Stonehaven Seawall, Aberdeenshire

Feasibility Study of Improvements

Report EX 3731
November 1998

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HR Wallingford

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Contract

This study was commissioned by the Transportation and Roads Department (South) of Aberdeenshire Council, Stonehaven, represented by Mr Alasdair J Smith. The work was carried out by staff from the Coastal Group at HR Wallingford. The HR job number for this project was CBM 4068. Further information on this report, and the work carried out, can be obtained from the authors, Dr Alan Brampton and Mr George Motyka.

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Approved by ....

Date...

Summary

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The coastline at Stonehaven on the east coast of Scotland experiences a vigorous wave climate. The beach of sand and shingle is regularly modified in response to these waves, especially during storm conditions, and this causes a number of problems. Overtopping of the seawall, and blocking of the mouths of streams, occurs from time to time, depending on tidal levels and wave conditions. Damage is caused over a relatively short time period, and results in expense in subsequent repair and maintenance works. Other problems are causing cumulative damage and will take longer to have an effect.

This study reviews the problems being experienced along the seafront at Stonehaven, and the processes that cause these problems. A number of possible methods for preventing damage and expense are reviewed. Recommendations are made both for relatively inexpensive palliative works, that can be deployed quickly, and for longer term methods for managing the coastline. In both cases the objective is to provide increased public safety and reduce the risks of damage to property, in a cost-effective manner that avoids adverse effects on the human and natural environment of Stonehaven Bay.

Contents

Contents *continued*

Plates

Appendices

1 Introduction and report outline

1.1 Background

In June 1997, the Transportation and Roads Department (South) of Aberdeenshire Council commissioned HR Wallingford to study problems of flooding and damage to coastal defences along the coastline of Stonehaven Bay, about 22km south of Aberdeen on the east coast of Scotland.

This report deals with two of the problems giving rise to concern in Stonehaven Bay. The initial commission was to consider the overtopping of a section of the seawall and promenade just north of the Cowie Water (see Figure 1). At a later stage, this commission was extended to provide advice on the problems of maintaining a channel for the River Carron across the beach. The river mouth is about 400m north of Stonehaven harbour.

As well as these two problems, there are a number of other concerns that may have to be addressed in the future management of the coastline and its defences against flooding. In our present study, we have kept in mind these other problems, to avoid worsening them. Proceeding from north to south, they are:

- Overtopping of seawalls in Cowie village during severe storms and possibly the longer-term problems of seawall abrasion and falling beach/ foreshore levels there as well.

- Abrasion of the surface of the stepped seawall and the access steps from the promenade.

- Periodic blockage of the Cowie Water leading to a reduction in its capacity to discharge fluvial flows, leading to the danger of flooding just upstream from its mouth.

- Variability in the width and height of the beach just south of the mouth of Cowie Water. When beach levels are low, properties just behind the seawall (including sheltered accommodation) suffer damage from overtopping, especially during severe easterly storms.

- Overtopping and the deposition of sand and shingle on the coastal path between the mouth of the River Carron and Stonehaven harbour.

However, as stated initially, this report concentrates on the two most pressing problems, namely the overtopping of the seawall and promenade north of Cowie Water and the improvement of the channel of the River Carron.

The study of the seawall overtopping is described in Chapters 2–4 of this report. Chapter 2 describes the problems that have occurred, and sets out the objectives of our investigation into methods of alleviating this problem. The first step in this work was to evaluate the tidal levels and wave conditions that occur in Stonehaven Bay, and these are described in Chapter 3. This chapter also discusses the movement of beach material by interpretation of data from the Council's beach surveying programme. Combining the wave and tidal information with details of the likely behaviour of the beach during storm conditions has allowed a better understanding of how the wave overtopping occurs.

Based on this understanding, Chapter 4 then describes the appraisal of various possible improvements to the defences, bearing in mind financial, environmental and land use constraints. A number of possible remedial options are suggested in the first part of the chapter, based on experience of similar problems elsewhere in the UK. These various options were then ranked in order of overall effectiveness, so that the most promising options could be identified and recommended to the Council at the draft final report stage of the study. The criteria used to perform this ranking were then discussed with the Council, and refined, prior to repeating the exercise for this final report. The best two options were still ranked in the same order after this approach.

Finally, an initial evaluation of the benefits of the preferred defence improvement option was carried out. Information is presented on the financial losses that might arise if the seawall is not improved both in terms of flood damage and future maintenance costs. This evaluation was carried out to decide whether the improvements might be eligible for Grant Aid from the Scottish Office (under the 1949 Coast Protection Act), and whether the preparation, by the Council, of a formal submission for such Grant Aid would be economically worthwhile.

Chapter 5 considers the problems of flooding along the lower reaches of the River Carron, and its causes. The blockage and meandering of its channel across the beach seem to be a major cause of such events. The chapter therefore sets out the options for alternative types of training wall to improve the channel, and makes recommendations on both short-term remedial measures and possible further studies over a longer time-period to optimise the management of the flooding problems.

Overall conclusions from the study and recommendations for future actions are presented in Chapter 6.

2 Overtopping of the seawall and promenade

The most pressing problem along the shoreline of Stonehaven Bay is the overtopping of the seawall and promenade north of the Cowie Water. This wall extends for a distance of about 325m from the mouth of the river towards Cowie village (see Figure 1).

The low-lying land behind this wall is relatively undeveloped and used primarily for leisure and amenity purposes. Part of this seawall is presently experiencing abrasion by shingle which is free to travel along its length, eroding the concrete steps on the lower part of the seawall. Occasionally overtopping leads to flooding of the promenade and the accumulation of shingle on the roadway. Under extreme conditions, several buildings fronting the promenade are affected by overtopping damage; low lying land behind them may also become flooded albeit infrequently (see Plates 1 and 2). Overtopping occurs when large waves arrive at the site at times of high water. Such events occurred several times in the first three months of 1996 during an exceptionally prolonged period of easterly winds. In more normal years significant overtopping may occur on 1-3 occasions, but with little or no flooding of the hinterland.

The crest of the wave return wall is 6.8mODN, giving considerable freeboard above normal high tide levels. Mean High Water Springs is 2.05mODN and the Highest Astronomic Tide of 2.45mODN is about 0.5m below the level of the top of the lowest step (2.9m/3.0mODN). In view of this freeboard and the cross-sectional profile of the wall (concrete steps extending up from 2.9mODN to 5.5mODN and above these a wave return up to 6.8mODN) it is surprising that serious wave overtopping occurs at all. Certainly the offshore wave heights during severe easterly gales can be large, but the mild nearshore seabed slope, the presence of rock headlands both to the north and south, and the high beach under normal conditions (higher than MHWS at the seawall toe) reduce waves considerably before they reach the wall.

There is anecdotal evidence that overtopping is worsened by unpredictable and substantial changes in the profile and planshape of the shingle beach in front of the wall. With the exception of the groyning effect produced by the training wall at the mouth of Cowie Water the alongshore movement of shingle is unrestricted. As with most shingle beaches the cross-sectional profile is volatile, changes of up to a metre vertically in a few hours being possible. One consequence of this is that shingle may be transported rapidly away from the most exposed part of the seawall leading to localised but substantial beach lowering. Another possibility is that onshore movement of shingle by wave action can lead to the formation of a ramp against the face of the seawall, extending over the concrete steps. Views have been expressed that such a ramp may increase the capacity of waves to overtop the wall.

2.1 Objectives of overtopping study

The Council is looking into ways of reducing the intensity and frequency of wave overtopping of the promenade wall and of reducing the risk of flood damage affecting backshore developments. In addition, it would be helpful to reduce the present abrasion of the wall steps, to reduce present maintenance costs. Further benefits of additional protection would be the prevention of drains becoming blocked with beach shingle and causing water to pond on the promenade.

It would be relatively straightforward, albeit very costly, to overcome these problems by increasing the height of the seawall by a substantial margin. However, it is necessary to take into account the importance of the beach in Stonehaven Bay as a recreational and tourism asset. Any proposed improvements must therefore be such that they do not detract from the use of either the beach or the promenade, especially during the summer months. This has implications for the aesthetic appearance of any modifications to the defences, for the access between the promenade and beach, and for public safety.

Local improvement to the defences must also be achievable without affecting other parts of Stonehaven Bay, not least because defences elsewhere are substantially lower than the promenade seawall. It would be counter-productive, for example, if protecting the wall worsened the present situation at the mouth of Cowie Water, where the alongshore movement of shingle sometimes impedes the flow from the river and in extreme cases can cause flooding just upstream. Thus any structures such as large breakwaters, which could be used to build up beach levels in front of the promenade, may not necessarily improve conditions elsewhere. Thus a major investment necessary to construct such structures is neither desirable nor appropriate. Finally the cost of any modification to the defences or to the beach and subsequent maintenance requirements needs to be given careful consideration. The Council has a limited budget for maintenance of coastal defences, and is unlikely to embark upon a major scheme to protect what is essentially low density, amenity land.

In view of these various constraints a study has been commissioned by the Council to review the various options and to provide a number of possible alternatives which are relatively inexpensive and which can be upgraded or extended if and when necessary. This aim is thus to provide protection at modest cost and also to reduce the deterioration of existing coastal defences. These have a service life which can be prolonged significantly by judicious “beach management”.

3 Evaluation of coastal conditions

3.1 Tidal levels

Predicted tidal levels in Stonehaven Bay are based on data from the A-Class tidal gauge in Aberdeen Harbour and harmonic analysis of this data is used by the Admiralty to derive the daily predictions of high and low water levels for each tidal cycle. Adjustments given in the Admiralty Tide Tables indicate that predicted water levels at Stonehaven (relative to ODN) will be virtually the same as those at Aberdeen, which is only 22km to the northward.

Using the Admiralty Tide Tables the following tide levels are predicted for Stonehaven:

Highest Astronomic Tide (HAT)	2.45mODN
Mean High Water Springs (MHWS)	2.05mODN
Mean High Water Neaps (MHWN)	1.15mODN

Atmospheric effects can cause substantial changes in tidal propagation around the coast of the UK, producing either higher or lower levels than those predicted from Admiralty Tide Tables. These differences, known as residuals or surges, are caused both by variations in atmospheric pressure and by wind induced forces. As a first estimate tide levels may exceed the predicted value by as much as 1.25m during a “fifty year surge” off Stonehaven (Department of Energy, 1985). The likelihood of such a surge coinciding with the time of a high tidal level is very low, so it would be unrealistic to apply the 1.25m increase to the predicted value of MHWS, for example, unless events with a very small probability of occurrence are being examined.

Information on extreme tidal elevations is derived from work by Dixon and Tawn (1995) and presented in a regional study of the Scottish coast between Fife Ness and Cairnbulg Point (HR Wallingford, 1996). This indicates that the tide level which is expected to occur once in every 50 years at Aberdeen is 3.02mODN. The value at Stonehaven for this and other extreme water levels is similar to those derived for Aberdeen. Hence values for extreme tidal levels at Stonehaven Bay are predicted to be:

Extreme tidal levels - Stonehaven

Return period	Water Level (mODN)
10 years	2.85
25 years	2.96
50 years	3.02

For the purpose of examining the problems associated with overtopping of the promenade wall, two still water levels, equal to the highest astronomic tide, and the mean high water of spring tides, have been chosen for analysis. Suitable wave conditions associated with these levels are considered below.

3.2 Wave conditions

The coastline at Stonehaven experiences a wave climate which is dominated by waves generated within the North Sea. For example, a period of prolonged, strong easterly winds in the North Sea will generate large waves along the long “fetch” across to the coast of Denmark. Such waves may cause erosion to take place at the base of the promenade wall if they were to coincide with a period of high tidal levels.

In the absence of measured waves off Stonehaven, synthetic data has been abstracted from the Met Office wave model archives for the period January 1992 to December 1996. The point used has co-ordinates 56.75°N 2.07°W; it lies some 15km offshore and about 28km south – southeast of Stonehaven. HR Wallingford is an “Authorised Data User” and is licensed to hold and apply this data for consultancy purposes (see Appendix 1).

It has been possible to obtain this synthetic offshore data for the present study at a very modest cost. It must be borne in mind, however, that to “transform” all this data inshore would require a major study. To minimise costs we have therefore chosen several storm events which occurred in the recent past, and which caused serious overtopping at Stonehaven. These events occurred in January/February 1996 for example, when most of the east coast of the United Kingdom was hit by a series of prolonged and damaging easterly storms.

Before analysing individual storms, however, it is necessary to examine the general wave climate off Stonehaven. This wave data is shown in Figure 2 in the form of a wave rose. Because this is an offshore location the rose incorporates wave conditions from the southwest which would not affect the east coast of Scotland. The predominant sector for evaluating wave conditions at Stonehaven, is 40° to 140°N. It can be seen that within this sector there is a considerable amount of wave activity in excess of 4m, with the largest waves being from 100°N. These waves will propagate perpendicular to the general trend of the seabed contours and losses due to wave refraction will be small. These waves have therefore been chosen to evaluate the damaging events at Stonehaven.

A more detailed breakdown of the “average annual” wave climate at this offshore location is presented in Tables 1 and 2, as diagrams of height versus period, and height versus direction respectively. It can be seen that the largest wave heights lie within the sector 90°N to 110°N, and are therefore virtually normal to the coastline at Stonehaven. The highest waves within this sector are in the 6.0 to 6.5m range and have a mean wave period of 8 to 9s.

It is now necessary to examine specific storms that are likely to cause problems at Stonehaven. Offshore waves from January to March 1996 are now examined since wave overtopping occurred at Stonehaven in this period. Using a threshold wave height, H_s = 5.0m, the following offshore wave conditions are identified within the UKMO data set:

Wave extremes for January to February 1996 (H_s > 5m)

Date	Wind speed (m/s)	Wind dir [n.] (°N)	Hs (m)	Tm (s)	Wave dir [n.] (°N)	GMT (hrs)
6/1/96	15.7	126	5.1	8.1	110	2100
7/1/96	16.1	127	5.4	8.3	109	0000
7/1/96	15.4	126	5.1	8.3	105	0300
9/2/96	17.2	144	5.1	8.4	122	1500
9/2/96	15.8	141	5.2	8.1	122	1800
18/2/96	16.1	39	5.9	9.2	59	1800
18/2/96	11.6	18	5.4	9.0	64	2100

It can be seen that waves in excess of 5m are predicted to have occurred on seven (3-hour) occasions in the period January/February 1996. By contrast the average annual occurrence of waves in excess of 5m (from Tables 1 and 2) is of the order five times per year. Clearly the early part of 1996 was a period of particularly prolonged and severe weather with a greater number of storms in two months than in an average year. These storms provide a benchmark against which to compare the feasibility of reducing overtopping, by raising the wall crest for example.

A more detailed analysis of the storm waves could have been made using HR numerical models to evaluate inshore conditions in the winter of 1995/6. However, this would require the offshore waves to be transformed to the site using wave refraction techniques. Such an exercise is costly and appears to be unnecessary given that these large waves will be reduced by breaking and will be “depth limited”, by the time they reach the seawall.

The vulnerability of the Stonehaven seawall to overtopping has been evaluated for one of the wave conditions in the storms of January/February 1996. However, as a precursor to such calculations it is necessary to obtain an understanding of beach behaviour under such conditions. This is essential as the water depth limiting criterion controlling inshore wave heights is strongly dependent on bed levels in front of the wall.

