in place, but this was reconstructed to a greater height and length, so as to prevent loss of material to the natural cliffed (and undeveloped) coastline to the east.

In 1987, the central and eastern end of Seaford was nourished with 1.5 million cubic metres of shingle won from offshore. The material was won by using a trailer-suction dredger, extracting the material from an existing licence area on the Owers Bank, south of Selsey Bill. The material was spread over a 2.5km frontage. The western frontage was left untouched, as the beach there was already wide.

Performance of mitigation measure
Following the initial period of adjustment the beach actually increased in volume within the active beach profile (taken as above –4m AoD). This is because the wave reflectivity was significantly reduced, causing pebbles to migrate landward from the hard chalk seabed on which material was very mobile.

Monitoring has been critical to the long-term viability of the scheme. In order to maintain sufficient beach width at all points on the frontage, recycling needs to be carried. Modelling indicated that the average annual recycling volume was likely to between 20,000 and 25000 cubic metres per annum. The volume that has had to be recycled has, in fact, varied considerably from year to year, and the average value has been considerably higher than anticipated. Nevertheless, the scheme has successfully protected the ageing seawall from wave attack and stopped the heavy wave overtopping that used to take place.

Possible improvement measures
It would appear that a nourished fill is considerably more mobile than the native beach material. The reasons for this are not particularly well understood, but it is considered likely that shingle transport on what is a relatively steep beach, may be enhanced by tidal current action. Certainly there is evidence in the form of shingle waves, showing enhanced mobility. This type of response has been observed in a number of other beach nourishment schemes involving shingle.

The disadvantages of beach nourishment are that it is difficult to predict the expected life-span of beach nourishment material, in view of the unpredictability of the UK wave climate. There may also be difficulty in obtaining the right grade of material, as offshore dredging operations are dependent upon a licence being available. Massive nourishment schemes, particularly where they involve recycling, also have an adverse impact on the usage of the beach. This can be minimised by targeting the recycling operations so as to avoid holiday periods etc.

CASE HISTORY 7 - Selsey Bill, West Sussex

Background history

Selsey Bill is situated at the southern tip of a low-lying headland that juts out into the English Channel. The coastline is formed of Bracklesham Clays, which are overlain by gravel deposits at Selsey Bill. These gravels form low cliffs that are easily eroded. Due to its open aspect the Bill is a focus for wave energy. It is also a drift divide, the cliff erosion having provided material for the development of the beaches east and west of the Bill. In addition, marine sediments, principally coarse sands and gravel, are driven onshore during storms, from barely submerged banks lying off the Bill. This movement takes place in pulse fashion, so that the thickness of the shingle cover at any location varies greatly with time. The geomorphology of the area is thus very complex and ever changing.

During the early part of the 1900's the coastline west of the Bill had undergone continued long-term recession, which reached an annual rate of the order of 6m per year. The erosion of the sandy clays and gravel provided large drift along the frontage to the west, providing sediments for the East Head spit at the western side of the entrance to Chichester Harbour, as well as for the ebb bar across the entrance. However, had erosion continued much of the shorefront development would have been lost (some had already been lost before the scheme was implemented).

Sea defences were begun in the 1950's. With an onshore supply near the Bill the beaches did not deteriorate as rapidly as might have been expected, in view of the earlier very rapid rates of retreat. However, by the late 1980's the walls had deteriorated through continuous wave action and falling beach levels. The photograph below shows the situation in 1988.

Low beach levels in front of stepped concrete wall, Selsey Bill (1988)

Mitigation measures
By the early 1990's, the stepped concrete wall at Selsey West Beach was in danger of being undermined. In addition, there was heavy wave overtopping on virtually every high tide, resulting in damage to developments on the immediate backshore.

In 1992 a major scheme was underway along much of the Selsey frontage. At Selsey West Beach, the seawall was reconstructed at strategic locations where heavy overtopping could not be tolerated. In other areas, the wall was strengthened. In places the wall was extended by the addition of concrete armour units and a rock berm, as shown in the photograph below. In addition, some shingle was added to the upper beach, to help fill the groyne compartments.

Performance of mitigation measures
This is a very exposed location and it is not possible to maintain a shingle beach in front of the seawall permanently (the photograph below shows the face of a groyne, against which the shingle beach has recently been drawn down). Shingle beach levels continue to fluctuate strongly from season to season. The level of the lower sandy foreshore appears to have been maintained.

While the problems of overtopping along this frontage have not been eliminated, the volume and frequency of overtopping has been significantly reduced. In addition, the stability of the toe of the wall has been secured.

Toe berm of rock and concrete armour units, Selsey Bill (1994)

CASE HISTORY 8 - Sidmouth, Devon

Background history
Sidmouth is situated at the mouth of a river valley that is flanked by cliffs of red marl. The net littoral drift is from west to east, pinching in the river Sid against the cliffs on the eastern end of the valley. (Sidmouth was a port, which became unusable when the river entrance was infilled with shingle). The shingle beach is formed of material derived from the erosion of the marl cliffs. These cliffs are largely unprotected, so that the supply has been maintained. Over the town frontage, the shingle beach is backed by a seawall, and is therefore vulnerable when waves are able to reach the wall.

Apart from changes due to fluctuations in the rate of west to east littoral drift, the shingle beach in front of the seawall is affected by draw-down during severe storms, leaving the wall exposed to wave attack. In the early 1990's, a severe storm caused serious beach lowering and major wave overtopping over the town frontage. After this storm, the beach did not recover its former levels.

