Poole Borough Coastal Strategy Study

Report EX 2881
February 1995

HR Wallingford

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Contract

This report was commissioned by Poole Borough Council (Drainage and Coastal Engineering Unit), represented by Mr Ronald Scholes and Mr Stuart Terry. The work was carried out in the Coastal Group at HR Wallingford which is managed by Dr K A Powell. The study was managed by Dr A H Brampton. The HR job number for the project was CBR 1609.

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Approved by

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Summary

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As the first step in establishing a Shoreline Management Plan, Poole Borough Council commissioned HR Wallingford to carry out a wide ranging coastal processes study. The study was based on a number of previous projects carried out for this area of the coast, but a substantial amount of new work was needed to produce this report. The overall purpose of the study was to provide an understanding of the present coastline, an assessment of its coastal defences, and to suggest suitable methods for managing the shoreline in the future.

This involved a number of tasks, first providing a quantitative understanding of the waves and tides affecting the Borough's coastline both within Poole Bay, and inside Poole Harbour. In view of the likelihood of further Shoreline Management Plans being carried out for adjacent stretches of coastline, the numerical modelling necessary to define the hydraulic environment of the coast was deliberately extended well beyond the limit of the Borough's boundaries. Information on both normal and extreme wave and tidal conditions was produced and validated against previous measurements.

Before considering the need for defending the coast, it was necessary to understand how it had evolved to its present position, and how it would develop naturally if its coastal defences were not maintained. Apart from an examination of the geological formation of the coastline, and its recent historical changes, this also involved calculations of the rates of movement of sediment along the beaches.

The present state of the coastal defences was examined, from a functional viewpoint as well as an appraisal of their remaining life. This information, together with information on the land-use behind the defences, and the hydraulic and sedimentary processes, was used to define a number of "Management Units" for the Borough's shoreline.

For each unit, the need for maintaining and/or improving the defences was examined. Where appropriate, possible options for the future management of each unit were identified and ranked according to importance, bearing in mind factors such as efficiency, cost and impacts on the environment. These options and their rankings were discussed with staff from the Borough Council to ensure that local sensitivities were taken into account.

The resulting management plans proposed here for each unit will be used in further discussions with local interest groups, and this will form the basis for a formal Shoreline Management Plan to be submitted to MAFF by the Council.

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1 Introduction

1.1 Background

The coastline of Poole Borough is very varied (see Figure 1.1). Much of it lies within Poole Harbour, one of the largest natural harbours in Europe. Although the River Frome discharges into the Harbour, its flows are small compared to the volumes of water entering and leaving on each tide. In that sense, the Harbour is best regarded as a tidal inlet. Although the area of the harbour is large, the entrance to Poole Bay is narrow and there are many islands and inlets, with the result that the coastline within it is generally well sheltered from severe wave activity. As a consequence of its hydraulic regime, the coastline has been developed for both commercial and recreational maritime activities. In addition, the harbour is of international ecological importance, especially because of its wading birds.

The Borough also has a more exposed sandy coastline, backed by dunes and cliffs, in Poole Bay, which is a highly regarded asset of the tourism and amenity industry, and also has considerable value as a site of geological importance.

The construction of engineering works to protect the coastline from erosion, and flooding from the sea, has been carried out for many years. Without such works, there would have been considerable loss of cliff top land along the shoreline of Poole Bay, and it is very likely that the narrow "neck" of the Sandbanks peninsula would have been breached. In addition, there have been a number of quite large reclamations within the Harbour, also requiring protection. The result of all these activities has been to leave the present Council with a wide range of coastal defences of different types, ages, and conditions. In places, there are presently problems of overtopping of the defences during severe storms, leading to flooding, and of beach erosion, leading to the exposure of the toe of defences and the threat of their collapse and subsequent erosion of the valuable land behind.

In order to manage these defences efficiently in the coming years, Poole Borough Council have decided to draw up a Shoreline Management Plan for their frontages. Such Plans are a recent concept, actively being promoted and supported by the Ministry of Agriculture, Fisheries and Food who administer the Coast Protection Act, and provide Grant Aid to local authorities carrying out coast protection schemes under this Act. The aim of a Shoreline Management Plan (SMP) is to set clear strategic guidelines for the management of defences along a coastline, bearing in mind the practical constraints, the appropriate standard of defence, and the environmental sensitivities of the coastline.

This report describes a wide ranging study of the hydraulic environment, coastal processes, and defence measures, commissioned by Poole Borough Council as the first stage of the development of a Shoreline Management Plan.

1.2 Terms of Reference

The large number of previous studies carried out by HR Wallingford for engineering schemes in Poole Harbour and Poole Bay provided a base on which to build the present investigation. For example, a number of numerical models of waves and tidal flows, validated against extensive field measurements, were made available with the approval of clients such as Poole Harbour Commissioners and British Petroleum.

The full terms of reference for the study are presented in Appendix 1 to this report, but can be briefly summarised as follows:

(i) **Hydraulic and sedimentary regime** - to review, model, and define the hydraulic processes affecting the coastline of Poole Borough, including wave conditions, tidal levels and tidal currents. From this to define design conditions for coastal defences. Also to review the sediment transport processes along the coastline of Poole Bay and within Poole Harbour.

(ii) **Long term development and character of the coastline** - to review the geological development of the coastline, its recent historical changes and describe its character.

(iii) **Present state of the coastline** - to define the areas of erosion or accretion, the present condition of the defences, and review historical flooding or damage events. Also to define the land use behind the defences, with the idea of setting appropriate standards of protection for the defences, and to help in initial cost-benefit analyses.

(iv) **Environmental impact review** - to examine the sensitivity of the environment to various shoreline management methods, and identify any measures to enhance the environment, or mitigate damage to, it as a result of defence schemes.

(v) **Recommending future monitoring and management measures** - based on a logical division of the Poole Borough Shoreline into "management units", to identify the most promising methods for maintaining adequate defences against the sea, to prioritise any necessary schemes, and to recommend appropriate monitoring as part of the management plan.

1.3 Report outline

The following chapter deals with the waves and tides affecting Poole Borough's coastline. Tidal modelling was carried out for a large area stretching from west of Portland Bill to the western Solent and the Isle of Wight. This provides a useful staring point for future Shoreline Management Plans for other authorities along this part of the south coast of England. The methods used, however, allowed very detailed resolution of the Poole Borough coastline. Waves were predicted both offshore, and at a large number of points along the shorelines of Poole Bay and Poole Harbour. Of considerable interest in planning coastal defences is the simultaneous occurrence of large waves and high tidal levels; this too is covered in Chapter 2.

Chapter 3 considers the geological formation of the coastline and its recent development, identifying present-day as well as historical erosion rates. This information is vital in considering one of the coastal management options, that is to allow the coastline to develop naturally, with no further expenditure on its defence - the so called "do nothing" option.

Chapter 4 describes how the Poole Borough frontage has been divided into 14 management units. The characteristics of each unit, in terms of hydraulic conditions, coastal geology, foreshore and coastal structures as well as land use and environmental sensitivity are described in detail.

Chapter 5 goes on to examine options for management of the Poole Borough coastline. Options for management of each unit are reviewed and recommendations are made on the most suitable means of managing and monitoring each unit.

2 Tides, waves and water levels

2.1 Tidal flow modelling

2.1.1 Introduction

The marked variations in tidal range and asymmetry and the complex patterns of tidal flow are an important feature of the study area. Off Portland, for example, the tidal range on mean spring tides is in excess of 3m; there the tidal curve is relatively symmetrical. At Poole, on the other hand the range is as small as 1.7m, and the tidal curve is strongly distorted, having a double high water and only a short "stand" at low water.

Offshore the main tidal streams are generally parallel to the coast, being in an easterly direction on the flood tide and in the opposite direction on the ebb. Closer inshore, tidal flows are very complex, particularly in Weymouth Bay, Swanage Bay, Poole Bay and Christchurch Bay, all of which are strongly indented. The flow patterns are further complicated by flows into and out of the Solent.

For shoreline management it is thus important to have detailed information about tidal flow patterns, particularly for sands and silts which are easily transported in suspension. The distribution of tidal flows not only affects the movement of material on nearshore bank systems but in areas such as Poole Bay also helps to shape the beaches.

Because of the importance of tidal currents in determining sediment transport, the tidal flow modelling has been extended to cover a much wider area than Poole Bay and Poole Harbour alone. At the request of MAFF the tidal model was extended to Portland in the west and the Isle of Wight to the east. The model will thus be useful in the formulation of shoreline management plans drawn up by neighbouring authorities in Dorset and Hampshire, as well as being applicable to the west coast of the Isle of Wight. It will also provide useful data for any offshore studies that may be carried out in this area.

The model has been designed so that its grid can be easily refined and can be made to give a high degree of detail in any specific area of interest, by patching in more detailed bathymetry by reducing the size of grid elements. This is precisely what is being carried out in a follow-up study by HR for PBC (HR, EX3083, 1994) where a detailed examination of tidal flows off Sandbanks is being made, to examine the best form of defence needed to combat erosion in this area. The head of the Sandbanks peninsula has been shown to be under threat as a result of very rapid inshore tidal currents, caused by the deepening of a tidal channel (the East Looe Channel).

2.1.2 Model description

The tidal flows between Portland Bill to the west, St Catherines Point on the Isle of Wight to the east, and Calshot at the entrance to Southampton Water to the east-north-east, have been modelled using a depth-averaged flow model called TELEMAC-2D. This model, which was developed by Electricité de France (EDF), employs finite element techniques with a completely "unstructured" grid, which allows one to resolve tidal flow in fine detail in an area of interest, but uses a coarser resolution at the model edges, thus enabling boundaries (and any undesirable boundary effects) to be distant from the area under study.

The model mesh is shown in Figure 2.1. It can be seen that the greatest degree of refinement is in and around Poole Harbour because of the complex bathymetry there, and because of the complexity of the coastal configuration. Poole Harbour includes a number of islands which had to be resolved in order to reproduce tidal flows correctly. There is also a finely resolved grid within the Western Solent up to Calshot at the entrance to Southampton Water because of the complex bathymetry (large intertidal flats bordering a deeply incised approach channel). Finally the whole of the mainland coast and that of the Isle of Wight are resolved in fine detail, because of the rapidly changing coastal configuration and nearshore bathymetry. The maximum grid size was about 100m around Poole Harbour, 150m along the coast and in the West Solent, increasing to 500m offshore and towards the seaward boundaries of the model.

The bathymetry of the model area is shown in Figure 2.2 at two levels of detail. The top part of Figure 2.2 shows the general seabed contours out to 60m depth; this was taken directly from a previous uniform 400m grid model of the area (HR, EX2228, 1991). This model, called TIDEWAY, was used to set up a mesh of water depths at the node points shown in Figure 2.1. HR Wallingford's TIDEWAY model also contained much of the information on boundary conditions, water levels, tidal levels etc which were needed for the setting up and calibration of the present model. Within Poole Harbour itself, and around the coastlines within the model, additional detail was obtained from contemporary bathymetric and land survey data.

Boundary conditions for the TELEMAC model were generated by creating tide curves at 9 locations around the edges of the model. Apart from the normal tidal constituents needed to represent tides in UK waters six other constituents were also used in this model to accurately define the varying tidal curves throughout this complex area. Each constituent represents one of the apparent rotations of the sun and moon around the Earth, having periodicities ranging from roughly 24 hours down to 4 hours. The tidal curves were then interpolated to define the tidal elevation at all intermediate boundary positions.

2.1.3 Model calibration and validation

The Poole Bay area is very well served with tidal data. On 24/25 April 1990 British Maritime Technology (BMT) carried out mid-depth and near-bed current observations simultaneously at three sites in Poole Bay, as well as making observations of tidal level at Bournemouth Pier. This tide, extending from one low water to the next (ie a full tidal cycle) was selected because it had a range of 1.8m at Bournemouth, which is very close to the mean spring range of 1.7m, and also because both low waters had approximately the same elevation. This data had previously been used to calibrate HR's TIDEWAY model of the area (HR, EX2228, 1991).

Apart from the 9 tidal curves synthesised on the model boundaries (and used for driving the model) tidal curves were also synthesised at 5 positions within the model itself for comparison with the model output. The synthesised curves can be considered as tidal data, in the same manner as Admiralty curves are used for tidal predictions.

Figure 2.3, which is the tidal level calibration, shows the comparison between the model results and tidal elevations synthesised and observed for the mean spring tide chosen. It can be seen that the model gives an extremely good representation of the highly variable tidal regime in the study area. In different locations the tides vary in range, phase and shape.

The tidal curve at Calshot (Western Solent/Southampton Water) is well represented despite being close to the eastern boundary of the model. Similarly Chesil Beach is very accurately represented on the western boundary of the model. Freshwater, which is situated on the south-west coast of the Isle of Wight, is also very well represented, the high water "stand" being very similar in both the synthesised and in the model tidal curve. At Poole Harbour entrance the double high water and the tidal range are also well represented, despite the very complex tidal regime in this area. Finally there is also a good correlation between observed (actual) and model observations in the central part of Poole Bay. The double high waters at the latter three locations are caused by the propagation of the tide up the English Channel and the presence of an amphidromic point (point of zero-tidal range) near Poole.

Having calibrated and checked that the tide was represented accurately in all parts of the study area, attention was then given to tidal current patterns in the western part of Poole Bay. The model results were compared with observations of tidal currents at the three BMT observation sites in Poole Bay and this is shown in Figures 2.4 and 2.5. Position 1 is situation on the east flank of Hook Sand while position 2 is a further 1½km eastward into Poole Bay. Position 3 is also situated in the west part of Poole Bay some 3½km north-east of Handfast Point (HR, EX 2228, 1991). The BMT observations were scaled by standard numerical methods to give depth-mean currents so as to be directly comparable to the depth averaged flows produced by the TELEMAC model. It can be seen from Figure 2.4 that both the direction and the speed of tidal flows at all three sites were very well represented by the model and gives us confidence in the interpretation of tidal flow patterns within the study area. Predicted tidal currents within Poole Harbour were not compared with measured values in the present study. However, such validation has previously been carried out in connection with studies of the navigation channels within the harbour for Poole Harbour Commissioners (HR, EX2122, 1990, HR, EX2160, 1990, HR, EX2532, 1992, HR, EX2903, 1993).

The model was also run for a mean neap tide (range about 0.7m) and this was achieved by scaling the nine boundary tide curves for springs, according to the ratio of the sum of the two principal tidal harmonics. The results of the comparison of tidal current velocities at the three BMT positions is shown in Figure 2.5. It can be seen that the comparisons are very good for all three locations.

The model was also compared for a number of Admiralty diamonds in the area from which peak ebb and flood velocities were abstracted and compared against the model results, see Figure 2.6. Again the comparison is very good within the study area with the exceptions of the western boundary in the vicinity of Portland Bill.

2.1.4 Model output

In order to provide a detailed coverage of tidal current patterns over the study area, predictions of water level, tidal current speed and direction, were made at the 103 locations, shown in Figure 2.7 which extend out to the 50m depth contour. For greater clarity tidal flow patterns within Poole Harbour itself are shown as tidal vectors, see below.

The predictions have been made for both mean spring and mean neap tides, with the time interval between readings being 20 minutes. A vast quantity of data has therefore been assembled and this has been passed on to Poole BC in digital form. For each prediction position the data includes the grid reference, the current speed (m/s), direction (° grid N) and tidal elevation (m ODN).

In this report information has been summarised in the form of tidal vectors which give a representation of "streamlines". The results are shown in Figures 2.8 to 2.19, at hourly intervals over a mean spring tidal cycle. It can be seen that the peak ebb flow generally occurs at around low water in the northern part of Poole Harbour (between Poole Town Quay and Sandbanks). An hour after low water the tide has turned and the flood tide increases to reach a peak within the harbour about three hours after low water. By six hours after low water (at about the time of the first high water) conditions are virtually quiescent and after seven hours the tide has turned in Poole Harbour and Poole Bay. Ebb flows in the Harbour drop significantly around nine hours after low water due to the "double high water" effect but are unaffected offshore in Poole Bay. At 10 hours after low water a strong ebb flow has set in again.

The high tidal velocities have an important influence in a number of areas. Around Lower Hamworthy there are strong tidal flows which affect ship movements into and out of the commercial port and the adjacent marina (Unit 12). Any developments in this area which are carried out in the intertidal zone should ideally examine tidal flows "before" and "after" construction. Currents are also very rapid in the Back Water Channel in the entrance to Holes Bay and these fast flows also affect the Town Quay frontage (Unit 8).

Flows around the rest of the periphery of Poole Harbour, Holes Bay and Lytchett Bay are generally low. This situation could be altered, however, by the construction of any major structures. Again we would advocate "before and after" tidal modelling for any future works in these areas, although we do not anticipate any serious problems arising from the construction of small scale structures (low short groynes etc).

The strongest currents occur in the entrance to the Harbour and Figure 2.20 shows peak ebb and flood flows in this area. It is evident that any coastal works carried out around the head of the Sandbanks peninsula would need to be considered in terms of their impact on tidal flows. Figure 2.20 demonstrates that there are significant tidal currents along the east face of the Sandbanks peninsula. Analysis of contemporary bathymetric charts has identified the growth of the East Looe Channel. Tidal flows in this area have a major bearing on conditions in the intertidal zone, due to the channel sides now intruding on to the lower part of the beach. The tidal flows in this area are being examined in a study of coast protection works which are needed to prevent further deterioration of beaches in this area (HR, EX3083, 1994).

Towards the neck of the Sandbanks peninsula, there is a marked reduction in tidal velocities and an increasing exposure to wave action towards the Bournemouth frontage. There is evidence that the East Looe Channel may be extending in this direction (HR, EX3083, 1994) so tidal currents may play an increasing role in beach management in the central part of Management Unit 1 in future years (see Chapter 4).

For the purposes of clarity colour maps have also been presented which show the general distribution of peak ebb and flood flows at the Harbour entrance on a mean spring tide, (see Figures 2.21 and 2.22). These flows are superimposed upon the seabed contours and show the main flows to be concentrated in the Swash Channel outside the harbour, and in the channel between Brownsea Island and the inner face of the Sandbanks peninsula. The strong flows in the East Looe Channel are also clearly illustrated. (Due to the time-lag between the entrance and the upper reaches of the harbour the maps do not show the rapid flows around Lower Hamworthy and along the Town Quay).

Figures 2.23 to 2.29 show spring tidal current velocity patterns on a more regional scale, extending from seawards of the Isle of Purbeck eastward to Hengistbury Head. These "streamlines" show the general patterns of offshore movements at a relatively coarse scale of model interrogation.

2.2 Wave modelling in Poole Bay

2.2.1 Review of sources of wind and wave data

(i) Wind data

Wind data is important as input to any wave prediction work. Ideally, accurate long-term sequential over-water wind conditions would be used. However, this is rarely available, and instead land wind data is often used, with wind speeds suitably increased to be representative of speeds over open water.

For UK studies, wind data from Met Office coastal (or near-coastal) anemographs is preferred. The perfect wind recording station would provide long-term sequential data in digital format, would be well exposed, for example on a cliff top, and would be in continuous operation. There are several possible anemograph stations near the region of interest (see Figure 2.30). Although all of them provide sequential wind data in digital format, none has all of the desirable characteristics identified above.

The anemograph at Portland Coastguard Station offers over 20 years of wind data from a well exposed site, but was withdrawn from service in February 1992. Hurn also offers over 20 years of data, and is still in operation, but being several kilometres from the coast, the wind speeds are not representative of open sea conditions. Calshot offers nearly 20 years of data at a site very representative of conditions in Southampton Water and the Solent, but was withdrawn from service several years ago. Lee-on-Solent offers about 15 years of data at a site representative of conditions in the Solent, but was withdrawn a few years ago. Thorney Island offers over 20 years of data, but is generally accepted to be under-exposed in comparison with winds over the sea. St Catherines offers many years of data, continuing operation and a well exposed location on the southern tip of the Isle of Wight, but the wind gauge is not of a continuously recording type. Wind data measured in Poole Harbour from 1991 to 1993 was supplied to HR Wallingford by Poole BC, but because of the relative shortness of the record, this data was not used directly, but only for comparative purposes.

Winds at each site were compared. Portland was chosen as the best source of wind data for simulation of waves generated in the open sea, although still with some modifications for the differences between recorded wind speeds and those expected over the sea. Because of its more sheltered position, and the assumption that all waves are locally generated, Hurn was chosen as the best source of data for prediction of waves within Poole Harbour, although still with some speed modifications for some directions. Wind roses, based on data as recorded at Poole Harbour, Portland and Hurn are shown in Figures 2.31 to 2.33.

(ii) Wave data

Poole Bay has been the subject of several wave recording and prediction studies. Bournemouth Borough Council funded the University of Southampton to record waves off Southbourne 1974-79 (Henderson and Webber, 1974-79). BP commissioned HR Wallingford (EX2373, 1991) to record waves off Poole Harbour entrance over the winter of 1990-91. HR Wallingford has carried out several numerical model studies of Holes Bay, Poole Harbour, Swanage Bay and the Bournemouth BC frontage. A further series of studies funded by BP in connection with the possible construction of an artificial island off Poole Harbour entrance considered waves, currents, sediment transport, coastal defences, and any impact that the proposed island might have.

Perhaps the most relevant reports in terms of wave prediction are those prepared for Bournemouth BC (HR, SR146, 1986) and for BP (HR, EX2228, 1991 and HR, EX2508, 1992). These reports describe the validation of numerical wave models against field wave data, and the derivation of wave climate data for about a dozen nearshore locations based on 15 years of wind data from Portland.

The existing measured and predicted wave data was not used directly in the present study, but experience, model calibration and digitised bathymetry were used in setting up the models for the present work.

2.2.2 Methodology for the offshore wave climate modelling in Poole Bay

(i) Background

Wave conditions along the coastline between St Catherines Point and Portland Bill are best evaluated by using a two-stage approach. In deep water, ie greater than about 25m in the present case, the effects of the seabed on the character of waves are small in comparison to the effects of winds. Closer to the shore, the effects of the seabed and headlands become more important, and dominate effects due to the wind. In this study, the proposed numerical simulation of the generation of waves in deep water, and of their transformation in shallow water, considers the two aspects separately. In a previous study in Poole Bay (HR, EX2508, 1992), wave measurements have verified that this approach provides reliable estimates of actual conditions.

(ii) Methods

HR Wallingford has already carried out several studies of wave generation in this area. A standard HR computer model, HINDWAVE, was used with input wind data measured at Portland Bill from 1974 to 1992. Most of this wind data was already held at HR, but to improve the joint probability study (see Section 2.5), a further approximately 2 years of recent measurements were obtained during this study to extend the wave "hindcasting" to cover the whole 19 year period. Five locations offshore from the coast were selected on about the 20mCD contour between Portland and St Catherines (see Figure 2.30). For each location, hourly wave conditions were predicted, and a synthetic wave climate, ie a distribution of wave height, period and direction, was evaluated.

Statistical methods were used to predict extreme wave conditions at each of the offshore locations.

(iii) Outputs
The offshore wave modelling provides:

Predicted hourly wave conditions at five offshore locations, which were made available on floppy disks to Poole BC at the end of the study.

Tables and diagrams (eg wave "roses") summarising the overall wave climate for each location.

Predictions of extreme wave conditions, for a variety of direction sectors, and for a variety of return periods.

2.2.3 The HINDWAVE offshore wave climate model

The HINDWAVE model takes as input, details of the geometry of the area in which the waves are generated and hourly wind records from a local anemometer station (in this case, Portland). Output from the model is in the form of hourly estimates of wave height, period and direction, which can be condensed into the required probabilistic description of the wave climate.

A detailed description of the HINDWAVE model (Hawkes, 1987) can be found in Appendix 2 of this report, but briefly the method works as follows. Information about the shape of the wave generation area is presented as a table of fetch lengths, drawn radially outward at, say, 10 degree separations from the point of interest. Using this information, and a wave forecasting model based on the JONSWAP method as modified by Seymour (1977), a set of site-specific offshore wave forecasting tables are produced. Each table gives the predicted wave height, period and wave direction for a wide range of wind speeds and directions, assuming a particular (fixed) duration of that wind condition.

A variety of such tables are computed corresponding to a chosen set of durations, in this case 1, 2, 4, 7, 10, 14, 18 and 24 hours. Previous experience of the model for this part of the English Channel showed no improvements in wave predictions if longer durations were considered. Once the set of forecasting tables has been completed, the second phase of the process begins. At every hour during the period being analysed (in this case 19 years), the hourly wind records are vectorially averaged over the same number of hours (ie duration) leading up to that hour as were used in setting up the forecasting tables (ie over the preceding 1, 2, 4, ... hours). For each duration, the corresponding wave height is obtained from the relevant table, and the largest of all these values is chosen. This value is stored together with the associated wave period and direction. This procedure is then repeated for the next hour, up to the end of the period of wind data. Once the hourly wave conditions have been produced it is possible to condense the results into tables giving details of, say, the probability of particular combinations of wave heights and directions, or wave heights and periods.

Now, although the HINDWAVE model uses a well accepted wave forecasting method, it is still possible, and indeed necessary to carry out some modifications to improve its performance (Hawkes and Jelliman, 1993). In the first instance, it is important to allow for the reduction in wind speeds over the land (where they are recorded) from their speeds over the sea surface. This reduction factor generally depends on both direction and speed.

Previous experience with the use of wind data from Portland in the HINDWAVE model had produced a set of wind speed adjustment factors. Although generally the recorded speeds have to be increased, the position of the anemometer on the top of the Isle of Portland tends to over-estimate winds from the south-east. More detailed information on the adjustments can be found in Appendix 2 of this report.

It has also been found that predictions from the model can be substantially improved by limiting the duration over which wind records are vectorially averaged. This is equivalent to making an assumption about the upper limit of the diameter of a weather system; a similar effect could be achieved by artificially limiting the very long fetch lengths to the south-west. Either method of adjustment is reasonable because of the inherent simplicity of the way the HINDWAVE model works. In the present study this limit was set to 24 hours.

2.2.4 Wave climate results

HINDWAVE was run separately for each of Offshore Points 1-5, (see Figure 2.30) using 19 years of Portland wind data. The time series results were retained for later transfer to Poole BC and for use in the joint probability analysis. To make the results easier to use they were collated in the form of annual probability distributions of significant wave height, mean period and mean direction.

Tables 2.1 to 2.10 are scatter diagrams of wave height against direction and against period for each prediction point. These diagrams show the probability, in parts per hundred thousand, of any particular combination of wave height with either wave period or direction. The same information is shown in the form of wave roses in Figures 2.34 to 2.38.

The prevailing wave direction, and the direction from which the largest waves come, is south-west to west-south-west. There is a secondary concentration of waves from the east-south-east direction. Offshore Point 3 is significantly less exposed than the other points due to its position. The annual offshore distribution of wave height is summarised as follows, averaged over four prediction points (ie excluding the under-exposed Point 3): significant wave heights of 2, 3, 4, and 5m are predicted to be exceeded approximately 12, 3.3, 0.65 and 0.08% of the time, respectively.

2.2.5 Extreme wave conditions

The significant wave height data given in Tables 2.1 to 2.10 was extrapolated to extremes using the method described in Appendix 3. A three-parameter Weibull distribution was fitted to the distribution of H_s predicted for each wave direction sector of potential interest at each offshore point.

Extreme significant wave heights were then determined, corresponding to probabilities of 3 hours occurrence, in each of several different return periods. These results are listed in Tables 2.11 to 2.15, together with a method for estimating the corresponding wave period based on a typical wave steepness. Tables 2.11 to 2.15 contain predictions of the "all directions" extremes. As would be expected, these values are slightly higher than the highest of the direction-dependent extremes.

The largest predicted significant wave heights offshore come from direction sector 210-250°N, with values of approximately 5½, 6½ and 7½m for 1, 10 and 100 year return periods, respectively. The extremes predicted for Offshore Points 1, 2, 4 and 5 are approximately equal. Since the Poole BC frontage is protected from the full force of waves from the south-west, extreme wave heights inshore will be significantly lower than offshore.

2.2.6 Methodology for nearshore wave climate modelling in Poole Bay

(i) Background

As waves travel into the interior of Poole Bay from the open waters of the English Channel, they are modified by the changing water depths. The partial protection provided by Durlston Head is also important, especially for Studland Bay and the eastern part of the Poole Borough frontage in Poole Bay. Wave conditions during a westerly storm are therefore very different at Hengistbury Head from those at Poole Harbour entrance. Numerical modelling was used to quantify these changes, as an essential step in producing "nearshore" wave climate information at a number of locations around Poole Bay.

(ii) Methods

The wave transformation modelling proposed in this study was heavily based on previous HR Wallingford studies of Poole Bay. An existing computer model was reinstated, and was combined with the offshore wave climate information (see Section 2.2.2) to calculate corresponding "nearshore" wave climates at a number of locations both along the Poole Borough frontage, and elsewhere.

This wave climate information was used in later stages of the study to predict extreme wave conditions for use in design studies, and to calculate longshore drift rates for sandy beaches in Poole Bay (see Sections 2.5 and 3.3.4).

This modelling assumed a fixed tidal level (MHWS), thus providing a good estimate of wave conditions occurring during the important upper part of the tidal cycle. Because of the modest tidal range in Poole Bay the effects of this choice (compared to say Mean Sea Level) on predicted nearshore wave conditions, are small. However, that at low water the actual wave heights may be slightly smaller than predicted (by choosing MHWS) is a somewhat conservative assumption.

The sequential wave conditions predicted for the nearshore locations were used at a later stage in the study to provide information on the year-to-year variability of longshore drift rates along the sandy coastline of the Borough.

Because of the choice of water level at MHWS for the wave refraction calculations, the resulting estimates of drift rates are likely to be slight over-estimates.

(iii) Outputs
The nearshore wave climate information is presented in the form of a number of tables and diagrams, in a similar manner to that used for the offshore wave climates.

In addition, standard statistical methods were used to predict extreme wave conditions, in the form of significant heights, periods and directions, at each nearshore location.

The sequential nearshore wave data was made available to Poole BC on floppy disks at the end of the study.

2.2.7 The OUTRAY inshore wave transformation model
The majority of wave generation will occur in deep open expanses of water. Whilst generation will not cease as waves reach shallower water, the effects of the seabed become increasingly important. Wave refraction and shoaling are usually considered together, as both are caused by spatial variations in water depth. Shoaling involves a change in wave height consequent upon the waves slowing down as they travel through water of decreasing depth. Refraction occurs when waves approach the coast at oblique angles of incidence. It involves a gradual change in wave direction as waves travel toward the coast. Both these processes are included in the standard refraction programs used at HR Wallingford.

Refraction analysis produces sets of transfer functions for wave energy and velocity, dependent upon frequency and direction. These take the form of tables of coefficients relating conditions at the inshore point to those at the offshore point, for each frequency and direction considered. The offshore wave predictions are produced in the form of a directional spectrum, ie an array of energy components as functions of frequency and direction. Each member of the array is multiplied by the appropriate transfer coefficient in order to derive the corresponding inshore spectrum. The spectrum can then be integrated as shown in Appendix 2, in order to calculate the usual parameters, ie significant wave height, mean period and mean direction.

For the refraction analysis, a standard mathematical technique based on the concept of wave rays was used. A full description of the model may be found in Appendix 4 of this report. However, a brief explanation is given here. The technique consists of following, or "tracking", rays seaward from an inshore point to the offshore edge of the grid system. Each ray, which is a line perpendicular to the wave crest, then gives information on how energy travels between the seaward edge of the grid system and the nearshore point of interest. By considering a large number of such ray paths a particular set of matrices may be constructed. This set of matrices are known as transfer functions because they provide a description of the transformation of wave energy between the edge of the refraction grids and the point of interest. Once the transfer functions have been evaluated, and because linear wave theory is being used, the refraction of a large variety of offshore wave conditions can be calculated fairly simply.

During previous use and validation of wave models in this area (HR, EX2508, 1992), diffraction of westerly waves around Durlston Head was found to have a significant impact upon waves in the north-western part of Poole Bay. A slightly modified version of OUTRAY, called OUTDIF, was therefore used for the refraction modelling in this study. This model simulates wave energy transmission around a "diffraction barrier", in this case Durlston Head at the south-west extremity of Poole Bay. The diffraction theory is that applicable to waves propagating into still water in the lee of a semi-infinite thin breakwater (defined by angle of breakwater and position of tip). The results from OUTDIF are in similar format to those from OUTRAY, ie a transfer function of energy multipliers dependent upon offshore wave frequency and direction.

2.2.8 *Setting up of OUTRAY for use in Poole Bay*

The OUTDIF model works by "back-tracking" a series of "rays" to an "offshore boundary". Each ray represents a line which is parallel to the wave direction and hence waves are said to be "tracked" by the ray paths. The density of rays in a particular area corresponds to the concentration of wave energy: a high density of rays means a high energy flux. It is therefore possible to calculate the transfer of energy from the offshore boundary to the inshore points, and to summarise the results in the form of a "transfer function".