3.3 Beach changes

The movement of beach material, which takes place under the action of both waves and tidal currents, is strongly dependent upon the geomorphology of the site. The coastline of the Stonehaven area is dominated by the underlying solid geology. At Stonehaven the bedrock consists of Old Red Sandstone, and this rock forms cliffs as well as abrasion platforms. The volume of beach material is limited and consists in general of poorly sorted shingle overlying bedrock. The beaches within Stonehaven Bay have been derived from the reworking of mainly fluvio-glacial deposits. These deposits are now protected from erosion by coast protection works so the amount of new material made available by coastal erosion is low. Some material is still derived from erosion of the cliffs and shore platforms north of Stonehaven Bay. This is only transported southwards into the Bay in small quantities, as the drift rate in the area is very low. Spate flows down the Cowie Water also bring a limited amount of shingle onto the beach. As the beach is constrained laterally both by rocky headlands and abrasion platforms, beach volumes are not decreasing significantly.

The dominant process is cross-shore movement of shingle, with beach levels at the toe of the promenade wall in particular depending on the most recent wave conditions.

There is also a nett movement of beach material southwards, resulting in a reduced volume at the north end and an increased volume of shingle at the south end of the frontage in the long term. This accounts for the wide beach south of Cowie Water which also receives a small input of sand sized material from the River Carron. As well as small progressive changes due to this weak southerly drift there are also rapid and large scale but short term fluctuations in beach plan shape, due to changing directions of wave approach (see below).

Recent beach changes
Information about the recent behaviour of the beaches in Stonehaven Bay was obtained by examining the Council’s topographic surveys covering the period 1990 to 1997. These

illustrate the predominant southward movement of beach material, with the outlets of Cowie Water and River Carron in the central and southern parts of the Bay being deflected southwards on all surveys. This southward drift has provided a healthy beach in the southern half of the Bay so that wave overtopping problems are less severe in that area, despite the seawalls there being lower than the promenade wall. North of Cowie Water a training wall has allowed a substantial shingle beach to accumulate there, giving a high level of protection to the southernmost 100m or so of the promenade wall. In this area, the beach width above MHWS (2.05mODN) is greater than 5m on all surveys except 1996. It is important to note that in 1996 (the year in which prolonged eastern storms occurred) the high water line retreated back to the wall immediately north of the training wall and beach material was transported northwards towards Cowie. A "drift divide" may have thus formed here in the short term.

Away from the training wall there is a reduction in beach width. At a position 200m to the north, in the vicinity of the café, this "upper beach" varied from a minimum width of 0m in 1992 and 1994 (MHWST being coincident with the toe of the seawall steps) to a maximum of 8m in 1996 and 1997. The relatively low beach levels opposite the café appear to be due to littoral transport being away from this zone, possibly due to the drift divide shifting northwards temporarily. Once material has been transported northwards it tends to accumulate in the shelter of The Toutties. Southward drift, on the other hand, tends to accrete against the training wall or at the mouth of Cowie Water, ultimately finding its way to the town frontage. The beach opposite the café is thus more prone to beach lowering than adjacent areas.

The volatility of the beach opposite the café is well illustrated by the annual variability of the beach profile in that area. At the toe of the wall maximum and minimum recorded levels are 3mODN and 2mODN respectively. Further offshore beach levels are even more volatile, with levels varying from a maximum of 1.5mODN to a minimum of 0mODN at 20m chainage, the variation then reducing offshore, being from 0.8mODN to –0.8mODN at 30m chainage. However, these annual surveys would not have picked up the maximum beach level variations; it is expected that at the toe of the wall beach levels could vary by as much as 2m between winter and summer conditions.

Because of the strong indentation of the embayment and the reduction in beach width at the promenade wall, the frontage in the vicinity of the café is clearly more prone to wave overtopping than the frontages northward and southward. In the following calculations, therefore, it has been assumed that beach levels at the toe of the wall may be as high as 3mODN under calm weather conditions but may drop by 1.5m to 2m during storms.

It has not been possible within the budget of the present study to carry out a detailed analysis of beach level changes from the Council's topographic surveys. It is recommended that such an analysis should be made and should be updated on a regular basis. The surveys should be analysed using ground-mapping software in order to determine:

a) the volumetric changes of the upper beach
b) the variations in beach level in front of the wall at various parts of the frontage.

As the present surveys are carried out once a year they "underestimate" the variability of beach levels. It would be sensible, therefore, to repeat such surveys, or alternatively, make simple longitudinal monitoring of beach levels at the base of the seawalls, after severe winter storms.

3.4 Wave overtopping

3.4.1 General

As part of this study, we have calculated the volumetric rates of overtopping of the promenade seawall at Stonehaven. This has allowed:

- a better understanding the circumstances under which flooding of the promenade occurs,
- guidance on the likely efficiency of various remedial options considered.

For these calculations, we have employed a standard computer program, developed at HR Wallingford, and called SWALLOW. This program is described in Appendix 2 to this report. In order to carry out calculations the program requires the following information:

- seawall geometry, i.e. crest level, slope of face, details of permeability, roughness etc.
- beach level (at time of storm conditions)
- tidal level (at high tide)
- wave conditions just offshore of the seawall.

Each of these topics is now considered in turn.

<u>Seawall geometry</u>
The SWALLOW program requires information on the geometry of the seawall, e.g. its crest level and slope of its face. This information was obtained from the surveys carried out by the Client. Because the main concern of this study was to understand the processes causing overtopping, and to compare alternative remedial measures, some simplification of the seawall profile was acceptable. Hence the fine details such as the recurve at the seawall crest were ignored in the operation of the model.

<u>Beach levels</u>
A preliminary analysis of the overtopping problems showed that the level of the beach was a crucial factor in the performance of the seawall. The annual surveys carried out by the Client indicated that beach levels at the toe of the promenade seawall could certainly fall to 2.0mODN, as indicated by the 1994 survey of the frontage. At this level, the beach crest is below MWHS. It is considered likely that during stormy conditions, beach crest levels may well fall below this minimum recorded value, even if only temporarily.

In the following calculations, therefore, we have assumed rather lower levels of the beach crest, and carried out calculations for these cases as well.

<u>Tidal levels</u>
For the purpose of this study, we have concentrated on calculating overtopping discharges for two tidal levels, namely 2.05mODN (MHWS) and 2.47mODN (HAT). Even higher levels will occasionally occur, as indicated in Section 3.1 above, although it would be extraordinary if waves conditions as severe as assumed in our tests occurred at the same time. A detailed analysis of the joint probability of large waves and high tidal levels is beyond the scope of this study.

<u>Wave conditions</u>
Analysis of storms during the winter of 1995/6 has shown that in deep water, significant wave heights of up to 5.5m and mean wave periods of the order of 9.5s can be expected to occur during easterly and north-easterly gales.

A storm having a significant wave height H_s = 5.4m and a mean wave period T_m = 9.6s was used as input to SWALLOW, this being one of the conditions which were reproduced by the UKMO model during early 1996 (when severe overtopping took place). In practice, wave conditions will change between deep water (i.e. at the UKMO wave model point used in this study) and the coastline. Refraction, diffraction and friction will all tend to decrease wave heights while shoaling will increase them. For the present study where we are comparing alternative types of improvements, however, it is not critical to include all these processes. We have therefore erred somewhat on the side of caution, and assumed the offshore wave conditions occur unaltered just seaward of the surf zone in our modelling.

Even at the high water levels examined here (HAT = 2.47mODN and MHWS = 2.05mODN), however, these wave conditions will be considerably reduced by breaking over the rocky foreshore in Stonehaven Bay. This reduction in wave energy is evaluated automatically using a wave-breaking criterion in the SWALLOW program, thus ensuring that appropriate wave conditions are used in the analysis.

Further, it was assumed that these storm conditions would persist for a sufficient part of the tidal cycle for severe wave action to coincide with high tide.

3.4.2 Overtopping results

In the remainder of this section, we present results of overtopping calculations carried out for assumed tidal levels of 2.05mOD (MWHS) and 2.45mOD (HAT). All the results quoted are for the mean discharge rate, in cubic metres per second per metre run along the seawall. This mean rate is an average over a few hundred wave periods. It should be noted, however, that peak instantaneous overtopping rates, i.e. over a few seconds during a large wave, may be several thousand times this mean rate.

Overtopping for a high tide level of 2.05m ODN (MHWS)
The initial calculations of overtopping were carried out for a tidal level of 2.05m ODN (MHWS). The results obtained are presented in Table 3. For a beach crest level of 2mODN, conditions on the promenade would be “wet, but not uncomfortable” for pedestrians; no remedial measures would be necessary.

A drop in beach level to 1.5mODN allows much bigger waves to reach the wall, increasing overtopping significantly and making it dangerous to use the promenade. Under these conditions, it was found that increasing the height of the rear wall by 0.5m would reduce the overtopping discharge by 300%.

Assuming a further drop in beach crest level, i.e. to 1mODN, the calculations show very high overtopping making it very dangerous to use the promenade. Changing the wall height by up to 1m (to 7.8mODN) would not reduce this discharge to an acceptable level. Table 3 shows that, under these conditions, the mean discharge over the wall would still be as high as $0.026m^3/s/m$ run of wall.

Photographs of the Stonehaven frontage, taken during severe storms, show that shingle can be thrown up onto the seawall steps by wave run-up, to form a shingle ramp over these steps (see Plates 1 to 3). The crest of this ramp can reach the height of the top step i.e. about 5.5mODN. At the early stages of this study it was considered that the formation of such a ramp could facilitate further wave runup and could be a contributory cause of overtopping.

In order to determine whether this might be the case, wave overtopping of this ramp was simulated, assuming that its roughness of the shingle ramp was similar to the roughness of the seawall steps, i.e. the permeability of the shingle was not taken into account. Table 4 illustrates that for a hypothetical shingle slope of 1:3 extending from the toe of the wall to the top of the steps, overtopping would actually be significantly reduced. At a beach level of 2mODN at the toe of this ramp, overtopping is predicted to be so small as to be only a minor nuisance.

If beach levels were to drop to 1.5mODN or lower, the rate of overtopping would reach a dangerous level for pedestrian usage, but would nonetheless be much lower than without the ramp. Increasing the wall height by 0.5m increments (to 7.3 and 7.8mODN respectively) improves the situation but still results in dangerous overtopping.

The calculations summarised in Tables 3 and 4 lead one to infer that the shingle ramp is formed as a result of wave run-up and actually helps to reduce overtopping. Without the ramp, the incident waves are likely to slam into the vertical faces of the steps, with much of their energy being reflected out to sea. The interactions between the incoming and reflected waves will result in water and spray being thrown into the air, and with an onshore wind, this will increase the overtopping discharge predicted here. However, the rates are still likely to be less than for the parts of the seawall without a shingle ramp.

Plates 1a and 1b suggest that a low wall would not reduce the volume of overtopping water significantly but might prevent some of the shingle being thrown onto the road. Plates 3a and 3b show two phases of beach development. In Plate 3a one can see the influence of a change from stormy to calmer weather conditions. Earlier storms have resulted in shingle being thrown onto the seawall steps. More recent calmer weather, with waves approaching from the north have resulted in the build up of a shingle berm seaward of the steps. Plate 3b shows the situation immediately after a period of rough weather with a berm being formed over the steps of the wall, with its crest at approximately the swash limit of the waves. There is evidence of pooling of water at the "blue bin" as a result of overtopping.

Overtopping at a high tide level of 2.47m ODN (HAT)
The above calculations, assuming a high tide level of 2.05M ODN, show that inshore wave heights and hence the amount of overtopping which occurs are strongly dependent upon the water depth near the toe of the wall

Having examined the overtopping under these conditions we next examined the situation under conditions of very high water level (HAT) plus storm wave action. The likelihood of beach material accumulating against the seawall steps is very small and therefore the impacts of a ramped beach were not examined. These latter calculations are primarily to demonstrate how low beach levels have to fall before wave overtopping under very high water levels becomes unacceptably high.

Table 5 presents the results of the overtopping calculations for the same wave conditions at a still water level of 2.47mODN, i.e. HAT. With assumed beach levels at 2mODN, there is nearly 0.5m water depth in front of the wall and the overtopping discharge is predicted to be so high as to make it impossible to use the promenade. A drop in beach level to 1.5mODN or lower would allow overtopping waves to cause damage to the road pavement.

3.4.3 *Summary of overtopping calculations*

From this analysis of wave overtopping it can be deduced that problems are most likely to occur under severe wave conditions when beach levels in front of the seawall are 2mODN or lower and when water levels are at or above MHWS (2.05mODN) or higher. Should the beach fall below 2mODN, serious overtopping can be expected. Waves impacting against the face of the seawall will result in spray being carried over the wall, as well as "green water". The ramping of shingle on the concrete steps and the dispersal of pebbles over the promenade (see Plates 1b and 2) seems to be the result of wave overtopping, rather than the cause. Wave overtopping can be seen to be as a result of wave impact in Plates 1a and 1b, while the carpet of pebbles could not be caused by ramping, unless the ramp built up right up to the crest of the wave return wall. This scenario is unlikely to say the least. The calculations reinforce our view that beach levels at the toe of the wall should be closely monitored, both before and after storms. Such information is as useful, if not more so, than detailed topographic modelling of the whole frontage on an annual basis. The distance from the seawall to the high water mark is also a useful indicator of the "health" of the beach. This too could be measured by simple surveying at regular intervals over the frontage (and could include the beaches south of Cowie Water).

Measures to overcome the overtopping problem are discussed in Chapter 4. However, to consider the suitability of the various options open one also needs to consider the implications of the "do nothing" scenario. This scenario is therefore also considered as a possible management option.

4 Improving the seawall - appraisal of remedial options

4.1 General

There are a number of measures, which might be adopted to reduce overtopping of the promenade wall, hence improving the amenity of the area and minimising damage to the hinterland as a result of flooding. Some of measures suggested may also be useful in reducing other problems, e.g. the abrasion of the seawall steps, and the changes to the beach south of the mouth of Cowie Water, although these are regarded as longer-term objectives with less of an immediate need for their resolution.

Broadly speaking the options considered here fall into one of three classes:

1. Linear, shore-parallel defences. These include seawalls, revetments, etc.

2. Beach management methods, including nourishment or recycling existing stocks of beach material, together with structures such as groynes to hold the material in place.

3. A combination of options designed to manage and retain the beach material and to reduce the severity of wave action. These include offshore breakwaters, rock sills, groynes, shore connected breakwater etc.

These possible schemes all have to be compared with the option of not intervening and allowing overtopping to continue i.e. the “do-nothing” option. In reviewing the do-nothing option, consideration has to be given as to how much damage or disruption overtopping under present circumstances is likely to cause, perhaps leading to a need for the re-siting of amenities further landward.

Other constraints on the choice of future management options are related to usage i.e. the need for maintaining the present amenity of this area. Although there are important SSSI’s in the area which are of geological interest (Crawton Bay and Fowlsheugh), both are “hard rock” areas, situated “updrift” of Stonehaven, and certainly would not be affected by minor alterations to the seawall within Stonehaven Bay.

A range of possible remedial measures is now discussed, chosen from the usual coastal defence management techniques, and their application to this site considered.

4.2 Beach nourishment

Methods of upgrading existing coast defences by beach nourishment can be useful not only in reducing the likely overtopping volume but also increasing the residual life of the wall itself. For nourishment to be cost-effective the frontage should experience only a modest longshore drift, which can be countered by recycling or the construction of modest control structures such as rock groynes. However, it is generally a less attractive option if only parts of a frontage require protection and where there are adjacent areas that need to be kept free from accretion. In Stonehaven Bay the addition of fresh material is likely to cause build up in the mouth of Cowie Water, and to a lesser extent further south as well. Keeping beach material in place by groynes would greatly increase costs of dealing with the overtopping problem, which presently affects only a small part of the frontage infrequently.