Mitigation measures
Following model testing at HR Wallingford a beach nourishment scheme was implemented using local beach material, together with the construction of two offshore breakwaters and a groyne at the eastern (downdrift) end. The purpose of the two breakwaters at the western end of the frontage is to protect the town frontage against the predominant westerly storms. The groyne is there to prevent material from being transported out of the area by the net west to east drift.

Beach recharge, groyne and offshore breakwaters, Sidmouth

Performance of mitigation measures
The scheme has eliminated beach draw-down and overtopping during westerly storms.

A succession of storms from the east caused shingle to migrate into the lee of the breakwaters, reducing the beach width at the eastern end of the frontage. This was remedied by constructing an additional rock groyne to reduce littoral drift from the east. Since the second groyne was added the beach has maintained an adequate width over the whole frontage.

The two photographs show the western and eastern ends of the nourished frontage. The improvement in beach width has not only effectively reduced the former problems of beach lowering but has also provided a more attractive beach. In addition, the offset breakwaters are also not visually intrusive, being below the horizon at promenade level.

Rock groynes and beach recharge, eastern end of promenade, Sidmouth

CASE HISTORY 9 - Portobello Beach, Edinburgh

Background history
In the United Kingdom, the justification for many beach nourishment schemes has been that the cost of nourishment is considerably less than the cost of reinstatement of existing hard defences. One of the first schemes to be justified on this basis was carried out at Portobello Beach, Edinburgh. This beach is situated on the western side of Edinburgh and faces directly into the mouth of the Firth of Forth. It became denuded as a result of sand abstraction for the glass industry, which began in the 19th century and continued up to the 1930's. The promenade seawall at Portobello dates back to 1860. By the late 1950s, the lowered beach meant that the wall was under continuous wave attack, with resulting frequent serious overtopping (see plate below). By this time the beach had flattened and the median sand grain size was 0.2mm.

Portobello Beach, Edinburgh (c1970)

Mitigation measures
Following studies by HR Wallingford the beach was nourished in 1972. The sand nourishment material had a median size of 0.27mm, considerably coarser than the beach material. The sand was obtained from a sub-tidal borrow area some 3km east of Portobello, in a sheltered part of the Firth of Forth. This material is so close to Portobello that it may well be the natural sand size for the area. (The beach at Portobello had become so eroded that the sand was no longer representative of the beach material under healthier conditions.)

Some 180,000 cubic metres of coarse sand was extracted from a borrow area by bucket dredger, transported by barge, and pumped over a 1.6km frontage to a foreshore gradient of 1 in 20. The material was placed over a depleted beach whose gradient had fallen as a result of beach lowering to about 1 in 42. The nourished beach was held in place by a number of timber groynes, with an (easily adjustable) gabion-type groyne at the eastern (updrift) end of the frontage.

Post-nourishment view at Portobello

Performance of mitigation measure
From the start, it was recognised that careful monitoring was crucial to the long-term success of the scheme. The beach was monitored in the early years of the scheme by HR Wallingford and then by the local Coast Protection Authority (first Lothian Regional Council, then City of Edinburgh Council).

The beach profile surveys show that after 18 months the beach slope had adjusted to 1 in 23, but other than the seaward movement of the toe of the nourished beach there were no significant offshore losses of beach material. Littoral drift in this area is low so that end losses are small.

Beach volumes remained relatively unchanged until 1978. By 1981 the losses had increased to about 50% of the original nourishment volume, as a result of severe storms. Following calmer weather, the beach recovered, so that in 1984 there was still some 75% of the renourishment volume remaining above the low water mark. By 1988 the nourishment volume had reduced to 70% of the placed volume. In late 1988 a further 102,000 cubic metres of sand were added as a topping up and improvement operation. The trend of beach erosion after storms and subsequent recovery has continued since. Despite a trend of gradually declining beach volume the beach remains above its pre-1988 renourishment level (HR Wallingford, 2002).

Possible improvement measures
This scheme has been so successful that no significant improvements to the mitigation technique employed can be envisaged. The beach has a low littoral

drift and the swell waves penetrating through the mouth of the Firth of Forth almost balance the destructive action of locally generated, hence short period and destructive waves.
Had finances been available then the scheme might have been extended westwards over the partly industrial frontage to Leith Docks.

Appendix 2

Scour depth design formulations

The key points of 10 approaches have been summarised in this appendix. For further information the reader is referred to the original reference.

Xie (1981, 1985) / Sumer and Fredsøe (2000)
Hughes and Fowler (1991)
O'Donaghue (2001)
Herbich and Ko (1968)
Song and Schiller (1973)
Fowler (1992)
Jones (1975)
Powell and Lowe (1994)
Powell and Whitehouse (1998)
McDougal, Kraus and Ajiwibowo (1996)

XIE (1981, 1985) / SUMER AND FREDSØE (2000)

Valid for:

- Non-breaking waves – fine sand suspension mode and coarse sand non-suspension mode
- Normally incident
- Regular waves
- Vertical or sloping walls
- Flat bed in front of structure
- Sand

Limits of applicability:

- Suspended mode for $\frac{(U_{max} - U_{*})}{w} \geq 16.5$;
- Non-suspension mode for $\frac{(U_{max} - U_{*})}{w} < 16.5$;

where U_{max} is the maximum horizontal velocity at the bed, U_{*} is the critical velocity for incipient motion and w is the sediment fall velocity.