Before running the model it was necessary to digitise the bathymetry in the area of interest. This had been largely completed during previous studies (HR, SR146, 1986 and HR, EX2228, 1991), but before use in the present study, the bathymetry of the Poole Bay coast was re-plotted and checked. The information is stored on a number of rectangular grids (see Figure 2.30), with a depth value being supplied at each grid point. Dimensions of the ten grids are given below:

Grid number	Distance between depth measurements (m)		Number of grid points	
(No)	X (E-W)	Y (N-S)	Columns	Rows
1	400	400	11	14
2	400	400	35	14
3	400	400	13	14
4	400	200	11	15
5	400	200	17	15
6	400	400	13	7
7	200	200	19	29
8	400	400	19	15
9	800	800	26	11
10	1600	1600	13	6

In the above table the 'rows' are parallel to the x-axis which runs west to east, and the columns are parallel to the y-axis which runs south to north.

The offshore boundary of the model was taken as the southern edge of the grid system plus the part of the western edge of Grid 8 extending south of Durlston Head. Deep water (or "offshore") wave conditions are assumed to be homogeneous along this offshore boundary.

Eight Inshore Points A-H were chosen along the Poole Bay frontage on the -2mCD contour (apart from Point D, where it was necessary to move the prediction point to -4mCD to obtain satisfactory results). The refraction model was run for each of Inshore Points A-H (see Figure 2.39) at a water level of +2mCD, corresponding to Mean High Water Springs and a depth of water of 4m (6m at Point D). A total of 15 wave periods were used in the refraction analysis, starting at 1.2 seconds and increasing in 1.2 second increments to a maximum value of 18 seconds. For each of these periods an inshore angular separation between adjacent rays needed to be chosen. The longer wave periods refract more strongly than the shorter wave periods and therefore require a greater density of rays. The angular separations used in this study were as follows:

Wave period (s)	1.2-6.0	7.2-13.2	14.4-18.0
Separation (degrees)	1.0	0.5	0.25

OUTDIF calculates a transfer function for each inshore point, showing the ratio between inshore and offshore wave energy as a function of wave frequency and direction. These matrices are in a format compatible with the offshore directional spectra produced by the first stage of HINDWAVE. When the two are multiplied together, the equivalent spectra at the inshore points are generated, from which the corresponding wave heights, periods and directions can be derived.

2.2.9 The LOCALGEN wave model for use in Poole Bay

The main part of the wave modelling assumes that waves are generated only outside Poole Bay, and that they are transformed by shallow water processes only within Poole and Christchurch Bays. Using this approach alone, wave energy locally generated within Poole and Christchurch Bays would be neglected. This energy could be significant when the wind is from either the north-westerly or north-easterly quadrants.

To account for this, a second HINDWAVE model was set up to hindcast the locally generated component of wave energy. For this purpose, fetch lengths were measured from the general area of the inshore points to the edges of the refraction grid. HINDWAVE was run to produce a time series hindcast of hourly wave conditions generated *within* Poole and Christchurch Bays. This wave energy was later added to that predicted by the main wave models.

2.2.10 The HINDRAY inshore wave climate model

An efficient method of combining refraction and HINDWAVE analysis (Hawkes and Jelliman, 1993) was used in this study. A large representative sample of wave conditions were put through the wave transformation procedure for each inshore point, from which the transfer of all other wave conditions was inferred by interpolation. Armed with this information, the wave condition "menus" produced by the first stage of HINDWAVE were transformed into equivalent inshore "menus" for each inshore point.

Retention of the efficient "menu" format for the inshore points allowed large quantities of sequential wind data to be processed with quite modest computational effort. Hindcasting of wave conditions for each inshore point was then a simple matter, and was carried out in the same way, as for the offshore points (see Section 2.2.4). The HINDWAVE part of the HINDRAY model was used to simulate offshore wave generation based on long-term wind data from Portland, whilst the OUTDIF element was used to simulate refraction effects.

Finally, the locally generated wave conditions (see Section 2.2.9) remained to be added on to those produced by HINDRAY. Both sets of predictions existed in simultaneous time series form. A composite (overall) wave prediction was derived for each hour by adding the energies of each wave component. Performed sequentially this yielded overall time series wave hindcasts in the same form as produced for the offshore points.

2.2.11 Wave climate and extremes results

Extreme wave conditions inshore could have been predicted by transforming the offshore extremes (see Section 2.2.3) to the inshore points using the refraction model. However, the approach adopted here, which gives better directional resolution, was to extrapolate the inshore wave climate data to direction-dependent extremes in the same manner as was used for the offshore data.

Results were collated and presented in the same format as used for the synthetic offshore wave data. Tables 2.16 to 2.31 show the distribution of the wave climate at each of the eight inshore points, whilst Figures 2.40 to 2.47 show the same wave climate information in the form of wave roses. Tables 2.32 to 2.39 show the extreme wave heights at a range of return periods.

Differences between results for the various prediction points are consistent with their positions along the coastline. The largest predicted waves are around five or six metres significant wave height at each of the points. The directions of the highest waves are clustered in a fairly narrow band around the beach normal at each point. The average direction of the waves at the shoreline gradually changes from about south-east at Inshore Point A (in Studland Bay) to about south-west at Inshore Point H (near Hengistbury Head). Also the range of directions is a little wider at the more easterly points. These small differences become important in the later littoral drift modelling.

Some of the wave heights predicted are high enough to have been reduced by wave breaking before arrival at the inshore wave prediction points. However, the wave heights are quoted *unbroken* in order to provide more generally applicable input to subsequent calculations, in which wave breaking is modelled where appropriate.

2.3 Wave modelling in Poole Harbour

2.3.1 Background and Methodology

Waves in Poole Harbour are almost entirely due to local wave generation within the Harbour itself. The height of the waves depends largely on the wind speed (over about the last hour), and the distance across the Harbour in the wind direction to the point of interest, ie the "fetch". It also depends on the fetch "width", calculated using fetch lengths at angles either side of the wind direction. Because of the substantial drying areas within the Harbour, fetch lengths, and waves, are smallest at low water, and highest at high tidal levels. The wind strength will also tend to be greatest at high water because the water surface is "smoother" than the saltmarsh and mud-flats at low tide. Because of this, even the relatively small waves on this part of the Poole BC coastline can combine with high tidal levels to cause damage and flooding.

The complex shape of the Harbour, with various islands which shorten some fetch lengths, and change the fetch widths, means that wave conditions will change along even a short stretch of coastline. However, because the waves generated within the Harbour have a rather short period, they will be affected only slightly by changes in water depth.

Because of the rapid spatial changes in wave conditions around the coastline of Poole Harbour, it was not possible in this study to provide wave climate information at every point. This part of the study was therefore carried out in three parts, as follows.

First the various sources of wind data available were assessed for use in calculating wave conditions within the Harbour. There are wind records from Hurn, Portland, St Catherine's Point and other locations. A recommendation on which wind data to use, and how it needs to be modified for the area was made, and a suitable wind rose (and frequency of occurrence table) was produced. Extreme wind conditions, from various directions, were also calculated.

In the second stage, nine locations along the Poole BC side of the Harbour were selected (see Figure 2.48), and a computer model of wave generation developed by HR Wallingford was used to produce wave climates and extremes at these chosen locations. This information provided an input to the joint probability study (see Section 2.5).

Finally, the computer model was passed over, together with user notes and worked examples, to Poole BC staff, for them to apply at any other location(s) of interest.

2.3.2 Wind conditions within Poole Harbour

Local sources of wind data were described in Section 2.2.1 and wind roses are presented in Figures 2.31 to 2.33. During previous studies of Poole Harbour (HR, EX1760, 1988 and HR, EX2122, 1990), winds from several local sources were compared, and a wave model dependent upon local wind data was approximately validated against observations of waves in Poole Harbour.

At that time, wind data from Hurn (Bournemouth Airport), with some small modifications, was taken as representative of conditions in Poole Harbour. The distribution of wind velocities as recorded at Hurn is shown in Figure 2.33. The modifications consisted of adding 10% to all wind speeds (U), a further 20% to wind speeds from directions 105-285°N, and then a further 10% to derived extreme wind speeds with return periods of 1 year or more. Some resulting representative wind conditions are given in Table 2.40: average U, U exceeded 10% of the time, U exceeded 1% of the time, 1 and 100 year extreme U, and percentage of data, for each 30 degree direction sector.

During the present study, 3 years of wind data recorded within Poole Harbour (see Figure 2.31) were supplied to HR. The distribution of wind direction in this data is quite similar to that at Hurn. The representative speeds derived from the Hurn data are about 15-20% higher than would be obtained directly from the Poole Harbour recorder. To allow for the land recorded wind speeds being a little lower than those over the water, to be conservative, and to be consistent with the earlier studies, the wind conditions listed in Table 2.40 were used as input to the wave predictions within Poole Harbour.

2.3.3 *The SAVILLE locally generated wave climate model*

The wave prediction model needed to be easy to apply, because of the number of locations and wind conditions to be tested. It also needed to be reliable across the whole range of commonly occurring and higher wave heights, and to be able to deal with locally generated waves.

The most suitable wave prediction models for a small wave generation area such as Poole Harbour is based on the Saville method. This method was suggested by the Institution of Civil Engineers (1975) for estimation of wave heights in reservoirs. It is suitable for fetch lengths up to 15km. In Poole Harbour the maximum fetch length is about 4km.

The wave generation area is divided into segments centred upon a wave prediction point. The procedure assumes that the effectiveness of any segment is indicated by the ratio of the actual length of the segment to the length it would be in a fetch of unrestricted width. This ratio is the same as that of the projection of these lengths onto the central radial from the point of interest. It is further assumed that the effectiveness of the wind in generating waves (ie in exerting its stress on the water surface) is proportional to the product of these two values. The total effectiveness of each fetch segment is proportional to the product of these two values. The total effectiveness of the entire fetch is the sum of these products divided by the sum of the cosines.

Fifteen radials are drawn from the wave prediction point with 6 degree intervals between them. The central radial is usually drawn along the line of longest fetch. Each fetch length is measured from the wave prediction point to the shore (zero total depth contour). Each radial is taken to represent the mean fetch for a 6 degree sector. The effective length of each segment is the component of the radial's length in the direction parallel to the central radial. Winds outside the sector of 45 degrees on either side of the central radial are considered to be completely ineffective. Unless there is specific evidence to the contrary the design wind speed is presumed to blow down the central radial.

Significant wave height (H_s) and wave period (T_s) are parameters in common use amongst coastal engineers to describe ocean waves. H_s represents the average height of the highest one third of the waves and T_s is the wave period associated with this wave height. The values of H_s and T_s for the Saville method are given by:

$$H_s = 0.0026 \frac{u^2}{g} A^{0.47} \qquad (2.1)$$

and

$$T_s = 0.46 \frac{u}{g} A^{0.28} \qquad (2.2)$$

where

u   is the wind speed (m/s)
g   is acceleration due to gravity (m/s^2)

$$A = \frac{gF}{u^2}$$

and F is the effective fetch (m).

The water in parts of Poole Harbour is shallow even at high tide, and so attenuation of wave energy due to bed friction will occur. The Saville method is intended for use in deep water and so does not include sea bed friction and percolation. Therefore, the values obtained from Equations 2.1 and 2.2 (above) need correcting for these shallow water effects. Equations (3-21) and (3-22) of the Shore Protection Manual (US Army, 1973) give values of H_s and T_s ignoring the effects of friction, and Equations (3-25) and (3-26) give values including friction. These are predicted using a different method, which is less suitable for very short fetches. However the reduction in H_s and T_s due to bottom friction and percolation can be extracted from these equations and used with the H_s and T_s values obtained from the Saville method. From the equations in the Manual the following equations can be obtained.

$$H_{sf} = \frac{H_s \ A2 \tanh (A1/A2)}{\tanh (A1)} \qquad (2.3)$$

and

$$T_{sf} = \frac{T_s \ B2 \tanh (B1/B2)}{\tanh (B1)} \qquad (2.4)$$

where

$A1 = 0.0125 \ A^{0.42}$, $A2 = \tanh (0.578 \ D^{0.75})$
$B1 = 0.0774 \ A^{0.25}$, $B2 = \tanh (0.520 \ D^{0.375})$

and $D = \dfrac{gd}{u^2}$

Here d is the average water depth along the fetch ray and H_{sf} and T_{sf} are the values of H_s and T_s corrected for bottom friction and percolation.

When the wave height was greater than 55% of the total depth at the wave prediction point, it was assumed that the waves would have broken. In this case, the wave height was limited to the maximum breaking wave height of 0.55 x depth.

Waves predicted by the SAVILLE model using wind data from Hurn were compared with field observations taken over several days during an HR Wallingford survey campaign in Poole Harbour in 1990 (HR, EX2122, 1990). Following the adjustments to the wind data described in Section 2.3.2, satisfactory agreement was obtained between model and measurements.

2.3.4 *Wave climate and extremes results*

For the purposes of deriving wave climate information, the representative wind conditions (and their frequencies of occurrence) listed in Table 2.40 were used as input to the SAVILLE model. Two representative water levels were used for the calculations: one to represent shallow water conditions and one to represent conditions of deeper water. The results were apportioned according to the percentage of the time for which these conditions apply. The results were collated in terms of an overall distribution of significant wave height and direction for each prediction point in Poole Harbour.

The results are shown in the form of wave roses in Figures 2.49 to 2.57. With the possible exception of Point C (in Holes Bay) and Point A (at the western end of Poole Harbour), the wave climates predicted for the nine points are quite similar. The prevailing wave direction is south-west and no predicted significant wave height exceeds one metre.

The distributions of significant wave height were extrapolated to extreme values in the same way as was used for the offshore points (see Section 2.2.5). However, in view of the small size of the waves and fact that the results were not to be used in subsequent wave transformation modelling, extremes were calculated only for all directions combined. The predicted extreme wave heights are listed in Table 2.41, together with a standard wave steepness from which the corresponding wave periods can be estimated. Again with the exception of Points A and C, extremes are quite similar at all the Harbour Points. Typically 1, 10 and 100 year significant wave heights are about 0.65, 0.80 and 0.95m, respectively.

The water levels assumed were slightly conservative for wave climate calculations but there was some concern that they may have under-predicted the wave heights occurring at extreme water levels in Poole Harbour. The sensitivity of the wave prediction model to the assumed water level was therefore tested. Sample wave predictions carried out for very high water levels (10 and 50 year return period) gave wave heights only very slightly higher than those predicted for the 2mCD water level. However, with friction and breaking effects "switched off" in the model, the highest wave heights were predicted about 10% higher. These tests confirm that it was important to consider wave generation in Poole Harbour as being limited by the water depth, but that the exact depth used was not important.

2.4 Analysis of Poole Harbour tidal level data

2.4.1 *Methodology*

(i) Background

The tidal range is low in Poole Bay, making it difficult to predict water levels based on measurements taken some distance away. An existing tidal model, re-commissioned and re-run for the present study, provided information on the variability of water level from one point to another in Poole Bay and Poole Harbour.

Long-term tidal data from Portsmouth is not an ideal source from which to infer water levels in Poole Bay because of its distance from the site of interest. Weymouth has an 'A Class' tide gauge but has only been installed recently, and cannot therefore provide long term records on extreme water levels. An earlier A Class tide gauge in Portland Harbour operated for several years, but the results are not available in digital format, limiting their usefulness in the present study. However, tidal levels have been recorded in Poole Harbour for many years, with data from 1991 onward being available in digital format. In the present study, water level analysis was based mainly on the Poole Harbour records combined with the tidal model results.

(ii) Method
Poole BC obtained tidal measurements from the Poole Harbour Commissioners, recorded within the Harbour, for use by HR Wallingford in this study. This data was the basis for the analysis of both extreme water levels and the joint probability of such levels occurring with large waves.

Information on the spatial changes in tidal levels inside Poole Harbour and in Poole Bay was obtained both from the Harbour Master and from (existing) numerical models of tidal flows.

The digital tidal level data was entered into the computer system at HR, and standard analysis software was used to determine the various tidal constituents. These constituents were then used to "hindcast" the expected (astronomical) tide. The differences (residuals) between predicted and measured tidal levels were then calculated. The times and levels of the (higher) high water values for each tide were also identified and used in calculations of extreme high water levels.

(iii) Outputs
The analysis of tidal levels provided:

A time history of the (higher) high water level for each tide (at the tide gauge site), with times and calculated residuals (ie surges) at those times. This data was used in the joint probability study.

A list of the highest recorded water levels, with dates and times, for subsequent use in reviewing historical flood/damage events.

Statistical estimates of extreme water levels (at the tide gauge site) for a variety of "return periods", for example the level expected to occur only once on average in 50 years.

An estimate of the differences between water levels at high tide at the tide gauge site, and at other locations within Poole Bay and Harbour. This information was used in extending the joint probability study to other locations along the Poole BC frontage (see Section 2.5).

2.4.2 The TIRA tidal analysis software
HR Wallingford operates the standard TIRA tidal analysis software under licence from the Proudman Oceanographic Laboratory (POL). The background, theory and some operational details are described in Appendix 5. However, a brief description is given below.

The oceans generally respond to forces such as the gravitational attraction of the Moon and the Sun, to the radiational input from the Sun, and to forces generated by the meteorology. About 90% of the energy contained in sea level records is caused by tidal movements. This is easy to separate from the other components because it is coherent in time, whereas, for example, wind waves occur randomly.

The gravitational attraction of the Moon and the Sun on the Earth produces diurnal and semi-diurnal tidal variations, which have different frequencies and are usually denoted by the symbols M_1 and M_2 for the Moon and S_1 and S_2 for the Sun. The main constituents are modulated at periods of 1 month, 1 year, 8.85 years, 18.61 years and 21,000 years. So although tides are regular they are only repeatable over a very long time scale. Compound tides are generated by the linear tides as they propagate into shallow water or as they encounter frictional forces. They are given names such as M_4 to denote a multiple harmonic of M_2, or MS_4 to denote interaction between the M_2 and S_2 components.

For each location there exists a unique set of harmonic constants which define the complex tidal regime at that place.

The process of analysis is one of reducing a set of tidal level (or current) measurements to a manageable set of physically meaningful parameters which completely specify the measurements. In the harmonic method of tidal analysis the objective is to fit the best possible sinusoidal wave of each frequency to the measured data. Harmonic constants are invariate with time in that they do not depend on the time at which the tide was recorded. Harmonic analysis consists of fitting a finite number of sinusoidal constituents to the data and minimising the residuals by the method of least squares.

The main input to TIRA consists of sequential measured water level data, in this case recorded over two to three years in Poole Harbour. The first stage of the analysis procedure consisted of derivation of sixty tidal harmonic constants, from which the main astronomical component of water level was re-constituted. The second stage consisted of a comparison between the actual water level data and that which would have been predicted from the derived harmonic constituents, to leave the separate astronomical and residual components. Once derived, the tidal constituents can be re-used for prediction of astronomical tides in Poole Harbour at any future times.

2.4.3 Water level and extremes results

Water level data from Poole Harbour was supplied on floppy disk to HR Wallingford in the form of ten minute mean values recorded between 1991 and 1993. On receipt, a few obvious "spikes" in the time series data were detected visually, and were either corrected or deleted. A few short periods of missing data were identified and then either noted or patched (important in the TIRA analysis, but not in the later statistical analysis of water levels). When the TIRA analysis proved to be sensitive even to single missing records or to slight changes in the interval between records, further quite extensive "polishing" of the measured data was carried out.

Time series water level data as recorded in Poole Harbour from 1991 to 1993 is plotted in Figures 2.58 to 2.66. Levels vary between about 0.0mCD and 2.9mCD. There were only 10 spells during the period of the measurements plotted in Figures 2.58 to 2.66 when the water level exceeded 2.5mCD. The dates of these spells are listed below, together with the associated peak water levels. These dates give some indication of when flooding might have been expected to have occurred along the Poole BC frontage.

Dates of spell	Peak water level during spell
am11/09/91-pm11/09/91	2.60mCD
pm24/09/91-pm28/09/91	2.85mCD
am07/10/91-am09/10/91	2.71mCD
pm27/08/92-am31/08/92	2.89mCD
am27/09/92-pm27/09/92	2.52mCD
am25/10/92-am28/10/92	2.61mCD
pm24/11/92	2.52mCD
pm21/02/93	2.55mCD
mid-day 09/03/93	2.50mCD
pm14/10/93-am16/10/93	2.59mCD

TIRA analysis yielded the tidal harmonic constituents, required for calculation of the astronomical tidal levels. These are listed in Table 2.42. The analysis also separated out the surge and astronomical components from the measurements of total water level.

Surges from 1991 to 1993 are plotted in time series form in Figures 2.67 to 2.74. The plots are quite spiky due to incomplete removal of the tidal component of water level. The TIRA analysis was very sensitive to slight irregularities of timing (of the order of just a few minutes) in the tide data, particularly in this area of low tidal range and double-peaked high waters. Surges can be seen in Figures 2.67 to 2.74 by following the general trend of the trace. They vary between about -0.45m on 2 July 1991 and +0.95m on 21 February 1993. The profile of the February 1993 surge is shown in greater detail in Figure 2.75

Since water levels and surges are of greatest interest at the peak of the tide, records corresponding to (higher) high water levels were extracted from the ten minute mean data. The distribution of total water level as recorded at high waters in Poole Harbour is given in Table 2.43.

Gumbel and Weibull distributions were fitted to the high water data in the manner described in Section 2.2.5. Extremes (at the location of the tide gauge) derived from extrapolation of those distributions are given in Table 2.44. Extreme water levels of 3.00 and 3.25mCD are predicted for return periods of 10 and 100 years, respectively.

Extrapolation from just two years records is somewhat uncertain. Subsequent discussion with Poole BC indicated that the highest water level ever recorded was 3.06mCD in December 1989, and that prior to that the highest level had been 2.96mCD, 30 years earlier. These observations suggest that the extremes prediction in Table 2.4.4 are about 10cm too high, perhaps due to the presence of two particularly high water levels (September 1991 and August 1992) in only a two year sample.

Tidal ranges increase slightly going eastward across Poole Bay. Data from the tidal model and information given in Admiralty Tide Tables were used to estimate the variation in high water level likely to occur along the frontage of interest. Analysis indicated that the variation within Poole Harbour and at inshore points A and B is negligible. 3cm should be added to extreme water levels at points C to E, 5cm at point F, and 7cm at points G to H.

2.5 Joint probability analysis of waves and water levels

2.5.1 Methodology

(i) Background

Flooding and damage to coastal properties rarely arises just because of a very high tide, or just because of severe wave conditions. In almost all cases it is a combination of these two factors that is involved. Because of the modest (astronomical) tidal range in Poole Bay, the highest recorded tidal levels always include a significant weather-induced residual or "surge". As such surges typically result from low atmospheric pressure and strong winds blowing along the English Channel it is likely that the waves will also be significant at the time. There is a likelihood of at least a partial correlation between high water levels and large waves in Poole Bay (Hague, 1992). In Poole Harbour, where waves depend on the local wind strength and its precise direction at the time of high water, any correlation may be different to that outside in Poole Bay.

By ignoring such correlations, there is a danger of under-estimating the chances of flooding or damage. On the other hand by taking a pessimistic view, and assuming the worst wave and tide conditions will always occur simultaneously, it is probable that coastal protection works will be over-designed and unduly expensive. An accurate assessment is therefore needed of the joint probability of these two factors, and a derivation of appropriate combinations of them for use in the design of new coast protection works (or the assessment of existing structures).

(ii) Methods

The most satisfactory way of establishing "joint probabilities" is by analysis of simultaneous recorded (or hindcast) wave conditions and tide levels.

The main danger of damage and flooding occurs at the top of each tide, ie at the peak water level. By combining the information on these peak levels for each tide with the hindcast (offshore) wave conditions at the same time, the probability of the joint occurrence of large waves and high tides in Poole Bay was calculated.

In Poole Harbour, the calculations were slightly different in that the information on peak levels was combined with the calculated nearshore wave conditions, which vary depending on the exact location within the Harbour.

Having derived the individual (ie marginal) probabilities both of wave conditions and of tidal levels, together with the joint probability, it was then relatively straightforward to derive appropriate combinations of these two factors, with a specified probability of occurrence.

(iii) Outputs

The output from the joint probability study is a series of tables which give the probabilities of high tidal levels and simultaneous large waves, both in Poole Bay and at a number of locations in Poole Harbour.

In addition, tables listing recommended combinations of high tide level and wave heights for design purposes are presented for each chosen location around the Poole Borough coastline.

Finally, these "present day" design conditions are adjusted to take account of predicted increases in sea levels caused by global warming.

2.5.2 The JOINPROB joint probability analysis software

The joint probability of two variables (X and Y) is given by the likelihood ($0 \leq P(X \geq x \text{ and } Y \geq y) \leq 1$) of variable X being not less than a given value x, at the same time as variable Y being not less than a given value y. In this case X and Y are high tide level and significant wave height. Two trivial cases of joint probability are complete dependence and complete independence. Two variables, Tide and H_s are completely dependent if a given tidal level always occurs at the same time as a given wave height H_s, ie:

$$P(\text{Tide} \geq x \text{ and } H_s \geq y) = P(\text{Tide} \geq x) = P(H_s \geq y)$$

On the other hand if they are completely independent then there is no correlation between them and the joint probability is simply the product of the two marginal probabilities, ie:

$$P(\text{Tide} \geq x \text{ and } H_s \geq y) = P(\text{Tide} \geq x).P(H_s \geq y)$$

In the case of waves and water levels, the assumption of complete dependence would lead to a very conservative design since the 100 year event would have to comprise a 100 year wave condition coupled with 100 year water level. Conversely, the assumption of independence would lead to under-design in some cases, since any increase in the probability of high wave heights at times of very high water levels would have been ignored. The correlation between waves and water levels will usually lie between the two extremes of complete dependence and complete independence. The two will be partially dependent, to an extent best determined from analysis of actual data. JOINPROB analyses the dependence, in an objective way based directly upon the data, and making no prior assumption about correlation (Hawkes and Hague, 1993).

From the file of inshore hourly wave and water level conditions, wave heights at each of the high tides are picked out from the predicted waves: scatter diagrams relating high tide levels and simultaneous wave height predictions are produced.

Probabilities are extrapolated to extremes using JOINPROB, which first fixes a number of threshold levels for water level and calculates the extreme wave heights conditional on water level being greater than these levels using the Weibull method of Appendix 3. Similarly wave heights are fixed and extreme tidal levels are calculated using a combination of the standard Gumbel annual maxima method (for return periods of over 10 years) and the "countback" method (for return periods of over 10 years or less) which is best explained with reference to the following example. If, for a 20 year data set, the 0.1 year return period water level (ie the level equalled or exceeded on average ten times per year) is required, then this is given by counting back to the 20 x 10 = 200th highest value recorded.

These extreme values of wave height and water level are then plotted on a graph, and smooth curves are drawn linking combinations of waves and water levels with equal return periods. The spacing between the contours should be approximately equal for each factor of ten (or any other convenient factor) increase in rarity of event represented. So, for example, the 1, 10 and 100 year return period contours should be drawn so as to have roughly the same spacing.

Also plotted on the graphs are the independent and dependent cases for each return period. These are obtained by applying the literal definitions of independence and dependence given above.

The extremes of wave height (or water level) alone are defined such that the given value is expected to be equalled or exceeded, on average, once in each return period. The definition of joint probability extremes is an extension of this but the exact meaning should be understood clearly before the results are used. For a given return period both wave height and water level should be equalled or exceeded once in each return period. Therefore all combinations of wave height and water level along a return period curve are expected to occur once in each return period, although they may well not involve separate events; ie one event will probably cover a section or even all of the curve.

Consequently when designing a structure for say, a 100 year return period, it must be able to withstand <u>all</u> of the wave and water level conditions on the 100 year curve. In practice this means testing a range of conditions for the return period considered.

2.5.3 Results of JOINPROB correlation and extremes analysis

Offshore Point 2 was chosen for detailed correlation and joint probability analysis. Approximately 13 months of simultaneous wave and water level data was available for use in JOINPROB. Still water levels as recorded at high water in Poole Harbour were coupled with wave predictions for the times of high water at Offshore Point 2, providing a total of 766 records for use in the correlation analysis.

It was expected that the degree of correlation between high waves and high water levels would be dependent upon wave direction. Because of this, the records were divided into two categories of wave direction, namely 330-210°N (for waves generated within the English Channel) and 210-330°N (which includes wave energy from further afield).

Figures 2.76 to 2.78 present scatter diagrams of significant wave height against high water level for each direction sector and overall, contoured with lines of approximately equal probability density. The general shape of the contours suggests little or no correlation between wave heights and water levels. However, three particular records (highlighted by separate contours in Figures 2.76 to 2.78) stand out as having high wave heights at the same time as high water levels. These records suggest a degree of correlation between the two.

The Poole Harbour data supplied proved to be unsuitable for calculating reliable extreme joint probability contours. This is because of the three "outlier" records in what is a fairly short sample of data (usually at least five years of simultaneous wave and tide data would be used in analysis).

Fortunately, in a previous JOINPROB analysis at a similarly exposed location off Christchurch Harbour and using a much longer data set (Hague, 1992), more reliable and consistent results were achieved. Figure 2.79, reproduced from Hague (1992), shows a significant correlation between waves and water levels off Christchurch Harbour. (Only the 'all directions' diagram is reproduced here but the direction dependent results showed similar levels of correlation.) An intuitive assessment of Figures 2.76 to 2.79 produced a 'correlation factor' of about 140 for a 100 year joint return period. This represents the number of times it is more likely that high waves and high water levels will occur simultaneously than the assumption of independence (see Section 2.5.2) would suggest. This degree of correlation should be reasonably representative of conditions in Poole Bay.

Joint probability extremes were evaluated based on the extreme wave predictions for Offshore Point 2 (Section 2.2.10), the extreme water level predictions (Section 2.4.3) and the "correlation factor" between them (this section). A representative sample of conditions with a 100 year return period are listed in Table 2.45. Note that *each* of these combinations is expected to be exceeded once, on average, in the given return period.

There is up to about a factor of five uncertainty in the degree of correlation assumed to exist between wave heights and water levels. This factor of uncertainty feeds through directly into the joint probability return period. In other words a combination of events estimated to have an overall return period of 100 years could really have a return period of anywhere between 20 and 500 years. At some time in the future, it may be worth repeating the analysis with a longer data set, perhaps making use of longer term tidal data collected by Bournemouth BC.

2.5.4 General results of the joint probability analysis

An extreme water level would affect the whole study area almost equally. The same type of wind conditions (ie high winds from east, through south, to west) would produce high waves throughout the study area. The quite high degree of correlation between deep water waves and water levels appeared not to be dependent upon wave direction. The correlation between high waves and high water levels was therefore assumed to be the same for all the Poole Bay and Poole Harbour wave prediction points as for Offshore Point 2 (see Section 2.5.3).

The table of joint probability extremes for each point could be determined from Table 2.45 derived for Offshore Point 2. However, the high water level is known to vary slightly over the area of interest (see Section 2.4.3). Therefore the best estimates of water levels at each point were substituted in place of the water levels at the tide gauge. Similarly, local extreme wave conditions, for each direction sector and overall, were used at each point.

Joint probability extremes for each nearshore point in Poole Bay are given in Tables 2.46 to 2.49 in the same format as Table 2.45, but now using locally applicable water levels and wave conditions. Corresponding results for the Poole Harbour points are given in Table 2.50. Water levels in Tables 2.45 to 2.50 are given relative to Ordnance Datum, as this is usually more convenient for use in design.

2.5.5 *Allowance for expected sea level rise*

Mean sea levels have been rising at a rate of 1-2mm/year for about the last century. The rate of rise is predicted to increase to about 5-6mm/year and to continue at that rate for the foreseeable future. As there is no particular evidence to the contrary, extreme water levels are assumed to rise in line with mean water levels.

The numerical allowance for sea level rise is a design decision, but a possible method of determining the allowance follows. Assume that a coastal defence has a design life of 50 years and that it should have a probability of failure equivalent to once in 100 years at the end of that period. The design would then be based on 100 year return period conditions, but with the addition of the sea level rise expected to occur over the next 50 years. In other words, the 100 year extremes calculations are undertaken with present day data, but with 275mm added to all of the water levels previously calculated.

There is some evidence that mean wave heights in both the Atlantic and the North Sea have increased in recent years, although no clear evidence either that the trend will continue in the future or that extremes are increasing. There is no corresponding increase in wind conditions and the relatively sheltered Poole BC frontage would probably not be affected by such changes. No allowance for future changes in wave conditions is therefore recommended.

2.5.6 *Comments on the use of the results in design*

The water level results are quoted without any allowance for expected sea level rise. Any such allowance would consist of simply adding a given amount to all water levels as described in Section 2.5.5. The exact amount to be added is a design decision depending mainly upon the design life of the defences.

Wave heights are quoted in terms of the significant wave height averaged over the duration of a single high water. The height of the highest individual wave height will be approximately twice the significant height.

The extreme wave heights should be checked for depth limitation before use at any particular point on the coastal defences. The depth used should be taken just offshore of the defences, following any addition for sea level rise.

There are a number of combinations of wave conditions and water levels for any given joint return period. Each one is expected to be equalled or exceeded once, on average, during that return period. It is therefore necessary to test each combination given in order to see which is the critical case for design. (This may depend upon whether run-up, overtopping or littoral drift is the more important parameter and whether or not wave heights are depth-limited before they reach the defences: the critical combination may vary from place to place.)