We have not considered the possible sources of gravel or small rounded stone for use as a beach nourishment material in this report.

4.3 Groynes

Groynes are structures which protect the shoreline indirectly by interrupting the alongshore movement of beach material. Whilst groynes have been widely used in conjunction with seawalls to maintain beach levels, there is still considerable debate about their best usage. In general, groynes will trap shingle more effectively than sand, since the latter may be carried in suspension and is therefore likely to overtop the groynes or be carried round them.

At Stonehaven we have considered the feasibility of using short groynes whose function would be to maintain beach levels at 2mOD, but to allow transport above this level. The groynes are only designed to interrupt the transport of shingle (i.e. they should not interfere with sand transport).

The main problem is that even short groynes may cause problems on adjacent frontages by reducing the supply of material to them. Also, during periods when waves do not break parallel to the beach contours, the groynes result in the beach becoming “zigzag” shaped in plan. Thus, additional protection to the seawall is provided immediately “updrift” of a groyne but a reduced level of protection occurs “downdrift”. Wave overtopping may therefore be eliminated in some parts of the beach but increased in others.

The construction of groynes is a feasible option, but one that would need to be examined in greater detail than is possible here, by mathematical modelling of the planshape of the beach that will evolve after their construction.

4.4 Rock sill at toe of wall

The advantage of a rock sill strategically placed in front of the existing seawall is that it will directly prevent a drop in levels during storm wave action. A sill can be constructed at the wall toe or some distance seawards. The best location can be determined by means of physical modelling in a hydraulic flume of the rock sill, seawall profile and the beach. In the present situation, however, amenity considerations dictate that the only sensible location for a rock sill is at the seawall toe. Suitably sized rock placed to a hydraulically efficient profile (generally 1:4 or flatter) can prevent waves reaching the wall directly. An important advantage of placing rock directly onto the beach to form a sill is that the armoured profile can be amended and extended if necessary. Other advantages are the dissipation of wave energy, and direct protection to the seawall, resulting in less wear and tear on its face.

The obvious disadvantages are related to amenity usage of the beaches, although it is common practice to construct permanent steps across such structures to provide beach access.

At this location, wave overtopping calculations have shown that except under the most extreme storm conditions (1:50 year tidal levels for example) wave overtopping is tolerable provided beach levels do not fall below 2mODN (MHWS). A rock sill designed to reduce overtopping would not require a crest height in excess of this value and would therefore be invisible for most of the time. Beach levels along this frontage seem to fall below 2mODN only rarely e.g. during storm events.

4.5 Raising the wall crest

Wave overtopping can also be reduced by increasing the effective height of the wave return wall. This can be achieved by major structural alterations, or more simply by the addition of, say, concrete blocks to the promenade deck during winter periods. At first sight this alternative would appear to be an attractive one. However, calculations have shown that a crest level of 8.3mODN would be required to reduce wave overtopping volumes to an acceptably safe level. Even with this crest level, wave impact against the steps during periods of low beach levels will cause spray to be carried onto the promenade. Providing a high vertical face, against which waves will impact directly, is therefore not necessarily the most effective means of protecting the promenade from flooding. Any structural modifications to the existing seawall will be costly and will result in a permanent loss of the seascape.

Temporary structures, such as concrete blocks, short lengths of “retaining wall” etc., would need to be substantial to be stable under wave impact and will therefore need mechanical plant to be available to place them and to remove them. Storage may also pose logistical problems.

4.6 Offshore breakwaters

In areas of low tidal range, breakwaters can be used to defend a coast or create a wide beach in their lee. However to be effective, the crest of such structures needs to be at or above the highest anticipated tidal levels, to ensure that wave energy is effectively reduced and not transmitted through or over them. The existing coastline in Stonehaven Bay has natural examples of low crest breakwaters formed by the abrasion platforms in the centre and both ends of the Bay. Where these occur there is usually an accumulation of beach material behind them. However this material rarely builds up much higher than the crest level of the platforms, so protection of the upper beach may not be significant.

A number of difficulties arise with such structures, limiting their usefulness. Because they are built in deeper water than the seawall they are to protect, they will be exposed to more severe wave action and consequently require large rock or concrete armour units for stability. Also, the quantity of material needed will be large because both a back face, a crest and a front face are needed. It must also be borne in mind that breakwaters are normally very effective in arresting alongshore drift, with consequent dangers for the stability of adjacent beaches.

From the viewpoint of amenity the shelter provided in the lee of a large breakwater may alter the beach regime to such a degree as to encourage fine silt to settle out, for example. On the positive side, breakwaters can be effective when only localised protection is required.

Construction of breakwaters in Stonehaven Bay is likely to be an extremely expensive option. This type of structure could be oriented to reduce wave attack at the promenade but its impact on coastal processes would be difficult to predict. Wave energy would tend to be “concentrated” at either end of the structure with a small zone of increased wave height being possible there. Movement of material into the lee of the breakwater would reduce beach levels to the north and south. There is an additional risk of the overtopping problems being transferred to adjacent parts of the frontage, as a result.

From an amenity viewpoint breakwaters may not be generally accepted as they are visually intrusive. Offshore breakwaters are therefore not recommended as an option in this particular instance, and are not considered further.

4.7 Ranking of alternative options

Sections 4.2 to 4.6 above have reviewed various options for managing the overtopping problem at the promenade wall. Attention is now turned to ranking these options in order of effectiveness. The method used to identify the most promising options is called a “Multi-Criteria Analysis”. This is a technique by which an assessment is made of options using various criteria, effectiveness, cost, environmental impact etc. Criteria that are considered to be more important than others have this reflected by using a weighting method.

Having chosen a set of criteria and having determined their respective weightings, each of the options are then ranked according to how well they fulfil each criterion. The overall performance of each option is then evaluated by a summation of the total weighted score.

In Stonehaven Bay the criteria which are considered to be of importance are hydraulic efficiency, modest cost, and low impact on adjacent frontages. As this is an area used for recreation the criterion of impact on the amenity value of the frontage is also a factor. These and various other obviously relevant criteria are compared below:-

Comparative weighting of criteria

Criterion		A	B	C	D	E	F	Total weight
A	Prevention of overtopping	-	1	2	1	2	2	8
B	Modest capital cost	1	-	2	1	2	2	8
C	Low maintenance cost	0	0	-	1	2	2	5
D	Low impact on adjacent beaches	1	1	1	-	2	2	7
E	Low impact on amenity	0	0	0	0	-	1	1
F	Decreased wall abrasion	0	0	0	0	1	-	1

In the above analysis the comparative weighting is arrived at as follows:

If the criterion in a row is

- more important as the criterion in the column, then the criterion in the row is given a score of 2 and the criterion in the column is given a score of zero.

- equally important than the criterion in the column, then both criteria are given a score of 1.

- less important than the criterion in the column, then the criterion in the row is given a score of zero and the criterion in the column is given a score of 2.

Having chosen a set of criteria, and evaluated their weighting factors, we now draw up a set of possible management options to reduce the problems of overtopping of the promenade seawall at Stonehaven. Five such options are sensible, including the “no-nothing” scenario, where the existing overtopping is not reduced, but accepted, possibly leading to a need to re-locate or protect facilities at risk. These options are summarised in the table below.

Option	Description
1	Beach nourishment with shingle, plus short groynes
2	Groyning of the promenade frontage
3	Construction of rock sill at toe of wall to prevent fall in levels and consequent wave height increase at toe of wall
4	Increase height of wall to reduce green water overtopping
5	The do-nothing option, accepting occasional overtopping and potential flood damage

These options are largely self-explanatory, and relate to Sections 4.2 to 4.5 above. Greater detail needs to be provided before any of these could be seriously presented as a viable scheme. However, it is possible to compare these options using the Multi-Criteria Technique, as follows.

Comparative assessment of defence options

Criterion		Weight	Effectiveness of options				
			1	2	3	4	5
A	Prevention of overtopping	8	7	4	8	7	0
B	Modest capital cost	8	4	6	6	6	10
C	Low maintenance cost	5	5	7	8	6	5
D	Low impact on beaches	7	5	4	8	8	10
E	Low amenity impact	1	5	4	4	2	5
F	Decreased wall abrasion	1	7	5	8	0	0
Total weighted score			160	152	220	192	180

From this analysis, the two most promising options (3 and 4) are found to be the rock sill and the blocks on the seawall. These are calculated to be of similar cost. Both should reduce overtopping, especially of shingle, but the former will reduce abrasion of the wall (at least locally) and should require less annual maintenance (i.e. compared to placing and removing the blocks).

Possible groyne schemes, with or without nourishment, are as expensive (or more so) and cannot be guaranteed to prevent overtopping/abrasion although it should be considerably reduced. There is, however, a concern that the groyning of 100m of this shoreline may result in problems elsewhere, making their advantages less clear.

Raising the wall crest will not only reduce the amount of green water but to a degree will cut out some of the spray. It will also reduce the shingle build up and hence the blockage of the drains. However this is not a cheap option and there is no guarantee that if the wall is raised in one area, beach volatility will not cause overtopping problems to be shifted to adjacent parts of the wall. This option is therefore feasible if the crest can be raised temporarily and over a localised frontage, say by placing concrete blocks on the promenade. The logistics of such an operation will however need careful consideration. One possibility would be to employ short lengths of concrete retaining wall such as those used on motorways to separate traffic flow. These could be fabricated to a height of say 1.5m and a length of 2.5m. Protection of a frontage of some 100m would thus require the deployment of about 40 such units, for which storage would have to be found “out of season”. The visual impact of such units would be considerable and given the difficulties of rapid mobilisation the units would probably have to be in place for most of each winter period. As easterly storms can be expected on a regular basis in the months of December to February, the concrete blocks would have to be left in place for a 3 month period and in severe winters possibly longer. At this stage it is possible that this option may prove to be unacceptable on amenity grounds. Amenity use of the promenade would be affected by their placement.

The most promising option as far as reduction of overtopping is concerned is the rock sill, built along the affected stretch of the promenade frontage. Because of the potentially high cost of this option, it is recommended that a simple design should be considered which would minimise the volume of rock needed to give the necessary protection to the seawall toe. It is recommended that only a 100m stretch of the frontage centred upon the café and adjacent buildings should be protected, monitored and extended at a later date, if necessary. The details of this scheme are outlined below (Section 4.9). The disadvantages of this option are the impacts on amenity usage of the frontage, by restricting access, producing a potential hazard to beach users, and perhaps appearing unattractive visually. These disadvantages can be minimised by keeping the crest level of the sill at about 2.0m ODN, when the natural shingle beach should cover it most of the time.

4.8 Evaluation of scheme benefits

The area between the Cowie Water and the village of Cowie is an important amenity asset to Stonehaven. It provides a focal point for beach usage as well as land based facilities (tennis courts, bowling greens, putting greens, etc). Parts of the area are also used as Caravan Parks and for other seasonal recreation activities. The promenade between the Cowie Water and the southern end of Cowie is popular with local residents and visitors as a place for walking and sightseeing activities. To consider undertaking works to improve the standard of flood protection along the promenade, the interaction of potential flood routes needs to be considered. For example, there would be little point in raising the standard of protection against wave overtopping (say by beach nourishment) if this encouraged blockage of the mouth of Cowie Water, thereby reducing the standard of protection against river flooding.

Based upon an inspection of the site there is a risk of land flooding resulting from waves overtopping the promenade wall in the vicinity of Beach Road (see Figure 1). The potential flood zone has not been evaluated but low lying land stretches northwards from the Cowie Water towards the southern outskirts of the village of Cowie. The area at greatest risk lies to the landward of the café and includes parts of the Caravan Park, several tennis courts, bowling greens and a putting green. How great a proportion of this zone may be affected by flooding can only be determined by a detailed land survey (see recommendations in Chapter 6). Within recent years the most severe overtopping took place in early 1996 when the southern part of the floodable zone was affected. The winter and spring of 1995/6 were exceptionally stormy and in the very much calmer winter of 1996/7 no serious overtopping occurred. Taking into account the increased instability of the wind and wave climate in recent years and a widely expected increase in mean sea level, it is likely that events which lead to flooding of this frontage will become more frequent and that the costs of flood damage will escalate. Also given the frequent use of the promenade “out of season” it is necessary to ensure that not only is the risk of flooding reduced but that the existing seafront walkway remains safe to use during the majority of frequently occurring storm events. This means that any scheme that deals with flooding should not only reduce the flood discharge severity and frequency of occurrence but that the possibility of anyone being hurt by pebbles hurled onto the promenade by waves should be taken into account.

In terms of direct damage caused by overtopping, an assessment has been made based on the flood damage that occurred in 1996. In that year the easterly storms persisted for a number of weeks and the public was thus better prepared for the situation. A number of buildings adjacent to the promenade were protected by sandbags etc and valuables were removed out of reach of floodwater.

Despite these precautions, waves overtopping the promenade during the storms of 1996 still caused significant damage. An appraisal of this damage was made as part of the present study. The main problems seem to have been as follows:

- seawater that accumulated on the promenade could not escape through the road drains that were blocked by shingle from the beach. As a consequence, water levels against the front walls of buildings at the rear of the promenade rose, and entered the cafe, flowing through it and out to the rear of the premises. The main damage was caused when water reached the level of the electricity sockets and/or the motors of the freezers within the property. One freezer had to be replaced, and there was flood damage to the walls and floors. The estimated repair/ replacement costs were in the region of £5,000.

- the impacts of overtopping water, and the accumulated volume, also damaged steel doors and caused flooding in the amusement arcade next to the cafe. Immediate damage was estimated at about £1000, although longer-term effects of corrosion of machinery were difficult to determine.

- the accumulated seawater on the promenade was sufficiently deep to prevent cars travelling through it. At least one vehicle, a police car, had to be pushed clear of the area because its engine failed, presumably by water affecting the car's electrical system or blocking the exhaust pipe. Such damage is hard to evaluate, but at the least is likely to cause long-term corrosion problems.

- seawater flowed into the grounds of the Bowls Club, partly flooding the putting green, but (fortunately) not affecting either the Bowling Greens or the artificial surface of the tennis courts.

- some seawater also flooded the open space to the north of the amusement arcade, including the car park in front of the leisure centre. Although parked cars were surrounding by floodwater, it appears that water levels did not cause significant damage to them, or reach the level of the doorstep to the leisure centre.

There was also a considerable effort spent by Council staff in clearing the promenade of shingle, unblocking drains and the like, on several occasions. The marginal cost to the Council of employees carrying out this work is again difficult to judge in financial terms.

It is probable, however, that a slightly worse event might have caused much greater economic losses, for example if flood waters reached the underside of parked cars at the Leisure Centre, or the surfaces of the Bowling Greens.

These direct costs are likely to have been accompanied by further, even less readily calculated losses, for example loss of trade and risks, e.g. the danger of people on the promenade suffering injury. However, it seems unlikely that a full evaluation of the damage actually caused in 1996 would produce a figure much greater than £10,000 - £15,000. Such a study itself would probably cost several thousand pounds. The economic benefit to the Council of such a study, in securing Grant Aid for improvements to any proposed improvement to the coastal defences is, in our opinion, dubious. A greater value, however, may arise when considering the potential future benefits of a coast protection scheme.