Design relationship: $\frac{S_{\max}}{H} = \frac{C}{[\sinh(kh)]^{1.35}}$

C=0.4 for fine sand suspension mode

C=0.3 for coarse sand non-suspension mode

C=0.3-1.77exp(-α/15) for sloping breakwaters (Sumer and Fredsøe, 2000) for coarse sand

where:

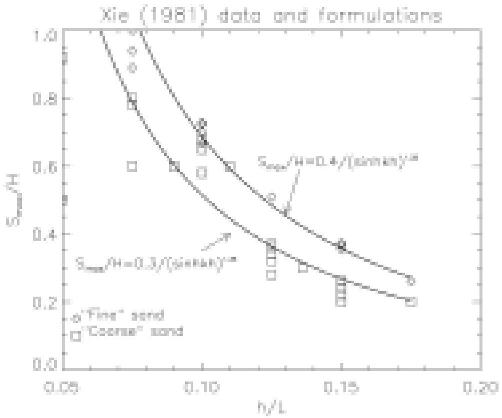
k: incident regular wave number (=2π/L)

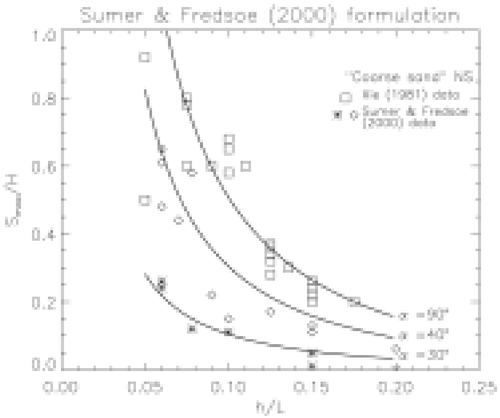
H: incident regular wave height

S_{max}: maximum scour depth at the node (a distance L/4 from wall)

h: water depth

α: breakwater slope in degrees above horizontal: 30º≤α≤90º





Experimental procedure:

- Small flume (38mx0.8mx0.6m) and large flume (46mx0.8mx1.0m)
- Regular waves and 3 cases of irregular waves. H=0.05 to 0.09m, T=1.17 to 3.56s, H/L=0.083 to 0.375.
- Four different grain sizes: 0.106, 0.15, 0.20 and 0.78mm

Other remarks

The design relationship is the same for prototype and model dimensions. Similitude between model and prototype is achieved if $\frac{(U_{max} - U_*)}{w}$ remains the same for both.

Xie S-L (1981) Scouring patterns in front of vertical breakwaters and their influence on the stability of the foundations of the breakwaters. Department of Civil Engineering, Delft University of Technology.
Xie S-L (1985) Scouring patterns in front of vertical breakwaters. Acta Oceanologica Sinica, vol4, n1; 153-164.
Sumer B M and Fredsøe J (2000) Experimental study of 2D scour and its protection at a rubble-mound breakwater. Coastal Engineering 40; 59-87

HUGHES AND FOWLER (1991)

Valid for:

- Non-breaking waves – fine sand suspension mode
- Normally incident
- Irregular waves
- Vertical walls
- Flat bed in front of structure
- Sand

Design relationship: $$\frac{S_{\max}}{T_p (u_{rms})_m} = \frac{0.05}{[\sinh(kh)]^{0.35}}$$

where:

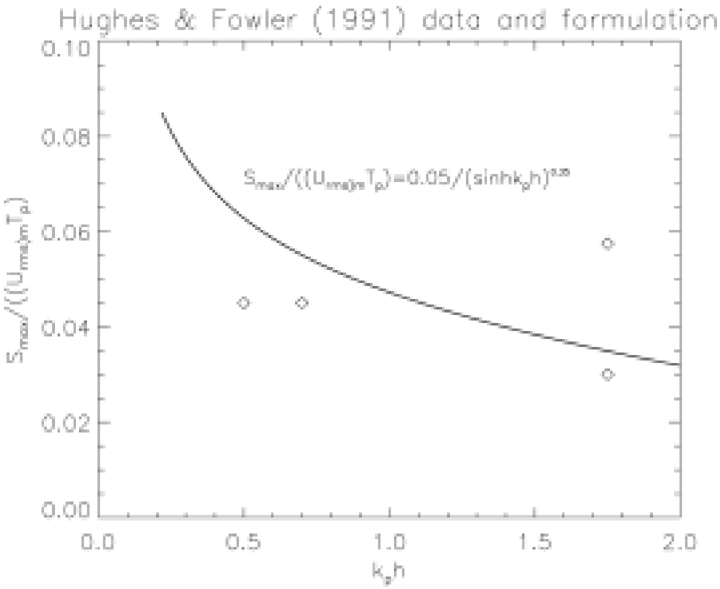
k: incident wave number (=2π/L)

T_p: peak period

S_{max}: maximum scour depth at the node (a distance L/4 from wall)

h: water depth

$(U_{rms})_m$: maximum rms velocity at the bottom. They give an empirical formulation: $$\frac{(u_{rms})_m}{g k_p T_p H_{m0}} = \frac{\sqrt{2}}{4\pi \cosh(k_p h)}\left[0.54 \cosh\left(\frac{1.5 - k_p h}{2.8}\right)\right]$$ valid for $0.05 < k_p h < 3$ where k_p is wave number (linear theory) associated with the frequency of the spectral peak at a given water depth and H_{m0} is the zeroth moment wave height defined as 4 times the standard deviation of the sea surface elevation.