Only high tidal levels are of interest. The wave conditions were averaged over a period of 3 hours on the assumption that they would persist over the duration of a high water level. Surges may last for several hours but a severe surge will not usually persist over two consecutive high waters.

It follows that each high water is an "independent event", and that the duration of the predicted conditions is equal to the duration of the high water level. In other words, overtopping or damage may begin up to an hour or so before high water, will become worse as the water level reaches its highest level, and will then gradually reduce for up to an hour or so after high water. (The wave conditions would persist for the remainder of the tidal cycle but less damage would occur at the lower water levels.)

3 Coastal evolution and processes

3.1 Introduction

At the heart of a coastal strategy study is an understanding of the morphological processes taking place on the coast. In order to understand present day processes an appreciation of the geology of the area and its geomorphological development is extremely useful. The following section of this chapter therefore describes and explains the geology of the Poole area and geomorphological developments within the area since the last Ice Age.

Moving closer to the present day, a study of shoreline changes that have taken place along PBC's Poole Bay shoreline over about the last 100 years is described. This information is interpreted in the light of the results of numerical sediment transport modelling and a considerable body of previous research to give guidance on future coastal behaviour.

Sediment transport on the sea bed of Poole Bay is also examined so as to identify any impact which sea bed processes may have on the coast.

The influences on shoreline change within Poole Harbour are somewhat different to Poole Bay. Waves and currents combine to create an unusual hydraulic environment which in turn has shaped sea bed and coastal sediment distribution. Both human development, for example by land reclamations, and the changing extent of saltmarsh vegetation have had a major impact on the morphology of the Harbour. Each of these issues is examined in the section on Poole Harbour by interpretation of the results of numerical modelling and by reference to previous research.

3.2 Geology

3.2.1 Background

Geological structure and lithological variability have both exerted a strong influence on the morphology and the evolution of the coastline of Southern England. Both the underlying sedimentary rock geology and the major shoreline changes which have taken place since the end of the last Ice Age have an important bearing on present-day processes. The most recent geological memoir for the Bournemouth area (Bristow et al, 1991) which benefits from recent borehole and survey results, reinterprets and supersedes previous publications. The following geological summary is based on the findings of Bristow et al (1991) with reference to previous publications.

The geology in the Poole and Bournemouth area is sedimentary, underlying beds having been laid down during the Cretaceous and Eocene.

The formations outcropping in the study area are summarised as:

Period	**Epoch**	**Formation**
Quaternary	Recent:	Blown Sand Alluvium
	Pleistocene:	River Terrace Deposits Head

Tertiary	Eocene:	Branksome Sand Poole Formation London Clay	} Bracklesham Group

The nomenclature for formations which appears on the most recent geological survey (Sheet 329) and has been reproduced above differs from previous classifications. The evolution of the nomenclature of the Eocene rocks is represented in Figure 3.1. The Plateau Gravel and Valley Gravel which are referred to in previous publications are now collectively known as the River Terrace Deposits

3.2.2 Solid geology

The Cretaceous chalk does not outcrop on PBC's frontage but forms the cliffs and Old Harry stacks on the south side of Studland Bay. Thick beds of chalk were formed by compaction of remains of shallow water marine organisms. The Upper Chalk is a firm white fossiliferous chalk with scattered layers of flint nodules and thin marl seams. There is an abrupt change in the stratigraphical succession above the chalk and the overlying beds of sands and clays which were deposited during Eocene.

The Tertiary formations, which underlie most of the district, total around 420m in thickness. Much of the higher part of the sequence is well exposed in the coastal cliffs from Poole Head eastward.

The Tertiary deposits comprise sediments ranging from clays to gravel beds. The sequence consists of a number of sedimentary cycles each of which commenced with a marine transgression.

The oldest of the Tertiary deposits is the London Clay which outcrops as the so called Creekmoor Clay bed on the north side of Lytchett Bay and Holes Bay. These are overlaid by the inter-bedded sands and clays of the Poole Formation which range from fluvial through deltaic to marine in origin, with lateral variations in material type occurring at the same stratigraphic horizon. The Branksome Sand is a dense, coarse to fine grained sand of fluvial origin which is white and yellow in colour (Golder Associates, 1990). It is a 70m thick bed and outcrops widely on the PBC frontage at its greatest height in the cliffs on Poole Bay.

Following deposition of these and subsequent (now eroded) beds the whole region experienced folding associated with the formation of the Alps (the Alpine orogeny). The folds are aligned in a roughly east-west direction (plunging towards the east) across Southern England. The trough of the Hampshire Basin syncline lies more or less across Poole Harbour. North of the Harbour the beds dip by 1° or 2° towards the south. The other limb of the fold dips more steeply towards the north.

The land raised up by this folding was gradually eroded, so older strata were exposed at the surface. Thus the youngest rocks are found in the trough of the syncline in Poole Harbour whilst the older chalk outcrops further south near Studland. The chalk forms a spine running east across Poole Bay to the Isle of Wight and was a dominant influence on subsequent morphological changes.

3.2.3 *Morphological changes since the Pleistocene Ice Age*

The Quaternary era saw many oscillations of climate, ranging from cold periglacial to warm temperate. During the more extreme cold periods extensive ice sheets pushed southwards across England but did not reach the Poole District. Marked changes in sea level were associated with these climate oscillations. During the Pleistocene, Britain was joined to Continental Europe and the coastline was further to the west, probably in a position between Cornwall and Brittany. The Channel was occupied by a large river system draining westwards, combining rivers from both sides of the Channel and may have included the outflow of the Thames and the Rhine at times when the ice blocked the North Sea.

The rivers of the Poole area, swollen with meltwater, drained into the Solent River which flowed across the floors of Poole and Christchurch Bays and thence through the Solent and Spithead (see Figure 3.2). Rising sea levels at the end of the last glaciation induced rapid coastal recession so that river valleys and low-lying areas were submerged. Gradually the valley of the Solent River and its tributaries became flooded leaving a narrow barrier of chalk hills running from the Isle of Purbeck across Poole Bay to the Isle of Wight. Eventually the sea breached this barrier and the Isle of Wight was separated from the mainland by the wide tidal channels of the East and West Solent.

Once the Purbeck-Isle of Wight ridge had been breached coastal erosion could proceed across the valley of the former Solent River. Rising sea-levels redistributed eroded sediment within Poole Bay. Breaching of the West Solent -Needles Channel initiated rapid erosion of Christchurch Bay and began the coastal evolution to the double bay form which now exists. The bays are stabilised by the anchoring effects of Handfast Point, Hengistbury Head and Hurst Castle Spit but, as is discussed in Section 3.3.1, they do not yet appear to have attained a natural equilibrium plan-shape.

In Poole Bay the chines are a noteworthy feature (see Figure 3.3). Branksome Chine shows two distinct valleys of different ages and the newer and steeper valley alone carries water. Steers (1969) proposes that the older, broader and longer chine valleys were formed in a cold period by large volumes of meltwater flowing over ground which was impervious owing to frost, and when the rocks in which they are cut would have been effectively harder and more resistant than now. The smaller inner valleys are the result of present climatic conditions and represent the power of the present tiny streams in cutting a small V-shaped valley in the relatively soft sediments

Poole Harbour owes its main outline to submergence (from about 8500 years before present) converting what was formerly a low moorland area with sandy knolls into a broad lagoon or estuary. The former knolls now form islands: Green Island, Furzey Island and Brownsea Island. The numerous bays and inlets of its margins represent former shallow valleys which might reasonably be expected to have characterised a lowland heathy area traversed by a big river. Submergence of the harbour is due to a combination of Holocene sea level rise, sinking land mass due to isostatic readjustment, and slow local subsidence. The maximum extent of the marine transgression, reached about 6000 years ago, is marked by a low bluff or by cliffs surrounding much of the Harbour above the present shoreline. The low cliffs at Rockley Sands are examples of continuing coastal erosion. Local sedimentary processes have now extended the shoreline somewhat seawards of the cliffs in sheltered areas with low-lying beaches or mudflats. For example the low white cliffs at the back of Parkstone Bay were, in the past, fronted by low-lying marsh which has now been reclaimed as a recreation ground.

3.2.4 Quaternary deposits

The melting of ice towards the end of the Ice Age produced large amounts of sediment. These were distributed by the swollen river systems around what is now the coastline and seabed. The oldest of these deposits is known as the Head deposits and is found in shallow valleys around the Poole Bay and Poole Harbour frontage. Head deposits vary markedly in their composition and thickness. A series of fluvial deposits known as the River Terrace Deposits occur extensively in the Poole area and consist mainly of flint gravel which is commonly very sandy. These River Terrace Deposits are readily visible overlying the cliffs of Branksome Sand on the Poole Bay frontage. Remnants of these deposits have also been recognised by offshore surveys in Poole Bay (Fitzpatrick, 1987).

Freshwater peat deposits found at a depth of -12.8m ODN during excavation works at Hamworthy are overlain by an extensive series of marine deposits. Radiocarbon dating of these deposits at about 7340 years before present indicates the onset of marine transgression in this part of the Harbour (Dyrynda, 1987).

Recent deposits comprise layers of alluvium laid down by existing streams and in marshy areas of Poole Harbour, for example in Lytchett Bay. Sedimentation of these fine materials has kept pace with rising water level to form the deep sediment column which now floors much of the harbour. Dunes of blown sand are still in the course of deposition in Shell Bay. Construction on the Sandbanks Peninsula has prevented growth of dunes in recent years.

It is nowhere reported at what depth beneath the dunes on Sandbanks are older Quaternary or Tertiary sediments to be found. Whilst the evolution of South Haven Peninsula around a core of Head gravel and clays of the Poole Formation has been studied by Steers (1969) and others, the development of Sandbanks is not so thoroughly reported. The width and stability of the peninsula between the Haven Hotel and North Haven Point suggest that, like the South Haven Peninsula, this part of Sandbanks may also be based upon a core of older deposits.

3.2.5 Geological SSSIs

It should be clear from the preceding discussion that the geology of the cliffs on the PBC frontage is of considerable interest in its own right. In recognition of this, Sites of Special Scientific Interest have been established, the extent of which are shown in Figure 3.4. Future management of the coastline will need to take the differing attributes and environmental importance of these sites into account. Recommendations in management of geological SSSIs are included in Appendix 6.

3.3 Morphology of Poole Bay

3.3.1 Morphological development

The morphodynamic mechanisms in Poole Bay are driven by waves, currents and wind. The morphological processes within the Bay depend on how these forces interact with the sediments and geological formations. Both southerly and south-easterly winds can generate severe waves within the English Channel, which can affect the plan-shape of the beaches. However, it is the waves generated by the more common westerly and south-westerly winds, together with the long fetches towards the Atlantic, which dominate the hydraulic environment of Poole Bay. In the eastern part of Poole Bay the coast is exposed to waves approaching from the south-west travelling up the English Channel from the Atlantic. The further one progresses westward from Bournemouth towards Studland, the greater is the shelter provided by the Isle of Purbeck and Handfast Point. Waves from the south-west are refracted and diffracted round these obstacles to approach from the south and south-east, having been much diminished in height and energy.

The wave climate towards the sheltered western side of Poole Bay thus tends to be less severe than further east. The currents induced by this wave height gradient are one of the forces driving the morphodynamic processes in Poole Bay. This tends to lead to a general accumulation of sediment in the western part of Poole Bay, eg. Hook Sand and the South Haven peninsula. Superimposed on these wave-induced currents are tidal currents. Because the tidal range is small the contribution of tidal currents to sediment transport is slight apart from near the mouth of Poole Harbour where strong currents are the dominant influence on sediment transport.

Both Poole and Christchurch Bays display crenulate plan form characterised by resistant headlands and a soft coastline in the indentations. Research by Silvester (1970) investigated the development and stabilisation of crenulate bays in sedimentary coastlines. Conformity of Poole and Christchurch Bays to Silvester's theory is poor (Webber, 1980) but some worthwhile conclusions regarding the long term development of the bays can be inferred:

- From the relationships derived for crenulate bays it would appear that in neither Poole or Christchurch Bays has an equilibrium state been established.

- Poole Bay has a potentially much greater indentation and Christchurch Bay a slightly greater indentation than at present before such an equilibrium is reached.

- Whilst the westward promontory of Handfast Point is fairly stable, Hengistbury Head prior to the construction of the long groyne in 1938 was eroding at a rate approaching 1m/year. Erosion of this headland would induce major instability and potentially rapid coastal retreat and readjustment within Poole and Christchurch Bays.

3.3.2 Shoreline changes

Although macro-scale coastal readjustment within Poole Bay has now been arrested by coast protection works, retreat of the sedimentary cliffs has in the past characterised the morphological development of the bay. Figure 3.5 gives an overall impression of the rates of cliff to recession in Poole Bay over the past century. Before extensive coast-protection works there was a steady recession of the cliff top along the length of Poole Bay. Coast protection works from about 1890 onwards have effectively prevented or reduced cliff erosion but the resulting progressive reduction in the supply of fresh sediment to the beaches has lead to a continuing problem of beach lowering in Poole Bay.

Erosion of the cliffs around the bay has in the past been the largest contribution of sediment to the coastal zone. By comparing historic cliff recession, height and particle size distribution Lacey (1985) assessed the supply of eroded sediment to the beaches of Poole and Christchurch Bays. The early part of the last century showed a contribution to the beaches of 91,000 m^3 year which reduced in the latter part to 66,000 m^3 year. The present day contribution of the cliffs from Southbourne westwards is negligible. The sediment contribution changes with time and also varies along the coastline as depicted in Figure 3.6. The increase in beach material contributed to the coastal system from 1925 onwards west of Canford Cliffs is due to the increased erosion of the cliff prior to the building of the promenade along these sections.

3.3.3 Analysis of shoreline changes on PBC's Poole Bay frontage

In order to supplement previous research work and to look in detail at PBC's frontage in Poole Bay an analysis of shoreline change was carried out specifically for this study.

The information used in this part of the study comprised published Ordnance Survey maps at a scale of 1:2500. The dates quoted throughout this section of the report refer to the published dates of the maps and not to the dates of the surveys (which are not indicated on the maps). With the exception of the latest map (1993) the earlier maps were photocopies of originals which may have some distortion and scale variation. Great care was taken when extracting information but some inaccuracies are inevitable when using photocopies. Registration was sometimes difficult especially on the earlier maps when reliable fixed features were not always available.

It is possible that, over the period covered in this study, the Ordnance Survey could have changed the designation of HW level for the purpose of defining the HW contour but this information is not shown on the maps.

The period examined has been divided into three sections, namely:

1901 - 1925
1925 - 1955
1955 - 1993

Although a map edition dated 1932 was available, a close examination indicated that both the cliff line and the HW contour were identical to those shown on the 1925 map. This does not necessarily mean that no changes occurred but is more likely to be the result of these features not being re-surveyed in the intervening period.

The description below is related to the sub-sections of Management Unit 1 (described in Section 4.4.2) which are shown on Figures 3.7 to 3.10.

(i) Changes in the position of the HW contour between 1901 - 1925

A clear pattern emerges from this first period with a landwards movement of the HW contour amounting to about 18m at sub-section 1/4 (see Figure 3.7) gradually declining to zero at sub-section 1/9. Towards the south-west there was a seawards migration of the HW contour gradually increasing to about 50m at the south-western end of 1/11 and then rapidly declining to zero at 1/14. Beach accretion on Sandbanks is associated with construction of groynes on the beach at the end of the last century. Between 1/14 and the south-western end of 1/15 the contour moved landwards by up to 12m.

During this initial period, between 1901 and 1925, the HW contour undertook a re-alignment rotating anti-clockwise about 1/9 by about 2 degrees. Positions to the north-east eroded whilst accretion occurred to the south-west. The results are shown on Figures 3.7 and in Table 3.1.

(ii) Changes in the position of the HW contour between 1925 - 1955

No clear pattern of change was identified during the second period studied - the changes were generally very small except in the groyne bays at 1/12 - 1/14 where the accretion noted during the earlier period continued (Figure 3.8).

Between 1/15 and 1/6 the HW contour moved landward by up to 5m, indicating erosion, but between 1/7 and the north-eastern end of 1/10 seaward movement by up to 7m took place. A small stretch in the centre of 1/10 accreted but the remainder of 1/10 and the whole of 1/11 exhibited no change between 1925 and 1955. As indicated earlier the most significant change during this period was one of accretion in 1/12 to 1/14 where the HW contour moved seawards by up to 17m. The length of coast between 1/15 and 1/16 eroded with an isolated change of 25m in the position of the HW contour located in the centre of 1/15.

(iii) Changes in the position of the HW contour between 1955 - 1993

No very clear pattern of change occurred during this latter period although there was a tendency for accretion over the north-eastern half of the study area and erosion closer to the point (Figure 3.9).

Between 1/4 and 1/5 the HW contour migrated seawards by up to about 8m but in 1/4 what little movement occurred was in a landwards direction. Further down the coast between 1/7 and half way along 1/10 the movement of the contour was seawards by up to about 9m. From the middle of 1/10 landward movement occurred which steadily increased from near zero at this point to about 24m at 1/14 where the change in position of the contour dropped back to near zero between here and the end of the Sandbanks peninsula.

(iv) Changes in the position of the cliff line between 1901 - 1925

At the north-eastern end of the study area slight erosion of the cliffs occurred amounting to up to 4m in 1/4 (Figure 3.7). No change could be measured in 1/5. In 1/6 the north-eastern end eroded by up to 8m, there was little change in the centre but substantial erosion amounting to a change in the cliff line of up to 28m at the south-western end (Canford Cliffs). No change in the location of the cliff line could be identified in 1/7 between the 1901 and the 1925 maps. A further stretch of the cliff was lost between the centre of 1/8 and its south-western end and just into 1/9. The maximum loss over this stretch was 24m near the centre of 1/8 but this reduced rapidly towards the south-west.

(v) Changes in the position of the cliff line between 1925-1955

In the period between 1925 and 1955 the cliff line in 1/4 retreated by up to 9m. Further to the south-west in 1/5 the change was less at up to 5m and in the north-eastern end of 1/6 even less change was identified. At the south-western end of 1/6 a 200m length of cliff retreated by up to 8m. No change occurred in 1/7 or in the north-eastern half of 1/8. The remainder of 1/8 exhibited a small change amounting to about 1m but 1/9 lost up to 10m of cliff. The results are shown diagrammatically in Figure 3.8 and are tabulated in Table 3.1.

(vi) Changes in the position of the cliff line between 1955-1993

A close examination of the 1993 map showed that with the exception of the 1/4 and 1/5 no change in the cliff line occurred between this and the earlier 1955 map. This is because by the end of this period the frontage was protected by a seawall/ promenade. The change amounted by up to 6m and up to 8m in 1/4 and 1/5 respectively. The results are shown diagrammatically in Figure 3.9 and are tabulated in Table 3.1.

(vii) Summary

As demonstrated in Figure 3.10 the largest changes in position of both the HW contour and the cliff line occurred during the period between 1901 and 1925. The HW contour moved landwards by as much as 18m at the north-eastern end of the study area (1/4) and seawards by a maximum of 51m at Sandbanks (1/11). At two locations (Canford Cliffs and just south-west of Flag Head Chine) the cliff retreated by about 1m per year on average during the period between 1901 and 1925 whereas in the latter two periods (1925 to 1955 and 1955 to 1993) the maximum rate of erosion was considerably less at 0.3 m/a and 0.2 m/a respectively.

3.3.4 Beach sediment transport

Long-term changes in sand or shingle shorelines are usually caused by variations from point to point in longshore transport of beach material. As with many other parts of the modelling of the Poole BC coastline, use was made of previous investigations (HR, SR 1460, 1986 and HR, EX 2228, 1991). The sediment transport modelling carried out for this benefited from wave rider data used to verify wave climates in Poole Bay and from representation of wave diffraction around Durlston Head (see Section 2.2.6)

Littoral drift is a continuous process occurring throughout the year in both commonly occurring and storm wave conditions. The rate of drift at any moment is sensitive to wave height, wave period and wave direction. There is usually considerable seasonal and annual variability in drift rate. The average gross drift over a year, ie the sum of the leftward and rightward drift rates (as seen by an observer facing seaward), is often very much higher than the average nett drift rate, ie the difference between the leftward and the rightward drift rates.

Different wave conditions produce different rates of drift and occur for different percentages of the time. To estimate annual drift volumes it is therefore necessary to have detailed wave climate data, comprising as a minimum the distribution of wave height against direction and a typical relationship between wave height and period.

The HR Wallingford DRIFTCALC model was used to calculate potential sand transport rates. Each wave condition in DRIFTCALC is transformed inshore to its breaker point, assuming parallel-contoured refraction and shoaling effects. An equivalent leftward or rightward rate of drift is calculated for each wave condition. A volume of leftward or rightward drift over a year is then calculated, with reference to the frequency of occurrence of that wave condition. The volumes are summed to give an estimate of the gross and nett drift rates.

Methods for assessment of long-term changes in littoral drift rates on UK beaches were developed during a previous HR Wallingford study of climate change (Jelliman, Hawkes and Brampton, 1991). Using the 19 years of simulated wave climate data produced by HINDWAVE, the potential littoral drift was hindcast for individual years from 1974 to 1992. As well as estimating the long-term annually averaged littoral drift rate, and its variability between the prediction points, it was possible to analyse the year-by-year variability and trends over the 19 year period.

It was originally intended that DRIFTCALC would be run using the annually averaged wave climate and the typical beach normal at each of the eight Poole Bay wave prediction points. However, as the study progressed it became obvious that useful results could not be obtained for some of the prediction points. Inshore Point A provided deep water wave data for use throughout Studland Bay, but because of the range of beach orientations in the Bay, it was impossible to define a realistic beach normal direction. The remaining seven locations (B-H) were used in the DRIFTCALC analysis.

The predicted annually averaged potential nett and gross drift rates are listed in Table 3.2. The units used are cubic metres of material moving through a typical beach cross-section per year. (To obtain the approximate equivalent rates in tonnes per year, multiply the figures by about 1.8.) For completeness, other information such as the beach normal and the mean inshore wave direction are included in the table. There is great variability from point to point and from year to year. The lowest predicted rate of drift in Poole Bay is at Inshore Point E where the average inshore wave direction is almost equal to the beach normal direction. Individual drift rate predictions are quite uncertain, but the results listed in Table 3.2 give a good idea of the volumes of material involved.

For each of the Poole Bay points, the gross potential littoral drift rates for each individual year of wave data were calculated, and the results are plotted in Figures 3.11 to 3.17. The corresponding results for *nett* drift rates for each year are shown in Figures 3.18 to 3.24. There is great inter-annual variability at all the prediction points, both in the gross and nett drift rates. There is a suggestion of long-term trends in some of the nett drift plots. In particular the rate of nett easterly drift at Points D to H is predicted to have increased substantially from 1974 to 1990. The nett drift is predicted to have changed direction from west to east during that period at Points E and F.

Discussion
For points E to H, in the eastern part of Poole Bay, the results concur with previous researchers (Henderson, 1979, Lacey, 1985) in predicting a drift towards the east. However Table 3.2 indicates that the littoral drift is towards the east around the whole of Poole Bay (east of Poole Harbour entrance). Eastward drift is confirmed by observation of build-up of sand on the western side of groynes on the PBC frontage (observed as long ago as 1903 by Vernon-Harcourt), by historic photographs (see Plate 4.1), and surveys from this century which show the same trend.

Previous researchers (HR, EX 2228, 1991, Henderson, 1979, Lacey, 1985) have indicated a westerly drift in this area. Their hypothesis is supported by the classic spit shape of Sandbanks and by the fact that the only source of sediment in Poole Bay is by erosion of the sand cliffs whilst there is large scale accretion west of Poole Harbour entrance in Shell Bay and Studland Bay, suggesting a westwards sediment transport pathway.

The results of the analysis carried out for this study suggest that the contribution to littoral drift of south-westerly waves diffracting round Durlston Head and into Poole Bay is evidently significant. These waves drive drift towards the east dominating the locally generated southerly to easterly wave climate. Furthermore, the wave refraction diagram shown in Figure 3.25 shows how storm waves from the south-east are reflected off Hook Sand so as to approach Sandbanks from the south contributing to an eastward drift along to the frontage. Waves reflected off the steep sides of Swash Channel will also generate an easterly drift component.

It would not be realistic however to imagine that these latest results are a precise representation of the littoral drift regime, particularly near Poole Harbour entrance. In calculating the drift rate no attempt was made to include effect which gradually reducing wave heights in the lee of the Isle of Purbeck will have on littoral drift. This wave height gradient will induce a westwards current which will make a contribution to carrying sand in that direction.

Although the effect of the channels and shoals near the Harbour entrance has been included in the wave refraction modelling, the processes of wave transformation and breaking in shallow water are complex and cannot be fully represented by existing models. One impact these shallow water effects will have is to drive sediment onshore over Hook Sand. High waves which pass over Hook Sand lose a proportion of their energy because of breaking. This loss of energy not only reduces inshore wave heights but also introduces a current over the shoals which will act to move fine material. The effects of wave breaking over the nearshore shoals has been studied by modelling radiation stresses (HR, EX2228, 1991) which confirms that during storm conditions radiation stresses contribute to onshore movement of sediments on Hook Sand and near Poole Head (Figure 3.26).

Near Poole Harbour entrance tidal currents make a significant contribution to sediment transport. Figure 3.27 indicates the nett potential sand flux over the course of a spring tide. The very large vectors in the Swash Channel are misleading because the channel bed is coarse material which will not be as readily transported as the sand for which the plot was computed. However, the pattern of nett transport on to Hook Sand and westwards down the flood-dominated East Looe Channel are instructive. The combined effect of currents and waves on sand transport in this area is much more difficult to compute.

Tidal currents also influence sediment distribution by interacting with waves to change their height and direction. This effect is well known in Poole Harbour entrance where waves steepen on a flood tide, but has not been included in the sediment transport modelling. However, despite some of the shortcomings in the littoral drift modelling, most of which are confined to the immediate vicinity of the Harbour entrance, the results presented in Table 3.2 are a reasonable estimate of drift rates elsewhere in Poole Bay.

The question remains that if longshore drift on Sandbanks is towards the east how did the peninsula originally form? At first glance Sandbanks appears to be a classic example of a spit formed by drift towards the west. However, since longshore drift is tending to force Sandbanks to retreat towards the east it seems more likely that the peninsula is a relic of a former morphological regime. Rising sea levels after the last Ice Age left former knolls in the moorland which covered the Poole Harbour basin as isolated islands. Brownsea Island, Green Island and Furzey Island remain isolated within the harbour, whilst the shape of Sandbanks suggest that it is of a similar origin. The abrupt change in the course of the fast flowing tidal channel at Brownsea Road and round the tip of Sandbanks suggest that resistant beds underlie Sandbanks and Brownsea Island. The peninsula's existence is explained more readily in terms of post-glacial sea level rise than in terms of relatively recent trends in longshore drift.

It is also reasonable to suppose that the direction of drift computed from recent wave climates did not necessarily prevail even in the historical past. Analysis of the potential littoral drift over the years 1974 to 1990 suggest a reversal of nett drift direction from west to east at point E in Poole Bay during the 19 year period (see Figure 3.21). Subtle changes in bathymetry or wave climate can have a significant impact on longshore drift. It is quite possible that at some time in the past westward drift did contribute to the formation of Sandbanks or at least to linking a former "Sandbanks Island" to Poole Head.

Another sediment transport influence which has obviously contributed to the formation of Sandbanks is that of dune formation. Wind-blown sand has built up substantial dunes on Sandbanks (and South Haven Peninsula). Though the dunes have now mostly been built upon, wind-blown sand is noticeable on the upper beaches at Sandbanks. Prevailing south-westerly winds encourage the observed build-up of sand on the western sides of groynes.

The best appreciation of the present day longshore drift regime along the western part of the Poole Bay beaches is as follows:

(i) Waves carry beach material westward, along the upper part of the beach, from the west of Bournemouth over the boundary into Poole Borough and toward Canford Cliffs.

(ii) From here onward, sand increasingly tends to travel offshore travelling south-west onto Hook Sand.

(iii) Some of the sand eventually 'escapes' over the dredged channel to Studland Bay (although this mechanism is rather more easy to deduce than to demonstrate by numerical models or field data).

(iv) Transport on or around Hook Sand is very complicated. There is strong evidence, however, that the nett transport of sand in East Looe Channel is westward, as a result of the flood tide dominance. There is also likely to be a periodic onshore transport of sand from the crest of Hook Sand in severe storms.

These two mechanisms provide a supply of material to the beaches from the Haven Hotel eastward.

(v) Finally the balance of evidence suggests a nett eastward drift along the inter-tidal beaches. This diminishes and eventually reduces to zero, at the meeting point with the westward-travelling drift from the Bournemouth frontage. This location will not be fixed but will tend to shift position depending on recent weather conditions, changes in Hook Sand and subtle changes in the wave climate from year to year.

3.3.5 *Seabed morphology*

Shown in Figure 3.28 are the findings of seabed sampling which revealed the seabed to be principally sand apart from a gravel bed to the Swash Channel. Strong tidal currents in Swash Channel ensure that except within the vicinity of the bar at its seaward end, sand within the Channel remains in suspension and is not deposited on the seabed. An area of fine sand extends from the north-east flank of Hook Sand into Studland Bay. The sediment on top of Hook Sand is coarser and therefore less mobile than that on the Bar. The bed is gravelly further offshore (HR, EX 1796, 1988).

Analysis of the seven charts (1848-1990) (HR, EX 2480, 1991) confirms that over the past 150 years the main features of Poole Bay have not changed. The main changes in bathymetry are as follows:-

(a) An increase in depth over the Bar at the seaward end of Swash Channel from 1.0m to 4.0m associated with the construction of the training walls between 1848 and 1878 and 1925-27. The construction of the training walls controlled the migration and spread of Swash Channel and induced a build-up of sediment in Studland Bay.

(b) In 1988-1989 Poole Harbour Commissioners (PHC) carried out capital dredging in the Swash Channel resulting in deepening and widening of the channel. This has affected localised accretion and erosion due to changes to the tidal flow through the channel and surrounding bank areas. However, checks were made that changes in the channel would not influence wave heights along the nearby coasts.

(c) The East Looe Channel has generally deepened over the past 150 years, and particularly over the last four years (see Figure 3.29).

(d) The crest of Hook Sand has been seen to migrate from one survey to another and is absent in the 1988 survey. The crest is removed by the action of storm waves and is likely to re-establish during the calmer summer months. This feature is an example of the transient seasonal differences that occur from one survey to another.

Surveys between 1785 and 1934 (Robinson, 1955) are compared in Figure 3.30. These historic surveys suggest the same conclusions as inferred above, that though principal bathymetric features of western Poole Bay are all recognisable in the first chart (1785) they have been continually developing, the main change being associated with construction of the training bank and dredging of the Swash Channel. Significantly, Vernon-Harcourt (1903) records that erosion on the Sandbanks frontage began upon construction of training bank west of the Swash Channel and consequent opening out of the East Looe Channel.

A comparison of surveys in 1990, 1992 and 1994 (Figure 3.31) suggests that at its western end the East Looe Channel has deepened noticeably, tending to steepen the beaches of Sandbanks. Material driven onshore from Hook Sand will tend to be carried in the flood dominated East Looe Channel into the Swash Channel rather than accreting on the beaches. The reasons for the deepening and onshore movement of the East Looe Channel are uncertain. It is not surprising that the bed where the East Looe joins the Swash Channel is tending to erode, in view of the differential in bed levels at this point. Increased tidal flows in the Swash Channel have in turn encouraged faster flow and hence deepening of the East Looe Channel. The tendency for tidal flows in the Swash Channel to increase is attributable to the construction of the training bank, and an increase in the tidal prism inside the Harbour (due to saltmarsh dieback). These changes have been examined in more detail in the feasibility study for Sandbanks Coast Protection Scheme (HR, EX 3083, 1994).

3.3.6 Sediment transport pathways

Numerical sediment transport modelling (HR, EX 1461, 1986) indicates that the tide generates a southward drift of water and suspended sand along the Hook Sand and across the Bar past old Harry Rocks. Preliminary tidal and sand transport calculations for a mean tide without the aid of wave action, indicate that about 20000m^3/year of sand passes southwards over the Bar. The stirring up of sediment by wave action on the Bar assists the predominant ebb tidal currents in the Swash Channel to carry sand along the side of Hook Sand even if there is no littoral component along the spit. In a subsequent study (HR, EX 1796, 1988) a computational model was used to simulate sand transport under the influence of both tidal currents and wave action. The calculations showed that most of the sand transport occurs during periods when typical wave conditions coincide with spring tides which happens about 15% of the time. There is a well defined net residual drift of water from Hook Sand southwards across Poole Bar. Sand has tended to accumulate in Studland Bay. Onshore wind transport of sand accreting on the foreshore of South Haven Peninsula has lead to the ongoing building of the sand dune system on the Peninsula which constitutes a sediment sink. From Handfast Point strong tidal currents rapidly flush sandy and fine gravels further south into Swanage Bay. Thus southward transport along this pathway is an output from the Poole Bay system (Bray et al, 1991).