4.9 Dimensions of rock sill

From the above analysis, the most promising option for a scheme to remedy the wave overtopping along the promenade seawall at Stonehaven is a low rock sill built along the toe of the seawall. This will need to extend along about a 100m long frontage, with a crest level at 2.0m ODN, or slightly higher. In order to avoid, or at least minimise its impacts on amenity usage of the frontage, the existing access steps down the seawall will need to be preserved, and the proposed rock sill built in between these.

The slope of the front face of the rock sill should be steeper than that of the shingle beach, so that it is covered by beach material under most conditions. However, it is recommended that the front slope should not be too steep, since this will tend to reflect waves and cause greater scour when and if it becomes exposed. A slope of between 1:3 and 1:5 is suggested as a reasonable gradient fulfilling these criteria.

The size of the armour stone used should be kept to modest proportions, with the intention that any voids between them will be small, and pose no danger to beach users. In this context, we would expect most of the voids to become filled with shingle except perhaps in severe storm conditions. Based on this, and bearing in mind the potential problems of obtaining, delivering and placing rock armour, we recommend using 3-5 tonne rocks placed in a double layer. This layer should extend seaward about 5-8m, measured horizontally from the 2m ODN contour on the seawall face. Figure 3 is a sketch of the proposed rock sill, showing its suggested dimensions. Further work is still required on the detailed design of this sill, for example to decide on the details of the seaward toe of the sill, the need for any stone under-layers and/or geotextile blankets beneath it etc. This detailed design is beyond the scope of the present study.

5 River Carron – Channel training works

5.1 Background

The River Carron reaches the coast about 400m north of the harbour at Stonehaven. Along its lower reaches it flows through the southern part of the town in a rather narrow channel, between mainly residential properties. The discharge of floodwater to sea is hampered, especially at times of high tides, by both a footbridge with a low clearance over the water surface and by the deflection and partial siltation of its channel across the beach. As a consequence, flooding of properties occurs along its banks sufficiently often to be unacceptable.

The nett southerly drift of sand and shingle along the beach (see Chapter 3 above) continually alters the most seaward section of the river. Mechanical excavation of a straight deep channel is only successful in the short-term (weeks or even days) depending on weather conditions. Thereafter, the longshore drift shifts the channel southward and partially fills the channel, raising the invert level. The footbridge and a typical state of the channel are shown in Plates 4a and 4b respectively.

This section of the report reviews the possible methods of training the channel of the River Carron across the beach, to allow a better discharge of floodwater. It also makes some recommendations for simple modelling of the lower reaches of the river to check that the proposed improvements will produce a significant reduction in flood risk upstream from the river mouth.

5.2 Training works – general observations

The provision of one or more training walls to provide good discharge across a beach is an option often adopted for situations such as that at the mouth of the River Carron. Indeed, just such a structure has been built on the northern side of the mouth of the River Cowie further north along the Stonehaven town frontage.

There are three options for such training works, namely:

1 A single structure, e.g. a groyne or wall, on the up-drift side of the river channel, designed to intercept and retain beach material that would otherwise have deflected the channel and partially filled it.

2 A single structure as in option 1, but on the down-drift side of the river channel. The idea of this is to prevent the down-drift migration of the channel. The result of the longshore drift is then to “pinch” the river, deflecting it to run along the up-drift side of the training structure and reducing its width. As a consequence the currents are increased, improving the capacity of the river to remove sediment and maintain its channel cross-section.

3 Two training structures, one on either side of the desired channel position, thus preventing the down-drift deflection of the channel and reducing its siltation by material brought from up-drift by the longshore currents.

It is important to note that all three options interfere with the normal longshore sediment transport along the coast, and the following problems will typically arise:

1 The sediment accumulating up-drift of the river channel will eventually start to pass beyond the seaward end of the training works, typically forming a “bar” across the river channel that impedes the discharge of floodwater. Wave action may subsequently move sediment up the channel leading to more general siltation problems.

2 The beach down-drift of the river mouth will be deprived of sediment and will thus erode. For option 1 above, this increases the tendency for the river channel to migrate down-drift. For all options, the possibility of damage to, or flooding of, properties along the coastline immediately down-drift needs to be considered.

3 For options 1 and 3 above, sediment accumulating on the up-drift side of the training works may also cross over the crest of those works, leading to siltation of the river channel, and impeding the outflow of flood-waters.

The only sensible solution to these problems is the periodic transfer of beach sediments from one side of the river mouth to the other. The frequency and intensity of such operations depends on the efficiency of the training works, the rate of longshore drift, and the capacity of the river to transport and mobilise the beach sediments.

There are no hard and fast rules for the choice of an appropriate training option for any particular situation. Successful examples of options 1 and 2 can be quoted in apparently very similar situations. Option 3 is clearly likely to be more expensive than the other two, but equally is likely to be more efficient in maintaining a satisfactory channel.

5.3 Training works for the River Carron

At Stonehaven, the main considerations in designing suitable training works, apart from improving the entrance channel, are as follows:

- The upper part of the beach profile is a mixture of sand, gravel and stones. When this sediment enters the channel of the river, it is much more difficult to move seaward than sand on its own would be. As a consequence, the channel deflects southwards since the floodwaters cannot erode the deposited beach material. However, the coarser sediment does stand at a steeper slope than a sand beach, so that a shorter training wall is needed to reach, for example, the mean sea level contour.

- A sewer pipe, and a manhole cover, runs across the most direct route of the river channel to the sea, i.e. across the beach normal directly out from the river mouth. While the sewer pipe is itself below the required invert level of the river channel, it has sheet steel piles protruding above it that are not. Training the river channel along this direct route will therefore involve excavating and cutting off these piles.

- It is important to minimise the effects of beach erosion south of the river mouth, and to provide protection in this area against storm wave action. This certainly will require periodic “by-passing” of beach sediment across the river channel, i.e. collecting material from the area(s) in which it accumulates to the north and placing it on the beach further south, but other mitigation measures may also be worth considering.

- The form of construction of training works should be compatible with the character of the Stonehaven seafront, recognising the importance of the beach areas for recreation and tourism. Consideration needs to be given, in the design of any works, to the potential dangers to the public; features such as groynes and training works “attract” people, especially children, to walk on and explore them.

- The training works should also be of modest cost, and capable of modification in the light of their performance, maintenance requirements etc. In the absence of any firm guidelines, and because the present study is very brief, it is likely that the recommended option will not be the optimum. Some flexibility to extend or shorten, lower or raise the works in the light of experience would be sensible.

5.4 Recommendations for training works

Bearing in mind the general and local considerations regarding the design of training works, the following options have been considered for the training works at the mouth of the River Carron (see the sketches presented in Figure 4).

<u>Option 1(a) – Up-drift training wall (straight)</u>
This option (see Figure 4a) appears to be the most straightforward in principle. The coarse beach material will accumulate on the northern side of the training wall. Provided the wall extends to mean water level (0m ODN), and it is built about 1.0 - 1.5 m above the present beach profile, it should store the material transported along the beach over a few months.

The obvious form of construction is of random rock; using articulated concrete armour mattresses is not recommended because they are unlikely to withstand either abrasion by shingle or potential vandalism on this beach.

One obvious disadvantage of this option is that it would be necessary to excavate and cut off the sheet steel piles along the sewer pipeline route that crosses this channel route. Also, for this option the training wall and accumulated beach material will provide no protection from wave action to the down-drift frontage. The river channel is likely to move southwards in an uncontrolled manner as a consequence of this erosion (helped by any sediment moving over or around the seaward end of the wall). This may in turn add to the possibility of waves running over the coastal footpath, and perhaps damaging seafront properties.

<u>Option 1(b) – Up-drift training wall (curved)</u>
This option is similar to option 1a, with the same general advantages. However, by curving the line of the wall, as shown in Figure 4b, the need to excavate and cut off the sheet steel piles is avoided. In addition, the wall, and the sediment accumulating on its up-drift side, will provide some shelter from wave action to the area further south. The disadvantages, compared to option 1a are that the length of the wall to the mean water level will be greater, and the river channel will be curved, with some inevitable loss of discharge efficiency. This latter point, however, may be unimportant – see below. There will be a tendency for beach erosion down-drift, nevertheless, and the river channel would be free to deflect southwards as in Option 1a.

<u>Option 2 –Down-drift training wall</u>
This option (see Figure 4c) seems unlikely to be successful at his location. During periods of low river flows, it seems likely that the coarse beach material would virtually close off the river channel, and that subsequent high river flows might not be strong enough to re-create a deep, wide channel as desired.

<u>Option 3 – Up-drift training wall (curved) plus down-drift groynes</u>
This option is as Option 1(b) but with extra training works on the south side of the river channel. The problems of down-drift erosion will still remain and there will therefore still be a need for periodic by-passing of sediment across the river channel. It is also likely that some siltation across the channel at the seaward end of the training wall will occur, as noted previously.

Two sub-options can be considered. The first involves building a “twin” structure extending along all or part of the riverbank opposite the up-drift training wall. By acting as an offshore breakwater, this second wall will also provide some protection to the beach to its landward side. This is likely to be the most efficient option considered, but also the most expensive.

The alternative is to install “flow-deflecting” groynes (see Figure 4d), projecting into the river channel along its southern bank. These should prevent the deflection of the channel, and be cheaper to construct. They would also reduce the loss of beach sediment from the most sensitive stretch of the frontage south of the footbridge. However, this is a “novel” approach, and there are no clear guidelines for the design of such groynes. Some degree of “trial and error” is therefore likely to be involved to obtain an optimum groyne arrangement.

5.5 Recommendations for further actions

At this stage, we feel that the best option for improving the discharge of the River Carron would be to install the up-drift training wall, Option 1b, and monitor the beach and river channel. This single wall may reduce the tendency of the channel to meander by preventing coarse sediment entering it from the north. If this is the case, then the channel may be more cheaply maintained by periodic mechanical excavation, and hence the down-drift works may prove to be unnecessary.

It has been assumed in this chapter that training the outflow of the River Carron will lead to a better channel across the beach, and that this in turn will reduce the problems of flooding further upstream. Although this seems very likely to significantly improve the present situation, it is not certain that all flooding problems will be prevented by such a scheme. In the longer-term, therefore, it would be sensible to consider further improvements, as outlined below.

The footbridge across the mouth of the Carron is believed to impede flows and hence lead to increased river levels upstream at high tide when flood discharges are high. The proposed training walls will not significantly affect this situation. However, it is also conceivable that the footbridge may in some circumstances improve river levels upstream, i.e. by impeding the inflow of the tide.

It may also be that the very narrow channel of the river between the houses of Stonehaven may, at times of very high tides and high fluvial discharges, result in unacceptably high water levels even without the footbridge and the problems of the channel across the beach.

To investigate these possibilities, with a view to providing further safeguards against flooding, we would recommend some simple modelling of the lower reaches of the River Carron, using a one-dimensional numerical model such as ISIS. This will enable flow velocities and river levels to be determined. The modelling could be extended to allow the cross-sectional channel profile to minimise flooding to be ascertained, and/or to provide an estimate of the scouring capacity of the river flows to “self-cleanse” a channel.

For such modelling, information will be required on the following:

- The tidal “hydrograph”, i.e. the time history of the rise and fall of a tide, for a number of tidal ranges (e.g. a mean spring tide, highest astronomical tidal range, spring tide plus surge). This information can be obtained with satisfactory accuracy from tidal tables/ measurements.

- Measurements or estimates of flood hydrographs i.e. time histories of fluvial discharges. It is believed there may be a flow-gauging station on the River Carron, from which information on “typical” and extreme flows may be obtained.

- Measurements of cross-sections at a number of points along the course of the river, starting at its seaward end (i.e. of the channel across the beach), and proceeding upstream, at about 100m intervals, levelled to ODN. These can be of rather “crude” accuracy, e.g. invert levels, width at bed, bank side slopes, for the modelling proposed here.

- Information on bridges (e.g. level of undersides) and other obstructions to flow, e.g. weirs, at the relevant points along the river course.

In addition to the above, measurements of flood discharges from the gauging station, together with river levels (especially either side of the footbridge) during times of high tide and substantial fluvial discharges would be useful. This would provide validation data for the modelling.

6 Conclusions and recommendations

The first element of this study investigated why the promenade seawall in Stonehaven Bay is occasionally seriously overtopped. When this overtopping occurs, the results can be quite dramatic. Not only is shingle deposited in large quantities on the roadway, blocking the drains, but also several commercial premises on the landward side of the road are affected by floodwater. In extreme cases the amenity land, together with several properties may become flooded. While this problem may occur two or three times in a winter there may be some years when no damaging overtopping is experienced.

Under normal conditions beach levels in front of the seawalls are healthy and topographic surveys made by Aberdeenshire Council show that levels in front of the promenade wall are usually about 1m above the level of mean high water of spring tides (i.e. between 2m and 3mODN).

Analysis of tidal records show that the 50-year water level is predicted at 3.02mODN, reducing to 2.85mODN for the 10-year extreme value. When beach levels are at their highest (3mODN) the amount of wave overtopping will be minimal. However, beach levels only have to fall by 1 metre for wave overtopping to become dangerously high. Under these conditions any sensible raising of the wall crest (by 0.5m to 1m) does not reduce the overtopping to an acceptable level.

Because of the sensitivity of the rates of overtopping discharge to maximum beach levels the most obvious solution to protect the backshore against unacceptable flooding is to ensure that these levels do not fall below a threshold value. This can be achieved by maintaining a high beach and ensuring beach drawdown does not affect the crest, or by armouring the beach with rock, for example.

A consideration of conditions along the rest of the Stonehaven frontage shows that maintaining levels by means of beach nourishment is not a sensible option. Recharging the beach with pebbles would increase the potential southward transport of beach material, tending to cause blockage at the mouth of the Cowie Water. (Recharging the beach with sand would do little to improve beach levels other than on the lower foreshore.)

A fall in beach level at the toe of the promenade seawall can best be prevented by the construction of a rock berm. This could be formed from two layers of rock with a median weight of 3-5 tonnes laid at a slope of about 1:4 and extending approximately 8-10m seawards from the toe of the wall. The crest of the sill could be set at 2mODN, which is the lowest recorded level at the toe of the wall. The sill would then be covered by shingle for most of the time, thus not seriously affecting the amenity value of the beach.

The construction of the rock berm should be carefully targeted and only the frontage from about 100m to about 200m north of the Cowie Water terminal groyne needs to be protected in this way at present. Elsewhere the beaches are presently healthy and extension of the rock berm either northwards or southwards would be difficult to justify.

The proposed sill will help dissipate wave energy when beach levels fall. This will help reduce wear and tear of the concrete steps. The sill will also tend to improve the shingle beach at the seawall toe, and hence help to reduce the water depths at the face of the wall, reducing both wave overtopping and the accumulation of pebbles on the promenade. Another advantage is that the wall will be subject to less severe wave impact forces.

It is doubtful whether the scheme proposed will be eligible for grant aid. Beach levels are presently healthy and the frequency of serious flooding and associated damage is small. There is therefore all the more reason why beach levels should be carefully monitored. The backshore area should also be carefully surveyed to determine the flood “footprint”. The land should be contoured at 0.25m elevation increments to accurately delineate the flood risk zone. Further studies, including a cost-benefit analysis are needed before any scheme can be justified on present day information.

The second issue considered in this report was the flooding of the lower reaches of the River Carron. It seems likely that these problems are largely caused by the low footbridge at the mouth of the river, and the blockage and deflection of its channel across the beach by sand and shingle. A number of alternative training works have been considered to reduce the latter effect, based on experience of similar problems elsewhere in the U.K. A curved “up-drift” training wall is concluded to be the most cost-effective solution to this problem, but must be accompanied by periodic mechanical “by-passing” of beach material from the north to the south of the river.