Erosion and deposition of sediment coincide with the high and low values of bottom u_{rms}, at least for the portions of the profile closest to the wall.

Experimental procedure:

- Physical experimental study (typical conditions H_{m0} = 0.09m, T_p = 1.26s, h = 0.3m);
- Horizontal bed from the wall;
- Irregular waves in bursts of 500-800s until scour profile reached equilibrium (approx.6000 waves);
- Sand with mean diameter of 0.13mm (calculated fall speed of 1.64cm/s).

Other remarks

It is a modification from Xie (1981, 1985) formula for regular waves, so that irregular waves scour to maximum depths that are about 55% of that predicted for regular waves.

Hughes S A and Fowler J E (1991) Wave-induced scour prediction at vertical walls. Proc.Coastal sediments, vol2; 1886-1900

O'DONOGHUE (2001)

Valid for:

- Non-breaking waves – coarse sand non-suspension mode. Valid for $u_n/\omega<14$, where ω is fall velocity calculated with Hallemeier equations (SPM, 1984) and u_n the maximum near bed velocity occurring at the node and calculated with second-order theory. The parameter u_n/ω was used to discriminate between N-type and L-type, so that N-type response occurs when $u_n/\omega<14$, L-type when $u_n/\omega>19$ (with some uncertainty in between);
- Normally incident;
- Regular waves;
- Vertical walls;
- Sand.

Limits of applicability

Three checks need to be made on the applicability of the method:

1. Mobility number at the node, ψ_n, must be greater than 2 for the bed to respond at all
2. Valid for $u_n/\omega<14$
3. Flow nonlinearity must be limited

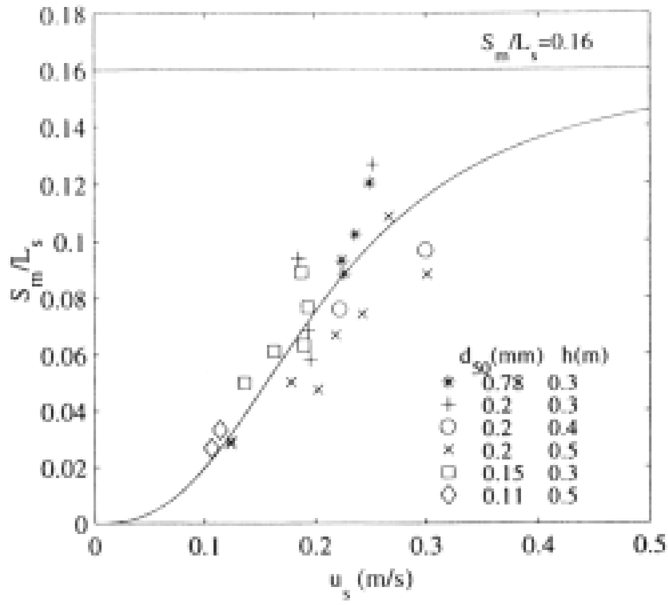
Design relationship: $\frac{S_{\max}}{L_s} = \frac{0.16 u_s^p}{q + u_s^p}$ with p=2.65 and q=0.016

where:

L_s: scour hole length

u_s: standing wave, near-bed, main flow horizontal velocity under node (m/s)

S_{max}: maximum scour depth



O'Donoghue (2001) formulation fitted to Xie (1981)

Scour occurred in the area of bed between the node and antinode of the standing wave field and accretion under the node, for bedload sediment transport.

Experimental procedure:

Theory developed analytically, best-fit from Xie (1981) experiments

O'Donoghue T (2001) N-type sediment bed response under standing wave. Journal of Waterway, Port, Coastal and Ocean Engineering, vol.127; 245-248.

HERBICH AND KO (1968)

Valid for:

- Non-breaking waves
- Normally incident
- Vertical or sloping walls
- Flat seabed in front of structure
- Sand

Limits of applicability:

- Equation derived theoretically (from conservation equation for water flow and the boundary layer equation for a plate)

Design relationship: $$S_{max} = \left(h - \frac{A}{2}\right)(1 - K_r)u_* \left(0.75 C_D \rho \frac{\cot\theta}{d_{50}(\gamma_s - \gamma)}\right)^{1/2} - 1$$

where:

K_r: reflection coefficient (H_r/H_i)
A: wave height at the wall $A=H_i+H_r$ (incident plus reflected wave heights)
S_{max}: maximum distance-averaged scour depth
h: water depth at the wall
u_*: horizontal velocity within boundary layer, under standing waves
C_D: sediment drag coefficient
ρ: fluid density
θ: angle of repose of the sediment
d_{50}: effective median sediment diameter
γ: specific gravity of fluid
γ_s: specific gravity of the sediment

The relationship predicts a distance-average scour over a sea bed line normal to the wall, rather than the depth of toe-scour. The reason for this is because it was found that the first scour hole developed some distance seaward from the structure.

Experimental procedure:

The theoretically derived equation was compared with experimental data, which characteristics are:

- Flume (2ft wide, 2ft deep and 67 feet long)
- d_{50}=0.25mm
- Seawall at 15 and 45 degrees
- Experiments conducted until scour depths became fairly constant with time

Other remarks

All experiments performed indicated that there may be a limit of scour depth which is approached asymptotically. The scour increases very rapidly during the first few hours and then the erosive process slows down and reaches a state of what is called ultimate scour depth.