Evidence of seabed sediment supply to Poole Bay from Christchurch Bay is conflicting. Sand-wave bed-forms indicate a westward supply from Dolphin Bank. By comparing Admiralty Charts Lacey (1985) calculated a net accretion of 727000m^3/year (1849-1977) in the central part of Poole Bay and measured a comparable erosion in Christchurch Bay. Westward transport to Poole Bay is a logical explanation for the observed features (Bray et al, 1991). However, sea bed drifter experiments (Tyhurst, 1976) indicate a transport in the opposite direction though the reliability of these studies is questioned by Bray et al (1991). It seem unlikely that significant amounts of material could travel westwards over Christchurch Ledge from Christchurch Bay against the prevailing wave direction (HR, EX 2228, 1991). Indeed littoral transport round Hengistbury Head is clearly in an eastward direction.

In addition to the recognised sediment path from Poole Bay into Studland Bay, evidence of bed form and the mineralogy of sediment samples indicates a southward supply from Poole Bay. No evidence of corresponding northward return feed was available so it is concluded that this pathway comprises a net output from the Poole Bay system. Much material may have been supplied from Christchurch Bay via Dolphin Sand and possibly only passes through Poole Bay en route to a sediment sink in the Channel (Bray et al, 1991).

The final contribution to the sediment system in Poole Bay is interchange with Poole Harbour. Sediment transport modelling of Poole Harbour entrance yielded a net sand transport for tidal currents alone of 113m^3 per tide out of the Harbour during spring tides (HR, EX 1796, 1988). Analysis of sediment transport by wave action indicated potential for sediment transport and deposition in the harbour entrance especially with storm waves. Calculated input on a spring tidal cycle was 250m^3 per tide for typical waves and 1625m^3 per tide for storm waves (HR, EX 2356, 1991). These results indicate that a critical balance must exist between sand input (typical storm wave conditions prevailing perhaps 30% of the time) and output (calm conditions prevailing for about 70% of the time). The long-term net trend could not be established from these studies (Bray et al, 1991). It is to Poole Harbour that we shall now turn our attention.

3.4 Morphology of Poole Harbour

3.4.1 Morphological development

The characteristic hydraulic conditions within the harbour have an important bearing on recent morphological development:

- The small tidal variation (1.1m neap tidal range, 1.8m spring tidal range) restricts the vertical range over which wave action influences the shore.

- The presence of water above mean sea level for 16 out of 24 hours means that conditions within the Harbour approach those of a marine lake or lagoon.

- The prolonged tidal stand exacerbates the very flushing characteristics of the Harbour which are fundamentally the product of the high degree of land-locking.

- Wave action within the Harbour is dominated by locally generated waves which are fetch-limited. Because of the intricate shape and bathymetry of the harbour, the fetch lengths and thus incident wave height can vary considerably along quite short stretches of shore. The maximum fetch at high tide within the Harbour coincides with the direction of prevailing winds from the southwest.

- Larger waves can penetrate the harbour entrance and spread into the harbour by refraction and diffraction, whereby wave height rapidly decreases. This only occurs when winds are from the south-east though this is the direction of largest storm waves in western Poole Bay.

- Tidal velocities increase towards the harbour entrance. The merging of the channels as well as constrictions and bends in the channels all serve to increase this tidal velocity. In the Middle Channel the flood tidal stream dominates whereas the ebb dominates the North and Wych channels.

It is believed that sediment processes within the harbour are more or less in equilibrium (Bray et al, 1991, Fahy, 1993, PHC, 1985). The equilibrium must be a dynamic one since the reducing extent of saltmarsh vegetation has released large quantities of sediment into the harbour during recent years and man has a significant impact on the system both by reshaping the shoreline with land reclamations and by dredging the harbour floor.

3.4.2 Sediment distribution

The underlying beds of the Bracklesham Group contribute sands, gravels and clays to the sediment regime in Poole Harbour. These have been re-exposed by erosion in several places around the harbour. Inter-tidally both sands and clays feature in cliff faces cut by wave action. Sub-tidally, natural exposures of the clays occur within several sections of the channel where tidal currents and scour are sufficiently strong to prevent settling of more recent sediments. The sand beds are too readily erodible to naturally feature intact. Hard natural bedrock is virtually absent within the Harbour with the exception of one area of the Haven Channel where outcrops of sandstone have been exposed by the strong currents and more recently by dredging (Dyrynda, 1987).

More recent fluvio-glacial deposits which overlie the Bracklesham Group contribute a wide range of sediments. Fluvial deposits which continue to be discharged into and redistributed within the Harbour contribute silts and clays to the sediment regime. The lower Frome, in particular, is characterised by a wide alluvial flood plain. However, on the PBC frontage the supply of fluvial sediment is much less than elsewhere in the Harbour being confined to the streams which discharge into Holes Bay and Lytchett Bay. Organic activity, as well as stabilising fine sediments, has laid down beds of peat which have been found in Holes Bay exposed by channel scour.

The pattern of sediment distribution reflects the varying wave / current environment. Generally the upstream gradient of declining tidal energy away from the entrance into the sheltered parts of the harbour, is reflected in decreasing sediment size, increasing incidence of net accretion as opposed to erosion, and increasing incidence of unconsolidated as opposed to consolidated beds. Similarly across the channel cross-sections there is an decrease in sediment coarseness and consolidation towards the channel margin, though this pattern is modified at channel bends. Thus within the Haven, the most current-scoured section of Poole Harbour, the channel is more than 15m deep and sediments range from medium and coarse sand at the channel peripheries, to stones, boulders and consolidated clays at the channel centre. At the other extreme, typical upstream profiles, for example in upper Holes Bay, may hardly bottom out below Chart Datum level and the profile may be entirely of soft mud.

Super-imposed on this pattern of tidal-induced sorting are the effects of waves on the northern and north-eastern shores which are exposed to the longest fetches. Sand dominates in these areas, occasionally with a narrow shingle or shell upper beach. Large cobbles and boulders are found at several locations derived from erosion of the seawalls and breakwaters built up to 120 years ago. In sheltered parts of the bays muds prevail.

Because of the restricted wave energy, and limited vertical extent of wave action, nearshore sediments can be poorly sorted. Below mid-tide level, shingle in a matrix of sandy muds is widespread on the PBC frontage. Furthermore, because of the large quantities of sand that have been released within the harbour by erosion, this sediment is widely prevalent and even visible in the low energy environments of the bays.

3.4.3 Saltmarsh stabilisation

Saltmarsh vegetation has made an important contribution to stabilisation and accretion of areas of mudflats. Indeed changes in the inter tidal area of Poole Harbour this century have been dominated by the arrival, spread and decline of the saltmarsh grass *spartina anglica*. Because of higher than average wave activity, saltmarsh is less extensive on the PBC frontage than on southern and western shores of the harbour. However, saltmarsh is the dominant foreshore type in Holes Bay and Lytchett Bay and also exists in Parkstone Bay and Blue Lagoon. Even where saltmarsh is not the dominant foreshore type its impact on sediment distribution and the tidal prism is of importance throughout the Harbour.

The sources of the clay and fine silt particles that have provided the raw material for mudflat construction include river suspended load, marginal erosion and (more debatably) reworking of deposits in the harbour bed. The colonisation of mudflats by saltmarsh vegetation has been decisive in accelerating rates of deposition. In this context the role of *spartina* over the past century has been especially important (Bray, et al 1991).

Spartina was first reported in 1899 at Ower and it thereafter invaded large areas with remarkable speed and vigour in the succeeding 25 to 30 years. Stratigraphical studies have indicated that the general marsh level was raised in some areas of the upper Harbour by more than 1.8m, and that *spartina* - accreted sediment depths of more than a metre are common with a possible gradient to shallower sediments towards the Harbour mouth (Gray, 1985). Since then there has been a gradual reduction in the extent of the vegetation leading eventually to widespread "dieback". The loss of marshland has become particularly noticeable in recent years in areas such as Holes Bay but detailed studies indicate that the rate of recession has accelerated only slightly within the Harbour as a whole (Gray, 1985). By 1980 around 360 hectares, more than 46% of the 1924 area, had been lost, with as much as 189 hectares loss since 1952.

The process of saltmarsh spread and dieback is not fully understood, nor has it been established how much sediment is trapped by saltmarsh vegetation. It is clear, however, that saltmarshes do represent a major sediment store and that where dieback has taken place considerable quantities of silt/clay have been re-released into the harbour. Re-deposition of sediment has shallowed the harbour in some areas, notably in the upper parts of the major channels. However, possibly as a result of the enhanced tidal volumes and scouring in the harbour because of the loss of spartina marsh, the seaward ends of the major channels have actually deepened.

3.4.4 Shoreline changes

Reclamation and shoreline changes in Poole Harbour were studied in detail by May (1969) whose principal findings are presented in Table 3.3. Changes in the intertidal area were also examined by May and the findings summarised in Table 3.5. Accretion of an estimated 1052 hectares due to natural or human land claim has significantly exceed erosion of an estimated 41 hectares.

Shoreline change has been measured for this study by comparing the first edition of the Ordnance Survey County Series (surveyed in 1887 and 1888) with subsequent resurveys (in 1900, 1937, 1953 and 1993). Changes in the Mean High Water Mark and, where appropriate, the cliff toe and cliff top were studied. Specific changes are discussed in the Management Unit descriptions in Section 4.4.

The general trend, which is illustrated in Figure 3.32, has been one of retreat of the High Water Mark at the southern end of the Sandbanks peninsula (Management Unit 2), at the cliffs on the Lilliput frontage (Management Unit 5), at Baiter Park (Management Unit 7) and at Rockley Sands (Management Unit 13). Erosion of the southern end of Sandbanks has been most marked near the Haven Hotel where the High Water Mark has retreated by 14m between 1888 and 1993. Moving towards North Haven Point the retreat has been less and opposite Brownsea Road there is no consistent trend in shoreline change the High Water Mark having retreated and advanced between various surveys.

The cliff toe at Lilliput (Management Unit 5) has not moved since 1955 by which time it was fixed by seawalls. Before then, cliff erosion was active and the High Water Mark retreated by up to 12m between 1900 and 1955. Photographs of the cliffs in 1952 show signs of active erosion.

Baiter Point has always had a beach though before substantial land claim in Parkstone Bay it only comprised a narrow spit. The beach is now backed by recreation ground (Management Unit sub-section 7/5 and 7/6). Comparison of surveys indicates a retreat of the High Water Mark by up to 28m between 1888 and 1993.

Analysis of shoreline change at Rockley, where erosion is still continuing, is hindered by the impact of past china-clay workings have had on the coast. The 1901 revision of the Ordnance Survey plan shows spoil from the china clay workings dumped on the foreshore east of Rocksey Point and this has masked trends in the natural processes. Nonetheless, natural erosion has persisted throughout the record of surveys. The High Water Mark has retreated by about 10m between 1888 and 1954 (in Management Unit sub-section 13/10) and the cliff edge by up to 16m over the same period.

3.4.5 Littoral processes

On several shores a modest sand, shingle or shell beach towards the backshore indicates potential for some littoral sediment transport. Wide flat foreshores on the north-east side of the bay have a surface layer of silt or sand but underlying widely graded materials from gravel to clay indicate minimal sorting by wave / current action.

Computation of potential wave-driven sand transport on the beaches inside Poole Harbour was carried out using the DRIFTCALC model which is described in Section 3.3.4. Five of the nine wave prediction points in Poole Harbour (see Figure 2.48) were employed for the DRIFTCALC analysis. Point C was not used since Holes Bay consists of mudflats and points E, H and I were not used because determination of a beach normal at those locations would have been too arbitrary and the rate of drift would not reflect the general potential drift in the area.

The predicted annually averaged potential nett and gross drift rates are listed in Table 3.4 and are shown on Figure 2.48. Typically the nett volume per year is a few thousand cubic metres. The potential transport can only occur in practice if a full beach is available to be acted upon by the waves. Clearly this is not the case in Poole Harbour where the backshore and in places part of the foreshore has been extensively reclaimed. Moreover, the dominant influence of tidal currents in sediment transport will modify and in places completely override wave-induced transport. However, the results from in Table 3.4 are still instructive in elucidating the longshore sediment transport regime on the north and north-east shores of Poole Harbour.

As would be expected potential sand transport rates (Table 3.4) are generally modest and from west to east on the Rockley, Lake, Baiter and Parkstone beaches. At Lilliput the local beach orientation of 232°N induces a northerly drift driven by prevailing south-westerly waves, a result confirmed by the build-up of sand on the south side of Salterns Marina.

3.4.6 Bathymetric development

Review of historic charts reveals that the basic pattern of channels in Poole Harbour has remained relatively stable since 1785 (Fahy, 1993). The major changes are:

- There has been considerable shallowing of the Wareham and Wych Channels mostly associated with the release of sediment by the decline in spartina between the 1930's and 1960's. Most of the other minor channels have also shallowed greatly though there has been some stability since 1950 (PHC, 1982).

- There has been considerable deepening in the Middle Mud area since the 1940s. This was first definitely observed in 1954 and the rate of deepening is now increasing (PHC, 1982).

- The North Channel has tended to migrate northwards and eastwards (Green, 1940, McMullen, 1979).

- Until recent dredging, the Middle Channel was naturally shallowing, especially immediately north of the eastern end of the Wych Channel where ebb streams are said to deposit sediment in the channel.

These channel changes have tended to be a consequence of rather than the cause of shoreline change, most notably in the case of spartina dieback which has released sediment into the upper channels.

The relative stability of the channel system is clearly related to the constraining effect of the promontories and islands which prevent the extensive lateral migration of low water channels which is a characteristic feature of large estuaries such as the Dee or Morecambe Bay (Grey, 1985). Channel stability, and indeed survival of the narrow inlet to the Harbour (Green, 1940), also depends on the small tidal range.

3.4.7 Human influences

Human development of Poole Harbour in recent centuries has had a major impact on the shoreline and bed of the harbour. Whilst on the southern and western shore of the PBC frontage natural processes still prevail, development of the majority of the shoreline dominates the pre-existing regime. Whilst human development of the Harbour takes many forms from boat moorings to navigational aids the two important influences on morphological development have been reclamation and dredging.

(i) Reclamation

Land claim on the shore of Poole Harbour has progressively reduced the area of the harbour and modified the shoreline environment. Natural stabilisation and build-up of mudflats has contributed to effects of manmade reclamations in causing a 20-25% or 1000ha reduction in the original surface of the harbour as measured at MHWS (Fahy, 1993). May (1969) quantified the area of land claimed by both natural and human influences and his findings are summarised in Table 3.5.

Human reclamation has tended to be concentrated on the northern side of the Harbour and has been carried out for the purposes of communication (wharves, jetties, roads, railways), housing, recreation (marinas, playing fields) and industry (power station, gas works and factories). Dyryndra (1987) identifies two types of reclamation:

- impoundment - the enclosure of inter-tidal areas using breakwaters, bunds etc;
- infilling - involves total loss of the claimed area, usually to above the high water mark, to create land for development.

Construction of railway embankments has partly impounded Lytchett Bay and inner Holes Bay and completely cut off the inner recesses of Parkstone Bay so forming the boating lake within Poole Park. The extension of berthage on both sides of Back Water Channel has lead to further isolation of all upstream areas within the Holes Bay. Much smaller areas of the harbour have been impounded for construction of marinas. Whilst the area of ground actually covered by impounding structures is fairly small, by reducing tidal exchange within the impounded area and increasing shelter from wave action, lagoon-like areas are created. These areas generally tend to become shallower due to an accumulation of fine sediments, whilst within the link-channel of the impounded area (if there is one), concentrated tidal flows cause localised bed scour.

Infilling often follows on from impoundment. Depending on the nature of the infilled area this can involve immobilisation of a sediment store. The reclamation work can act as a barrier to littoral processes and can constitute a more reflective shore thus altering the hydrodynamic environment in front of the reclamation. This is the case between Town Quay and the Ferry Terminal where reflections from the vertical wharves amplify the local wave climate. In general, however, infilling has not had much impact away from the immediate area of development.

The principal infilled areas on the PBC frontage have been around the sides of Parkstone Bay for marinas, recreation and the former gas works; on either side of the Back Water Channel and at the Hamworthy docks; for the A350 on the eastern side of Holes Bay.

(ii) Dredging

Occasional capital dredging and periodic maintenance dredging has been carried out to improve the navigability of Poole Harbour for commercial shipping. The following average annual dredging volumes are quoted by Fahy (1993):

1969 to 1975 23103m^3/yr
1975 to 1981 34448m^3/yr
1981 to 1984 47679m^3/yr

Table 3.6 shows the capital and maintenance dredging undertaken within the harbour from 1984-93.

The main capital dredging project in recent years has been deepening of the Middle Channel to accommodate the cross-channel vessel M V Barfleur. Shipping bound for Poole Port which used to navigate the North Channel now uses the straighter Middle Channel. Associated with this work has been deepening of the roll-on/roll-off berth. Sediment has also been removed by dredging from Fisherman's Dock, Poole Harbour Commissioners Quays, the Royal Marines Slipway, and Chapman's Peak, though the latter shoal had substantially returned only three months after dredging (Appleton, 1994).

The ecological impacts of dredging are discussed in some detail by Dyrynda (1987). In terms of the impacts on shoreline processes, dredging within the harbour has shown not to have any discernible effect. The hydraulic regime has been modified since currents have tended to be concentrated within the dredged channels. Higher waves may be locally transmitted in the deeper channels but will be attenuated on the channel banks. By increasing the subtidal volume, the tidal prism ratio will be reduced. However, there is no conclusive evidence of the impact, if any, of these changing influences on conditions at the shoreline.

4 Coastal characteristics and existing defences

4.1 Introduction

This chapter describes in detail the important characteristics of the PBC frontage which will influence decisions on future management of the coastline. Management will be influenced by hydraulic and morphological conditions which have been discussed in preceding chapters and are summarised in this Chapter. It will also be influenced by the nature and condition of the existing coastal defences and by issues of land use and environment, so these characteristics are also discussed.

The approach taken here has been to divide the coast into fourteen Management Units, each covering between 0.3 to 4.6km of shoreline (see Figure 4.1 and Drawings 1 to 4). The Management Units have been defined on the basis of consistent beach type and hydraulic regime. Usually the coastal defence and the land use behind the shoreline is also homogenous within each Unit. The justification for extent and limits of each Unit is explained in the Management Unit descriptions in Section 4.4.

The nature and condition of the existing defences are covered in detail in the Management Unit descriptions. In the next section the existing defences on the PBC coastline are discussed in general terms, particularly in relation to the protection they afford against coastal erosion and flood damage. It was hoped to build up a comprehensive body of information regarding specific flooding events. In practice, despite wide-ranging enquiries, evidence of specific events was scarce. However, during the course of this investigation it was possible to identify sites where flooding and erosion has in the past been a problem. These sites are noted in the individual Management Unit descriptions.

One of the most important issues to be considered in a coastal strategy study is the consequences of <u>not</u> intervening. In many areas of the PBC frontage the present defences will continue to be adequate for the foreseeable future, and these stretches of coast may require no more than periodic monitoring. In this sense, "adequacy" is taken to mean providing the appropriate level of protection, which on certain stretches of shoreline may be rather less than provided today. Equally well, from the analysis of present erosion rates, of the performance of existing defences, and the sensitivity of the performance to a modest rise in sea level, one may deduce that the "do nothing" option might cause unacceptable flooding and/or permanent loss of land. An appraisal of the consequences of not intervening in the medium term future (ie up to 10 years) is therefore necessary as a first step in assessing the need for, and type of management needed. For each management unit, a number of factors have been taken into account. These include:

- the present and historical rates of shoreline change;
- the calculated present performance of the defences;
- the residual life of the defences;
- the past history of flooding or damage;
- the sensitivity of the defence standards to climate change; and
- the land use, and value behind the defences.

A full economic evaluation of the "do-nothing" option would require each of these factors to be carefully looked into, and quantified, and this is well beyond the scope of the present study. It has been possible, however, to provide a reasoned argument about whether or not such an approach is safe in the medium term future (ie the next ten years). Depending on the particular frontage, the most crucial factor will differ. For the eroding cliffs at Rockley Sands, for example, the likely cost and environmental impacts of installing defences will probably outweigh concern about slow shoreline retreat. Conversely on Sandbanks where the beach is lowering in front of vulnerable sheet pile walls which protect valuable properties, the situation will worsen in the coming years and warrants immediate attention. For each Management Unit (Section 4.3), therefore, we have made an assessment of the likely consequences of a "do nothing" approach. This has been a primary guide to the management options proposed in the following chapter.

4.2 Review of existing defences and coastal structures

The standard of defence provided by a section of seawall depends on the margin of safety it provides against structural collapse or unacceptably high overtopping discharge. The two distinct limit states are related. Seawall collapse is a consequence of hydraulic forces as well as structural and geotechnical aspects. Overtopping discharge, as well as depending on the severity of storm conditions, is also a function of crest level and cross-sectional profile. Both overtopping and structural failure are highly influenced by the level and condition of the beach in front of the seawall.

The assessment of the existing defences around the frontage has therefore examined two criteria: the structural condition of the wall and its capacity to prevent overtopping.

(i) Structural condition of existing defences.

The seawall type and material has been classified for each management unit and subsection. The defence condition has then been classified using the standard NRA classification of "good", "fair", "poor" or "bad".

Of approximately 25.5km of defence, 8.6km is in a good condition, 12.3km is fair, 3.8km is poor and about 0.8km is bad. The defences in a bad condition are to be found in Units 5 (at Evening Hill), 6 (Blue Lagoon) and 13 (Rockley Sands). Of these the structures across Blue Lagoon should not be cause for concern provided they are sufficiently intact to prevent a significant change in hydraulic conditions inside the lagoon. The other problem areas are discussed in the Unit descriptions and solutions are recommended in the following chapter.

(ii) Protection against overtopping and flooding

The capacity of the seawall to resist overtopping depends primarily on the seawall crest level but also on the seawall slope and material, and on the beach level at the toe of the wall. At each major section of seawall around the frontage, information on crest height, cross-section, material and toe level has been combined with design wave and tide conditions (derived in Chapter 2) to calculate wave overtopping of the seawall.

Analysis of overtopping at seawalls is based on the findings of flume model research studies carried out at HR Wallingford and elsewhere (HR, EX924, 1980 HR, SR316, 1993). For seawalls up to a 1:1 slope these empirical findings now form the basis of the HR Wallingford seawall overtopping model SWALLOW.

Overtopping was calculated at 17 seawall cross-sections on the Poole Harbour and Poole Bay frontages. Dimensions of the seawall cross-sections were obtained from the MAFF (1993) Coast Protection Survey. The cross-sections analysed are shown in Figures 4.2 to 4.7.

Overtopping discharges which are sufficiently large to cause discomfort or danger to passing pedestrians or vehicles, or actual damage to seawall embankments were investigated by Goda (1971) whose findings are presented in Figure 4.8. Goda assessed the critical discharge at the crest of the seawall and 10m behind the crest, and found that 10m behind the crest each of the limiting discharges could be increased by a factor of about 10. These critical overtopping discharges can be compared with the predicted overtopping discharges calculated at the various sites around Poole Bay and Poole Harbour (Table 4.1 and 4.2). The results are shown in diagrammatic form in Figure 4.9.

Overtopping discharge in the present study has been tested for two events, a 1:10 year and 1:50 year storm (based on joint probability return periods for waves and water levels), to give an indication of the frequency at which inconveniencing or damaging events may be expected. So, for example, a discharge sufficient to damage the seawall embankment during a 1:50 year event represents an unacceptable risk of breaching whilst it is probably acceptable that on a road near a seawall cars will have to slow down because of overtopping during a 1:10 year storm.

At two sites (Unit 1/14 near Midway Path on Sandbanks, and at Unit 8/2 at Poole Town Quay) the combination of hydraulic conditions and seawall dimensions is outside the bounds of the empirical equations on which overtopping computations are based. However, whilst the overtopping at these two sites cannot be quantified without recourse to site-specific research work, experience at these sites proves that wave overtopping in rare storms is excessive. Clearly both sites warrant attention to deal with this problem though at Unit 1/14 the problem of overtopping is secondary to that of coastal erosion.

Elsewhere the model results provide instructive guidance on the adequacy of the defences. On the Poole Bay frontage the defences have similar crest levels (varying between 2.9 and 3.2m ODN, with the exception of sub-section 1/11 which is at 2.55m ODN). The protection against overtopping is closely related to the beach level in front of the walls. Thus at sub-sections 1/10 and 1/11 where the beach in front of the wall is high the overtopping, even during a 1:50 year storm has been calculated to be sufficiently close to zero to be negligible. Meanwhile at sub-sections 1/1 to 1/6 where the beach is up to 1.5m lower, overtopping during a 1:10 year storm is enough to make walking along the promenades dangerous, and to damage beach huts. Overtopping at these cross-sections is still not sufficiently high to seriously damage the promenade itself even during a 1:50 year storm. These calculations show that raising beach levels (to say 2.5m ODN at the toe of the sea wall) will limit overtopping to a negligible rate. This response is more practicable and more acceptable, in terms of the amenity use of the promenade and beach, than raising the seawalls.

On the Harbour side of Sandbanks where wave conditions are mild, and the backshore is occupied by private gardens and houses set well back, overtopping is calculated to be insufficiently high to be a problem even during a 1:50 year storm.

Two lengths of wall, at sub-sections 4/2 and 5/2 are below the 1:10 year water level so sea water will flow over them. At sub-section 4/2 the road is set back and is well above the wall crest level so whilst the wall overtops, flooding of the road is not reported to be a problem. At sub-section 5/2 a footpath runs parallel to the sea defence at the same level and is regularly flooded causing an inconvenience.

The wall along Shore Road (sub-section 4/5) is somewhat higher, but overtopping is still calculated to be sufficiently high to be a hazard to traffic during a 1:10 year storm.

At Lilliput (sub-section 5/4) overtopping into private gardens can be sufficiently high during a 1:10 year storm to be hazardous to someone standing directly behind the wall. However, even during a 1:50 year storm it will not be damaging to the seawall. The houses themselves are situated well back from the defence.

In sub-section 6/3 at Parkstone, the same land use of private gardens, with houses set well back from the wall, prevails. Here overtopping is more severe. The still water level during a 1:50 year storm, after 50 years of sea level rise at predicted rates, will be above the crest of the seawall. Calculations suggest that constructing these walls to a uniform crest level of 2.25mOD will reduce overtopping to an acceptable minimum even after predicted sea level rise.

Overtopping onto the footpath at Parkstone recreation ground will be hazardous to pedestrians immediately behind the rock revetment more frequently than once in ten years. However the overtopping will not damage the integrity of the defence. The rock revetment in Holes Bay (sub-section 9/2) provides good protection against overtopping for the road which is set well back.

At Hamworthy recreation ground, whilst the beach is relatively high, affording some protection against wave attack, the seawall itself is very low. It is regularly overtopped. Beach huts and the cafe have been damaged by overtopping during storm surges. To limit overtopping during a 1:10 year storm to an acceptable level the seawall would need to be raised to a level of 2.3m.

Further west the private seawalls vary in crest level. The typical wall surveyed has a level of 2.57m ODN, has a high beach in front of it (above MHWS), and affords good protection against overtopping.

Table 4.3 demonstrates that predicted sea level rise will, over a period of fifty years, have a marked impact on the capacity of the seawalls to resist overtopping. This is particularly true inside the Harbour where seawall crest levels are, in many places, already in danger of being overtopped by extreme surge water levels. Sea level rise is therefore a significant consideration when examining standards of flood protection, which differ somewhat from the recommended standard of protection against overtopping.

Low-lying areas on most of the PBC frontage should be protected against an extreme water level with a 1:50 year return period whilst the urban area of Poole should be protected against a 1:200 year still water level (MAFF, 1993) which have been calculated to be 1.89 and 1.92m ODN respectively. Sections of frontage which do not achieve this standard are noted in the Unit descriptions. For example Town Quay is at 1.85 - 1.90m ODN and so fails to provide the required 1:200 year standard. It should be noted that even a modest degree of wave activity on top of these extreme water levels will lead to overtopping.

4.3 Coastal Management Units

4.3.1 Format of Management Unit descriptions

The description of coastal characteristics for each Management Unit draws upon on the findings of numerical modelling, desk studies previous work by PBC, and a comprehensive literature review. The entire frontage was inspected by staff from HR Wallingford on foot and by boat from 23 to 25 February 1994. During this visit the Management Units were defined and principal coastal characteristics noted, upon which the following descriptions are based. HR Wallingford is grateful to PBC for providing a boat and other assistance during the site visit.

Each description begins with a statement of the Ordnance Survey grid references at the ends of the Management Unit, and the length of shoreline measured at Mean High Water Level.

PBC have already sub-divided the coastline as the basis of a Shoreline Management Plan so the Management Units defined here are cross-referenced with the corresponding PBC drawing numbers and reference codes. For the sake of consistency the sub-sections within each Management Unit generally correspond to the PBC divisions of each sub-section and are summarised in a spreadsheet table (Tables 4.4 to 4.17) for each Management Unit.

An introductory description of each Management Unit explains the reasons for the Unit definition and extent. Important characteristics are briefly noted and, where significant, interaction with adjacent Units is described.

Hydraulic conditions within the Unit are then described in detail with reference to the degree of exposure, the wave conditions, the nearshore bathymetry and tidal current conditions.

The geological sediments outcropping within the unit are noted, followed by a detailed description of beach type (material, width, slope) and condition (eroding / accreting / stable). The beach level at the toe of the seawall is in some areas, quite volatile. The level quoted in the summary table is, therefore, only indicative and, for the sake of consistency, has been taken as the level given in the MAFF Coast Protection Survey (1993). Survey work for the MAFF (1993) study was carried out during August to September 1993. Where applicable, estimates of potential longshore sand transport obtained from the HR Wallingford model DRCALC are summarised. The effect of coastal structures on the beach is also noted.

The type and condition of the coastal structures within the Management Unit is then described. An assessment of the standard of protection against coastal erosion or flood damage is made. Where a structure is strongly dependent on the condition of the foreshore this is noted.

Land use immediately adjacent to and further behind the shoreline is described to give an indication of the type of property which is being protected. The vulnerability of the land to erosion or flooding is noted. Historic erosion rates or records of previous flood damage are evidence of vulnerability and are included where appropriate and where available.

The environmental sensitivity of the frontage has been described with reference to its value as an amenity, its visual attractiveness and its importance as an ecological habitat. Poole Harbour is nationally and internationally significant for nature conservation. In recognition of this Poole Harbour marshes, mudflats and islands are a biological SSSI and a Nature Conservation Review site which has important implications for future coastal management. Specific areas of importance are examined within the individual Unit descriptions.

Finally the consequences of carrying out no further works on the Management Unit are examined. This is based upon an assessment of the hydraulic conditions, present trends in beach erosion or accretion and the condition of the existing coast protection. A prediction is made of the effect of a "do nothing" approach on the coastal area during the next ten years. Over this period the consequences of sea level rise at present rates is unlikely to be noticeable. This appraisal of the consequences of not intervening in the medium term future is the first step in assessing the need for and type of management needed.

4.3.2 *Management Unit 1 Poole Bay*

Grid References SZ 0709 8999 to SZ 0372 8704
Length of frontage 4.55km

Drawing 1	PBC Map reference D065/01 and D065/02
Summary Table 4.4	Not included in PBC unit references.
Plates 4.1 to 4.4	

Description

The Poole Bay frontage is a continuous littoral regime. Since there is active sediment exchange between adjacent sections along the whole frontage, it should be considered as a single management unit. Intervention in the updrift sediment supply at one end of the Unit can and has affected beaches at the other end of the Unit. By the same reasoning it is contrary to the principles of good coastal management to separate the coast on the basis of administrative boundaries. On the other hand the purpose of this study is to develop management strategies for the frontage for which PBC has responsibility so it has not been extended beyond the PBC boundaries. It must be stressed however, that to manage this Unit an understanding of the coast processes throughout Poole Bay (as discussed in Chapter 3) and cooperation with neighbouring Bournemouth Borough Council is essential.

At the south-western end of the unit, the coastal characteristics change significantly. The beach narrows and eventually disappears, the end of the Sandbanks Peninsula being in deep water. The hydraulic conditions become dominated by the strong currents in the mouth of the Harbour. Thus a new Unit has been defined on the end of Sandbanks, the Haven Hotel being at the boundary.

Hydraulic conditions

Numerical modelling of wave conditions is discussed in Chapter 3. The inshore wave climates at points B to E along Unit 1 are shown in Figures 2.41 to 2.44. These wave climates reflect the increasing exposure to the south-west as one moves along the frontage towards the east. Nonetheless, waves from the south-west are diffracted around the Isle of Purbeck and drive an eastward drift along the beach. East of Poole Head the refraction modelling of waves is an accurate representation of the hydraulic conditions. However, as one moves towards Poole Harbour entrance the bathymetry becomes increasingly complex. In particular wave breaking and shoaling over the Hook Sand (which is mobile, changing from year to year and indeed from season to season) make computation of an accurate inshore wave climate more challenging. Furthermore, currents in the Harbour entrance interact with waves to change their height and direction. The steepening of waves in the Harbour entrance on an ebb tide is well observed.

Seabed morphology and tidal flow conditions in Poole Bay have been discussed in Chapter 2. The tidal flow modelling has demonstrated the principal tidal characteristics at Management Unit 1:

(i) Tidal currents are concentrated in the Harbour entrance and Swash Channel, but tend to fan out from the narrowest point in the Harbour Entrance.