In the longer-term, further improvements to the discharge of the river might be investigated, for example further training works along the southern side of its channel, and raising the footbridge. The need for more improvements can best be judged after experience has been gained of the efficiency of the proposed new training wall. It is recommended that if further work on the River Carron is proposed, a simple numerical model of its lower reaches should be commissioned to ensure that any works would have the desired effect.

Finally, on a more general theme, managing beaches is an important and cost-effective component of coastal defence here as along most parts of the U.K. coastline. The frontage in Stonehaven Bay needs to be monitored annually and after particularly severe storms. Beach monitoring should be carried out after each survey, flagging up any areas where the beach crest falls below 2mODN. This monitoring should include the southern half of the Bay. This area has a wide shingle beach and the risk of flooding is presently small. Should beach levels fall, however, the defences would be severely overtopped. The mouth of the Cowie Water tends to be blocked with shingle. This material could be dredged and usefully placed on the upper beach in the southern half of the Bay, thereby helping to maintain a healthy beach. Blockage of the river mouth could be avoided by building a groyne on the south side also. Such a structure might affect the redistribution of beach deposits in the southern half of the bay. This potential future management technique could be examined by means of modelling the planshape evolution of the beach. Modelling could be made using a model similar to HR's BEACHPLAN model (see Appendix 3).

7 References

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Tables

Table 1 Offshore wave heights and periods

Wave heights and periods, 1/1/92 - 31/12/96 at UKMO point 56.75N 2.07W

Data in parts per hundred thousand
Hs is the significant wave height in metres
P(H>H1) is the probability of Hs exceeding H1

Annual

Total number of hours = 43848
Based on UKMO predictions for January 1992 - December 1996

H1	To H2	P(H>H1)	Mean wave period in seconds (Tm)															
			0.0 1.0	1.0 2.0	2.0 3.0	3.0 4.0	4.0 5.0	5.0 6.0	6.0 7.0	7.0 8.0	8.0 9.0	9.0 10.0	10.0 11.0	11.0 12.0	12.0 13.0	13.0 14.0	14.0 15.0	15.0 16.0
0.00	0.50	0.99685	0	0	0	8703	3099	623	178	27	14	21	14	14	7	0	7	0
0.50	1.00	0.86980	0	0	0	16947	10612	3318	807	253	68	7	0	0	0	0	0	0
1.00	1.50	0.54967	0	0	0	4064	16270	3633	1204	171	27	7	21	0	0	0	0	0
1.50	2.00	0.29570	0	0	0	205	5015	7423	910	130	14	0	0	0	0	0	0	0
2.00	2.50	0.15873	0	0	0	0	89	6069	1136	123	14	14	0	0	0	0	0	0
2.50	3.00	0.08429	0	0	0	0	0	766	2928	157	21	21	14	0	0	0	0	0
3.00	3.50	0.04522	0	0	0	0	0	21	1854	260	14	14	0	0	0	0	0	0
3.50	4.00	0.02360	0	0	0	0	0	0	287	1054	41	0	0	0	0	0	0	0
4.00	4.50	0.00978	0	0	0	0	0	0	7	575	0	0	0	0	0	0	0	0
4.50	5.00	0.00397	0	0	0	0	0	0	0	89	130	0	0	0	0	0	0	0
5.00	5.50	0.00178	0	0	0	0	0	0	0	7	130	7	0	0	0	0	0	0
5.50	6.00	0.00034	0	0	0	0	0	0	0	0	21	7	0	0	0	0	0	0
6.00	6.50	0.00007	0	0	0	0	0	0	0	0	7	0	0	0	0	0	0	0
Parts per thousand for each wave period			0	0	0	299	351	219	93	28	5	1	0	0	0	0	0	0

Table 2 Offshore wave heights and directions

Wave climate at UKMO point 56.75N 2.07W

Data in parts per hundred thousand
Hs is the significant wave height in metres
P(H>H1) is the probability of Hs exceeding H1

Annual

Total number of hours = 43848
Based on UKMO predictions for January 1992 - December 1996

H1	To H2	P(H>H1)	-10 to 10	10 to 30	30 to 50	50 to 70	70 to 90	90 to 110	110 to 130	130 to 150	150 to 170	170 to 190	190 to 210	210 to 230	230 to 250	250 to 270	270 to 290	290 to 310	310 to 330	330 to 350
			Wave direction in degrees North																	
0.00	0.50	0.99685	472	1320	2449	1485	992	903	1040	821	452	424	520	445	328	198	192	205	267	192
0.50	1.00	0.86980	1286	2271	5439	2217	2135	2230	1875	1861	1300	1423	1177	1327	1320	1485	1389	1163	992	1122
1.00	1.50	0.54967	1286	1704	3701	1327	1341	1533	1471	1444	1444	1389	1464	1690	1587	1197	787	759	527	746
1.50	2.00	0.29570	390	766	1690	609	725	965	876	978	937	828	1143	1341	1184	541	301	192	82	151
2.00	2.50	0.15873	123	342	705	294	431	623	541	794	541	568	759	650	595	205	130	62	41	41
2.50	3.00	0.08429	55	233	205	151	212	465	472	431	376	376	287	260	205	55	34	48	21	21
3.00	3.50	0.04522	14	82	123	103	246	342	308	192	171	151	198	137	27	7	27	7	7	21
3.50	4.00	0.02360	0	34	82	96	164	356	144	164	123	82	55	62	21	0	0	0	0	0
4.00	4.50	0.00978	0	0	7	14	82	205	62	103	27	27	21	27	7	0	0	0	0	0
4.50	5.00	0.00397	0	0	0	7	14	62	62	62	7	0	0	0	7	0	0	0	0	0
5.00	5.50	0.00178	0	0	0	7	0	62	34	27	0	14	0	0	0	0	0	0	0	0
5.50	6.00	0.00034	0	0	0	7	0	21	0	0	0	0	0	0	0	0	0	0	0	0
6.00	6.50	0.00007	0	0	0	0	0	7	0	0	0	0	0	0	0	0	0	0	0	0
Parts per thousand for each direction			36	68	144	63	63	78	69	69	54	53	56	59	53	37	29	24	19	23

Significant wave heights for given exceedence levels

P(H>Hs)	All dir.	-10 to 10	10 to 30	30 to 50	50 to 70	70 to 90	90 to 110	110 to 130	130 to 150	150 to 170	170 to 190	190 to 210	210 to 230	230 to 250	250 to 270	270 to 290	290 to 310	310 to 330	330 to 350
		Wave direction in degrees North																	
0.50	1.10	1.02	0.95	0.94	0.88	1.02	1.25	1.18	1.26	1.32	1.29	1.38	1.35	1.31	1.07	0.95	0.94	0.85	0.93
0.20	1.85	1.44	1.57	1.49	1.52	1.92	2.47	2.23	2.25	2.16	2.14	2.13	1.98	1.92	1.56	1.45	1.38	1.28	1.35
0.10	2.39	1.78	2.02	1.91	2.08	2.70	3.40	2.92	2.84	2.72	2.66	2.50	2.42	2.28	1.91	1.84	1.67	1.46	1.51
0.05	2.94	2.04	2.52	2.29	2.72	3.38	3.95	3.43	3.54	3.17	3.03	2.99	2.86	2.51	2.20	2.19	1.99	1.83	1.89
0.02	3.63	2.48	2.96	2.82	3.52	3.91	4.49	4.16	4.26	3.70	3.61	3.41	3.39	2.89	2.47	2.56	2.56	2.36	2.44
0.01	3.99	2.79	3.30	3.28	3.85	4.20	5.09	4.72	4.66	3.92	3.93	3.67	3.74	3.16	2.73	2.98	2.82	2.69	2.94
Average	1.27	1.06	1.08	1.05	1.06	1.29	1.58	1.43	1.48	1.48	1.45	1.47	1.44	1.37	1.15	1.08	1.03	0.92	1.00

Table 3 Overtopping of promenade wall at MHWS

Still water level (mODN)	Return wall height (mODN)	Beach level (mODN)	Predicted overtopping Qm^3/s/m	Comments
2.05	6.8	2.0	0.36×10^{-5}	Wet but not uncomfortable
2.05	7.3	2.0	0.26×10^{-5}	Wet but not uncomfortable
2.05	7.8	2.0	0.13×10^{-5}	Wet but not uncomfortable
2.05	6.8	1.5	0.02	Dangerous for pedestrians
2.05	7.3	1.5	0.014	Dangerous for pedestrians
2.05	7.8	1.5	0.007	Dangerous for pedestrians
2.05	6.8	1.0	0.073	Very dangerous for pedestrians
2.05	7.3	1.0	0.053	Very dangerous for pedestrians
2.05	7.8	1.0	0.026	Very dangerous for pedestrians

Table 4 Overtopping of promenade wall at MHWS – shingle ramp

Still water level (mODN)	Return wall height (mODN)	Beach level (mODN)	Predicted overtopping $Qm^3/s/m$	Comments
2.05	6.8	2.0	0.07×10^{-7}	Wet but not uncomfortable
2.05	7.3	2.0	0.05×10^{-7}	Wet but not uncomfortable
2.05	7.8	2.0	0.02×10^{-7}	Wet but not uncomfortable
2.05	6.8	1.5	0.35×10^{-2}	Dangerous for pedestrians
2.05	7.3	1.5	0.25×10^{-2}	Dangerous for pedestrians
2.05	8.3	1.5	0.12×10^{-2}	Dangerous for pedestrians
2.05	6.8	1.0	0.022	Dangerous for pedestrians
2.05	7.3	1.0	0.016	Dangerous for pedestrians
2.05	8.3	1.0	0.008	Dangerous for pedestrians

Note: Slope of shingle ramp is 1:3, extending seawards from the level of the top step of the wall.

Table 5 Overtopping of promenade wall at HAT

Still water level (mODN)	Return wall height (mODN)	Beach level (mODN)	Predicted overtopping $Qm^3/s/m$	Comment
2.47	6.8	2.0	0.022	Structural damage to buildings
2.47	7.3	2.0	0.013	Structural damage to buildings
2.47	7.8	2.0	0.006	Structural damage to buildings
2.47	6.8	1.5	0.092	Damage to promenade possible
2.47	7.3	1.5	0.057	Structural damage to buildings
2.47	7.8	1.5	0.026	Structural damage to buildings
2.47	6.8	1.0	0.183	Damage to promenade possible
2.47	7.3	1.0	0.113	Structural damage to buildings
2.47	7.8	1.0	0.052	Structural damage to buildings