Herbich J B and Ko S (1968) Scour of sand beaches in front of seawalls. Proc. 11th Coastal Engineering Conference, Vol I; 622-643

SONG AND SCHILLER (1973)

Valid for:

- Normally incident
- Vertical walls
- Beach in front of structure
- Sand

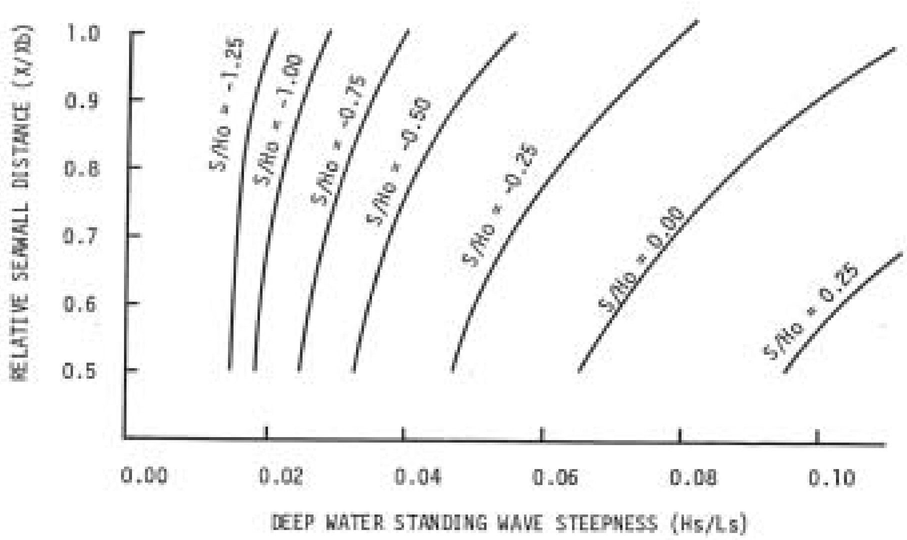
Limits of applicability:

- $0.5 < X/X_b < 1.0$ (Powell, 1987) $0.67 < X/X_b < 1.38$ (Fowler, 1996)

Design relationship: $\frac{S}{H_0} = 1.94 + 0.57 \ln\left(\frac{X}{X_b}\right) + 0.72 \ln\left(\frac{H}{L}\right)$

where:

L_0: deep-water wavelength
H_0: deep water wave height
H: deep water standing wave height
S: scour depth
X: horizontal distance from the original shoreline
X_b: horizontal distance from the original shoreline to the breaker point (i.e. original surf zone width)



Song and Schiller (1973) contours of non-dimensional scour depth

Experimental procedure:

- 2D movable physical experimental study (Flume dimensions: 40ft long, 8in.wide and 13in. deep)

- T=0.8, 1.0 and 1.3s. (10 Tests)
- Regular waves
- Initial beach slope is the stable beach profile formed after 48 hours of that wave climate without the seawall
- D50=0.17mm

Other remarks

The major problem lies in its derivation from small scale model tests (that may make no attempt to correctly reproduce prototype behaviour) and its relative simplicity, according to Powell (1987), as the equation does not take account of:

- The reflection characteristics of the wall, other than that implicit in the standing wave height
- The 3-dimentional aspects of scour
- Beach sediment size

Song W O and Schiller R E (1973) Experimental studies on beach scour due to wave action. Texas A and M University CoE Report n 166, TAMU-SG-73-211.

FOWLER (1992)

Valid for:

- Breaking waves
- Normally incident
- Vertical walls
- Beach in front of structure
- Sand

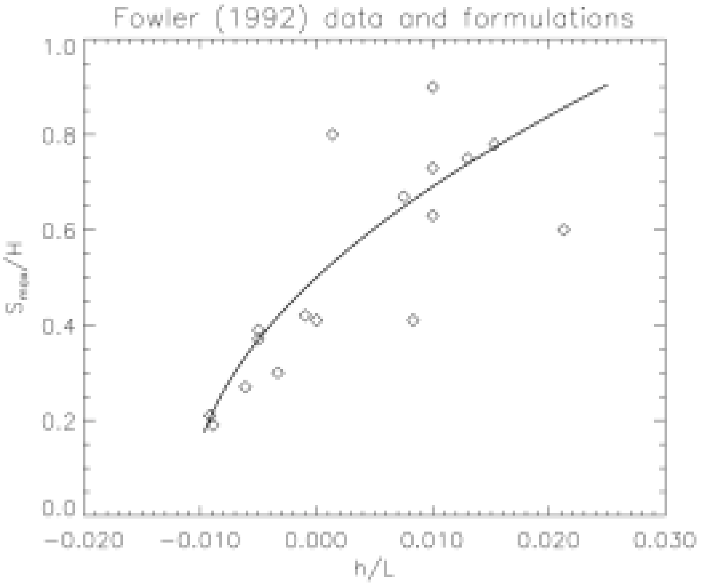
Limits of applicability:

- $-0.011<h_w/L_0<0.045$ and
- $0.015<H_0/L_0<0.040$

Design relationship: $\frac{S}{H_0} = \left(22.72\frac{h_w}{L_0} + 0.25\right)^{1/2}$

where:

L_0: deep-water wavelength
H_0: significant wave height
S: scour depth
h_w: water depth at the wall



Design relationship and data from Fowler (1992)

Maximum scoured measured immediately seaward of seawall (in some tests this value did not correspond to the maximum depth of erosion).