(ii) Beyond the immediate vicinity of the Harbour entrance and East Looe Channel, nearshore tidal currents are small.

(iii) There is a divide in nearshore tidal direction near Poole Head. East of Poole Head the currents follow the general flow within the English Channel of a flood towards the east and ebb towards the west. However, the effect of north/south tidal exchange with Poole Harbour is to generate nearshore flood tidal flows towards the west and ebb tidal flows towards the east between Poole Head and the Harbour entrance.

The important bathymetric features adjacent to this Unit are East Looe Channel, Hook Sand and Swash Channel. Analysis of historic charts (HR, EX3083, 1994) indicates that the East Looe Channel has generally deepened over the past 100 years. Comparison of recent surveys also suggests that the eastern end of the East Looe Channel has shifted towards the shore. Being actively worked on by breaking waves, the crest of Hook Sand tends to migrate but continues to be an important bathymetric feature. The deepest section of the Swash Channel (about - 18m ODN) is only 250m from the shore at the Haven Hotel. The scoured channel bed is gravel, cobbles and larger rocks: sand which is transported into the channel here is carried in suspension either into the Harbour or seawards towards the Bar and across Hook Sand.

Coastal geology

The Poole Bay frontage is of considerable geological interest displaying high exposures of Branksome Sand topped with beds of fluvio-glacial gravels. Erosion of these beds, which has continued until recently has, as discussed in Chapter 3, contributed large quantities of sediment to the littoral zone.

The cliffs of Branksome Sand extend along almost all of the PBC frontage as far as Poole Head. Only at the chines are the cliffs discontinuous, the sediments in the bottoms of the chines tending to be more recent alluvium, rather than the Tertiary Sands. At Poole Head the sands give way to a restricted outcrop of Parkstone Clay.

Beyond Poole Head, the surface deposits on Sandbanks are of windblown and drifted sand.

Foreshore

The unit is fronted by a sloping sand beach. The Bournemouth Borough Council groyne field terminates with four long groynes, more closely spaced than on the frontage further east. Moving west onto the PBC frontage the beach level falls noticeably. The eastern 2.4km of PBC frontage has 30 to 35m long groynes spaced at about 120m. Moving westward through this groyne field the beach level rises so that at the western end of the groyne field the groynes are more or less full.

The groyne field on the PBC frontage has been surveyed at bi-monthly intervals by PBC since 1991. Although the data set is not sufficiently long to enable long term trends in beach behaviour to be identified it does give a clear indication of the volatility of the beach. During the period for which records exist the average beach level has varied by up to 0.5m within the space of two months. Given this volatility, the beach level quoted on the summary table is only indicative. In the absence of a long record of beach cross-sections, long term trends in erosion or accretion can best be inferred from the behaviour of the high or low water mark. Analysis of historic maps indicates a general recession of the beach east of Poole Head whereas towards the west the trend has been for accretion (see Figures 3.7 to 3.10).

West of Poole Head the open beach is wide with a sloping backshore of windblown sand. On the lower beach the remains of old rock groynes have become exposed during the last year and beach monitoring confirms this recent erosion trend. In view of the lowering beaches (towards the west) this erosion can be explained by a reduction in supply.

Continuing towards the west the beach reduces in width and dunes on the backshore in sub-section 13 are showing signs of erosion. Further west in sub-section 14 the dunes have been claimed as private gardens by constructing steel sheet pile retaining walls. The beach in this area is volatile and its average level has fallen by more than 1m in recent years.

The beach was nourished with sand in 1992 but rapidly retreated to its previous alignment and has continued to retreat since. For example the 0m ODN contour on a cross-section in sub-section 15 monitored by PBC has retreated by approximately 12m since 1992. The beach has continued to fall during the course of 1994 when it would normally be expected to recover during summer months.

The beach in this area is generally steepening and contains an increasing amount of shell and shingle. It is believed that deepening of the East Looe is preventing onshore transport of sand from the Hook Sand. Increased tidal flows in the flood-dominated East Looe will carry sand into the Swash Channel where it is effectively lost from the beach system.

The rock groyne between sub-sections 13 and 14 has built up narrow sand and shingle beaches at its root but has not trapped a beach as wide as the one which historic photographs show was retained by the former groynes at this point on the frontage. Where reclamation for the Haven Hotel squeezes still further onto the foreshore from the original dune line, the beach becomes submerged at all tidal states.

As explained in Chapter 4, sediment transport models give a reasonable indication of longshore drift rates along most of the frontage but fail to represent the complexity of combined wave/current action near the end of Sandbanks peninsula. Potential drift rates on this frontage vary between $121000m^3$/year and $415000m^3$/year as shown in Table 3.2. By the principle of continuity there must be a circulation of material to "feed" this potential drift.

Structures

With the exception of a short stretch of dunes on the Sandbanks frontage, the whole of Management Unit 1 is protected with seawalls which have been built from about 1890 onwards.

From the PBC boundary to Poole Head a variety of stepped or near-vertical concrete walls are in fair to good condition. Although the sand beach is low (more so towards the east) these seawalls are not yet in danger of undermining. However, their prolonged life does depend on an adequate depth of sand at the toe. The foreshore level also has a direct influence on the wave conditions at the wall and hence it susceptibility to overtopping and scour. Where the beach is at 2.4m ODN the overtopping is negligible whilst where it is at 1.1m ODN overtopping during a 1:10 year storm is calculated to be 16 to 21l/s/m (depending on the seawall slope). The walls are backed by a continuous length of promenade which in places is in a rather poor condition and shows signs of subsidence. Along much of the frontage there are beach huts in front of the cliffs. A toe wall has been constructed along the foot of much of the cliff face to resist erosion and prevent eroded sand from being washed onto the promenade.

East of Poole Head the beach is groyned with timber structures 30 to 35m long, spaced at about 120 m. Though generally in fair condition, some of these structures are in poor condition, timbers being decayed and metal fixings corroded.

West of Poole Head there is a series of sloping, or vertical seawalls, some built to protect private properties, the remainder along the edge of the Sandbanks promenade. Because of the width of the beach at sub-sections 10, 11 and 12, some rather poor walls still provide an adequate level of protection. Moving west a short section of dunes (sub-section 13) is now showing signs of erosion.

Beyond sub-section 13 the dunes have been claimed by construction of a variety of vertical seawalls. Although structurally in a fair condition the stability of these structures depends on the foreshore level, which is falling. In order to safeguard sub-section 14 PBC placed emergency rock armour along the toe of the steel sheet pile wall in February 1994. By August 1994 this rock armour protection had settled by more than 0.5m. Rock armour has been used to protect the walls further west and the walls of the Haven Hotel are protected by a low-level rock revetment.

Land use and vulnerability

Land use along the Unit falls into two areas, east and west of Poole Head. To the east, the narrow area at the toe of the cliffs is occupied by beach huts, cafes and other amenities. These non-residential properties are at risk from both occasional overtopping damage and potentially from shoreline erosion. However, experience of similar coastal sites proves that capital works cannot be justified in economic terms solely on the basis of protecting such properties. The possibility of shoreline erosion initiating rapid cliff top retreat represents a greater financial risk. The cliff tops are occupied by valuable private residences. In places access roads lie close to the cliff edge.

To the west of Poole Head the sand dunes forming Sandbanks Peninsula are now occupied by private houses, apartments, hotels, beach huts, cafes and other amenities. These properties are potentially vulnerable to damage by wave overtopping and erosion. Actual flood damage on this gently landward-rising land is not a risk. Where the beach is wide, vulnerability to overtopping and erosion is slight, but moving westwards where the beach is narrower and private properties have claimed the backshore, overtopping is more frequent and seawalls are vulnerable to erosion.

Environmental sensitivity

The Poole Bay frontage is an important recreational amenity, its promenade and sandy beach being very popular with visitors and local residents. Preservation of this amenity will be an important consideration in planning management of the Unit. Towards the western end of Sandbanks, where the beach narrows, it is somewhat less popular. The large rock groyne is a barrier across the beach. Because of its mobile sand beach and concrete seawalls the Poole Bay shore is not of significant ecological interest. The dune backshore on the Sandbanks Peninsula has mostly been claimed by seafront property owners. Remnants of sand dunes are habitat to sand lizards. Where not claimed within seawalls this natural backshore tends to be eroding. The cliffs east of Poole Head are of considerable importance: where ongoing erosion reveals fresh exposures these are of geological interest, and where vegetated the cliffs support important flora and fauna. In recognition of their importance Canford Cliffs at the PBC/BBC boundary have been notified as geological SSSI's. Management of these geologically and ecologically important cliffs is discussed in Appendix 6.

Consequences of "do nothing"

Because of the dependence of the existing defences on beach conditions, the consequences of no further maintenance or capital works on the frontage depend on continuation in trends in shoreline change. The general trend is for beach lowering east of Flag Head Chine (sub-sections 1 to 7). West of Flag Head Chine (sub-sections 8 to 10) the observed trend is one of equilibrium or accretion. Further west (sub-sections 11 to 16) erosion becomes increasingly acute. If these trends are allowed to continue, then the seawalls in sub-sections 1 to 7 and 11 to 16 will be subject to increasingly severe hydraulic conditions as a consequence of beach lowering. At sections 8 to 10 beaches will continue to be healthy provided onshore transport of material from Hook Sand is not interrupted.

Although the concrete walls in subsections 1 to 8 are not in a bad condition it is not inconceivable that within ten years in an aggressive hydraulic environment, undermining could initiate collapse. Seawall collapse and subsequent rapid erosion will, in the course of time, induce cliff failure and cliff-top retreat. Before collapse, the walls will suffer from increasingly frequent overtopping, damaging beach huts and other properties.

The consequence of a "do-nothing" on sub-sections 9 to 12, which are protected by a wide beach, are not as immediate but are potentially more serious. The Sandbanks peninsula is at its narrowest here, so shoreline erosion threatens to breach this narrow section of land, cutting off Sandbanks from the mainland. Until construction of groynes in the 1890's there was a real threat that this would occur (Vernon-Harcourt, 1903). The seawalls on this section of frontage are generally in a worse condition than elsewhere, so shoreline erosion could initiate a breach in the sea wall. Given existing trends in shoreline change it seems unlikely that seawalls will breach in this area within the next ten years, but the risk of them doing so will continue to be a major concern in the longer term.

It is in sub-sections 14 and 15 that the consequences of a "do nothing" policy will be most immediate. Here the beach is continuing to fall exposing vulnerable sheet pile walls to increasingly aggressive hydraulic conditions. Emergency rock armour protection has helped to stabilize the walls but is by no means a permanent solution. If no further works are carried out increasing damage by wave overtopping can be expected to occur. Even more seriously, if beach levels continue to fall the sheet pile walls will fail initiating loss of property by coastal erosion. The risk of failure of these seawalls cannot be computed without knowledge of the depth of penetration of the sheet piles. However, if, as reported, the piles are only 4m long they are already highly unstable.

4.3.3 Management Unit 2 Haven Hotel to North Haven Point

Grid References SZ 0372 8704 to SZ 0352 8278
Length of frontage 0.31km.

Drawing 1 — PBC map reference number D065/01
Summary Table 4.5 — PBC unit reference numbers 1 to 11
Plate 4.5

Description

Flanking the Poole Harbour entrance the shore of Management Unit 2 is submerged in most tidal states only being exposed during low spring tides. It is subject to not only strong tidal conditions but also waves from both within and outside the Harbour.

East of the Haven, deep water gives way to the sandy beaches which characterise Management Unit 1. The northern limit of this Unit is at North Haven Point where conditions change from deep water to a narrow sandy beach.

Hydraulic Conditions

Tidal exchange with Poole Harbour means that currents of up to 2m/s flow within 100m of the shore of this unit. Scouring by these currents means that the channel is up to 16m deep, only 200m offshore. The channel decreases in depth moving into the Harbour.

Swash Channel (now dredged at the Bar) can propagate high waves from the southeast into the deep waters of the Harbour entrance. Interaction with ebb tides will steepen waves in the Harbour entrance. These waves do, however, run parallel to the shoreline of this Unit. In terms of wave action at the shore, locally generated waves from the south-west (the dominant wind direction) are also an influence.

A sandy shoal, known as Chapman's Peak, is located offshore of sub-section 4. Dredging was carried out by PHC in 1992 to remove the shoal in order to improve navigability of the Harbour entrance. Predictions (HR, EX 2356, 1991) had indicated that removal of the top of the shoal would not have adverse effects on the adjoining shoreline but that it was likely that material would return within a year. In the event, the shoal had substantially reformed only three months after dredging (Appleton, 1994).

Coastal geology

The Sandbanks Peninsula is formed from recent sandy sediment which have been deposited by marine and wind action.

Foreshore

The sandy shore of the Unit is submerged during most tidal states and is only exposed at low water during spring tides. The sand is highly mobile. Exposed seawall foundations indicate that the foreshore has lowered by as much as 1.5m. Anecdotal evidence suggests a healthy beach in once extended along this whole unit and that the beach levels have dropped by more than 0.5m in the last decade or so. Photographs taken during the 1920s show a narrow beach at sub-section 3 which is now eroded.

Comparison of historic surveys confirms this trend in shoreline erosion. At the southern end of sub-section 2/4 the High Water Mark retreated by 14m between 1888 and 1993. The retreat has been progressively less towards the north. At the boundary between Units 2 and 3 there was in fact a 28m advance between 1888 and 1933 though this was followed by a retreat of 13m between 1933 and 1993.

Construction of seawalls has insolated the sandy backshore and dune system in private gardens. Now that the foreshore has eroded the seawalls "squeeze" the coast and erosion of the dune system to supply the beach is prevented. The various jetties along the frontage have negligible effect on the shore.

Structures

The frontage is protected by a variety of privately constructed vertical walls. Lowering of the foreshore has exposed the foundations to these structures leaving them susceptible to washout of material. Some residents have observed subsidence behind the walls due to undermining. Remedial underpinning or protection with sheet piles has been carried out on some frontages. Privately maintained, the walls are in a state of repair varying from bad to fair.

Under such active tidal and current conditions the walls are subject to significant hydraulic loads. The stability of these vertical walls is dependent on the level of the foreshore which, under volatile sediment transport conditions can vary. A combination of high foreshore dependency with some inadequate seawalls threatens the integrity of this section of defence.

Land use and vulnerability

The shoreline in this Unit is lined with private properties the gardens of which slope down towards the seawalls.

Thus, whilst not at risk from flooding, without the protection of the seawalls, the private gardens would tend to erode, to attain a more natural configuration of a somewhat retreated shoreline.

Environmental sensitivity

Public access to the frontage is possible only at the chain ferry which attracts a large number of visitors. The appearance of the shore to onlookers from this point is therefore a consideration. Fishing is a recreational activity here. The remainder of the frontage is privately owned. Private jetties are used for mooring pleasure craft.

The (submerged) mobile sand bed and vertical seawalls are not of significant ecological importance.

Consequences of "do-nothing"

Without major maintenance work or replacement, local collapse of some of the seawalls facing this active hydraulic environment is likely within the space of ten years. The expected life of a few sections of wall is less than five years. Collapse will be followed by shoreline readjustment to revert towards the natural beach now claimed by private properties. Study of historic plans by Robinson (1953) (see Figure 3.30) indicates that Sandbanks Peninsula is reasonably stable in the long term. However, if unprotected, this Unit would to be susceptible to occasional, quite major, shoreline changes associated with occasional storm events.

4.3.4 *Management Unit 3 North Haven Point to Whitley Lake*

Grid References SZ 0352 8728 to SZ 0429 8768
Length of frontage 1.00km.

Drawing 1	PBC map reference number D065/01
Summary Table 4.6	PBC unit reference numbers 12 to 25
Plate 4.6	

Description

Extending between North Haven Point and Whitley Lake, Management Unit 3, like Unit 2, is tide dominated, but has been separated from Unit 2 because its beach is more exposed at low tide and its orientation is towards the north west rather than the south west.

The beaches of Unit 3 continue uninterrupted into Unit 4. However the course of the Middle Channel means that Unit 4 is not as dominated by currents as Unit 3. Furthermore, land use differs between the two Units.

Hydraulic conditions

Hydraulic conditions offshore are dominated by strong tidal flows through Brownsea Road. The channel is naturally scoured to as deep as - 12m ODN so has not needed to be part of capital or maintenance dredging programmes. Closer inshore tidal currents and bed scour are less marked.

Orientated between about 320° and 355°, the frontage is sheltered from the principal south-westerly wave direction. However, north westerly winds can blow over a deep-water fetch about 3km long. The wave climate computed at point I (see Figures 2.57) is closest to the site. The maximum 1:10 year significant wave height is 0.7m.

Coastal geology

The frontage forms part of the Sandbanks Peninsula, based on recent sand deposits which, above high water level, are wind-blown.

Foreshore

The Unit comprises flat sandy foreshores. The sloping sandy backshore which, in the past, would have surrounded to Sandbanks Peninsula, has now been claimed as the gardens of private properties, as part of the coastal squeeze from which this and other frontages suffer. Some narrow sloping sand backshores with shingle patches do remain. East of the Royal Motor Yacht Club the shore tends to be sandy mud and is scattered with boulder debris. Transects of the main tidal channels (Dyrynda, 1987) indicate that sediments become coarser towards the centre of the channels, though this trend was not observed closer inshore during walkover inspection of the beach.

Since the first edition of the Ordnance Survey in 1888 the location of the High Water Mark has both advanced and retreated at varying rates along the Unit frontage.

The steel sheet pile jetties of the Royal Motor Yacht Club constitute a barrier to littoral transport and sand and shingle has built up at the jetty roots but is not of morphological significance as there are no significant downdrift impacts. The several wooden jetties on the frontage have no impact on the beach. At North Haven Point a few short groynes stretch across the sloping sand and shingle beach. A narrow beach has built up, suggesting a net south-easterly drift along the frontage.

Structures

The frontage is protected by a variety of privately constructed sea walls in fair to good condition. A typical sea wall cross-section (see Figure 4.4) has been analysed and found to provide good protection against overtopping. Although old the occasional groynes are also in fair condition.

Land use and vulnerability

The shore in Unit 3 is backed by private houses and marinas. Gardens of the properties, built on former sand dunes, slope down to the shore. Thus the properties are not threatened with flooding and the sea walls protect the backshore from erosion. The exception is the boat house at the southern end of sub-section 1 which has been converted into a private residence. This property is situated at the high water mark and has suffered overtopping damage and flooding during storms.

Evidence of shoreline change is inconclusive. Plans presented by Robinson (1946) suggest that the Sandbanks Peninsula has not significantly changed over the period studied. However, a sand spit is a volatile environment and, like all dune systems, can be rapidly eroded in severe storm conditions. For this reason, the seawalls of necessity protect the properties from the possibility of erosion.

Environmental sensitivity

Public access to the shoreline is restricted and the beaches are not generally used by promenaders or bathers. The water front is very popular with pleasure craft users and there are many moorings and jetties.

Because of the firmer and somewhat muddy substrate the shore east of the Royal Motor Yacht Club is of greater ecological significance than further west where the more mobile sandy bed cannot support burrowing species.

Consequences of "do nothing"

Although some of the sea walls are ageing, most are in a fair condition and it seems unlikely that collapse would be initiated in the space of ten years. If, however, collapse did occur, shoreline readjustment is to be expected though not as rapidly as in the more active hydraulic environment of Unit 2.

The converted boat house at the southern end of Unit 3 will continue to suffer flood damage.

4.3.5 *Management Unit 4 Whitley Lake*

Grid References SZ 04298768 to SZ 0431 8906
Length of frontage 1.97km

Drawing 1	PBC map reference number D065/02
Summary Table 4.7	PBC unit reference numbers 26 to 30.
Plate 4.7	

Description

The shoreline flanked by Shore Road and Banks Road forms a wide flat, west-facing embayment.

The channels of Brownsea Road which dominate Unit 3 are further offshore from Unit 4. Further north, beyond the modest barrier of East Dorset Yacht Club jetty, the land slopes gently upwards directly from the shore.

Interaction with shores on adjacent Management Units is active but hydraulic, foreshore and land use characteristics justify Unit 4 being a separate unit.

Hydraulic conditions

The open bay which forms the inside of Sandbanks Peninsula is exposed to a narrow but relatively long fetch of 9.5km to the west-north-west. Waves from the south-west passing through Brownsea Road also approach the Unit. Whilst the fetch to the south-west is limited in length by Brownsea Island the wave climate at points H and I (see Figures 2.56 and 2.57) is still dominated by south-westerly waves corresponding to the direction of most frequent wind.

The nearshore bathymetry is flat, as far as the North Channel which is over 500m offshore. Before dredging of the Middle Channel in 1986, the North Channel was observed to have been shifting in a north-easterly direction, i.e. towards the shoreline (Green, 1940, PHC, 1984). Since dredging of the Middle Channel, the bed and sides of the North Channel have been accreting suggesting that is migration has been halted. In any case the Unit is in quiet current conditions, flows being concentrated in the Middle Channel well offshore.

Coastal Geology

The Luscombe Valley at the north end of the Unit is bedded with alluvium, whereas the ground which leads from the shore towards Poole Head is Branksome Sand. More than half of the Unit, which runs along Banks Road; is formed from the wind-blown sand formations of Sandbanks Peninsula.

Foreshore

The foreshore is a wide, flat expanse of sands and muddy silts with occasional gravel. Saltmarsh vegetation persists in the centre of the bay. On the sandy backshore at the centre of the bay there is small area of upper saltmarsh and dune vegetation. The area of saltmarsh is a remnant of more extensive marsh. In recent years the sediments have become increasingly sandy. Aerial photographs taken in 1924 show a network of meandering intertidal creeks typical of fine mud sediments (Gray, 1985) and these are also apparent on the

1889 and 1901 Ordnance Surveys. Thus the character of the bay has changed during the course of this century but that does not necessarily indicate that there is a natural tendency towards shoreline erosion. Indeed the general trend has been for advance in the location of the High Water Mark between 1888 and the latest surveys. Concentration of currents in the Middle Channel, reducing flows in the North Channel, means that sediment transport by tidal flows in Whitley Lake is now reducing.

Structures

A steep-sided concrete seawall has been constructed on the backshore along the edge of Shore Road and Banks Road. The older section of wall was built in the 1950's and is probably founded on sandy deposits. The concrete walls in sub-section 5 have been rehabilitated in 1994. In sub-section 2 the walls are in need of repair with vertical gaps in places, whilst in sub-section 3 the walls are low.

Overtopping of the walls in this Unit during occasional storms has closed the adjacent road. Overtopping calculations corroborate this evidence indicating that during a 1:10 year storm the overtopping discharge at sub-section 4/5 will be 2.0l/m/s. The worst overtopping is experienced in sub-sections 3 (where the wall is low) and 5 (which faces the dominant wave direction). South of sub-section 3 the road rises above the low walls which are sheltered from high waves, and overtopping has not in the past been a problem.

Land use and vulnerability

Shore Road and Sandbanks Road connect the Sandbanks Peninsula with the rest of the Borough. This important road is liable to being closed by overtopping during storms. The houses and commercial properties behind the road are set well back and are not susceptible to flooding.

The Luscombe valley is low-lying, much of it below 1.5m ODN. It is therefore susceptible to flooding and is threatened by overtopping at sub-section 5. The Luscombe valley is not developed though a few properties on its edge are at risk from extreme surge water levels.

Environmental sensitivity

The wide sandy bay is a focus for water-based recreation. The intertidal area is used for mooring pleasure craft and the bay is popular for wind surfing. Although the area of saltmarsh is relatively small it contains most of the common intertidal plants found in the Harbour. Similarly the numbers of intertidal birds are relatively small but most of the commoner species occur (Gray, 1985). At the same time, easy public access from the road which fringes Whitley Lake and broad views across the Harbour entrance and beyond attract large numbers of people throughout the year. Consequently this section of the shoreline is one where people and wildlife interact to an unusually large degree.

Consequences of "do nothing"

The seawalls are in reasonable condition having been well maintained. An end to maintenance work can however, be expected to be followed by fairly rapid decay of these old structures.

A more immediate threat is from overtopping flooding Shore Road and cutting off access to Sandbanks. Sea level rise will mean that these occasional flood events become more frequent.

4.3.6 *Management Unit 5 East Dorset Sailing Club to Salterns Marina*

Grid References SZ 0431 8906 to SZ 0378 8970
Length of frontage 0.83km

Drawing 2 — PBC map reference number D065/02
Summary Table 4.8 — PBC unit reference numbers 30 to 40
Plate 4.8

Description

Management Unit 5 is bound by the substantial structure of Salterns Marina in the north. At the southern boundary the sloping cliffs which form the coast within the Unit give way to the low-lying land of the Luscombe Valley. Land use and seawall type also change so that whilst within the Unit the shore is backed by variety of walls and sloping gardens or common ground, in Unit 4 Shore Road is immediately behind the concrete seawall.

The Unit's southern boundary is at the jetty of the East Dorset Sailing Club which forms a modest barrier to sediment transport on the upper shore.

Hydraulic conditions

Brownsea Island (2km to the south-west) affords shelter from waves from between 206° and 252°. Through Brownsea Road the Unit is exposed to a fetch of 5.2km in a south westerly direction. To the west fetches increase from 6.5km to a maximum of 11.5km down the Wareham Channel. Salterns Marina affords some shelter from westerly waves to the northern end of the unit. A wave climate has been calculated at point G (see Figure 2.55) close to the centre of the unit. Analysis of extremes indicates a 1:10 year significant wave height of 0.78m.

The centre-line of the North Channel is 420m from the shore but passes close to the end of the jetties of Salterns Marina. The North Channel is ebb dominated with spring tidal currents of more than 0.5m/s. However, closer to the shore velocities are much lower reaching a maximum of only 0.1m/s 100m offshore of the MHWS tide mark. Tidal modelling indicates locally increased velocities between the jetty of Salterns Marina and the north end of the Unit.

Before dredging of the Middle Channel in 1986 the North Channel was observed to have been shifting in a north-easterly direction ie towards the shore (Green, 1940, PHC, 1984). Since dredging of the Middle Channel the bed and sides to the North Channel have been accreting suggesting that its migration has been halted. Dredging of the Middle Channel has not had a noticeable effect on the shallows or shore.

Coastal Geology

The sloping shores of the Unit are predominantly composed of Branksome Sand the steeper sections of bluff having been eroded by wave activity, possibly before the Sandbanks Peninsula provided shelter from waves generated in Poole Bay. Parkstone Clay, which is the upper bed in the Poole Formation, outcrops on the shore towards both ends of the Unit. At the northern end of the Unit, immediately south of Salterns Marina, deposits of Head outcrop.

Foreshore

The foreshore is mainly muddy sands. At the southern end of the Unit there is a narrow sand and gravel upper beach and towards the north patches of gelatinous mud appear in the foreshore. Occasional remnants of eroded saltmarsh are also noticeable towards the north of the Unit. The beach, particularly towards the south, is strewn with rock remnants of walls and gabions.

Apart from the narrow sloping sand beaches at the limits of the Unit the foreshore is quite flat, widening towards the north of the Unit where the Low Water Mark is about 200m offshore.

Calculations of longshore drift indicate a nett potential drift of about $4000m^3$/year towards the north. A northerly drift is confirmed by the observed widening of the upper beach next to Salterns Marina.

The existing foreshore is currently stable though remnants of eroded saltmarsh indicate previous erosion associated with saltmarsh dieback. The low bluffs which form the coast suggest a tendency for erosion in the past and old photographs indicate that erosion of this cliff face was still taking place during the 1950s. Gaps beneath the outfall pipe near the centre of the Unit indicate that the beach has lowered since construction of this structure, during the 1890s. The backshore is fixed by gabion revetments over the southern 320m and private seawalls over the northern 500m.

Sand has accreted around the concrete root of the jetty at the East Dorset Sailing Club. The wooden piled end of this jetty and several other jetties have no impact on the beach. Neither does the sewerage outfall pipe. The northern end of the Unit is formed by the steel sheet pile wall of Salterns Marina. This affords increasing shelter from westerly waves towards the northern end of the Unit. Immediately to its south is a sand upper beach above the muddly shore which is backed by grass banks. The vertical sides of the marina do, however, reflect locally generated waves from the south or south west onto the shore.

Structures

The concrete seawall at the East Dorset Sailing Club, which is a continuation of the wall in Unit 4, gives way after 60m to a promenade protected by gabion baskets on its seaward side. These gabion baskets are close to the end of their useful life and are below the 1:10 year water level. The promenade is regularly overtopped which is not only dangerous to pedestrians but damaging to the structure and the cliff behind.

A variety of privately constructed seawalls protect the properties along the northern 500m of the unit. These walls are generally in a moderate condition though the condition of some is poor. For 80m near the centre of the unit an iron sewerage pipe protects the toe of the private seawalls.

Land use and vulnerability

Land slopes uphill from the shore eliminating flooding risk. Over the southern 320m of frontage the promenade is backed by vegetated slopes which lead up to Evening Hill. Over the northern 500m the sloping gardens of private properties back directly onto the shore.

The High Water Mark along most of the frontage retreated between 1900 and 1955 after which there has been no change. The average retreat was 11m in 55 years. This is considerably less than the cliff top erosion rate recorded by May (1969) of 42.7m between 1886 and 1952 at Lilliput, though May does not specify which section of the Lilliput frontage (which is included in this Unit) this rate applies to. A land slip, which is reported by PBC to have occurred during the 1950s, may account for the cliff-top retreat recorded by May (1969) whilst not appearing on the Ordnance Survey plans. Photographs taken in 1952 show the cliff obviously eroding or regraded where private properties have been constructed on the cliff-top, whilst the toe, at least on the central section of frontage, is fixed by the shore-parallel sewer pipe (still in-situ today).

Therefore, whilst now stabilised by seawalls, the potential for modest erosion exists. The risk of erosion and slip failure of the cliff is greatest in sub-section 2 where the cliff is protected by gabion baskets which are in a bad condition. Cliff failure on this frontage will jeopardise the road on the cliff top which is an important thoroughfare.

Environmental sensitivity

Public access to the Unit is along the promenade in the south. The promenade and Evening Hill affords good views across the Harbour towards Brownsea Island. Further north there is no direct public access apart from along the shore at low water. Pleasure craft use the various private jetties along the frontage. The beaches are extensively dug for bait.

The muddy parts of the shore particularly towards the centre of the Unit attract wading birds feeding on the intertidal foreshore.

Consequences of "do nothing"

If capital works are not carried out failure of the gabion promenade in sub-section 2 can be expected within the next two or three years. The promenade will become increasingly unsuitable for public access. The rock debris which will be deposited on the shore from the collapsed gabions will afford some protection to the cliff toe. It is unlikely that major cliff erosion will be initiated within ten years. However, the risk of cliff failure encroaching upon Evening Hill will be increasingly threatening.

Though some are in bad or poor condition it is not likely that any of the private seawalls will collapse within the next ten years. They will continue to suffer overtopping but at a tolerable rate. Sea level rise over a ten year period will not have a noticeable impact on the frequency of overtopping.

4.3.7 *Management Unit 6 Salterns Marina to Parkstone Yacht Club*

Grid References SZ 0378 8970 to SZ 0298 9042
Length of frontage 1.49km

Drawing 2	PBC map reference number D065/03
Summary Table 4.9	PBC unit reference numbers 1 to 9a
Plate 4.9	

Description

The coast between the littoral barriers formed by Salterns Marina in the east and Parkstone Yacht Club in the west is relatively straight and orientated towards the south-west at an average angle of 220°N. The sloping hinterland is occupied by private properties.

A narrow inlet allows tidal exchange with Blue Lagoon and during high water levels the various structures on the bar across the lagoon are submerged. However, apart from a narrow tidal channel, the coastal regime along the frontage is largely unaffected by the presence of Blue Lagoon. Historic plans indicate that the once-marshy lagoon was drained during the 19th century but was flooded again in the early part of this century.

The jetties of Salterns Marina and Parkstone Yacht Club constitute substantial barriers to longshore transport past the ends of the unit. Exchange with adjacent units is minimal.

Hydraulic conditions

The frontage is exposed to fetches of up to 6km in a south-westerly direction so significant wave heights during a 1:10 year storm can reach 0.78 m. The flat sea bed near the shore does limit the height of waves at the shore.

The North Channel is closest to the shore at Salterns Marina where its centre line is 600m offshore. The jetties of Salterns Marina prevent tidal flows from reaching close to the shore at the eastern end of the unit though they do generate some eddy effects in the main tidal flow past their end. The current flow conditions at Unit 6 are further influenced by the small tidal exchange with Blue Lagoon which is through a narrow current-scoured channel. The wide mouth of Parkstone Bay is of less influence.

Coastal geology

The frontage is formed by low bluffs of Parkstone Clay. These slope steeply directly to the shore suggesting that until construction of the seawalls the shoreline was eroding. Towards the back of Blue Lagoon beds of Branksome Sand and alluvium outcrop.

Foreshore

The foreshore is flat and sandy with some shingle and shells.