Figures

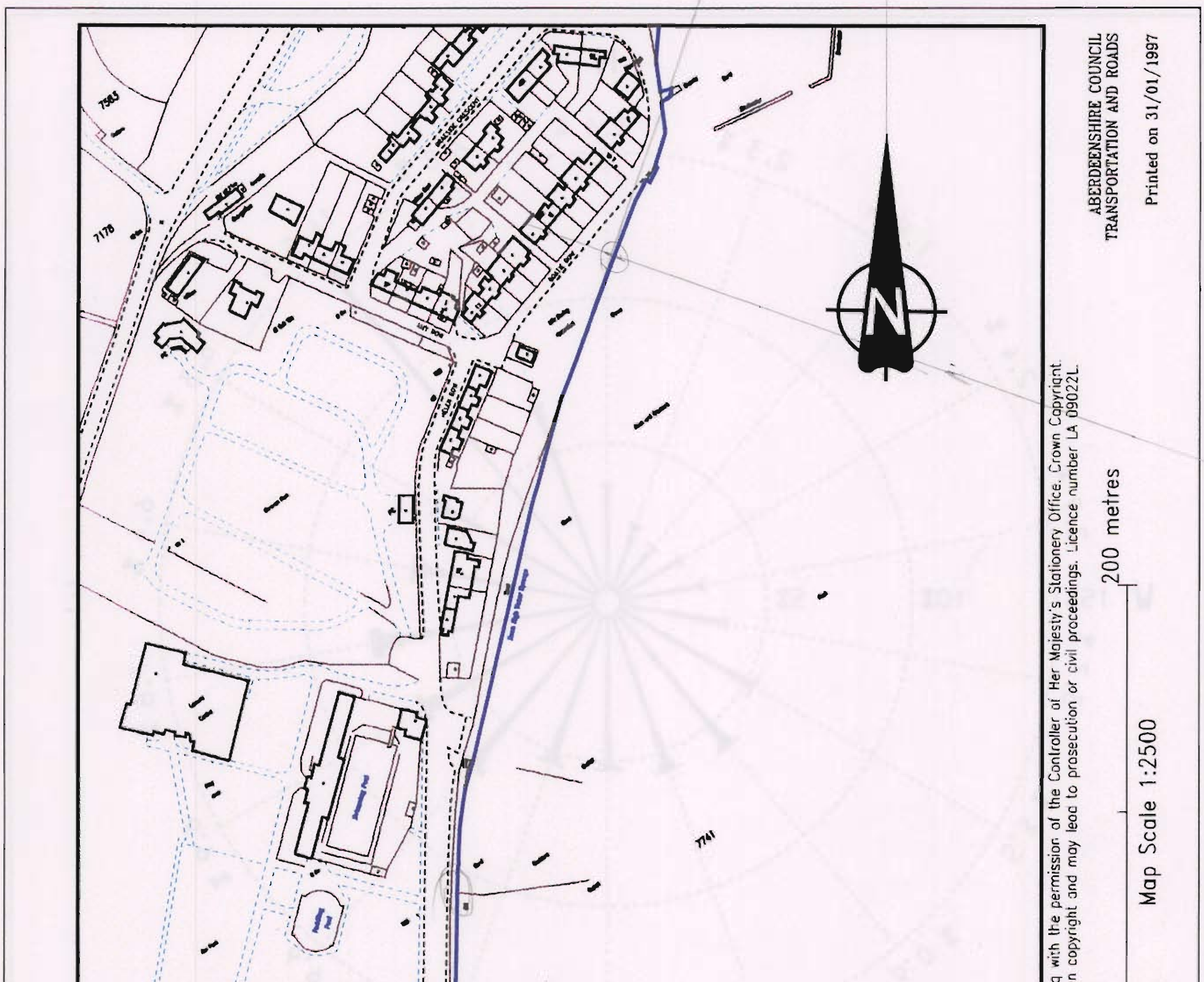


Figure 1 Location map

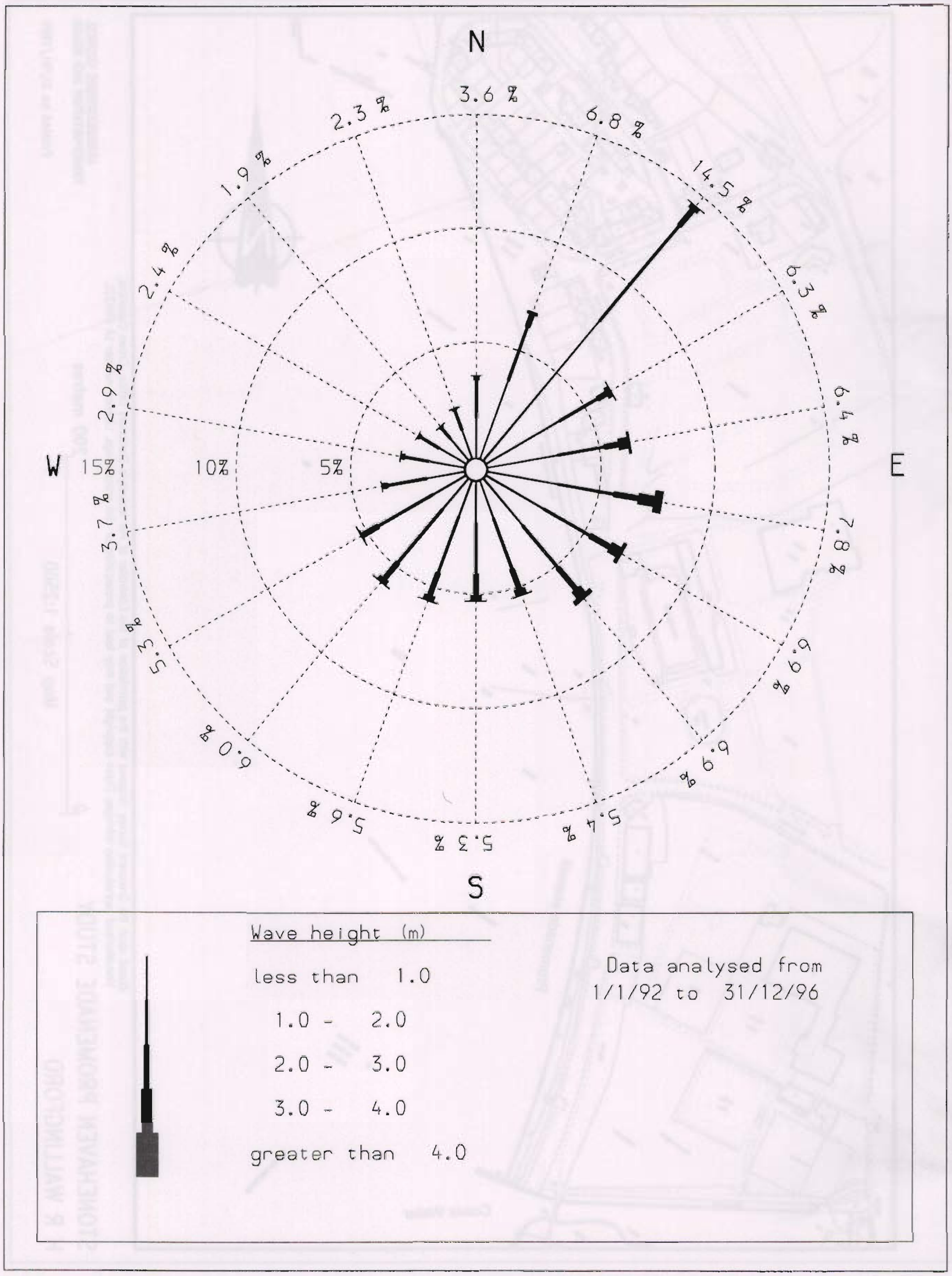


Figure 2 Wave rose at UKMO point 56.75N 2.07W

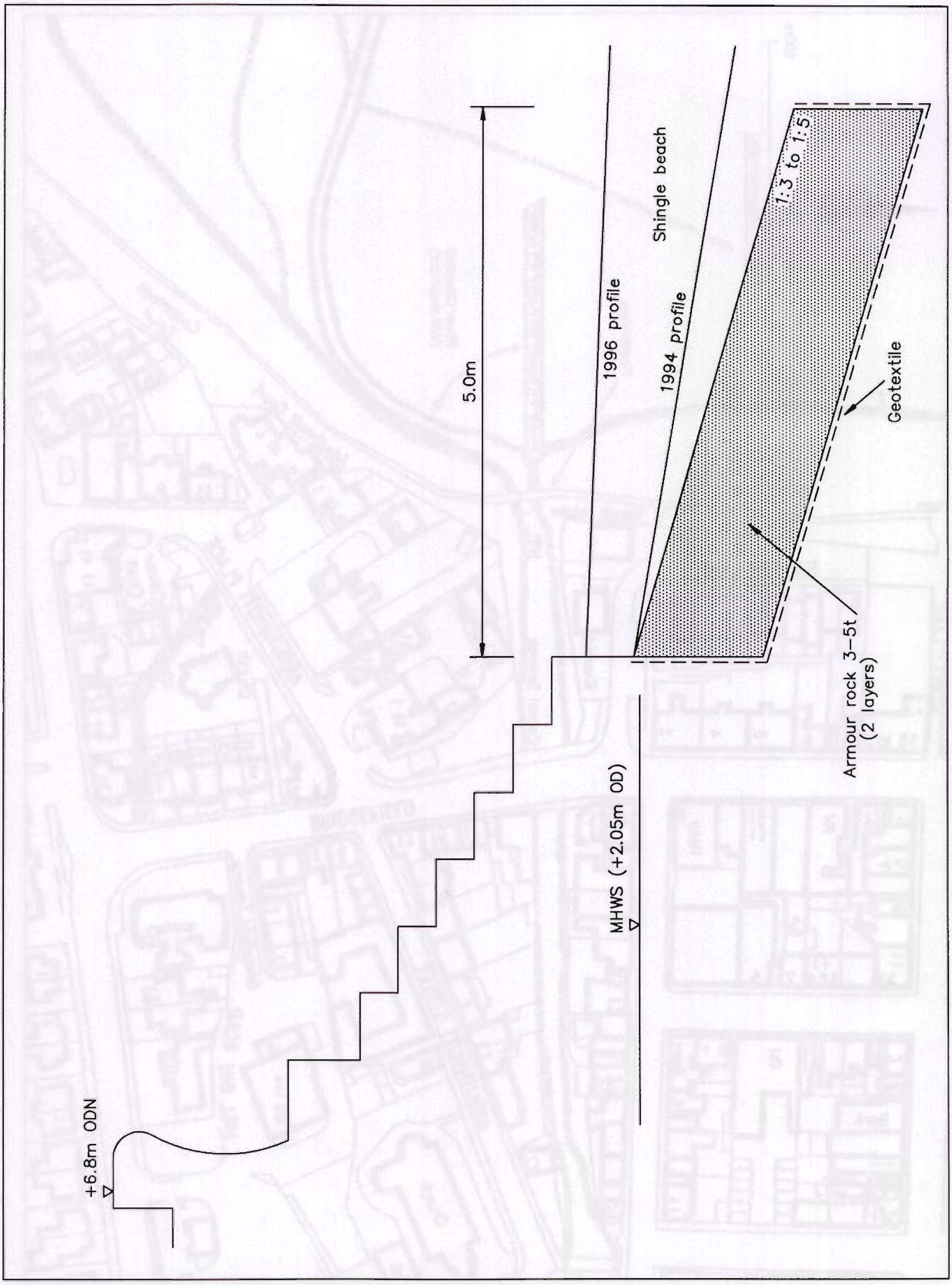


Figure 3 Outline design of rock sill

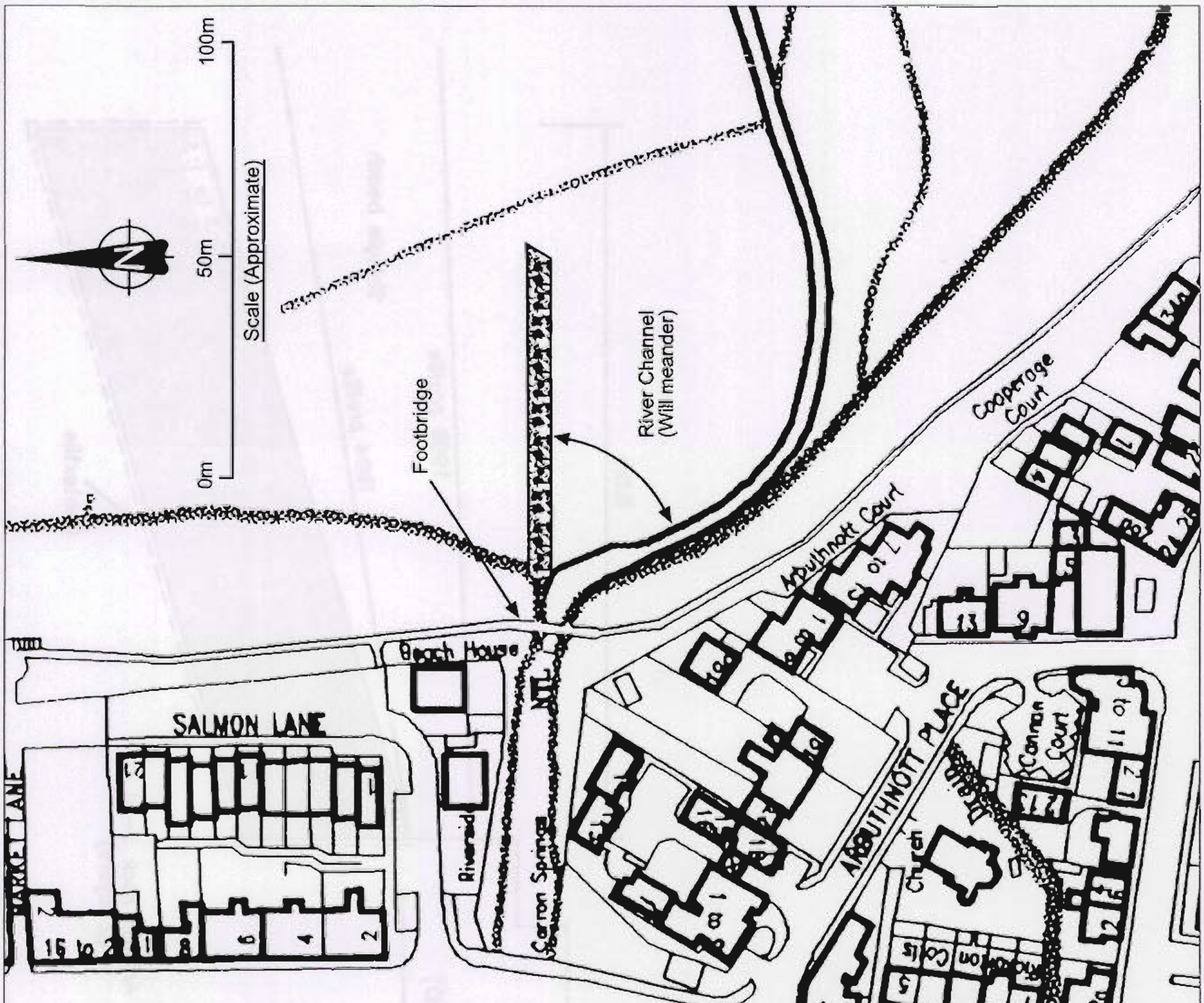


Figure 4a Option 1a - Straight updrift training wall

Figure 4b **Option 1b - Curved updrift training wall**

Figure 4c **Option 2 - Downdrift training wall**

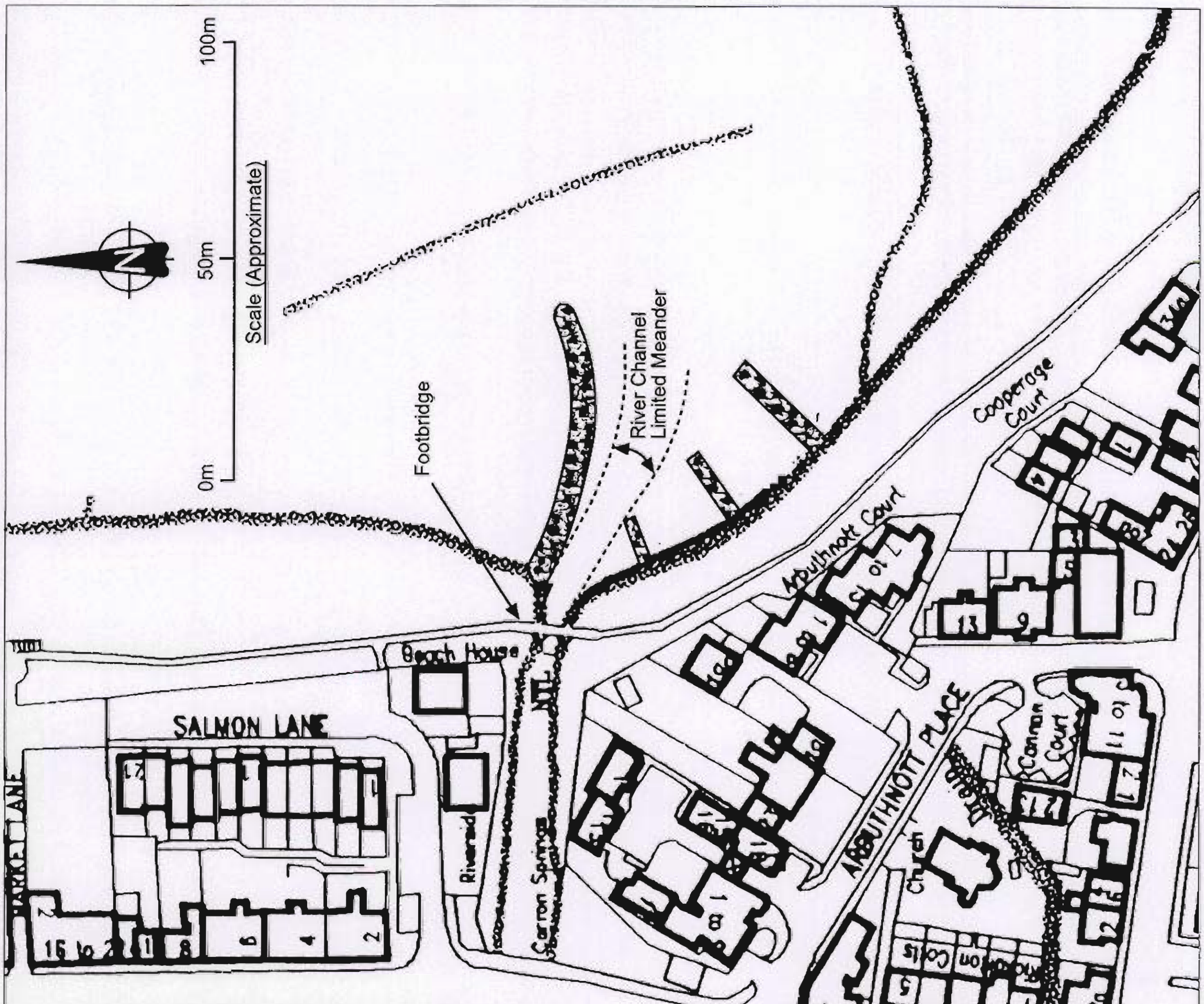


Figure 4d **Option 3b - Curved updrift training wall and downdrift groynes**

Plates

Plate 1a Overtopping of seawall – early 1996

Plate 1b Overtopping of seawall – early 1996

Plate 2 Accumulation of shingle on promenade – early 1996

Plate 3a Healthy shingle berm during calm weather

Plate 3b Shingle build up on seawall steps during stormy weather

Plate 4a Footbridge at mouth of River Carron

Plate 4b Southward deflection of channel – River Carron

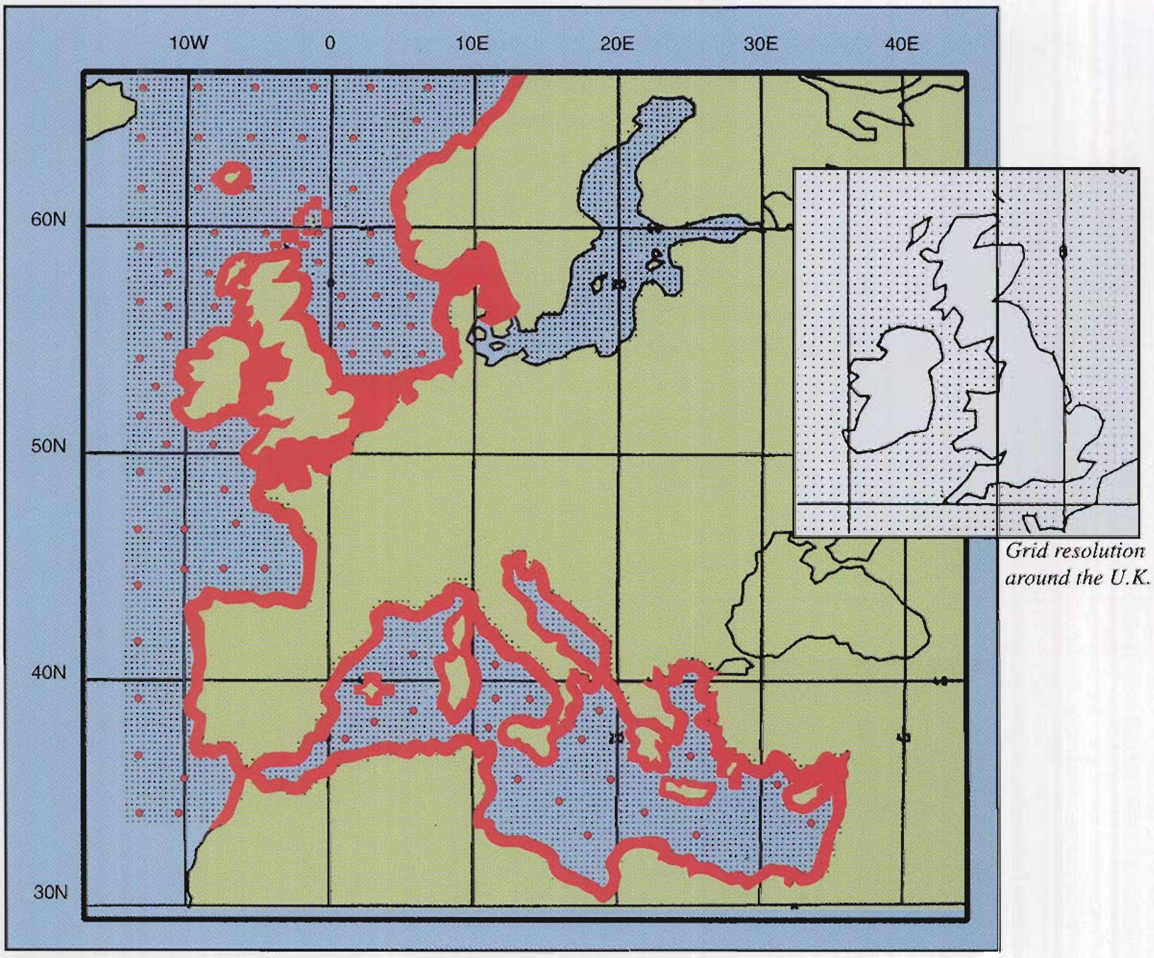
Appendices

Appendix 1

UK Met Office European Wave Model

UK Met Office European Wave Model

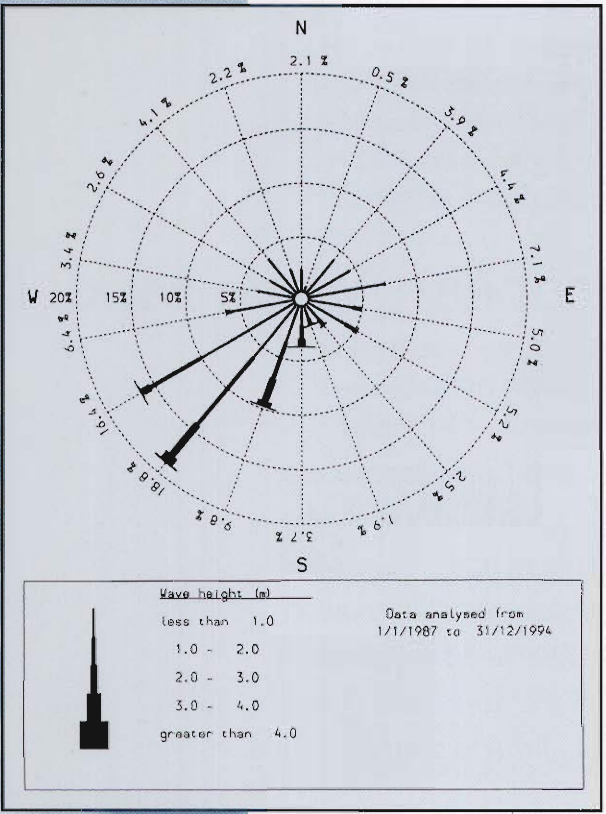
The UK Met Office European Wave Model covers European waters on a 25-30km grid. The model archive provides good quality synthetic sequential wind and wave data, in consistent format, from October 1986 onwards. It is a reliable and inexpensive source of information on deep water wave conditions and over-water wind conditions. HR Wallingford holds a copy of the archive for the areas and points shown in red below, and is licensed by the Met Office to prepare and supply wind and wave data analyses.



Grid resolution around the U.K.

Model grid indicating locations of data held by HR Wallingford

HR Wallingford



Wave rose

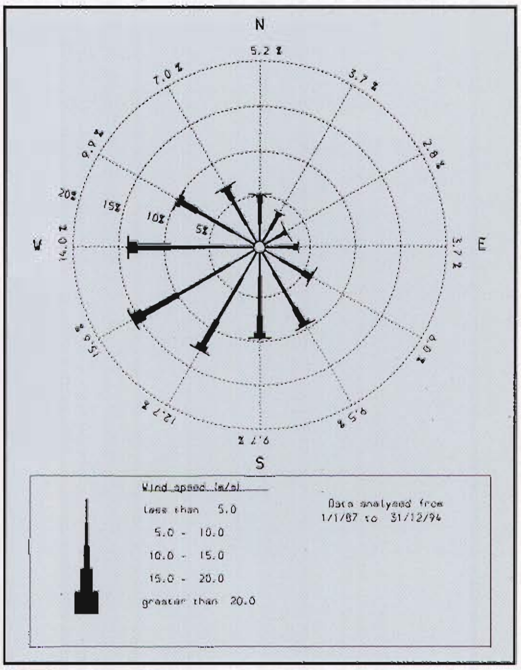
National meteorological agencies began to operate spectral wave prediction models in the mid 1970's. Effectively the models run continuously in real-time, taking as input spatially and temporally varying surface wind field data from weather forecasting models.

The grid systems for some of these models cover all of the Earth's oceans. Different components of the wave energy spectrum, for example wind-sea and swell, can grow, propagate, and decay, naturally and separately, each responding realistically to changes in the wind fields.

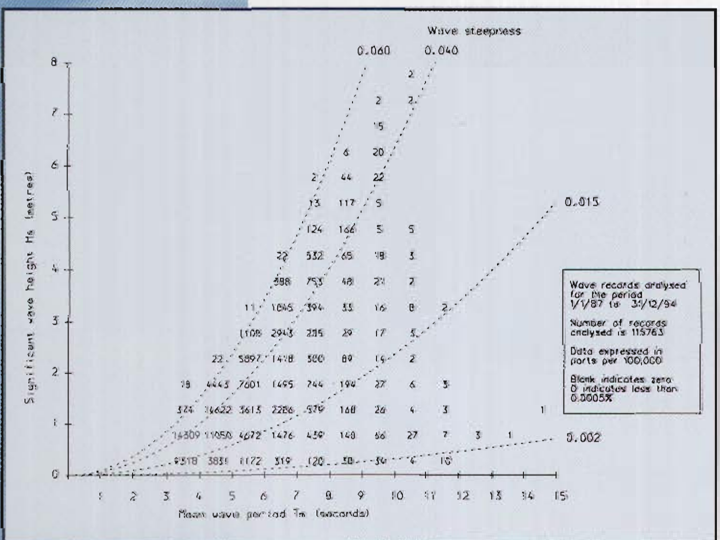
The UKMO Global and European Wave Models are known as *second generation* wave models because of the way the non-linear interactions in the wave spectrum are parameterised. In each 30 minute time step, the wave energy components move in their respective directions through the grid. The significant wave height (H_s), mean wave period (T_m) and direction (θ_m) at each grid point can be integrated out from a directional spectrum which has 16 direction and 13 frequency components.

The two surface wave models at present run on an operational basis at the UKMO are the result of an evolving series of such models, in use since 1976. The model calculations are carried out on a polar stereographic grid, whose exact spacing therefore varies from one latitude to another.

The European Wave Model (grid spacing 25-30km) is nested within the Global Model (grid spacing about 150km) from which it takes its boundary wave conditions.



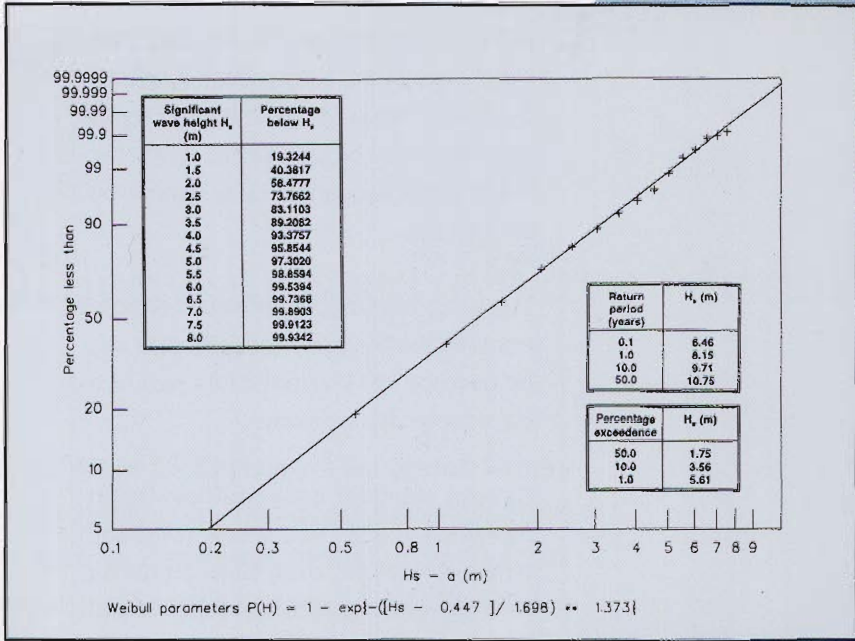
Wind rose



Wave height and mean period - Scatter diagram