Experimental procedure:

- Physical experimental study in a flume (100m long)
- Tested 3 locations of the seawall, x_w=0.9,0 and -0.9m
- 18 irregular wave tests and 4 regular wave tests
- Initial planar beach at a slope m=1:15
- Ottawa sand, d50=0.13mm (fall speed=1.64cm/s)
- Waves for bursts of 300s, but as many bursts as needed to reach equilibrium

Other remarks
The design equation was compared with data from regular wave experiments of Barnett (1989) and Chesnutt and Schiller (1971) – where H_0 was taken as the wave height from the regular tests. Although there was large scatter, they followed the trend.

Drawbacks
In a detailed review of scour processes, McDougal *et al,* (1996) identified that the equation proposed by Fowler includes an inverse dependency between the dimensionless scour depth and the deepwater wavelength, or wave period. As a result, Fowler's equation implies that the dimensionless scour increases with increasing wave steepness; a result which runs contrary to every other scour prediction equation.

Fowler, J E (1992) Scour problems and methods for prediction of maximum scour at vertical seawalls. Technical Report CERC-92-16, U.S. Army Engineer Waterways Experiment Station, CERC, Vicksburg, MS.

JONES (1975)

Valid for:

- Breaking waves
- Oblique angle of incidence
- Vertical walls
- Sand

Limits of applicability:

- $X_s<1$

Cross-shore depth profile: $h = \left(h_n^{5/2} + R^2 h_s^{5/2} + 2R h_s^{5/4} h_n^{5/4} \cos 2\beta\right)^{2/5}$

Design relationship: $\frac{S_{\max}}{H_b} = 1.60(1 - X_s)^{2/5}$

where:

h = water depth
h_n = water depth in absence of structure (assumed to be a Dean-type profile)
h_s = water depth at structure location, calculated in absence of structure
$R = K_r/(1+K_r)$ with K_r the reflection coefficient of the seawall
H_b: breaking wave height
S_{max}: maximum depth of toe scour below original (pre-seawall) level at structure
X_s: X/X_b where X is the horizontal distance from the original shoreline and X_b is the horizontal distance from the original shoreline to the breaker point (i.e. original surf zone width)

Theoretical derivation:

- Linear wave theory
- Applied wave stress is uniform over the surf zone (i.e. radiation stress varies linearly from peak at breakpoint to zero at shoreline)
- Maximum scour depth is maximum difference between h and h_n, obtained by setting $2\beta = 0$ and R = ½ (its maximum value) and recognising that S_{max} is calculated at the structure so $h_s = h_n$.

Drawbacks

The usefulness of this equation is somewhat limited by the assumptions that the seawall is indefinitely long and reflects waves perfectly (Powell, 1997) Moreover, the assumption of linear variation in radiation stress is too simplistic. Also, the zero scour is predicted when the seawall is located at Xs=1 (at the shoreline), which is contradicted in every study examined (SPM, 1984); in fact some have found that this seawall location corresponds to the greatest scour condition.

Jones DF, (1975) The effect of vertical seawalls on longshore currents. PhD Dissertation, University of Florida.

POWELL AND LOWE (1994)

Valid for:

- Breaking waves
- Normally incident
- Vertical, smooth sloping walls and rubble mound structures
- Beach in front of structure
- Shingle

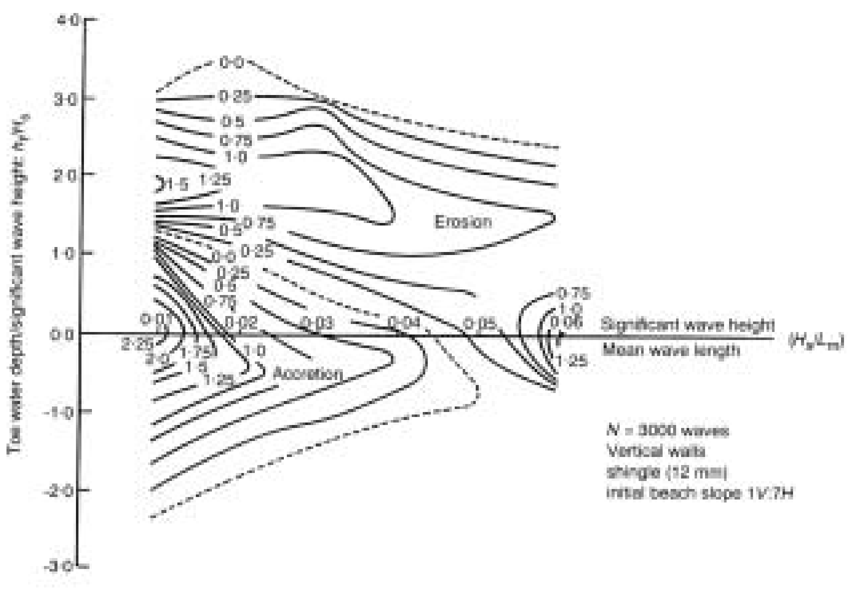
Limits of applicability:

- 5mm<d_{50}<30mm (modelled at 1:17 scale)

Design relationship: Isoparametric plots $\dfrac{S}{H_0} = f\left(\dfrac{h_t}{H_s}, \dfrac{H_s}{L_m}\right)$

where:

h_t/H_s: relative water depth,
h_t is the initial water depth at the wall,
H_s is the extreme unbroken deep water wave height,
Hs/L_m: wave steepness,
L_m is the mean wavelength of the unbroken wave (using $T^2g/2$),
S: scour depth after 3,000 waves.