The net potential longshore sand transport rate has been calculated to be about 2000m^3/year in an easterly direction. The direction is confirmed by the eastward shift of the channel leading into Blue Lagoon but the volume will be less than potential since the beach is far from full. Supply from the west is limited by the jetties of Parkstone Yacht Club. Transport past the eastern end of the unit is restricted by the jetties of Salterns Marina. Sand, which is transported past the end of Salterns Marina, can be influenced by the main tidal flows in the North Channel and will tend to stay in the channel system rather than passing onto the shores of Unit 5.

The low bluffs which form the shore suggest a natural tendency towards erosion in the past. There has been negligible natural change in the location of the High Water Mark since the 1888 Ordnance Survey suggesting that the low cliffs are a relic of erosion which took place when larger wave could penetrate the Harbour through a wider mouth than presently exists. Seawalls have been built on the foreshore claiming the backshore of the natural beach. A number of short groynes intersect the shore, inhibiting longshore transport.

Old editions of the Ordnance Survey indicate that Blue Lagoon was drained during the last century with a network of drainage ditches evident on the 1889 edition. A continuous sand and shingle beach stretched along what is now the bar across the lagoon. It is shown in the same way on the 1902 and 1932 revisions. By 1932 the area had been flooded, tidal exchange with the lagoon being controlled by a sluice gate which, more recently, fell into disuse. Flow in and out of the lagoon is now through a breach in a sand and shingle bar (possibly artificially initiated) southeast of the former sluice. Apart from this narrow channel the lagoon is very shallow with a bed of soft muddy and sandy sediments.

Parkstone Marina has gradually expanded. What was once the narrow Weston's Island in the mouth of Parkstone Bay has been included in the impoundment of the marina.

Structures

The properties on the frontage are protected by a variety of privately constructed walls in varying states of repair. The walls in sub-section 3 were upgraded during the 1950s by construction of concrete and steel toe protection.

Across the mouth of Blue Lagoon the bar is protected with a variety of concrete, steel and gabion structures, mostly in bad condition. The sluice which controlled flow in and out of Blue Lagoon is now disused and its channel is blocked with gabions and debris.

The groynes in front of sub-section 3 are in poor condition.

Overtopping of a typical private seawall on the frontage has been calculated to be 0.7l/m/s during a 1:10 year storm. Though quite high this discharge is tolerable onto the grassy sloping private gardens adjacent to the seawalls.

Land use and vulnerability

Land adjacent to the shoreline is occupied by large private properties. Either end of the unit is occupied by a marina. Because their gardens slope down to the shore, the properties are not threatened with flooding. However, the seawalls do prevent continuing erosion of the shore from which the properties are at risk.

The bar across Blue Lagoon shelters the shoreline within the lagoon. Within Blue Lagoon most of the land slopes down to the shore so flood risk is restricted to a few properties on the eastern side of the Lagoon. Since some of the shoreline within Blue Lagoon is not protected but merely vegetated a more active hydraulic environment as a result of breaching the bar could initiate shoreline erosion.

Environmental sensitivity

Public access to the frontage is restricted. The jetties and marinas along the frontage constitute the principal amenity. The muddy sand beaches are not a particularly important ecological habitat. Blue Lagoon with its mudflats and naturally vegetated shoreline is more significant.

Consequences of "do nothing"

Though some are in poor condition it is not likely that any of the private seawalls will collapse within the next ten years. When collapse does ultimately occur shoreline retreat and backshore erosion is to be expected but private gardens are sufficiently long to delay loss of property for several years. Wave overtopping of the private walls will continue at a fairly high rate during occasional storms but sea level rise will not have a noticeable impact on this currently tolerable problem.

Because of the poor state of repair of its various protective structures, the bar across Blue Lagoon could breach within the next ten years. The breach is unlikely to propagate rapidly but there will be a general increase in wave climate within the lagoon which will threaten the unprotected shoreline with erosion, albeit at a modest rate.

4.3.8 *Management Unit 7 Parkstone Bay and Baiter Park*

Grid References SZ 0298 9042 to SZ 0136 9023
Length of frontage 2.42km

Drawing 2 — PBC map reference number D065/03
Summary Table 4.10 — PBC unit reference numbers 9 to 15
Plates 4.10 and 4.11

Description

Despite a varying hydraulic environment associated with a wide range of exposure conditions, Baiter Park and Parkstone Bay constitute a continuous littoral environment. Longshore drift from the exposed Baiter frontage passes unhindered towards Parkstone Bay. Management Unit 7 is also characterised by a hinterland of low-lying reclaimed recreation ground.

To the east, marinas and vertical sea walls "squeeze" the narrow foreshore. The jetties of Parkstone Marina inhibit sediment interaction between Units 6 and 7. The boundary with Unit 8 is marked by a change in foreshore (from beach to deep water), seawall type (from sloping revetment to vertical wall), and land-use (from recreation ground to quays).

Hydraulic conditions

The orientation of the shoreline within the unit varies from 111° to 14°N. South-westerly facing shores are exposed to fetches of up to 6km in the direction of prevailing wind so the wave environment, particularly near Baiter Point can be quite active. The wave climates computed at points D and E (Figures 2.52 and 2.53) are both of relevance to Unit 7, indicating 1:10 year significant wave heights of up to 0.8 m. However, the south and south-easterly facing shores of Parkstone Bay are exposed to shorter fetches of between 2.2km and 3.2km. The consequent gradient of wave heights round Baiter Point drives littoral processes in this area.

The shallow bathymetry of Parkstone Bay limits wave heights in the back of the bay so that whilst exposed to south-westerly fetches, the back of the bay is more sheltered than Baiter.

The impact of currents in Parkstone Bay is minimal with no defined system of channels. Nearshore tidal flows are low. The Middle Channel, the centre-line of which is as much as 700m off Baiter Point, has been dredged along its natural alignment concentrating tidal flows offshore.

Coastal geology

The shores of most of the unit have been reclaimed. Underlying beds are alluvium and clay. The previous extent of Parkstone Bay before sedimentation with alluvium and subsequent reclamation is indicated by the low clay bluffs on the landward edge of the reclamation area. The railway embankment has separated Poole Park boating lake from the rest of Parkstone Bay, so it is no longer obvious that the shoreline previously extended to the beds of Branksome Sand on which Poole town centre is built.

Foreshore

The sloping upper beach along the frontage of Baiter Park is sand with a shingle crest. Beyond the narrow sand beach the foreshore is flat, comprising poorly sorted sediments, ranging from sand to shingle with clay in places. The sediments are black in appearance and the surface is "armoured" with a layer of pebbles.

In Parkstone Bay the shore is flat and exposed only at low water. It comprises mostly red or black sands with some clay and shingle. Near the shore the surface is littered with debris and seaweed. Towards the centre of the bay the bed is more muddy. In the eastern corner a small fringe of vegetation stabilises the shore.

Before the back of Parkstone Bay was claimed as a recreation ground the shoreline was naturally advancing. Between the 1888 and the 1900 editions of the Ordnance Survey the High Water Mark advanced by an average of about 20m.

Before major land claim (for construction of Poole Gas Works) the beaches of sub-sections 7/5 and 7/6 formed a narrow spit extending into Parkstone Bay. Despite the changes in the immediate hinterland this beach system still exists though it is now starved of sediment from the heavily built up frontage updrift. These beaches have been slowly eroding. The High Water Mark 50m west of the public slipway retreated by 28m between 1888 and 1993 and at Baiter Point it retreated by 12m during the same period.

Longshore drift calculations (see Table 3.4) indicate a potential sand transport rate of about 7000m^3/year in an easterly direction along the south-facing Baiter shore. Actual drift will be less than the calculated potential since there is not a full sand beach profile, and moreover, supply from the west is minimal. However, the calculations confirm the observed trend of sediment transport, and explain the previous tendency for erosion on the section of the south-facing Baiter frontage which was, until recently, unprotected.

Once sediment has been transported along the beach into Parkstone Bay, the potential drift reduces in the more sheltered environment. Tidal current circulations also make minimal contribution to sediment transport in the bay so it can be considered to be a sediment sink.

The beach is unimpaired by cross-shore structures along most of the frontage. The public slipway between sub-sections 4 and 5 has led to a build-up of beach on its western (updrift) side.

Structures

The majority of the frontage is protected by sloping revetments constructed of rock, concrete, or blockwork. The previously unprotected and eroding sub-section 6 was protected with a new stepped concrete wall in 1994. Sub-section 5 has no "hard" defence but the existing shingle bank affords protection and there is no evidence of serious erosion. In the silted back of the bay the unprotected sub-section 2 is naturally vegetated and stable.

The rock, stepped concrete, and interlocking concrete block revetments are in fair to good condition whereas the concrete sloping concrete revetment along sub-section 3 is rather poor.

The structures are occasionally overtopped, a discharge of 6.1l/s/m during a 1:10 year storm having been calculated at sub-section 4. This overtopping discharge is sufficiently high to represent a hazard to pedestrians but is not damaging to the path or recreation ground behind the revetment.

Land use and vulnerability

The shore is backed by low-lying land which has been progressively claimed for a variety of purposes. Construction of the railway line across Parkstone Bay separating what is now the boating lake from the rest of the Bay was the first major work. The 1887 Ordnance survey shows a sluice channel beneath the railway line to permit drainage from the enclosed part of the Bay. Subsequent land claim seaward of the railway line completely separated the boating lake from the rest of the harbour.

The recreation grounds and a car park which are now adjacent to the shore are susceptible to occasional overtopping. Whitecliffe recreation ground was identified in the NRA Sea Defences Survey (1991) as being susceptible to flooding.

Further back are roads, houses and a railway line. Most of these properties are at a high enough level not to be at risk from flooding though there are residential areas in Parkstone and at Green Gardens which are below 1.7m ODN and are therefore at risk from extreme surge water levels. The NRA Sea Defences Survey (1991) identified Green Gardens as being under threat from flooding and work was carried out to improve the defences in sub-section 7. In January 1993 Green Gardens was flooded by overtopping water from Town Quay and this problem is discussed in relation to Management Unit 8.

Environmental significance

Baiter Park and Parkstone Bay are popular throughout the year with promenaders. The narrow sandy beach at Baiter is used for recreation during the summer. There is a slipway for the public to launch boats at Baiter. The beaches are dug for bait.

Erosion of sub-section 6, which occurred until recently, deposited glass and the remains of shredded household refuse (which was used as fill for the Baiter reclamation) on the foreshore.

The wide inter-tidal flats are a feeding ground for numerous bird species. The grasslands, on reclaimed land, at Baiter and Whitecliffe, form an integral part of this area and are used by intertidal birds for roosting and feeding (Gray, 1985).

Consequences of "do nothing"

The frontage is now adequately protected to prevent damage from erosion which continued until recently. The standard of protection is adequate for the next 10 years.

4.3.9 *Management Unit 8 Town Quays*

Grid References SZ 0136 9023 to SZ 0093 9105
Length of frontage 1.65km

Drawing 2	PBC map reference number D065/03
Summary Table 4.11	PBC unit reference numbers 15 to 29.
Plate 4.12	

Description

The frontage surrounding the historic part of Poole has been reclaimed by works to construct the various quays. Management Unit 8 comprises mostly vertical sea walls built for boat access so that the shore at their toe is submerged during all tidal states. The boundary with adjacent Unit occurs where these vertical walls give way to sloping beaches in the east (Unit 7) and mudflats in the north (Unit 9). The Unit is also characterised by the strong current conditions in the Back Water Channel. The waterfront at Town Quay is open to a significant wind fetch across Poole Harbour.

Hydraulic conditions

Fetch lengths and direction vary markedly throughout the unit. The eastern end of the unit is exposed to waves generated in Poole Harbour from a southerly and south-westerly direction over a length of up to 4km. On the reach of the Back Water Channel near the swing bridge, the frontage is sheltered from wave action whereas further north the frontage is exposed to fetches of up to 1.7km in a north westerly direction. Wave conditions relating to this frontage have been computed at two points (C and D, see Figures 2.51 and 2.52). Extreme waves towards the eastern end of the unit are almost twice the height (1:10 year H_s = 0.77 m) of waves at the northern end of the unit (1:10 year H_s = 0.42 m).

Tidal exchange with Holes Bay is through the Back Water Channel (adjacent to this Unit). The channel has been narrowed by construction of the quays so that currents reach a maximum velocity of 0.7m/s at its narrowest point on spring tides (HR, EX2160, 1990). At its opening, the so-called Little Channel flows past the Hamworthy Quay so that currents at the eastern end of the Town Quay are limited and consequently the bed shoals near the Fisherman's Quay breakwater. Dredging is therefore carried out in the western approach to Fisherman's Quay

Coastal geology

The Poole Town Quays have all been reclaimed and natural strata of alluvium only outcrop at the eastern end of the unit near the Fisherman's Quay.

Foreshore

The unit is characterised by a stable sandy sea bed which is submerged at all tidal states.

Structures

The vertical quay walls along the frontage are constructed from a variety of materials (steel, timber, masonry, concrete) and are in widely varying conditions of repair. The Town Quay walls are in fair to good condition, whereas further north the various private quays are in a worse condition.

The water at Town Quay is over 5.5m deep during a 1:10 year storm which means that the hydraulic conditions are well outside the limits of usual seawall overtopping calculations. Overtopping rates cannot, therefore, be estimated without recourse to physical model studies. However, evidence suggests that overtopping is a problem at Town Quay with damaging incidents having occurred on 17 December 1989 and 13 January 1993.

The Fisherman's Quay wall is low (1.42m ODN) and is apparently occasionally overtopped, but is sheltered from direct wave attack by the breakwater.

Land use and vulnerability

The land behind the Poole Quays is occupied by industrial and commercial properties as well as roads, car parks and amenities. Overtopping of Town Quay threatens these properties with flooding; much of the area is below 2.0m ODN and some is below 1.5m ODN. The surface drainage on the Quay has been designed to discharge peak rainfall run-off volumes and does not have sufficient capacity to simultaneously discharge overtopping waters. Thus when overtopping coincides with heavy rainfall (which it tends to do) the low-lying areas of the old part of Poole Town can be flooded.

During 1993 Green Gardens (east of Town Quay) were flooded by water overtopping Town Quay. The Town Quay (1.85 - 1.90m ODN) does not achieve the indicative standard of protection against a 1:200 year surge (1.92m ODN).

Environmental sensitivity

Discharge of effluent from the industrial works on the south-eastern shore of the Holes Bay has contaminated sediments on the bed of the Back Water Channel. This environmentally damaging practice is now prohibited but the contamination remains.

Poole Town Quay is of historic importance and attracts many visitors. Its function as a commercial quay constrains the types of sea defence structures which would be suitable.

Consequences of "do nothing"

The existing Town Quay wall will continue to be subjected to overtopping. Occasionally discharges will be sufficiently high to represent a risk to passers-by or motorists, and there is a risk of more widespread flood damage to property. Overtopping damage will become more severe with rising sea levels predicted in the future.

Decay of the structures themselves, which towards the north are already in a poor state of repair, will threaten collapse and consequent damage to adjacent properties. However, the walls in the poorest state of repair are in a quiet hydraulic environment and probably still have a residual life of about ten years under these conditions.

4.3.10 *Management Unit 9 Eastern Holes Bay*

Grid References SZ 0093 9105 to SZ 0066 9222
Length of frontage 1.91km

Drawing 3	PBC map reference number DO65/04
Summary Table 4.12	PBC unit reference numbers 1 to 3
Plate 4.13	

Description

Management Unit 9 covers the length of reclaimed frontage on the eastern side of Holes Bay between the deeper water frontages on the Back Water Channel and the Holes Bay viaduct. The Unit is fronted by mud flats and remnants of salting but because of its south-westerly aspect and proximity to Creekmore Lake it is the least sheltered of the three (10 to 12) Holes Bay Management Units. Interaction with adjacent units is minimal being separated from northern Holes Bay by the viaduct and being quite different in character to the relatively deep-water frontages in Unit 8 to the south.

Hydraulic conditions

Fetches within Holes Bay are limited to a maximum of about 1.3km in a westerly to south-westerly direction. A wave climate has been computed at Point C within Holes Bay (see Figure 2.51). Analysis of extreme wave heights indicates a 1:10 year significant wave height of 0.42m.

Protection of this section of frontage by saltings is less extensive than elsewhere in Holes Bay so higher waves can approach the shore. The environment is nevertheless sheltered.

Back Water Channel and Creekmore Lake carry the main tidal exchange in Holes Bay close to the shore. The stable tidal channels are closer to the shore in the northern part of the unit.

Coastal geology

The frontage along Management Unit 9 is entirely reclaimed land with no natural outcrops. The underlying strata are clays and alluvium.

Foreshore

The flat foreshores are formed from a poorly sorted material comprising sediments ranging from muds to sands and gravels. The muds on the whole are very soft and mobile indicating accretion may still be taking place. The shore is in places stabilised by saltmarsh but this is less extensive than in northern and western Holes Bay. The poorly sorted sediment indicates a low-energy stable environment. Shoreline change has been as a consequence of reclamation and saltmarsh dieback.

Alongside the embankments the shoreline is littered with debris.

Structures

The frontage is protected with recently constructed rock armour revetments protecting reclaimed land. These are in good condition and in this quiet hydraulic environment provide an entirely adequate protection. Overtopping is limited to 0.1 l/m/s during a 1:10 year storm. Since the road is more than 10m from the crest of the revetment this discharge is acceptable.

Land use and vulnerability

Most of the frontage is formed by the A350, built on land reclaimed in 1983. This is separated from the sea wall by a footpath and an area of wasteland of varying width. The roadway is sufficiently high above maximum water levels not to be at risk from flooding and to prevent flood water from reaching the lower-lying land behind. The southern part of the unit is occupied by industrial properties, on low-lying land. Rock revetments prevent the risk from flooding or erosion.

Environmental sensitivity

The shelter of Holes Bay, and the safe moorings it provides, attract a large number of boat owners. A footpath runs along the edge of the reclamation but the littered foreshore and wasteland between the path and the road does not make this shore of Holes Bay a particularly attractive amenity.

All of Holes Bay is ecologically very sensitive (Dyrynda, 1983) and the saltmarshes are biological SSSI's. The environment in this Unit has been much affected by land reclamation as well as by the spread and subsequent decline of *spartina* saltmarsh. Although the invertebrate fauna is relatively impoverished the mudflats marshes and surrounding reclaimed land support large numbers of intertidal birds (Gray, 1985).

In the past discharge of industrial effluent near the south of the Unit has contaminated sediments on the bed of Holes Bay.

Consequences of "do-nothing"

The shoreline environment of this unit will remain stable though the long-term extent of saltmarsh vegetation is impossible to predict. Thus the existing rock revetments will continue to provide an adequate standard of protection.

4.3.11 *Management Unit 10 Northern Holes Bay*

Grid References SZ 0066 9222 to SY 9926 9214
Length of frontage 2.51km

Drawing 3	PBC map reference number D065/04
Summary Table 4.13	PBC unit reference numbers 4 and 5
Plate 4.14	

Description

Almost completely enclosed by the Holes Bay viaduct which forms the eastern and western boundaries of Unit 10, the northern part of Holes Bay comprises a very sheltered environment.

Hydraulic conditions

The Holes Bay viaduct almost completely separates the northern and southern parts of Holes Bay. In the north, therefore, the hydraulic conditions are very sheltered. Tidal flow in and out of the northern part of Holes Bay is through two open spans in the viaduct. Thus, whilst there is locally faster flow near these spans on the whole flushing is poor and this areas is quite lagoonal in character. Dyryndra (1983) suggests that the northern part of Holes Bay can be considered to be a "tertiary" bay, since it is separated from the remainder of Holes Bay which itself only exchanges with the primary bay (Poole Harbour) through the narrow Back Water Channel.

Coastal Geology

The oldest beds on the PBC frontage are to be found in the northern part of Holes Bay. The Creekmoor Clay forms part of the London Clay formation and underlies the whole frontage, apart from where the land has been reclaimed in the east.

Foreshore

The shore is fronted by a wide area of stable saltings. The saltings in this Unit have probably been less affected by saltmarsh dieback than elsewhere in Poole Harbour because of their sheltered location. To the west the saltings gradually grade into natural terrestrial vegetation whilst further east the abrupt change to rock revetment means that the natural saltmarsh environment in this area is somewhat degraded. Beyond the saltmarsh fringe the floor of the Bay comprises very soft muds.

Structures

The western section of the unit is unprotected by structures. In the east the embankment of the A350 is formed from rubble and earth. It shows a little localised weathering but is stable.

Land use and vulnerability

The western part of the unit comprises farm land and the Upton Park Country Park whilst in the east land has been reclaimed for construction of the A350. In this stable environment the shoreline is not threatened by erosion. The

A350 is sufficiently high not to be affected by surge water levels whilst the low-lying areas of the Upton Park Country Park are not damaged by occasional high water. The land between the rock revetment and the A350 is occupied by a footpath and wasteland.

Culverts flow south from the low-lying residential areas of Oakdale and Creekmoor and discharge into Northern Holes Bay. There is a risk of surge water levels in Holes Bay flooding the properties in Creekmoor via the western culvert. The drainage system is such that flooding from Holes Bay via the eastern culvert is prevented.

Environmental sensitivity

The western side of Management Unit 10 is an attractive coastal environment and Upton Country Park attracts numerous visitors. The *spartina* marshes, including those around Pergin's Island are backed by transitions from brackish marsh to semi-natural grasslands or farmland. The marshes are a biological SSSI.

Towards the east the natural backshore has been claimed and is marked by a limestone embankment with a small area of *spartina* marsh remaining north of the railway. The shore affords pleasant views towards the west but is littered with debris.

The mudflats, marshes and surrounding land support large numbers of intertidal birds.

The poor flushing characteristics, combined with discharges from sewage treatment works at Creekmoor have given rise to particular concerns about the levels of pollution in this northern area of the Bay (Dyrynda, 1983, Gray, 1985).

Consequences of "do nothing"

In the stable environment of Unit 10, changes in the shoreline environment or decay of the coastal structures is not to be expected within at least ten years. The saltmarsh vegetation in this sheltered environment is stable.

Surge water levels in Holes Bay are likely to cause flooding in the low-lying residential areas to the north via a culvert which discharges into the bay.

4.3.12 *Management Unit 11 Western Holes Bay*

Grid References SY 9926 9214 to SZ 0054 9070
Length of frontage 2.56km

Drawing 3	PBC map reference number D065/04
Summary Table 4.14	PBC unit reference numbers 5 to 9
Plate 4.15	

Description

The western side of Holes Bay comprises a saltmarsh shore which has, to a varying extent, been developed for housing and industrial purposes. Management Unit 11 is bound to the north by Holes Bay viaduct and in the south by the relatively deep water quay on the Black Water Channel to the now disused Poole Power Station. Interaction with adjacent Units is minimal.

Hydraulic Conditions

The Unit is exposed to fetches a maximum of 1.3km in length in a north-easterly direction so wave conditions are mild.

Generally speaking the constriction of the entrance to Holes Bay by construction of the wharves has restricted tidal flows. Silting of upper channels, particularly in Unit 10 since its enclosure, has reduced tidal flows throughout Holes Bay. Tidal flows increase towards the inlet into Holes Bay at the Back Water Channel.

Coastal Geology

The low lying shores of the western side of Holes Bay comprise beds of alluvium laid down by the streams draining into the bay.

Foreshore

The shore generally comprises saltmarsh vegetation which has stabilised the underlying sands, silts and clays. Although in the long term the trend has been for saltmarsh dieback in Holes Bay (Dyrynda, 1983), the shores are, in the medium term at least, stable. Beyond the saltmarsh fringe the floor of the Bay comprises soft muds.

Structures

The land on the western side of Holes Bay which has been developed for housing or industrial purposes is raised slightly above the fringe of saltmarsh by which it is fronted. These properties have not needed any further protection.

The Cobbs Quay comprises the principal structure within the Unit. Saltmarsh adjacent to the walls of the marina has died back or been removed. The presence of vertical walls and vessels passing in the approach channel increases hydraulic activity in this otherwise very sheltered coastal environment

Land use and vulnerability

Moving from the industrial land use in the south of the Unit, most of the remaining frontage is backed by housing estates. These have been built behind areas of saltmarsh and are high enough above surge water levels not to be at risk from flooding. Cobbs Quay extends into the deeper Upton Lake channel yet does not suffer from flood damage.

Shoreline erosion which is normally associated with saltmarsh dieback has not been quantified. It is probably very low and does not threaten properties.

Environmental sensitivity

Although not easily accessible to visitors the saltmarshes in the west of Holes Bay are an attractive natural environment. The marshes are part of the Poole Harbour biological SSSI. This part of the Bay is used for mooring of many small boats and Cobbs Quay is a focus for boat-building and a marina for larger private pleasure craft.

The marshes to the west of Holes Bay show a transition from spartina to other plant communities notably in the middle of the bay and around the promontory immediately south of the railway in an area of residential development (Grey, 1985). As elsewhere in the bay, the mudflats and marsh vegetation are important bird habitats.

Consequences of "do nothing"

Although the extent of saltmarsh vegetation is difficult to predict, the fringe of vegetation protecting most of the properties of Unit 11 can be expected to do so for the next ten years without any further intervention.

4.3.13 Management Unit 12 Hamworthy Quays

Grid References SZ 0054 9070 to SZ 0014 9013
Length of frontage 2.62km

Drawing 2	PBC map reference number DO65/03
Summary Table 4.15	PBC unit reference numbers 30 to 31

Description

Management Unit 12 comprises the various quays, marinas and commercial frontages at Lower Hamworthy. It extends from the northern end of the former power station wharf to the western breakwater of the Lower Hamworthy Yacht Marina.

Construction of the quays means that the foreshore is now submerged in all tidal states.

Hydraulic conditions

The Hamworthy Quays are exposed to fetches of up to 3km towards the south-west - the direction of highest waves. The Quays are exposed to longer fetches from the south and east-south-east Waves from the south to south-east can also be reflected off Poole Town Quay so that quite high waves can develop between the Quays. Regular passage of vessels to the quays, marinas and ferry terminal also induces wave activity.

The channel between Hamworthy Quay and Town Quay has been progressively narrowed by port developments thereby concentrating tidal flows. The approach channel and turning area for the Ferry Terminal have been extensively dredged.

Coastal Geology

Most of the shoreline within the unit has been reclaimed. The underlying beds are alluvium and clay but these are not exposed on the frontage.

Foreshore

The seabed is submerged at all tidal states. Previous studies (HR, EX2160, 1990) report it to include sand, silts and stones. It is worked upon by relatively high current flows close to the quays and has been significantly affected by capital and maintenance dredging.

Structures

The unit is largely fronted by vertical quays constructed from concrete, steel or masonry. The marina breakwater which forms the western end of the Unit consists of rock armour. The condition of these structures has not been assessed in detail for this study though they have been reported to be fair or good.

Land use and vulnerability

The Unit is fronted by land with various commercial uses. Upstream of the swing bridge are boat-yards and the now derelict power station. Poole Ferry Terminal and other PHC quays occupy much of the Poole Harbour frontage and the western end of the unit comprises a marina. All of this frontage has to a greater or lesser extent been reclaimed. PHC (1982) describes the progress of reclamation at Hamworthy.

Although the quay walls are relatively high (2.4 to 2.5m ODN) these exposed structures do occasionally suffer overtopping. However, PHC state that overtopping discharges are insufficient to significantly disrupt port activities.

Ecological sensitivity

Public access to the Hamworthy Quays is restricted as they are used for commercial purposes. The visual aspect of the site from the swing bridge and for the many passing pleasure craft should be a consideration in future development plans. Where there former power station is now being demolished there are opportunities for environmental enhancement on this degraded industrial site.

In terms of environmental impact, the mobile sandy bed is actively worked by tidal currents. Being submerged it provides little intertidal or supra-tidal habitat.

Consequences of "do nothing"

The Hamworthy Quays are expected to remain stable and to continue to provide an adequate level of protection over the next ten years

4.3.14 Management Unit 13 Lower Hamworthy Marina to Rockley Viaduct

Grid References SZ 0014 9013 to SY 9734 9195
Length of frontage 3.44km

Drawing 4	PBC map reference number DO65/05
Summary Table 4.16	PBC unit reference numbers 1 to 18
Plates 4.16 and 4.17	

Description

Management Unit 13 is bound by the breakwater to Lower Hamworthy Marina (which forms a substantial barrier to littoral transport) in the east, and Rockley Viaduct in the west. Rockley Viaduct forms the entrance to the partially enclosed Lytchett Bay.

Although subdivided by groynes, jetties and marina walls Unit 13 comprises a continuous littoral regime in which adjacent sections of beach interact.

Hydraulic Conditions

The Arne Peninsula limits fetches to the south-west to as little as 700m towards the centre of the Unit. However, towards the west end of the Unit the fetch in a south-westerly direction is up to 4.5km and is about the same length in a southerly direction at the eastern end of the Unit.

Two wave climates A and B (see Figures 2.49 and 2.50) have been computed near the shore of this Unit. Towards the eastern end of the Unit a 1:10 year wave height of 0.75m is predicted whilst at the western end a 1:10 year wave height of 0.59m is predicted.

The Wareham Channel flows parallel to the unit and close inshore between Lake Pier and the Moriconium Quay. Towards the west the narrower channel to Lytchett Bay flows past Rockley Point and through Rockley Viaduct, but tidal current flows are driven by a much lesser tidal volume.

Major capital dredging has not been carried out upstream of Hamworthy Marina though the entrance to the Royal Marines slipway has been dredged.

Coastal Geology

The natural shoreline geology alternates between alluvium and the Oakdale Clay of the Poole Formation. However, these natural deposits have been greatly disturbed by china clay workings at Rockley Sands and to a lesser extent by the reclamation at Hamworthy. The Oakdale Clay is visible in the cliffs near Rockley Point. Further east where it outcrops on the frontage at Lake, it is not exposed.

Foreshore

The Unit is characterised by sandy beaches. At Rockley Sands these display a natural profile, in places backed by a fairly healthy shingle crest. At Rockley Point a sandy spit has been formed adjacent to the channel leading into

Lytchett Bay. A sheltered pocket of muddy sand, strewn with debris, is found between Rockley Point and the railway viaduct.

The beaches on the central part of the Unit tend to be flatter, whilst to the east they have been nourished with sand, so that at the recreation ground a sand beach lies above a more muddy bed. The sand beach here has been observed to be quite mobile, significantly lowering after storm action and subsequently recovering. Some shingle is to be found on the upper beach and in the poorly sorted sediments near the low water mark.

Calculations of potential longshore sand transport driven by wave climates A and B indicate that at the western end of the Unit the potential net longshore drift rate is $1000m^3$/year towards the east. At the eastern end of the unit drift is also towards the east but the calculated potential drift is about $3600m^3$/year. These drift directions are confirmed by a build-up of sand or gravel on the western side of some groynes and other cross-shore structures on the frontage. A continuous easterly longshore drift regime is an important characteristic of Unit 13.

Structures

A variety of coast protection structures have been built, ranging from the concrete walls at Hamworthy recreation ground, to the privately constructed walls on the Lake frontage. The condition of these structures varies. The walls at the eastern side of Hamworthy recreation ground are rather old and have recently had to be repaired. The private seawalls on the Lake frontage vary widely in condition with some recently rebuilt or upgraded whilst others are poor. Most of Rockley Sands is unprotected and gradually eroding whilst towards Rockley Point the cliff toe is protected with gabion baskets. The lifetime of this type of protection is limited.

Sub-section 2 is protected by a very low wall which regularly overtops. Further west the private seawalls provide adequate protection. A typical seawall in sub-section 14 has been calculated to limit overtopping to 0.03l/s/m during a 1:50 year storm.

Land use and vulnerability

Towards the eastern end of the Unit, low-lying land has been reclaimed to form the Hamworthy recreation ground. Beach huts have been sited along the shore and the beach is a popular amenity. The recreation ground is poorly drained and is as low as 1.5 m ODN in places. During the highest recorded tide flood, water reached as far as the cafe and road at the back of the recreation ground.

Moving westwards, private houses and marinas and the Royal Marines Amphibious Training Unit back directly onto the shore. The westernmost 1.5km of the Unit is formed by Rockley Sands. Formerly worked for china clay, the east of this area is now a local nature reserve, whilst in the west it is occupied by a caravan site and a marina at Rockley Point.

Study of historic surveys shows that the shoreline of the Rockley end of the Unit has a natural tendency to erosion whilst towards the Hamworthy end the shoreline has been more stable. Some shoreline change (of the order of 10m in 100 years) is apparent in sub-section 13/1 and 13/2, though there is no dominant trend of advance or retreat. A stream used to discharge into the harbour close to the boundary between 13/2 and 13/3. The bed and mouth and this stream has now been claimed as recreation ground and the shoreline straightened somewhat. In sub-section 13/4 the High Water Mark retreated by up to 14m between 1900 and 1953 but has been stable since. Natural shoreline change in sub-section 13/7 and 13/8 is difficult to interpret from historic maps because of the impact of china clay workings east of Rockley have had on the shoreline. Natural retreat can be studied in sub-section 13/10 and is demonstrated in the table below.

Chainage move west along sub-section 13/10	Retreat between 1888 and 1954 surveys (m)	
	MHW	Cliff edge
0	10	16
100	7	8
200	10	9
300	7	5

Where unprotected, the low cliffs at Rockley Sands continue to erode. Tension cracks observed about 20m landward of the cliff top by PBC staff suggest potential for slip failure, and are evidence of this continuing erosive trend.