The models are run twice daily, driven by wind fields extracted from operational global weather forecasting models. They produce wave forecasts from 12 hours prior to the datum time (T) up to 36 hours ahead, at 3 hourly intervals.

As well as noting the time, date, latitude and longitude, each forecast gives the wind speed and direction, and H_s, T_m and θ_m for the separate wind-sea and swell components and overall. The data from T-12 hours to T+0 hours is permanently stored in an archive, whilst the data from T+0 hours to T+36 hours is immediately disseminated for forecasting purposes.

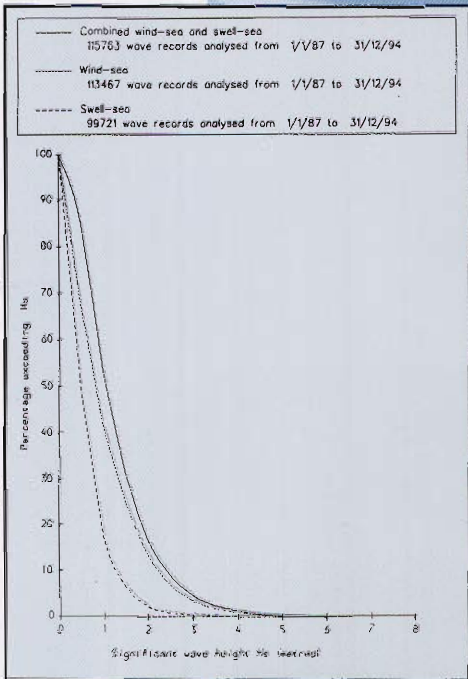


Weibull plot for deriving extreme wave heights

Sea state observations from fixed buoys, oil platforms, Ocean Weather Ships, and more recently satellite measurements, are used for real-time "calibration" of the models, and also for periodic validation exercises.

The European Model has been run in its present configuration since October 1986. The archive contains records at 6 hourly intervals from October 1986 to July 1988, and at 3 hourly intervals thereafter

The volume, resolution and consistency of results available in these archived hindcasts provides a good database of wave information for use in engineering studies. However, the spatial resolution and model physics mean that the data can only be considered representative of deep water conditions at least 20km offshore.



Wave height exceedence curves

Data in parts per hundred thousand
Hs is the significant wave height in metres
P(H>H1) is the probability of Hs exceeding H1

Total number of hours = 70128
Based on UKMO wave model predictions for January 1987 - December 1994

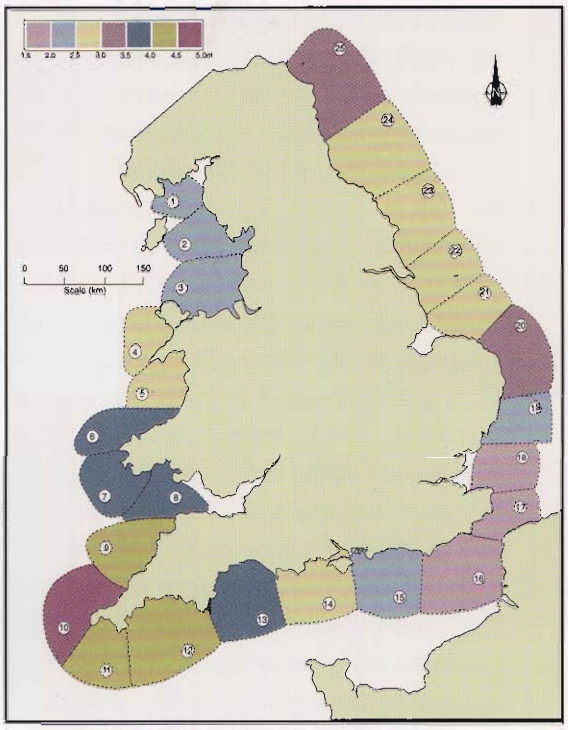
H1	To H2	P(H>H1)	Wave direction in degrees North											
			-15 15	15 45	45 75	75 105	105 135	135 165	165 195	195 225	225 255	255 285	285 315	315 345
0.00	0.50	0.98722	76	58	36	21	21	23	96	104	225	268	130	145
0.50	1.00	0.97519	1687	1198	278	275	335	506	719	1109	2872	2587	1617	1483
1.00	1.50	0.82853	2669	1369	331	217	610	1253	1606	1697	3083	1996	1367	1654
1.50	2.00	0.65000	2505	1176	275	183	459	1253	1584	1802	2813	1620	1112	1413
2.00	2.50	0.48802	1781	994	234	164	475	991	1308	1648	2487	1149	794	1209
2.50	3.00	0.35568	1236	607	100	123	351	943	1000	1420	1945	793	513	925
3.00	3.50	0.25612	766	396	60	84	167	810	841	1119	1654	510	215	518
3.50	4.00	0.18471	609	271	17	76	197	652	660	1038	1248	299	144	262
4.00	4.50	0.12998	419	161	6	51	155	491	589	790	964	180	73	170
4.50	5.00	0.08949	315	94	9	29	111	389	442	632	789	91	47	113
5.00	5.50	0.05889	170	101	13	9	134	382	338	476	476	71	30	83
5.50	6.00	0.03606	57	44	0	0	56	281	238	274	284	40	10	23
6.00	6.50	0.02300	39	13	0	0	40	191	195	187	180	16	11	30
6.50	7.00	0.01399	41	10	0	0	21	56	93	163	98	19	11	14
7.00	7.50	0.00873	29	1	0	0	14	39	70	76	103	7	4	13
7.50	8.00	0.00518	41	1	0	0	10	21	31	31	48	1	10	10
8.00	8.50	0.00311	23	4	0	0	4	16	24	54	33	0	0	1
8.50	9.00	0.00151	20	0	0	0	0	7	17	11	11	0	6	3
9.00	9.50	0.00076	21	0	0	0	0	4	4	11	4	0	0	1
9.50	10.00	0.00029	6	0	0	0	0	0	0	6	0	0	1	0
10.00	10.50	0.00016	6	0	0	0	0	0	0	6	0	0	0	0
10.50	11.00	0.00004	4	0	0	0	0	0	0	0	0	0	0	0
Parts per thousand for each direction			125	65	14	12	32	83	99	127	193	96	61	81

Wave height and mean direction - Scatter diagram

HR Wallingford is an "Authorised Data User", licensed to hold and apply UKMO European Wave Model Data. Analysis of the data can be supplied to clients but the original sequential data cannot be sent off site.