Powell and Lowe (1994) iso-parametric scour plot for shingle beaches

To select the worst possible scour, look at the dimensionless scour values for all h_t/H_S values below the maximum relative water depth, corresponding to the wave steepness, H_s/L_m and select the greatest relative scour height.

The plot gives the scour after 3000 waves; a correction must be used to predict scour for time intervals other than 3000 waves.

Experimental procedure:

- Physical experimental study at a scale 1:17
- Irregular waves (20 different spectra) and 8 different water depths at the seawall
- Initial planar beach at a slope m=1:7

Other remarks

For vertical seawalls, four types of scour/reflection behaviour were observed with the occurrence of a particular type depending on the local wave conditions and water depth. The dependence of relative scour depth on wave reflection coefficient was also mapped for different wave relative steepnesses.

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

POWELL AND WHITEHOUSE (1998)

Valid for:

- Breaking waves
- Normally incident
- Vertical, smooth sloping walls and rubble mound structures
- Beach in front of structure.
- Sand

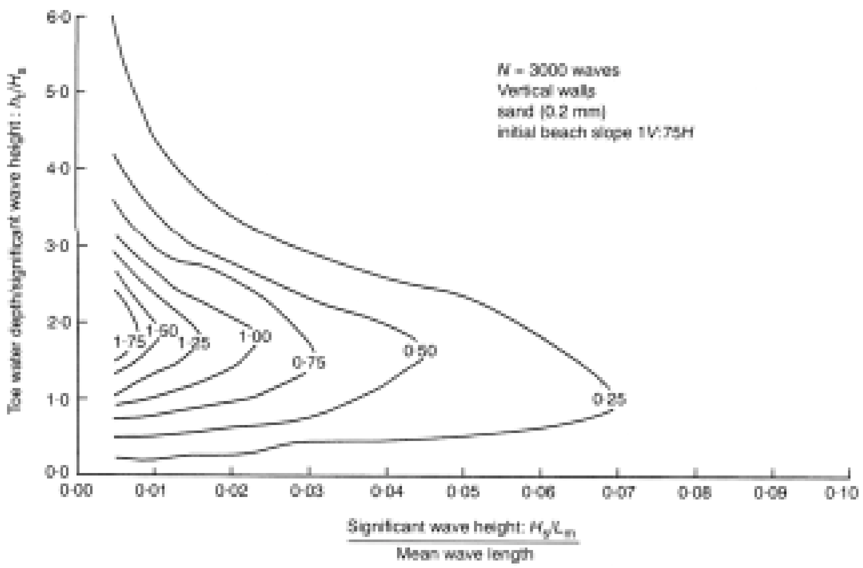
Limits of applicability:

- $0.005 < H_s/L_m < 0.075$
- $0 < h_w/H_s < 6$

Design relationship: Isoparametric plots $\frac{S}{H_s} = f\left(\frac{h_t}{H_s}, \frac{H_s}{L_m}\right)$

where:

h_t/H_s: maximum relative water depth,
h_t is the water depth at the wall,
H_s is the extreme unbroken deep water wave height
H_s/L_m: wave steepness,
L_m is the mean wave length of the unbroken wave (using $T^2g/2\pi$)
S: scour depth after 3,000 waves.



Powell and Whitehouse (1998) iso-parametric scour plot for sand beaches

To select the worst possible scour, look at the dimensionless scour values for all h_t/H_s values below the maximum relative water depth, corresponding to the wave steepness, H_s/L_m and select the greatest relative scour height.

The plot gives the scour after 3000 waves; a correction must be used to predict scour for time intervals other than 3000 waves.

Experimental procedure:

- numerical model COSMOS-2D used for all predictions
- Irregular waves (20different spectra) and 8 different water depths at the seawall
- d_{50}=200microns
- Initial planar beach at a slope m=1:75

Other remarks

COSMOS-2D (Nairn and Southgate, 1993, Southgate and Nairn, 1993) was validated with data from large-scale laboratory tests performed at SUPERTANK (McDougal *et al*, 1996) and field data from Blackpool. COSMOS-2D is generally unable to predict toe accretion and no accretion was predicted for any of the water level/wave condition combinations tested. Several water level/wave combinations produced accretion in coarse sediment experiments (in particular for most conditions when the beach crest level is above the water line). This conclusion was also reached by the SBeach modellers (McDougal, Kraus and Ajiwibowo 1996)

Powell K, Whitehouse R.J.S. 1998. The occurrence and prediction of scour at coastal and estuarine structures. 33rd MAFF Conference of River and Coastal Engineers, 1-3 July 1998. Keele University. UK.