Environmental sensitivity

The beaches between Hamworthy and Rockley are a pleasant and popular amenity. At Hamworthy Park in the east, the beach has been nourished and a promenade and beach huts constructed. In the west, Rockley Sands holiday camp is a large caravan and chalet complex. Slipways, piers and boat-yards towards the centre of the Unit are for public and private use. Angling is a popular activity and ragworm is dug for bait along the shoreline.

From Rockley Point extending south, natural cliffs of tertiary sands and clays are of geological and ecological interest. Where eroding, fresh exposures of the deposits are visible above a natural and unimpaired beach - a rare example of active geomorphological processes within the Harbour. The cliffs and heath vegetation in this area have been designated as a Nature Reserve in recognition of their ecological importance. Ham Common is designated as a biological and geological SSSI, which also includes the Lake Geological Conservation Review site.

Although the gravel shores are rich in polychaete worms they are not extensively exploited by intertidal - feeding birds at least in the eastern part of this Unit (Gray, 1985).

Consequences of "do nothing"

The gabion protection near Rockley Point will continue to deteriorate and within a few years will cease to function as a coast protection. Small-scale cliff degradation near the Rockley caravan site is to be expected within ten years. The cliff top where a line of caravans is now sited will be threatened by erosion. However, as caravans are a mobile asset erosion of the cliff top cannot be assumed to result in a loss of property. Erosion of the cliffs near Rockley Point will supply sediments to the beaches at the cliff toe and downdrift. It will also expose cliffs which are recognised for their geological interest.

Overtopping on the Hamworthy frontage will continue to flood the recreation ground on an occasional basis.

4.3.15 *Management Unit 14 East Lytchett Bay*

Grid References SY 9734 9195 to SY 9820 9220
Length of frontage 1.73km

Drawing 4	PBC map reference number D065/05
Summary Table 4.17	Not included in PBC unit references
Plate 4.18	

Description

Management Unit 14 comprises that section of the PBC frontage which lies within Lytchett Bay. Lytchett Bay has been partly enclosed by Rockley Viaduct which now forms the boundary between the semi-lagoonal Lytchett Bay Unit 14 and the more exposed Unit 13.

Although this Unit extends only along the PBC frontage the shore of Lytchett Bay has similar characteristics throughout, as does the hydraulic regime.

Hydraulic conditions

The enclosed bay limits fetches to a maximum of 1.6km (at the northern end of the unit and in a westerly direction). The open span of Rockley Viaduct is sufficiently narrow to prevent any significant wave activity from Poole Harbour from penetrating Lytchett Bay. Wave conditions are therefore very mild.

Similarly to other partially enclosed areas of Poole Harbour, the viaduct across the mouth of Lytchett Bay has reduced tidal flows within the Bay. Only at the narrow opening of the viaduct have tidal flows been increased. However, the main tidal channel is sufficiently far from the shore for currents near the shore to be minimal. Spartina dieback has, in general, resulted in a shallowing in the upper reaches of tidal channels within Poole Harbour.

Coastal Geology

The low-lying shore comprises beds of alluvium laid down by the streams draining into the bay, principally the Rock Lea River. Slightly higher ground comprises the Oakdale Clay of the Poole Formation and is closer to the shore in the south of the Unit.

Foreshore

In the lower part of Lytchett Bay a thin layer of sand overlies mud and there is a narrow sand and shingle beach at the shore. In the upper bay the sediment is muddy and stabilised by vegetation. The existing foreshore is generally stable as a consequence of sheltered hydraulic conditions and modest tidal flows. Nonetheless in the south-east of the bay saltmarsh dieback does seem to prevail whilst in the upper bay the saltmarsh is healthy. The intertidal flats are principally sandy muds and muds and are firmer than on the west shore (Gray, 1985).

No structures have been constructed for beach control purposes but Rockley Viaduct has had an impact on coastal processes by sheltering the southern part of the unit and forcing currents offshore, concentrating them in the narrow entrance to the Bay.

Structures

Within this stable shoreline environment it has not been necessary to construct "hard" coastal structures. The Turlin Moor reclamation is surrounded by a grassed earth embankment but is separated from the shoreline by a sufficient width of vegetation to warrant no additional protection.

Land use and vulnerability

A housing estate has been constructed on higher ground behind the natural saltmarsh shoreline at Turlin Moor.

Because of the stability of the shoreline, the Unit is not at risk from erosion. Where situated on low-lying land, existing embankments provide adequate protection from flooding.

Although backed by low bluffs, the natural shoreline comprises saltmarsh vegetation, suggesting that gentle post-glacial cliff erosion in the past was reversed by colonisation with saltmarsh vegetation. May (1969) suggests shoreline accretion, particularly within the northern part of the unit, between 6000BP and 1807AD. Subsequently the human activity has dominated shoreline changes with reclamation of Turlin Moor and construction of Rockley Viaduct. Saltmarsh dieback and sedimentation within the harbour, since partial enclosure do not appear to have markedly changed the shoreline.

Environmental sensitivity

The playing fields and sandy parts of the shoreline attract local visitors. Around the edge of the playing fields are fine views of the natural marshlands environment. Small craft are moored along the eastern shore.

The marshes display a natural transition from brackish saltmarsh to wet heathland and mixtures of plant species from both biotopes occur (Gray, 1985). These marshlands are habitats for important populations of birds and are part of the Poole Harbour biological SSSI.

Consequences of "do nothing"

The frontage of Unit 14 is currently effectively managed with a "do nothing" strategy and there will be no adverse effect if this strategy is continued in future.

5 Management options and recommendations for future action

5.1 Introduction

In this chapter an assessment is made of the various alternatives for protecting the frontages within the Management Units identified in Chapter 4.

For some Management Units no serious problems have been identified and the course of action which is appropriate is continued maintenance of the defences, together with an appropriate level of monitoring. For such Management Units where the "minimum maintenance approach" is appropriate a brief description of monitoring requirements is given here, under the title "Management Plan".

In a number of Management Units, problems have been identified which require remedial action. For these Units we have carried out an "Assessment of Defence Alternatives" before going on to the "Management Plan". The assessment has been carried out using a multi-criteria analysis. This should not be confused with a cost-benefit analysis which is carried as part of an actual design procedure.

A multi-criteria analysis is a technique which is used to assess the effectiveness of various options under various "performance criteria". These criteria assess the relative importance of various aspects of a coastal protection scheme, such as standard of protection, cost, impact on the environment, benefit to amenity etc. In a multi-criteria analysis these aspects are synthesized to provide reasoned decisions for selecting the best option(s) which provide the greatest overall benefit. This technique is essentially a qualitative one, which relies on feedback from the client and other interested parties with regards to the performance of value of the various criteria. Indeed quite often feedback from the client may result in additional criteria being considered and on repeating the analysis it is possible to produce an alternative assessment (perhaps with a different outcome).

In the multi-criteria analysis, various aspects of a scheme are compared against each other and ones which are deemed to be the most important have this reflected by using a higher weighting in the overall analysis.

The procedure used is best illustrated by considering as an example Unit 1 (sub-sections 1-9) at the east end of the Poole frontage within Poole Bay. Here the chosen criteria and their weighting are illustrated in Section 5.2.1 under the title "Comparative weighting of criteria". On this particular section of coast the most important aspect (Criterion A) is the protection of the coast, this being the prime function of the existing or future coastal defence works. Other criteria which rank high in importance are capital cost (Criterion B) and maintenance cost (Criterion C). Other important criteria include the impact of the defences on beach levels in front of them (Criteria D). For example, there is little point in constructing a massive seawall, which gives total protection, but which results in a dramatic loss of beach and hence reduces amenity and/or the future safety of the wall. Another important criterion (Criterion E) in Unit 1 is the impact a coast defence would have on adjacent beaches. A massive groyne system or system of offshore breakwaters may, for example, result in a local improvement of beach levels, thus giving additional protection to any existing coastal defences. Such a system might be useful in prolonging the life of old seawalls but may exacerbate the situation elsewhere. The impact of such a system on the adjacent beaches needs to be carefully judged, since the improvement of beach conditions by such structures may merely result in the problem being shifted to adjacent areas by changing the patterns of littoral drift. This criterion is particularly important on open stretches of coast subject to relatively high rates of transport. Another criterion which has relevance to an amenity beach (Criterion F) is that of public acceptance. Finally there is the aspect of environmental impact (Criterion G), which may or may not be relevant to open coast beaches heavily used by the public but has considerable importance in muddy semi-sheltered areas which are usually important habitats for flora and fauna.

In order to obtain a comparative assessment of these criteria their relative importance is determined by giving them a weighting as follows:

The criteria are set out in rows and columns and if the criterion in a row is:

- more importance than the criterion in the column then the score = 2
- equally important than the criterion in the column then the score = 1
- less important than the criterion in the column then the score = 0

Having compared all the criteria against each other in this way the numbers in each row are totalled (horizontally) to arrive at a ranking of importance by which to judge the various coastal defence options.

For the east end of Unit 1 the coastal defence options which have been considered are:

Option 1 Repair existing seawall and groynes.

Option 2 Upgrade existing seawall to give a higher standard of protection against backshore erosion and repair groynes.

Option 3 Renourish the beach and replace groynes by more effective ones (ie ones able to hold the nourishment material effectively).

Option 4 Protect the beach and hence prolong the life of the existing seawall by means of a series of detached offshore breakwaters.

Option 5 Remove existing groynes and install beach drainage system.

Various scoring systems were examined during the study, but for the sake of simplicity a system with positive integer scores on a scale from 0 to 10 was finally adopted. The disadvantage of this system is that a relatively small difference in scores can reflect a significant difference in option suitability. The method does, however give a clearly understood means of ranking scheme options. The scores of each option with respect to each criterion (A to G) are then multiplied by the relative importance (or weight) of each criterion to give a number which reflects the weighted performance of each option. This procedure is illustrated for the east end of Unit 1 by the table with the heading "Options".

Having carried out this assessment for the various units, a Management Plan is then drawn up. A summary of the Management Plan is shown diagrammatically in Figure 5.1 which indicates priorities of future works.

5.2 Unit 1 Sub-sections 1 - 9 Poole Bay - cliffed frontage

5.2.1 Assessment of defence alternatives

Problem : Low beach levels give rise to wave overtopping and possibility of cliff toe erosion.

Constraints : Beach is continuous hence coastal defences may have a knock-on effect. Sand beach has a high amenity value. Foreshore is exposed to wave activity and has a low ecological sensitivity. Wave induced longshore drift is high.

Comparative weighting of criteria

Criterion		A	B	C	D	E	F	G	Total
A.	Protection of coast	-	2	2	2	2	2	2	12
B.	Capital cost	0	-	1	1	2	1	2	7
C.	Maintenance cost	0	1	-	1	2	1	2	7
D.	Impact on beach levels	0	1	1	-	1	2	2	7
E.	Impact on adjacent beaches	0	0	0	1	-	1	2	4
F.	Public acceptance	0	1	1	0	1	-	1	4
G.	Low environmental impact	0	0	0	0	0	1	-	1

Defence alternatives:

Option	Description
1	Repair existing seawall and groynes
2	Upgrade existing seawall and repair groynes
3	Renourish the beach and replace groynes
4	Protect beach by offshore breakwater
5	Remove existing groynes and install beach drainage system

Comparative assessment of defence alternatives

Criterion		Wt	1	2	3	4	5
A.	Protection of coast	12	8	8	7	6	5
B.	Capital cost	7	6	3	6	3	6
C.	Maintenance	7	8	7	6	6	5
D.	Impact on beach levels	7	3	4	8	6	8
E.	Impact of adjacent beaches	4	5	5	8	5	8
F.	Public acceptance	4	7	6	8	3	9
G.	Low environmental impact	1	5	5	6	6	6
			268	243	294	215	267

5.2.2 *Management plan*

At present the frontage is adequately protected by the existing seawall and groynes. However, there is a localised damage to the promenade decking and the timber groynes show signs of wear and tear. The costs of maintaining an adequate defence standard will rise in the future should beach levels fall.

The frontage has a shortfall in sand supply, with groynes on the Bournemouth frontage being more substantial and better stocked with beach material. There is also a deterioration in beach conditions at the western end of the frontage and the problem of falling beach levels is spreading east. It is likely that at some stage in the future beach levels along this frontage (sub-sections 1-9) will also fall.

A comparative analysis of five defence options indicates that renourishment, together with groyne replacement is the best option. Beach nourishment has the advantage of providing a supply of material to downdrift beaches also, thereby providing additional protection for adjacent beaches.

Construction of offshore breakwaters can be ruled out on a number of counts, including high cost and adverse impact to adjacent beaches (material will tend to migrate into the lee of such structures). Upgrading the existing seawall and repairing the existing timber groynes is not a favourable solution for the long term because of high cost and because it does not deal with the underlying problem of beach loss. Repairing the existing sea wall and groynes is a similar option and one that may be acceptable in the short term, but again does not address the problem of beach lowering. Removal of the existing groynes and installation of a beach drainage system is attractive from an amenity viewpoint, but the standard of protection it will provide is uncertain.

It is recommended that for the present time, maintenance repairs to the existing seawall and groynes should be continued. The existing programme of beach monitoring at two monthly intervals should be continued in order to:

(i) identify rates of beach loss;
(ii) identify erosion hot spots;
(iii) determine the effectiveness of the existing groyne system.

When sufficient data has been collected to identify beach trends, plans should be drawn up for nourishing the coastline and replacing the existing groynes. It may be necessary to carry out additional studies in the meantime, to examine:

(i) nourishment efficiency;
(ii) behaviour of timber as compared with rock groyne designs.

5.3 Unit 1 Sub-sections 10 - 12 Poole Bay - Sandbanks peninsula

5.3.1 *Assessment of defence alternatives*

Problem : Threat of breaching at neck of Sandbanks peninsula. Seawalls protect much of the frontage but there are also stretches of duned backshore at the western end of the frontage which are at risk of erosion. Defences are in poor condition.

Constraints : Beach is continuous hence coastal defences have a strong knock-on effect. Sand beach has a very high amenity value. Wide sand foreshore has a low ecological sensitivity. Area relies on mobile nearshore banks for protection against water attack; at present their sheltering effect is considerable. Wave induced alongshore drift is moderate to high. There is evidence of recent beach erosion which is probably due to the deepening of the East Looe channel.

Comparative weighting of criteria

Criterion		A	B	C	D	E	F	G	Total
A.	Protection of spit	-	2	2	2	2	2	2	12
B.	Capital cost	0	-	1	1	1	1	2	6
C.	Maintenance cost	0	1	-	1	1	1	2	6
D.	Impact on beach levels	0	1	1	-	1	1	2	6
E.	Impact on adjacent beaches	0	1	1	1	-	2	2	7
F.	Public amenity	0	1	1	1	0	-	1	4
G.	Low environmental impact	0	0	0	0	0	1	-	1

Defence alternatives:

Option	Description
1	Upgrade existing sea defences and infill gaps
2	Provide rock protection in gaps
3	Renourish the beach and re-establish groynes
4	Protect spit neck by offshore breakwaters
5	Beach nourishment and beach drainage

Comparative assessment of defence alternatives

Criterion		Weight	1	2	3	4	5
A.	Protection of spit	12	8	7	7	6	6
B.	Low capital cost	6	7	8	8	2	8
C.	Low maintenance cost	6	8	7	7	6	6
D.	Low impact on beach levels	6	4	5	8	6	8
E.	Low impact on adjacent beaches	7	5	6	8	3	8
F.	Public amenity	4	6	5	8	3	8
G.	Low environmental impact	1	5	5	5	5	5
			274	271	315	194	297

5.3.2 Management plan

The wide sand beach in this area is showing signs of erosion with falling beach levels during the winter of 1993/4 having shown no signs of relenting despite the very calm weather during the summer of 1994. There is a threat of breaching taking place through the narrow neck of the Sandbanks peninsula. Historic evidence shows that construction of groynes in the past had a dramatic impact on the beach considerably increasing the width of the foreshore. The groynes have subsequently fallen into disrepair and are now an amenity hazard.

The coastal defences are not continuous and upgrading these would involve major capital investment. Protection by offshore breakwaters can be discounted because of high capital cost, bad impact on beach amenity and uncertain performance. An alternative would be to infill the gaps in the existing defences, by relatively low cost rock revetment protection. However, such a course of action could seriously damage the few remaining sand dunes at the west end of the frontage and may be unacceptable from an environmental viewpoint. An alternative is to renourish and re-establish groynes in this area also, and this is our preferred option.

Installation of a beach drainage system, in association with beach nourishment, will preserve the attractive, unobstructed character of the beach. Relatively sheltered wave conditions, low tidal flows and low tidal range are suitable for the system. However, beach drainage is as yet unproven on UK beaches and cannot be recommended without further study or pilot scheme.

It is recommended that beach monitoring should be continued at two monthly intervals as a continuation of the monitoring of the frontage to the east. Profiling should include transects through dunes sections so as to monitor pre and post-storm behaviour of the backshore.

Because of the deteriorating condition of the beaches, plans should now begin to be drawn up for improving the protection provided to the backshore defences. The most favoured option is beach renourishment or other forms of beach improvement, since this provides a natural protection against the sea, and will prolong the life of existing seawalls which only give a low standard of protection.

5.4 Unit 1 Sub-sections 13 - 16 Poole Bay - Sandbanks head

5.4.1 Assessment of defence alternatives

Problems : Low and rapidly falling beach levels at the head of the Sandbanks peninsula. Complex interaction of waves and tidal currents plus changing bathymetry make beaches unstable. Sea defences reflect waves and destabilise beaches. Threat of seawall undermining at west end of frontage.

Constraints : Deep water and rapid tidal flows close inshore make it impractical to hold beach levels by detached breakwaters. Traditional groynes may be ineffective in retaining beach material. Access difficult hence amenity value is reduced. Ecological sensitivity probably low. Nourishment material may be dispersed into harbour or to adjacent coast via the East Looe Channel, which is very close to the shoreline at this point.

Comparative weighting of criteria

Criterion	A	B	C	D	E	F	G	Total
A. Protection of backshore	-	2	2	1	2	2	2	11
B. Capital cost	0	-	1	0	2	2	2	7
C. Maintenance cost	0	1	-	0	2	2	2	7
D. Impact on beach levels	1	2	2	-	2	2	2	11
E. Impact on adjacent beaches	0	0	0	1	-	1	2	4
F. Public amenity	0	0	0	0	1	-	1	2
G. Environmental impact	0	0	0	0	0	1	-	1

Defence alternatives:

Option Defence

1 Upgrade existing defences
2 Provide rock berm in front of existing defences
4 Construct rock groynes and nourish beach
5 Reconstruct existing defences

Comparative assessment of defence alternatives

Criterion	Weight	1	2	3	4
A. Protection of Backshore	11	8	7	6	9
B. Low capital cost	7	6	7	5	3
C. Low maintenance cost	7	6	6	5	8
D. Low impact on beach levels	11	3	4	7	3
E. Low impact on adjacent beaches	3	5	5	7	5
F. Public amenity	2	4	4	8	5
G. Low environmental impact	1	5	5	6	5
		233	240	256	239

5.4.2 Management plan

The narrow sand beach is very volatile, with beach levels varying by up to 1m between surveys. The instability is the result of a complicated nearshore bathymetry with major shifts in sand banks and nearshore channels taking place in this area. The presence of deep water and rapid tidal currents in the East Looe Channel removes sand in suspension from even the upper part of the beach. This is leading to beach instability and permanent sand losses.

Recently beach levels have fallen so dramatically that there is now a danger of the seawalls becoming undermined during storm drawdown.

A comparative analysis of four possible defence options indicates that provision of a rock berm at the toe of the existing seawall to reduce beach scour plus the construction of rock groynes plus sand nourishment are very much more favourable than any "hard defence" options. Upgrading or reconstructing sea defences may only be expected to provide backshore protection in the short term.

It is recommended that beach monitoring at two-monthly intervals should continue. In an ideal world the effect of the present emergency rock protection would be assessed before further works are put in hand. However, the last four years have seen considerable deepening of the East Looe Channel and the beach in this area is now virtually the "side slope" of the Channel. Rapid beach losses which occurred following the removal of several old groynes have not been made good following the subsequent reconstruction of a massive rock groyne of the "fish tail" type which was built east of the Haven Hotel. It is recommended that plans for additional protection to this frontage should be put immediately into action. It is possible that further lowering will occur, and that the emergency rock protection at the toe of the sheet steel wall will slump further or may completely disappear into the beach. Under these conditions undermining of the seawalls within the next one or two years could occur.

It is also recommended that a hydrographic survey as part of design studies for further protection be put in hand. Our recommendation is that additional rock groynes should be considered, as a means of deflecting the rapid tidal currents away from the beach.

It may also be necessary to examine other ways of stabilising the frontage possibly by means of detailed modelling and cost benefit studies. The existing tidal flow model will be useful in achieving an optimum design since our study has shown that currents rather than wave action are the primary cause of beach destabilisation.

5.5 Unit 2 Haven Hotel to North Haven Point

5.5.1 Assessment of defence alternatives

Problem : Privately constructed walls of varying condition protect the head of Sandbanks spit. Wave action and strong tidal currents mobilise beach material very easily and as a result beaches are submerged at virtually all tidal states. Channel migration could lead to sudden bed level changes and possible wall undermining.

Constraints : Private frontage which has little public usage and low ecological sensitivity. There may be local opposition to any large scale defences, particularly if these affect the frontager's boat usage. Seawall reconstruction or repair may be made difficult by the exposure and degree of submergence of the foot of the defences. Wave and current induced longshore transport is low.

Comparative weighting of criteria

Criterion		A	B	C	D	E	F	G	Total
A.	Protection of backshore	-	2	2	1	2	2	2	11
B.	Capital cost	0	-	1	1	2	1	2	7
C.	Maintenance cost	0	1	-	1	2	1	2	7
D.	Impact on beach levels	1	1	1	-	2	2	2	9
E.	Impact on adjacent beaches	0	0	0	0	-	1	2	3
F.	Disturbance to boat use	0	1	1	0	1	-	1	4
G.	Environmental impact	0	0	0	0	0	1	-	1

Defence alternatives:

Option	Description
1	Repair existing walls/footings
2	Protect existing walls with rock toe
3	Hold beach levels by groynes
4	Reconstruct seawalls

Comparative assessment of defence alternatives

Criterion		Weight	1	2	3	4
A.	Protection of backshore	11	8	6	5	8
B.	Low capital cost	7	4	6	5	2
C.	Low maintenance cost	7	8	7	5	9
D.	Low impact on beach levels	9	5	6	6	5
E.	Low impact on adjacent beaches	3	5	5	5	4
F.	Low disturbance to boat use	4	8	6	4	3
G.	Low environmental impact	1	5	5	5	5
			269	255	215	239

5.5.2 Management plan

The condition of the defences along this frontage is very variable, the seawalls having been privately constructed at various times in the past.

Wave overtopping poses a nuisance rather than a serious threat to the properties as the backshore generally rises landward from the seawalls.

A comparative assessment of defence options suggests that repairing the existing seawalls as and when necessary is the most economical and probably an acceptable solution. Addition of concrete footings may be necessary in certain areas also. Large scale reconstruction of the defences to a common standard of protection cannot be justified on the grounds of very high cost set against a relatively low return, as regards increased protection. Alternative options involving alteration to the toe at the walls (eg rock toe protection) or groyning of the foreshore probably have little economic justification and would also seriously disrupt water front usage.

The stability of the seawalls is to a large degree dependent on the foreshore levels being maintained. Any migration of tidal channels could pose a threat to the stability of the defences. It is thus recommended that the hydrographic survey recommended for the western end of Unit 1 should be extended to encompass Unit 2 also. Should there be any evidence of landward migration of tidal channels then the hydrographic surveys may need to be repeated, say after a year, to check that the sea defences are not endangered.

In view of the varying condition of the defences an inspection of their structural integrity needs to be carried out on an annual basis.

5.6 Unit 3 North Haven Point to Whitley Lake

5.6.1 Management plan

The frontage is protected by privately constructed seawalls and the foreshore is also intersected by groynes in several places. While the condition of these defences is variable is seems unlikely that anything but localised damage to the defences is likely to occur in the space of the next ten years. Wave exposure from the north-east is relatively mild and properties are not threatened by serious overtopping or backshore erosion.

No capital works are presently envisaged. However given the variable condition and character of the defences it is recommended that an annual inspection should be made of each of the sub-units identified by Poole B. C. A number of groynes and marinas also provide reference points for photographic evidence of beach build up / erosion at these locations.

5.7 Unit 4 Whitley Lake

5.7.1 Assessment of defence alternatives

Problem : Seawall is low and the Banks Road and Shore Road are occasionally overtopped, preventing vehicular access.

Constraints : Seawall is old and its wholesale reconstruction will be costly. Reconstruction could also impact on the saltmarsh which is undergoing biological degeneration. Frontage attracts tourists and wildlife alike. Foreshore is important for craft mooring, sailboarding and other water recreations. Area has a high ecological sensitivity. Sand nourishment may smother saltmarsh. Littoral drift in this area is low and to centre of bay.

Comparative weighting of criteria

Criterion		A	B	C	D	E	F	G	Total
A	Protection against flooding	-	2	2	2	2	2	2	12
B	Capital cost	0	-	1	2	2	1	1	7
C	Maintenance cost	0	1	-	2	2	1	1	7
D	Impact on foreshore	0	0	0	-	2	2	1	5
E	Impact on adjacent coast	0	0	0	0	-	1	1	2
F	Public acceptance	0	1	1	0	1	-	1	4
G	Environmental impact	0	1	1	1	1	1	-	5

Defence alternatives:

Option	Description
1	Repair and raise existing wall
2	Reconstruct existing wall
3	Nourish backshore
4	Add rock berm to tow of wall

Comparative assessment of defence alternatives

Criterion		Weight	1	2	3	4
A.	Protection against flooding	12	9	9	7	7
B.	Low capital cost	7	5	2	5	5
C.	Low maintenance cost	7	7	9	6	8
D.	Low impact on foreshore	5	5	4	7	6
E.	Low impact on adjacent coast	2	5	5	7	5
F.	Public acceptance	4	9	8	9	6
G.	Low environmental impact	5	5	4	3	5
			288	267	261	264

5.7.2 Management plan

The road linking the Sandbanks peninsula with the rest of the Borough will continue to experience occasional flooding. To date such flooding has not posed a serious threat to properties but has caused major disruption for vehicular access to the peninsula.

Older sections of the seawall are now in poor condition and collapse of these walls would constitute a serious threat of erosion at the narrow neck of the peninsula.

While beach levels are low at the southern end of this frontage they appear to be stable, hence serious overtopping is only likely to occur during a combination of high tidal levels and extreme westerly or southwesterly winds.

Examination of the various options for preventing the onset of serious overtopping indicates that the situation can be remedied effectively in a number of ways.

Renourishment of the backshore is attractive at first sight on the grounds of improved public amenity, but against this there is the potentially damaging impact on the small area of surviving saltmarsh in the centre of the bay, which could become smothered by the nourishment material either as a result of wave action or by strong winds. This option is probably least effective under extreme tidal conditions as a high beach cannot be guaranteed following a series of severe storms, for example.

The addition of a rock berm in front of the wall would prolong its life by reducing the hydraulic loads imposed upon it during high tidal levels and storm wave action. However, there would be no improvement in public amenity and no environmental "gain".

A scheme involving a major reconstruction of the existing seawall scores highly in terms of affording a high level of protection for many years to come as well as low maintenance cost (the climate in the area is not particularly severe). However reconstruction will inevitably cause disruption to the upper foreshore and may also result in an adverse impact on the remaining saltmarsh area.

The most attractive solution of repairing and raising the existing seawall scores highly on the grounds of providing a high level of protection in future years at relatively low capital cost. It is also unlikely to have any detrimental impact with the regard to amenity and environmental issues. Parts of the existing seawall have in fact been refurbished during the course of the present study. The need for raising the seawall crest in areas of overtopping should however be given consideration also.

It is recommended that the foreshore should be monitored by annual visual inspections, the incidence of overtopping monitored and the disruption of traffic assessed. A full structural survey of the existing wall should also be made, if this has not already been carried out.

5.8 Unit 5 East Dorset Sailing Club to Salterns Marina

5.8.1 Assessment of defence alternatives

Problem : Steep cliffs of Branksome Sand were eroding before the construction of a promenade wall at their base. The concrete promenade deck is fronted by plastic coated wire gabions which are rapidly reaching the end of their useful life. The promenade level is also too low to prevent overtopping. There is thus a danger to pedestrians and to the stability of the cliff behind if nothing is done soon.

Constraints : North of the promenade the frontage is protected by piecemeal privately constructed defences. The presence of a sewer outfall at their toe poses constraints with regard to coast protection. At present coastal defences should be concentrated on the promenade, but may need to be extended northwards in future years. The flat sandy/muddy foreshore is used extensively by bait diggers and the area is popular with small craft users. Defences which intrude significantly onto the foreshore should avoided. Littoral drift in this area is very low.

Comparative weighting of criteria

Criterion		A	B	C	D	E	F	G	Total
A.	Protection of cliff toe	-	2	2	2	2	2	2	12
B.	Capital cost	0	-	1	2	2	1	1	7
C.	Maintenance cost	0	1	-	2	2	1	1	7
D.	Impact on foreshore	0	0	0	1	-	1	1	3
E.	Impact on adjacent coast	0	0	0	1	-	1	1	3
F.	Public acceptance	0	1	1	1	1	-	2	6
G.	Environmental impact	0	1	1	1	1	0	-	4

Defence Alternatives:

Option Description

1 Repair gabions
2 Replace gabions by rock toe
3 Nourish and groyne the foreshore
4 Reconstruct promenade

Comparative assessment of defence alternatives

Criterion		Weight	1	2	3	4
A	Protection of cliff toe	12	3	8	5	9
B	Low capital cost	7	9	7	6	3
C	Low maintenance cost	7	3	7	6	9
D	Low impact on foreshore	3	6	7	8	6
E	Low impact on adjacent coast	2	8	8	7	7
F	Public acceptance	6	4	6	7	7
G	Low environmental impact	4	4	6	5	4
			202	299	251	289

5.8.2 Management plan

The eastern part of Unit 5 has a promenade protected by gabion baskets on its seaward side, and backed by high unstable cliffs. To the west the frontage is protected by private seawalls, some of which are in a poor condition but which nevertheless are considered to give an adequate level of protection for the next ten years. However, the gabions on the promenade frontage are reaching the end of their useful life and defences here need reinstatement in the next one or two years.

The options for protecting the promenade range from reinforcement of the gabions or their replacement, provision of a sand beach as a buffer zone, or total reconstruction of the promenade. A number of other options were also considered and disregarded at the initial stage of studies. Offshore breakwaters for example were considered to be an inappropriate solution and which would seriously affect water based leisure activities in this area.

The option of repairing the gabions has been considered in the multi-criteria assessment because of low cost, but is considered to be the least favourable solution because of high maintenance costs and susceptibility to damage or indeed total collapse during an extreme event.

Renourishment and groyning of the foreshore is at first sight an attractive solution. However, this too, is considered to be liable to give an unacceptably low level of protection during extreme storms.

Reconstructing the promenade with a concrete seawall is considered to be a promising alternative given the general low level of wave activity in the area (as evidenced by the lack of serious erosion infront of the private seawalls at the west end of the frontage). This, however, is an expensive option.

Replacing gabions by a rock toe is the option which provides the best balance between an acceptable level of protection, moderate costs and a relatively low impact on foreshore conditions. Protection by means of rock armouring has already proved to be a good solution to coastal erosion elsewhere in the Borough (eg west of the Baiter, Parkstone Bay, Holes Bay).

It is recommended that the costs of reinstating the seawall and providing rock protection should be examined within the next year with a view to implementing a solution to the problem within the next two years.

5.9 Unit 6 Salterns Marina to Parkstone Yacht Club

5.9.1 Management plan

With the exception of Blue Lagoon the shorefront properties in Unit 6 are protected by privately constructed seawalls. These walls are in varying degrees of repair. They have been upgraded by the frontage owners over the years and are unlikely to suffer sudden collapse. Although wave overtopping will continue to take place on a regular basis the properties are not at risk of flooding because most rear gardens slope upwards from the shoreline. Collapse of the defences is unlikely to take place within the next ten years.

An inspection of the condition of the walls and the foreshore itself should be carried out on an annual basis to ensure that the walls continue to maintain an adequate level of protection against coastal erosion.

The waterfront properties within Blue Lagoon are only lightly protected but are at a relatively low risk to erosion. The bar across the entrance to Blue Lagoon provides shelter against wave activity at all stages of the tide. Because of the poor state of repair of the structures surmounting this bar, the level of protection within Blue Lagoon may fall within the next ten to fifteen years. It is recommended that as well as the annual shoreline inspection a detailed examination of the bar should also be carried out, but on a less frequent basis, say every three years.

5.10 Unit 7 Parkstone Bay and Baiter Park

5.10.1 Management plan

Most of the shoreline within Parkstone Bay and around Baiter Park consists of reclaimed land and the majority of the frontage is now protected by a variety of revetment types (rock, armour, concrete blocks, masonry etc). Some stretches of revetment, particularly the older masonry protection situated within Parkstone Bay, are in need of regular maintenance. However, there is little likelihood of serious damage or collapse taking place within the next ten years. The remaining eroding section of earth bank at Baiter Point has recently been protected.