The analyses can be purchased directly from HR Wallingford and/or the data can be used by HR Wallingford as part of a larger consultancy study.

Several standard analyses have been developed, e.g. for wave (or wind) climates, wave (or wind) roses, extremes, swell, persistence, and particular storms. Results can be smoothed over an area, and can be presented separately for different years, different seasons and/or different direction sectors.



1 year return period H_s for swell waves

The archive presently contains data from October 1986 to December 1995. Annual updates are added each January.

HR Wallingford is a UK registered, independent company limited by guarantee. The Company offers high technology applied research, specialist consultancy, software, equipment sales, training and technology transfer services across the entire spectrum of civil engineering and environmental hydraulics. As well as being the UK national centre of expertise in hydraulics and aquatic environmental modelling, HR has an international reputation for scientific and engineering excellence having worked in more than sixty countries in the last ten years.

Based at Wallingford, near Oxford, UK, the Company has a staff of over 250 including engineers, scientists, mathematicians and support staff. The HR Wallingford Group also has offices in Belgium, Hong Kong and Malaysia.

For further information, please contact Dr. Peter Hawkes (e-mail: pjh@hrwallingford.co.uk) of the Coastal Group at HR Wallingford.

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Appendix 2

The SWALLOW seawall overtopping model

Appendix 2 The SWALLOW seawall overtopping model

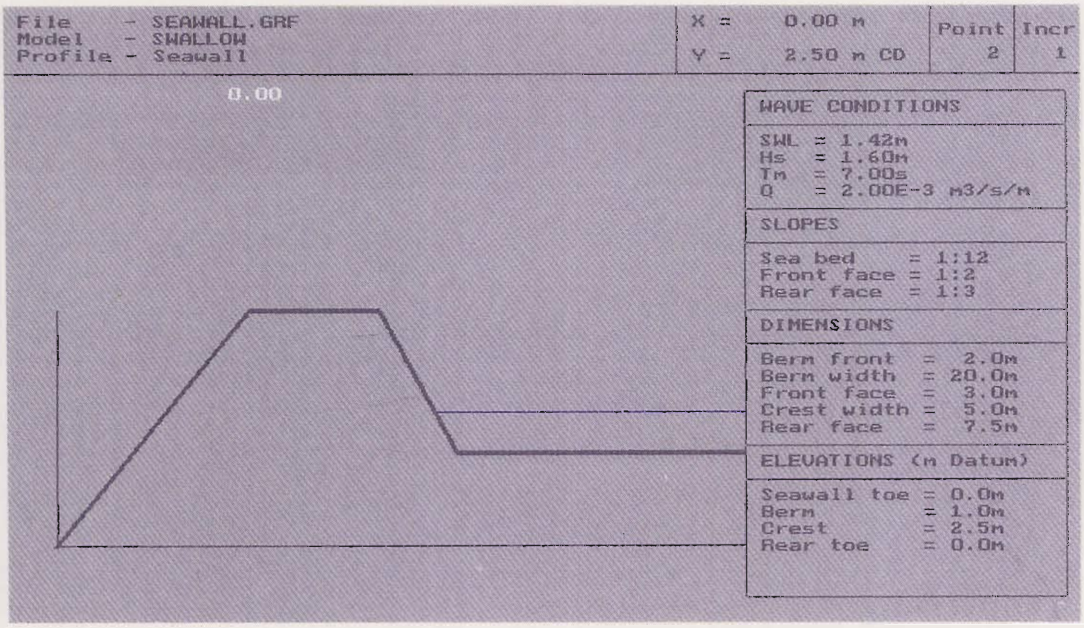
HR Wallingford

Software

SWALLOW

Sea Wall Overtopping by Waves

Swallow is a PC- based package for calculating sea wall overtopping by waves, based on the results of extensive tests conducted at HR Wallingford since 1977. A wide variety of sea wall geometries and wave conditions provide a reliable database for overtopping predictions. **Swallow** has been used successfully on many site specific studies worldwide and applied to research into sea level rise for the UK Government.



SWALLOW capabilities:

- applicable to embankment-type sea walls or revetments with a sloping seaward face
- includes effects of horizontal berms, wave return walls and roughness of the seaward face
- calculates overtopping discharge for specified fixed water level and wave conditions
- calculates total overtopping volume for a specified tidal cycle

SWALLOW features and benefits

- easy to-use graphical interface
- comprehensive documentation
- no training required
- on-going development programme to include latest research results

Example SWALLOW applications:

- Chek Lap Kok, Hong Kong
- Dabhol, India
- Cardiff Barrage UK
- Kuala Lumpur, Malaysia

For further information on **Swallow** please contact:

Gavin Eadie, Software Sales Manager, HR Wallingford, Wallingford, Oxon, OX10 8BA, UK
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Appendix 3

The BEACHPLAN sediment transport model

Appendix 3 The BEACHPLAN sediment transport model

Introduction
BEACHPLAN is a state-of-the-art model that simulates the beach evolution in plan shape. It was developed in HR Wallingford 20 years ago and has been in continuous development since, to become one of our most important tools on beach protection studies. BEACHPLAN uses a formulation of total longshore transport rate based on the widely used CERC formula. The model changes the coastline every time step, allowing for the correct simulation of the changing drift rates with time. BEACHPLAN models the following processes:

- Wave transformation:
  refraction,
  shoaling,
  diffraction

- Structures:
  transmission through structures,
  bypassing of groynes and breakwaters,
  effect of seawalls on the sediment transport.

- Sediment transport:
  CERC formula,
  longshore drift due to alongshore variation of breaking wave height,
  cross-shore distribution of the longshore drift,
  limited toe of the beach.

- Active beach management techniques:
  beach renourishment,
  beach mining.

The beach plan shape is specified by the position of a single contour, usually either Mean Water Level or a particular high-tide level. The model assumes an average beach slope and does not consider short-term changes in the beach profile. Offshore wave conditions are refracted into the position of breaking at each point along the beach. These breaking wave conditions are used to calculate the longshore drift at each of these points. The change in position of the specified contour is calculated from differences in the wave induced longshore transport.

In the presence of groynes, the process of diffraction has to be added to bypassing of the structure to accurately assess the evolution of the coastline either side of the groyne. BEACHPLAN models the change in bypassing of groynes by varying the rate of bypassing depending on the distance between wave breaking and the tip of the groyne. Hence, the bypassing will change for different wave heights and different locations of the beach profile.

In the presence of detached breakwaters the processes of diffraction and transmission of the structure will create a rapid change in wave height and direction. In such cases, the second term of the CERC formulae, introduced by Brampton (1980), gains increasing importance, as the gradient of wave height will introduce a substantial change in the longshore drift. The simulation of these processes in BEACHPLAN allows an accurate representation of the beach behaviour behind detached structures. The detached structures can have any shape, which allows features such as artificial or natural islands to be represented.

The BEACHPLAN model has been designed as a first-stage tool in understanding the behaviour of a coast and the impact of engineering works upon it. Its relative simplicity and ease of use allow the model to be used by non-specialist engineers with a minimum of data, as well as allowing more detailed investigations by more experienced users.

The beach planshape mathematical model
The model is essentially a finite difference solution of the following equation that expresses the continuity of the volume of sediment moving along the shoreline,

$$\frac{\partial Q}{\partial x} + \frac{\partial A}{\partial t} = 0 \qquad (1)$$

where:

Q is the volume rate of alongshore sediment transport,
x is the distance along the shore,
A is the beach cross-sectional area,
and t is time

The basic equation can be modified to

$$\frac{\partial Q}{\partial x} + \frac{\partial A}{\partial t} + q = 0 \qquad (2)$$

where q is used to express the volume of material brought onshore by wave action, added to the beach by artificial nourishment or removed from the beach by mining. By denoting the co-ordinate perpendicular to the beach by y, the beach cross-sectional area, A, can then be expressed by the product of y and a depth D. If D is assumed not to vary with time, then equation (2) can be written

$$\frac{\partial Q}{\partial x} + \frac{\partial A}{\partial t} = 0 \qquad (3)$$

Starting from some initial position, y = y (x), the model evaluates successive beach positions at time intervals δt, at points along the shore separated by δx. So for each ordinate x_i (separated from its neighbour $x_{i+1}$by δx) we have $y_i(0)$, $y_i(\delta t)$, $y_i(2\delta t)$ and so on. The model used is of a type known as 'one line', that is to say that the beach position is given by the location of a single contour which represents, say, the high water line. An important factor in the accuracy of the model is the representation of the alongshore rate of sediment transport, Q, which is dominated by the breaking waves. For waves of small unevenness in height along a beach with nearly straight contours, Q can be well approximated by

$$Q = K_1 (Y_s)^{-1} E_b (nC)_b \left(\sin 2\,\alpha_b - 2K_2 \frac{H_b}{\partial x} \cot\ \beta \cos \partial_b \right) \qquad (4)$$

where

K_1,K_2	are non-dimensional coefficients
E	is the wave energy density = (0.125/2) ρ gH^2
H	is the significant wave height
g	is the acceleration due to gravity
ρ	is the water density
γ_s	is the submerged weight of beach material in place
nC	is the group velocity of the waves
α	is the angle between their crests and the local depth contours
tanβ	is the mean slope of the beach face, and where used as a subscript
b	denotes breaking wave conditions

The first term in equation 4 is the well-known CERC (Scripps) formula and describes the alongshore sediment transport due to obliquely breaking waves. Other well known formulae can be substituted for this in the model. The second term takes into account the transport created by any alongshore variation in breaking wave height, which becomes important for beaches in the lee of headlands or breakwaters where diffraction effects are significant. Very little practical work has been carried out into the assessment of k_2. Purely theoretical calculations can produce a value of 3.2 (Ozasa and Brampton 1980), but work by Kraus & Harikai (1983) has suggested a low figure may be more correct, in the range from 0.3 to 0.7. For the sand beaches a value of $K_2 = 0.5$ is normally used.

The height, period and direction of the breaking waves, however, are more difficult to prescribe. Although it is occasionally possible to represent the mean annual wave activity at a site by a single breaking wave condition, typically several such conditions are required. Often, it is necessary to supplement such wave data, either with results from the analysis of previous beach plan shape changes in the study area, or by using offshore wave conditions and predicting the resulting conditions at wave breaking by means of wave refraction analysis.

Results

Output from the model is in the form of tables (displayed on the screen or listed to a printer), or as plots showing beach plan shape ranges. Files are also created allowing easy continuation of model runs if required. In addition, the results are also stored in a LOTUS 1-2-3 compatible file, so that further analysis can be done by the user.

References

Ozasa H and Brampton a h. Mathematical Modelling of Beaches Backed by Seawalls. Coastal Eng. 1980

Kraus N C and Harikai S. Numerical model of the shoreline change at Oarai Beach. Coastal Eng No 1, 1983.

HR Wallingford

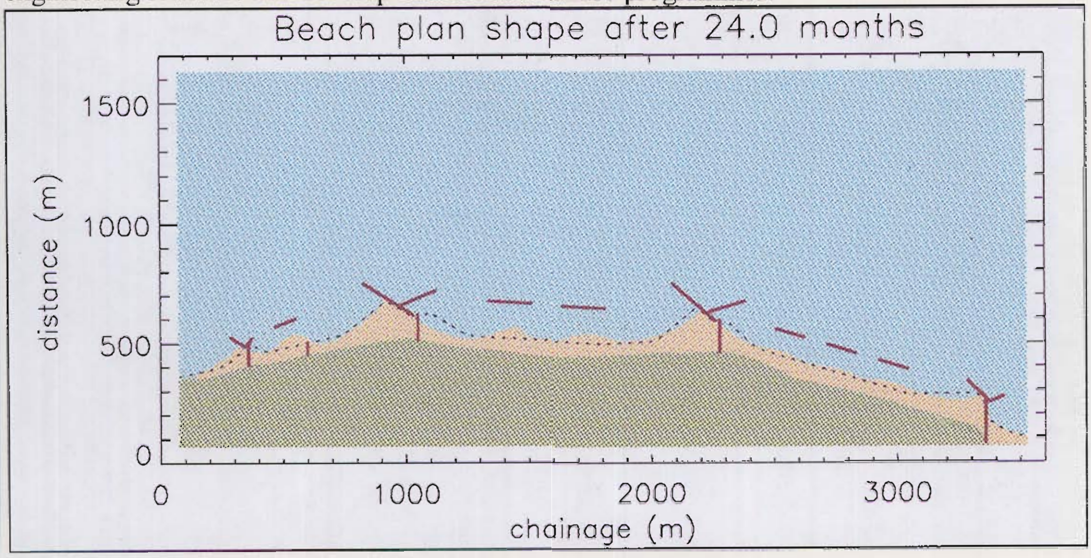
Software

BEACHPLAN (SeaWorks5)

Prediction of shoreline changes due to engineering works

BEACHPLAN is a state-of-the-art model that simulates beach evolution in plan-shape. It predicts trends in shoreline behaviour due to engineering works such as groyne construction, detached structures, recycling or renourishment.

BEACHPLAN enables coastal engineers and managers to assess the impact of proposed engineering schemes and develop beach maintenance programmes.



BEACHPLAN simulates the following coastal processes:

• wave transformation:

- refraction
- shoaling
- diffraction
- breaking

• structures:

- bypassing
- transmission through structures
- seawalls
- detached structures

• sediment transport:

- CERC or other sediment transport formulae
- Cross-shore distribution of longshore drift
- limited beach material

• active beach maintenance techniques:

- beach nourishment
- beach recycling

Typical applications of BEACHPLAN:

- assessing impacts of groynes, breakwaters, jetty or harbour construction
- evaluating effects of beach material lost due to extraction
- planning beach nourishment schemes

Examples of BEACHPLAN studies:

- Portman Bay, Spain
- Cavallino Littoral, Italy
- Quiryat, Oman
- Clacton, UK
- Lekki Harbour, Nigeria
- Zeebrugge Harbour, Belgium

BEACHPLAN is available for PCs and UNIX workstations.

For further information on **BEACHPLAN** or other modules in the **SeaWorks** system please contact HR Software Sales Manager, HR Wallingford, Wallingford, Oxfordshire, 0X10 8BA, UK, Tele: +44 1491 835381 Fax: +44 1491 832233

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