McDOUGAL, KRAUS AND AJIWIBOWO (1996)

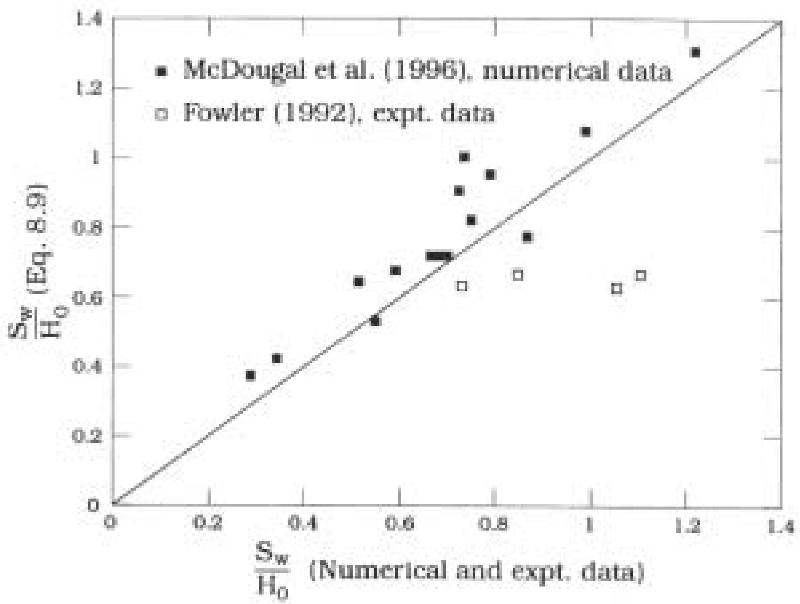
Valid for:

- Breaking waves
- Normally incident
- Vertical walls
- Beach in front of structure
- Sand

Relationship[1]: $$\frac{S}{H_0} = 0.41 m^{0.85} \left(\frac{L_0}{H_0}\right)^{1/5} \left(\frac{h_w}{H_0}\right)^{1/4} \left(\frac{H_0}{d}\right)^{1/3}$$

where:

L_0: deep-water wavelength
H_0: significant wave height
S: scour depth at the seawall
h_w: water depth at the wall
m: beach slope
d: sediment size



McDougal *et al* (1996) predicted versus measured scour depth

It gives scour at the seawall

[1] The authors emphasise that this is not a design equation as 'scour depth relationships are specific to the initial profile conditions; a point not well noted in the literature'. However, the equation may be useful to investigate the qualitative effect of grain size.

Experimental procedure:

- Numerical model SBEACH modified to include the effect of reflection at the seawall

Other remarks

The numerical model results were compared to the SUPERTANK data and Fowler (1992) data.

Steeper beaches may tend to have more scour at a seawall because wave energy dissipation is concentrated in a narrower surf zone. This increased response is also observed on the upper profile of steep, unwalled beaches.

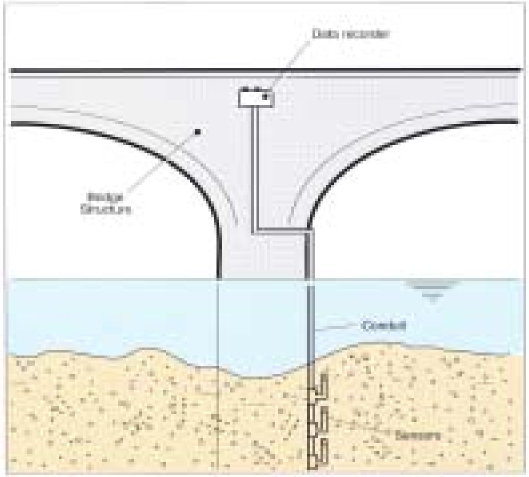
Fowler JE (1992) Scour problems and methods for prediction of maximum scour at vertical seawalls. Techncal Report CERC-92-16, US Army Corps of Engineers Waterways Experiment Station, 3909 Halls Ferry Road, Vicksburg, MS.
McDougal WG, Kraus N and Ajiwibowo H (1996) The effects of seawalls on the beach: Part 2, Numerical modelling of SUPERTANK seawall tests. Journal of Coastal Research, vol.12, n 3; 702-713

Appendix 3

Details of Tell-Tail scour monitors

The "Tell-Tail" Scour Monitoring System*

In response to increasing concern about the threat of scour, engineers at HR Wallingford have developed a system which is able to detect and monitor scour. The "Tell-Tail" scour monitoring system can be installed at new or existing structures and gives a clear indication of the depth of scour under all flow conditions. The system records the onset of scour, the depth of scour reached, and in-filling of scour holes following high flow events. It is reliable, economical and operates in extreme flow conditions - when it would be dangerous or impractical to use other methods.



The HR Wallingford 'Tell-Tail' system

How the system works

The system is based on omni-directional motion sensors, buried in the river or sea bed adjacent to the structure. The sensors are mounted on flexible "tails" and are connected via cable through protective conduit to the surface. Under normal flow conditions, the sensors remain buried and do not move. When a scour hole begins to develop, the sensors are progressively exposed and each begins to oscillate in the flow, triggering an alarm signal at the surface. Use of a multi-level array of sensors provides a more accurate measurement of the depth of scour.

Signals from the sensors are continuously monitored and logged on a solid state data recorder. Inspectors can assess the state of the sensors at any time using a portable alarm indicator so that an instant assessment can be made of the scour depth. This type of information may be used for example to decide when to close a bridge to traffic during floods. The system also indicates whether scour hole re-fill has occurred.


Components of the 'Tell Tail' monitor system

Standard set of equipment

The standard set of equipment consists of:

- Eight sensors with cables*
- a waterproof junction box
- an 8-channel, hand-held indicator unit
- an 8-channel, self-powered data logger
- a hand-held computer with software for downloading and transfer of data

Whilst this set of equipment will be suitable for many applications, the system can be tailored to individual circumstances. HR Wallingford can offer a complete supply, installation and monitoring service if required.

* Patent Pending

PB 11207

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