At the head of Parkstone Bay there is a short stretch of unprotected shoreline. Due to the shelter against wave action and relatively high foreshore levels this stretch is naturally vegetated and stable. An increase in still water levels due to extremely high surge levels could result in some flooding of the backshore making the footpaths a temporarily unusable. However there is presently no justification for "upgrading" the coast defences in this area.

An inspection of the condition of the various defences should be carried out to ensure that they continue to maintain an adequate level of protection against coastal erosion and flooding. Incidents of overtopping should be recorded in order to verify the standard of protection afforded by the defences. The remaining unprotected frontages should be carefully monitored to detect evidence of erosion.

5.11 Unit 8 Town Quays

5.11.1 Management Plan

The Town Quay which is situation in the historic centre of Poole is an integral part of the charm of the area. The frontage has been reclaimed over a number of years and quay walls have been constructed to provide access to the quayside at all stages of the tide. This frontage is the responsibility of the Poole Harbour Commissioners, as a commercial port. Overtopping at Town Quay is a regular occurrence, as a result of the low freeboard and deep water location of these walls at high tide. Flood and spray damage is a risk and could pose a threat to pedestrians. Fisherman's Quay to the east is a particularly low area and is occasionally overtopped, but is sheltered from direct wave attack by the detached rock breakwater. North of the Town Quay and within the Back Water Channel there are a number of quays which are in a poor condition but which are not affected by overtopping due to their sheltered location. These walls should be repaired so that there is no danger of local collapse affecting the integrity of adjacent structures.

To achieve a higher level of flood protection would require raising wall levels over virtually the whole frontage. The risk of flooding which has been identified is involved in the National Rivers Authority capital works programme for this area. Preliminary proposals comprise construction of a flood defence wall between the footpath and the road.

In addition to flood defence works, it is recommended that a warning system should be instigated to ensure that no danger to cars using the area for parking takes place and that the danger to pedestrians is eliminated. Plans should be made to restrict access to the area if severe flood events are forecast. Synthetic wave information can be produced from wind data and this linked to tidal gauge information should form the basis of an early warning system. It is recommended that as part of this system the extent and frequency of flooding should be monitored and entered onto a computer database. The events leading to flooding should be "hindcast" using this information and the reliability of the system thus monitored and upgraded if necessary.

5.12 Unit 9 Eastern Holes Bay

5.12.1 Management Plan

The reclaimed frontage forming the shoreline on the east side of Holes Bay is protected by a rock armour revetment. This revetment is sufficiently high and comprises sufficiently large armour rock so as to provide an adequate standard of protection in the foreseeable future (certainly for 10 years or more).

The flat foreshore on the eastern side of Holes Bay consists of mudflats and remnants of saltmarsh (the latter suffering from biological degeneration or so-called dieback). The intertidal zone is an important area for wading wildfowl. Despite the mudflats being relatively impoverished with regard to invertebrate fauna these mudflats, saltmarshes and the surrounding areas of reclaimed land support a large bird population.

While the defences in the area require little attention beyond an annual inspection, the strip of land between the shore and the road will require considerable attention to improve its amenity value. In view of the amount of bird life using this area, clearing up debris from the foreshore and judicious planting with appropriate trees and shrubs would yield considerable dividends both from an environmental and amenity viewpoint.

Discharge of industrial effluent at the southern end of the frontage has been identified as an environmental concern in the past. This discharge has now been curtailed and the levels of pollution within Holes Bay now appear to be within acceptable limits.

5.13 Unit 10 North Holes Bay

5.13.1 Management Plan

The northern part of Holes Bay is almost completely enclosed by the railway viaduct and comprises a very sheltered environment. The shoreline is fronted by extensive saltings which, unlike other areas, appear to have been unaffected by biological degeneration, probably because of the lack of "hydrodynamic stress" in this sheltered location. There is also no evidence of any coastal erosion.

The east part of the shoreline has been reclaimed for the construction of the A350 road. The embankment is very high and not unaffected by surge levels. There is an amenity area at the foot of this embankment with a footpath skirting the shoreline. Riprap protection to the edge of the path is in poor condition as much due to the small rock size as due to impact by wave action. Recovering the stone and replacing it at the footpath edge would improve the appearance of the area. There is also some scope for improving the visual appearance by tree planting, filling in of waterlogged depressions etc.

However, as far as coast protection is concerned little is needed beyond an annual or biannual inspection of the frontage.

The west part of the shoreline comprises agriculture land which unaffected by shoreline erosion or serious flooding. There is no necessity for works to be carried out in this area but the shoreline should be inspected on a biannual basis.

5.14 Unit 11 Western Holes Bay

5.14.1 Management Plan

Much of the western shore of Holes Bay has been developed for industrial and housing purposes. The land which has been reclaimed has generally been raised above the level of the saltmarsh, and there is little interaction between the two, in what is a very sheltered environment. There is no evidence of any coastal erosion. Large areas which have been developed for housing in recent years have been done so sympathetically. For example, the greensward between the houses and the saltmarsh fringe is now a valuable amenity.

The greatest disturbance to the foreshore is the result of marina development and the setting out of moorings on the fringes of channels. Boat wash has caused some damage to the saltmarsh, though on a minor scale.

Further south, industrial areas are being dismantled, noticeably the Power Station which lies on the west shore of the Back Water Channel. Until plans for redevelopment of this area are finalised little can be said as to the future coast protection requirements of this particular length of frontage.

In general, however, there is little need for intervention in the area and no upgrading of the defences in the areas used for housing development for the next ten years.

With regard to the southern end of the frontage which is earmarked for redevelopment it will be necessary to carry out further, more detailed studies of the tidal regime and of the ability of the new defences to withstand wave overtopping or erosion. This can be carried out with the models prepared during the course of this strategy study.

5.15 Unit 12 Hamworthy Quays

5.15.1 Management Plan

All of the shoreline in this Unit has been reclaimed and comprises various quays, marinas and other commercial frontages. The Hamworthy Quays are the responsibility of the Poole Harbour Commissioners, as a commercial port. Construction of these quays in conjunction with dredging means that the foreshore is now submerged at all tidal stages. The seabed is now subject to high tidal currents and the quayside is also exposed to considerable wave activity. Thus, although being considerably higher than the Town Quay, the commercial quays in Unit 12 do occasionally suffer wave overtopping. However, the overtopping discharges are rarely sufficiently high to disrupt commercial activities. The frontage is expected to maintain an adequate level of protection against flooding or erosion for at least the next ten years. Poole Harbour commissioners monitor and maintain the sections of Hamworthy Quay for which they are responsible.

The impact of any planned redevelopment of the southern part of Unit 11 on conditions at the quaysides of Unit 12 can be studied by models of numerical models of waves and tidal flows, which have been developed during the course of this study.

5.16 Unit 13 Lower Hamworth Marina to Rockley Viaduct

5.16.1 Assessment of defence alternatives

Problems : The frontage is relatively stable with the exception of the high cliffs of Bagshot Sands at the west end of the frontage. Where these cliffs have been protected the foreshore is narrow and has clay exposed on the surface. By comparison where cliff erosion has been allowed to continue there is a wide sand and shingle beach.

Constraints : There is a serious conflict between tourist and ecological interests at Rockley. Rockley Park is a popular caravan site. The adjacent cliffs and heathland form an important local nature reserve. Protection of the cliffs has a detrimental effect locally and at the coast to the west. Littoral drift is low but strongly unidirectional, from west to east.

Comparative weighting of criteria

Criterion		A	B	C	D	E	F	G	Total
A	Protection of cliffs	-	2	2	1	1	2	2	10
B	Capital cost	0	-	1	1	1	2	2	7
C	Maintenance cost	0	1	-	1	1	2	2	7
D	Impact on foreshore	1	1	1	-	1	1	1	6
E	Impact on adjacent coast	1	1	1	1	-	1	1	6
F	Public acceptance	0	0	0	1	1	-	1	3
G	Low environmental impact	0	0	0	1	1	1	-	3

Defence alternatives:

Option	Description
1	Reinstate gabion revetment
2	Reinstate gabions by rock toe
3	Nourish and groyne frontage
4	Remove gabions and relocate caravan (managed retreat)

Comparative assessment of defence alternatives

Criterion		Weight	1	2	3	4
A	Protection of cliffs	10	6	7	6	2
B	Low capital cost	7	6	6	5	9
C	Low maintenance cost	7	2	8	5	10
D	Low impact on foreshore	6	4	5	8	7
E	Low impact on adjacent coast	6	4	5	7	7
F	Public acceptance	3	5	5	7	4
G	Low environmental impact	3	4	5	6	8
			191	258	259	273

5.16.2 Management Plan

The relatively long frontage of Unit 13, although subdivided by groynes, jetties and marina walls comprises a continuous littoral cell in which adjacent sections of beach interact.

The eastern end of the Unit constitutes low lying land, reclaimed to form a popular recreation area. It is protected by a low concrete seawall/promenade and has a wide artificial beach intersected by several timber groynes. Overtopping which occurs occasionally only affects the recreational area itself and there is little risk to properties which are situated well to the landward of the recreational ground. The beach material appears to be relatively stable and the situation here could be improved by adding further quantities of sand or fine gravel. At the same time the short frontage towards the marina, which is protected by old walls and having an unprepossessing foreshore of mud could also be improved by nourishment and groyning.

Further west, private houses, a marina and the Royal Marines base back directly on to the shore. The structures along this frontage afford a reasonable level of protection and no major work is expected to be necessary in the next ten years.

The eastern part of the Unit, Rockley Sands and Ham Common, is formed of sandy and clay cliffs and stretches of heathland to the rear. This is an important nature reserve and an area of geological interest. The area is somewhat despoiled at its west end at the Rockley caravan site. Clifftop development at Rockley caravan site is expected to be affected by coastal erosion within the next ten years. The cliff toe at the west of Rockely Sands is protected by gabions, as are parts of the cliff face itself. The gabions require regular maintenance, without which their design life would be very short (very much less than ten years).

Except for the cliffed frontage below Rockley caravan site existing defences within Unit 13 will continue to provide the existing standard of protection during the next ten years.

A comparative assessment of various options for protecting the cliffs at Rockley caravan site has been made and this indicates that the least favoured option is the reinstatement of the existing gabions, not only because of expected future maintenance cost but because of the impact that these structures have had on the foreshore locally and on the downdrift beaches. By cutting off the supply of material from cliff erosion the gabions have resulted in low beach levels and the exposure of the underlying clay which new outcrops over a wide area. Where cliffs have been allowed to erode naturally, further east, there is a healthy sand beach and a relatively healthy lower foreshore.

Reinstating the beach by nourishment and groyning to protect Rockley is also considered to be a poor option. Sand nourishment would have to be carried out on a massive scale if cliff toe protection is to be carried out successfully. Construction of an energy dissipative rock toe, whilst improving the standard of protection and reducing maintenance requirements, is not favoured because it will not enhance sediment supply to downdrift beaches.

The favoured option is therefore to remove the gabions while relocating some of the caravans and allowing the cliffs to erode naturally. This clearly is the cheapest option and one which is probably the most acceptable from an environmental viewpoint. It has the advantage of restoring the supply of material to the adjacent coasts, hence increasing the level of natural protection to them.

It is recommended that a detailed study should be made to assess the viability of the managed retreat option and of the impact that this would have on general usage as well as on the viability of maintaining the caravan park usage. The possibility of relocating existing caravans or reducing their numbers should form an integral part of such a study. An environmental impact assessment should also be made to determine the affects of managed retreat on the environmental, attributes of the area. One also needs to take into account the need to maintain access to the boat facilities at Rockley viaduct at the western extremity of the frontage. (The access road runs through Rockley Point caravan site and is close to the cliff edge in places). Finally a coastal impact study will be required to determine how much supply to the beaches will be produced by renewed cliff erosion.

It is recommended that annual inspections of the whole of Unit 13 frontage should be made and any maintenance works carried out where necessary to observe the impact of construction of the marina and other water front structures on the beaches to ensure that no serious downdrift erosion problems develop.

The beaches at Hamworthy Park have in the past been nourished to improve their amenity value. This scheme is likely to require occasional small scale top-up operations in the future to maintain the amenity of this small beach. The presence of healthy beach also protects the concrete promenade. Beach monitoring will help and planning of top-up operations.

5.17 Unit 14 East Lytchett Bay

5.17.1 Management Plan

The enclosed nature of Lytchett Bay and the small wind fetch in all directions means that wave action is limited as is tidal current action. The presence of a sandy beach and some saltmarsh erosion in the south-east of the Bay does suggest modest wave activity in this area. However, erosion is very local and the Unit is stable for all intents and purposes. The high saltmarsh content in the Bay is an important environmental feature.

The existing stable coastal environment does not require any protection work to be made. With the exception of the railway viaduct there are no coastal structures which require maintenance. The extent and rate of retreat of the saltmarsh vegetation should be determined through an annual monitoring programme.

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Tables

Table 2.1 Distribution of wave height and direction for offshore point 1

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 = 159216
Based on HINDWAVE predictions for January 1974 - February 1992

H1	To H2	P(H>H1)	-10 10	10 30	30 50	50 70	70 90	90 110	110 130	130 150	150 170	170 190	190 210	210 230	230 250	250 270	270 290	290 310	310 330	330 350
0.00	0.20	0.97741	806	527	482	705	787	821	759	660	624	519	616	717	734	599	590	686	745	913
0.20	0.40	0.85452	1749	1174	1185	1205	1603	1984	710	161	164	156	216	870	1889	1390	924	463	349	895
0.40	0.60	0.68366	1119	538	568	754	1142	463	610	656	620	592	771	670	1988	1339	621	416	365	435
0.60	0.80	0.54697	392	241	406	477	1283	2240	361	246	246	251	311	835	1516	923	368	156	131	176
0.80	1.00	0.44138	97	85	86	231	688	1343	727	308	312	357	356	1196	2346	596	102	53	43	58
1.00	1.20	0.35154	57	53	92	65	303	1403	269	166	171	239	462	1178	2960	594	85	27	19	17
1.20	1.40	0.26993	29	18	14	14	104	438	250	215	119	219	276	943	2244	187	23	10	5	3
1.40	1.60	0.21881	4	4	1	4	107	355	138	87	116	160	449	1125	1073	95	14	5	0	0
1.60	1.80	0.18141	1	0	0	3	31	309	123	84	8	138	283	1049	2635	81	6	0	0	0
1.80	2.00	0.13391	0	0	0	0	9	69	68	57	157	126	162	92	689	13	0	0	0	0
2.00	2.20	0.11948	0	0	0	0	14	142	84	4	1	1	155	1277	1391	20	0	0	0	0
2.20	2.40	0.08858	0	0	0	0	6	89	26	75	94	152	285	224	406	14	0	0	0	0
2.40	2.60	0.07486	0	0	0	0	2	71	55	2	0	1	75	806	1344	4	0	0	0	0
2.60	2.80	0.05126	0	0	0	0	0	8	2	42	30	58	168	410	113	0	0	0	0	0
2.80	3.00	0.04295	0	0	0	0	0	63	33	2	1	0	4	521	799	5	0	0	0	0
3.00	3.20	0.02868	0	0	0	0	0	38	1	19	4	14	40	384	53	0	0	0	0	0
3.20	3.40	0.02316	0	0	0	0	0	9	3	1	7	31	83	285	487	0	0	0	0	0
3.40	3.60	0.01411	0	0	0	0	0	17	2	8	1	3	11	82	61	0	0	0	0	0
3.60	3.80	0.01225	0	0	0	0	0	9	0	1	7	10	24	185	161	0	0	0	0	0
3.80	4.00	0.00827	0	0	0	0	0	9	1	0	3	3	40	92	79	0	0	0	0	0
4.00	4.20	0.00600	0	0	0	0	0	13	9	2	1	3	6	97	82	0	0	0	0	0
4.20	4.40	0.00389	0	0	0	0	0	4	3	1	1	4	41	52	45	0	0	0	0	0
4.40	4.60	0.00238	0	0	0	0	0	2	2	3	1	3	0	8	53	0	0	0	0	0
4.60	4.80	0.00166	0	0	0	0	0	0	1	0	1	1	3	45	19	0	0	0	0	0
4.80	5.00	0.00098	0	0	0	0	0	0	3	0	0	0	5	9	4	0	0	0	0	0
5.00	5.20	0.00077	0	0	0	0	0	0	0	0	3	3	4	7	2	0	0	0	0	0
5.20	5.40	0.00058	0	0	0	0	0	0	1	0	0	1	0	8	6	0	0	0	0	0
5.40	5.60	0.00043	0	0	0	0	0	0	0	0	0	0	0	4	3	0	0	0	0	0
5.60	5.80	0.00036	0	0	0	0	0	0	0	0	0	0	2	2	1	0	0	0	0	0
5.80	6.00	0.00031	0	0	0	0	0	0	0	0	0	0	2	3	6	0	0	0	0	0
6.00	6.20	0.00021	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
6.20	6.40	0.00021	0	0	0	0	0	0	0	0	0	0	1	4	6	0	0	0	0	0
6.40	6.60	0.00010	0	0	0	0	0	0	0	0	0	0	0	1	0	0	0	0	0	0
6.60	6.80	0.00009	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
6.80	7.00	0.00009	0	0	0	0	0	0	0	0	0	0	0	5	4	0	0	0	0	0
7.00	7.20	0.00001	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
7.20	7.40	0.00001	0	0	0	0	0	0	0	0	0	0	0	0	1	0	0	0	0	0
Parts per thousand for each direction			44	27	29	35	62	101	43	29	28	31	50	135	237	60	28	19	17	26

Significant wave heights for given exceedence levels

P(H>Hs)	All dir.	-10 10	10 30	30 50	50 70	70 90	90 110	110 130	130 150	150 170	170 190	190 210	210 230	230 250	250 270	270 290	290 310	310 330	330 350
0.50	0.69	0.35	0.33	0.35	0.36	0.50	0.74	0.60	0.57	0.57	0.78	1.04	1.41	1.19	0.53	0.36	0.29	0.24	0.27
0.20	1.50	0.55	0.55	0.61	0.64	0.81	1.15	1.15	1.23	1.20	1.51	1.95	2.48	2.05	0.94	0.62	0.54	0.52	0.48
0.10	2.13	0.67	0.71	0.75	0.78	0.99	1.51	1.57	1.63	1.84	1.96	2.43	2.97	2.54	1.14	0.77	0.68	0.64	0.60
0.05	2.63	0.79	0.86	0.91	0.92	1.18	1.90	2.02	2.23	2.23	2.37	3.07	3.35	2.97	1.33	0.98	0.81	0.77	0.74
0.02	3.27	1.02	1.08	1.11	1.05	1.49	2.51	2.50	2.70	2.62	3.18	3.82	3.95	3.58	1.65	1.17	1.04	0.95	0.89
0.01	3.71	1.17	1.18	1.17	1.16	1.60	3.00	2.89	3.06	3.04	3.39	4.23	4.25	3.98	1.79	1.33	1.17	1.07	0.98
Average	0.93	0.38	0.37	0.40	0.41	0.55	0.81	0.75	0.75	0.77	0.94	1.20	1.55	1.35	0.61	0.41	0.34	0.31	0.30

Table 2.2 Distribution of wave height and period for offshore point 1

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 = 159216
Based on HINDWAVE predictions for January 1974 - February 1992

H1 To H2		P(H>H1)	Zero-crossing wave period in seconds (Tz)															
			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	0.20	0.97741	483	11807	0	0	0	0	0	0	0	0	0	0	0	0	0	0
0.20	0.40	0.85452	0	9021	8065	0	0	0	0	0	0	0	0	0	0	0	0	0
0.40	0.60	0.68366	0	9	13659	0	0	0	0	0	0	0	0	0	0	0	0	0
0.60	0.80	0.54697	0	0	8166	2394	0	0	0	0	0	0	0	0	0	0	0	0
0.80	1.00	0.44138	0	0	801	8183	0	0	0	0	0	0	0	0	0	0	0	0
1.00	1.20	0.35154	0	0	131	8029	0	0	0	0	0	0	0	0	0	0	0	0
1.20	1.40	0.26993	0	0	0	2812	2300	0	0	0	0	0	0	0	0	0	0	0
1.40	1.60	0.21881	0	0	0	1016	2724	0	0	0	0	0	0	0	0	0	0	0
1.60	1.80	0.18141	0	0	0	19	4731	0	0	0	0	0	0	0	0	0	0	0
1.80	2.00	0.13391	0	0	0	1	1441	0	0	0	0	0	0	0	0	0	0	0
2.00	2.20	0.11948	0	0	0	0	2426	663	0	0	0	0	0	0	0	0	0	0
2.20	2.40	0.08858	0	0	0	0	948	425	0	0	0	0	0	0	0	0	0	0
2.40	2.60	0.07486	0	0	0	0	14	2346	0	0	0	0	0	0	0	0	0	0
2.60	2.80	0.05126	0	0	0	0	1	829	0	0	0	0	0	0	0	0	0	0
2.80	3.00	0.04295	0	0	0	0	0	1428	0	0	0	0	0	0	0	0	0	0
3.00	3.20	0.02868	0	0	0	0	0	552	0	0	0	0	0	0	0	0	0	0
3.20	3.40	0.02316	0	0	0	0	0	215	690	0	0	0	0	0	0	0	0	0
3.40	3.60	0.01411	0	0	0	0	0	41	144	0	0	0	0	0	0	0	0	0
3.60	3.80	0.01225	0	0	0	0	0	28	369	0	0	0	0	0	0	0	0	0
3.80	4.00	0.00827	0	0	0	0	0	6	221	0	0	0	0	0	0	0	0	0
4.00	4.20	0.00600	0	0	0	0	0	0	212	0	0	0	0	0	0	0	0	0
4.20	4.40	0.00389	0	0	0	0	0	0	141	9	0	0	0	0	0	0	0	0
4.40	4.60	0.00238	0	0	0	0	0	0	24	48	0	0	0	0	0	0	0	0
4.60	4.80	0.00166	0	0	0	0	0	0	35	33	0	0	0	0	0	0	0	0
4.80	5.00	0.00098	0	0	0	0	0	0	14	7	0	0	0	0	0	0	0	0
5.00	5.20	0.00077	0	0	0	0	0	0	5	13	0	0	0	0	0	0	0	0
5.20	5.40	0.00058	0	0	0	0	0	0	0	16	0	0	0	0	0	0	0	0
5.40	5.60	0.00043	0	0	0	0	0	0	0	7	0	0	0	0	0	0	0	0
5.60	5.80	0.00036	0	0	0	0	0	0	0	4	0	0	0	0	0	0	0	0
5.80	6.00	0.00031	0	0	0	0	0	0	0	6	4	0	0	0	0	0	0	0
6.00	6.20	0.00021	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
6.20	6.40	0.00021	0	0	0	0	0	0	0	3	8	0	0	0	0	0	0	0
6.40	6.60	0.00010	0	0	0	0	0	0	0	1	0	0	0	0	0	0	0	0
6.60	6.80	0.00009	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
6.80	7.00	0.00009	0	0	0	0	0	0	0	0	9	0	0	0	0	0	0	0
7.00	7.20	0.00001	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
7.20	7.40	0.00001	0	0	0	0	0	0	0	0	1	0	0	0	0	0	0	0
Parts per thousand for each wave period			5	213	315	230	149	67	19	2	0	0	0	0	0	0	0	0

Table 2.3 Distribution of wave height and direction for offshore point 2

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 = 159216
Based on HINDWAVE predictions for January 1974 - February 1992

H1	To H2	P(H>H1)	Wave direction in degrees North																	
			-10	10	30	50	70	90	110	130	150	170	190	210	230	250	270	290	310	330
			10	30	50	70	90	110	130	150	170	190	210	230	250	270	290	310	330	350
0.00	0.20	0.97741	423	408	410	658	748	796	737	637	602	497	537	541	494	531	875	844	1119	516
0.20	0.40	0.86371	1679	1060	950	1101	1296	1669	585	153	167	153	203	580	1588	1416	988	844	697	995
0.40	0.60	0.70248	592	987	964	1007	1131	305	532	664	657	603	757	676	880	1115	561	400	312	313
0.60	0.80	0.57793	431	310	423	672	1238	1847	1013	263	256	251	327	639	1767	1418	708	394	314	384
0.80	1.00	0.45140	288	189	200	362	583	850	401	335	339	378	350	1149	2005	1048	227	117	81	90
1.00	1.20	0.36145	75	107	139	129	433	1062	431	165	184	240	509	977	2183	661	202	103	65	52
1.20	1.40	0.28427	42	57	43	28	175	444	323	247	136	253	251	853	746	573	94	35	17	22
1.40	1.60	0.24089	22	9	15	30	72	140	187	67	126	168	460	948	2917	434	58	16	11	7
1.60	1.80	0.18401	7	6	1	8	53	257	163	36	0	0	27	724	2028	222	16	8	3	1
1.80	2.00	0.14842	4	0	0	4	24	34	109	121	185	312	368	345	621	112	15	4	1	0
2.00	2.20	0.12583	1	0	0	1	18	145	62	13	1	1	155	1068	2098	77	3	0	0	0
2.20	2.40	0.08940	0	0	0	0	14	53	53	70	117	188	238	215	477	58	1	0	0	0
2.40	2.60	0.07455	0	0	0	0	4	23	41	15	0	1	75	675	1163	21	1	0	0	0
2.60	2.80	0.05435	0	0	0	0	0	35	7	42	4	4	136	406	326	4	0	0	0	0
2.80	3.00	0.04471	0	0	0	0	0	1	45	3	35	72	14	486	914	6	0	0	0	0
3.00	3.20	0.02895	0	0	0	0	0	34	19	6	1	5	38	366	25	3	0	0	0	0
3.20	3.40	0.02399	0	0	0	0	0	1	9	14	10	56	87	33	439	1	0	0	0	0
3.40	3.60	0.01749	0	0	0	0	0	10	10	0	2	4	4	308	112	1	0	0	0	0
3.60	3.80	0.01298	0	0	0	0	0	1	9	8	0	9	65	203	193	0	0	0	0	0
3.80	4.00	0.00810	0	0	0	0	0	8	1	0	11	14	8	110	45	0	0	0	0	0
4.00	4.20	0.00614	0	0	0	0	0	6	17	1	0	0	13	87	93	0	0	0	0	0
4.20	4.40	0.00398	0	0	0	0	0	4	3	1	2	16	35	65	30	0	0	0	0	0
4.40	4.60	0.00242	0	0	0	0	0	1	3	0	0	0	0	3	4	0	0	0	0	0
4.60	4.80	0.00231	0	0	0	0	0	0	1	3	1	3	0	36	55	0	0	0	0	0
4.80	5.00	0.00131	0	0	0	0	0	0	0	0	0	3	6	30	9	0	0	0	0	0
5.00	5.20	0.00085	0	0	0	0	0	0	3	0	0	0	2	3	4	0	0	0	0	0
5.20	5.40	0.00073	0	0	0	0	0	0	0	0	3	3	1	17	4	0	0	0	0	0
5.40	5.60	0.00045	0	0	0	0	0	0	1	0	0	0	0	4	3	0	0	0	0	0
5.60	5.80	0.00037	0	0	0	0	0	0	0	0	1	0	2	0	0	0	0	0	0	0
5.80	6.00	0.00034	0	0	0	0	0	0	0	0	0	0	2	3	4	0	0	0	0	0
6.00	6.20	0.00024	0	0	0	0	0	0	0	0	0	0	0	3	1	0	0	0	0	0
6.20	6.40	0.00021	0	0	0	0	0	0	0	0	0	0	0	2	4	0	0	0	0	0
6.40	6.60	0.00014	0	0	0	0	0	0	0	0	0	0	1	3	1	0	0	0	0	0
6.60	6.80	0.00009	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
6.80	7.00	0.00009	0	0	0	0	0	0	0	0	0	0	0	6	3	0	0	0	0	0
7.00	7.20	0.00001	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
7.20	7.40	0.00001	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
7.40	7.60	0.00001	0	0	0	0	0	0	0	0	0	0	0	0	1	0	0	0	0	0
Parts per thousand for each direction			36	32	32	41	59	79	49	29	29	33	48	118	217	79	38	28	27	24

Significant wave heights for given exceedence levels

P(H>Hs)	All dir.	Wave direction in degrees North																	
		-10	10	30	50	70	90	110	130	150	170	190	210	230	250	270	290	310	330
		10	30	50	70	90	110	130	150	170	190	210	230	250	270	290	310	330	350
0.50	0.72	0.36	0.41	0.44	0.44	0.54	0.71	0.69	0.58	0.59	0.84	1.04	1.45	1.45	0.70	0.39	0.32	0.25	0.33
0.20	1.54	0.67	0.62	0.68	0.72	0.87	1.13	1.26	1.25	1.28	1.82	1.96	2.54	2.16	1.18	0.76	0.66	0.57	0.64
0.10	2.14	0.85	0.85	0.88	0.89	1.09	1.38	1.68	1.81	1.89	2.25	2.42	3.05	2.66	1.47	1.01	0.80	0.74	0.76
0.05	2.69	0.98	1.03	1.05	1.00	1.27	1.77	2.13	2.25	2.27	2.85	3.12	3.59	2.99	1.70	1.20	1.05	0.91	0.91
0.02	3.32	1.21	1.22	1.19	1.18	1.59	2.26	2.90	2.69	2.85	3.35	3.72	4.06	3.62	2.03	1.46	1.24	1.13	1.13
0.01	3.72	1.39	1.34	1.33	1.42	1.81	2.72	3.37	3.15	3.22	3.88	4.20	4.37	3.99	2.25	1.59	1.40	1.25	1.25
Average	0.96	0.44	0.45	0.47	0.47	0.58	0.77	0.83	0.78	0.81	1.03	1.21	1.62	1.48	0.77	0.49	0.40	0.33	0.39

Table 2.4 Distribution of wave height and period for offshore point 2

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 = 159216
Based on HINDWAVE predictions for January 1974 - February 1992

H1	To H2	P(H>H1)	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
			Zero-crossing wave period in seconds (Tz)															
0.00	0.20	0.97741	483	10888	0	0	0	0	0	0	0	0	0	0	0	0	0	0
0.20	0.40	0.86371	0	6155	9968	0	0	0	0	0	0	0	0	0	0	0	0	0
0.40	0.60	0.70248	0	8	12447	0	0	0	0	0	0	0	0	0	0	0	0	0
0.60	0.80	0.57793	0	0	11224	1430	0	0	0	0	0	0	0	0	0	0	0	0
0.80	1.00	0.45140	0	0	1514	7481	0	0	0	0	0	0	0	0	0	0	0	0
1.00	1.20	0.36145	0	0	79	7639	0	0	0	0	0	0	0	0	0	0	0	0
1.20	1.40	0.28427	0	0	0	3759	580	0	0	0	0	0	0	0	0	0	0	0
1.40	1.60	0.24089	0	0	0	1369	4318	0	0	0	0	0	0	0	0	0	0	0
1.60	1.80	0.18401	0	0	0	52	3508	0	0	0	0	0	0	0	0	0	0	0
1.80	2.00	0.14842	0	0	0	18	2242	0	0	0	0	0	0	0	0	0	0	0
2.00	2.20	0.12583	0	0	0	1	2780	861	0	0	0	0	0	0	0	0	0	0
2.20	2.40	0.08940	0	0	0	0	806	680	0	0	0	0	0	0	0	0	0	0
2.40	2.60	0.07455	0	0	0	0	89	1931	0	0	0	0	0	0	0	0	0	0
2.60	2.80	0.05435	0	0	0	0	8	956	0	0	0	0	0	0	0	0	0	0
2.80	3.00	0.04471	0	0	0	0	0	1576	0	0	0	0	0	0	0	0	0	0
3.00	3.20	0.02895	0	0	0	0	1	496	0	0	0	0	0	0	0	0	0	0
3.20	3.40	0.02399	0	0	0	0	0	387	263	0	0	0	0	0	0	0	0	0
3.40	3.60	0.01749	0	0	0	0	0	58	393	0	0	0	0	0	0	0	0	0
3.60	3.80	0.01298	0	0	0	0	0	6	482	0	0	0	0	0	0	0	0	0
3.80	4.00	0.00810	0	0	0	0	0	8	188	0	0	0	0	0	0	0	0	0
4.00	4.20	0.00614	0	0	0	0	0	0	217	0	0	0	0	0	0	0	0	0
4.20	4.40	0.00398	0	0	0	0	0	0	156	0	0	0	0	0	0	0	0	0
4.40	4.60	0.00242	0	0	0	0	0	0	11	0	0	0	0	0	0	0	0	0
4.60	4.80	0.00231	0	0	0	0	0	0	43	56	0	0	0	0	0	0	0	0
4.80	5.00	0.00131	0	0	0	0	0	0	14	32	0	0	0	0	0	0	0	0
5.00	5.20	0.00085	0	0	0	0	0	0	2	10	0	0	0	0	0	0	0	0
5.20	5.40	0.00073	0	0	0	0	0	0	0	28	0	0	0	0	0	0	0	0
5.40	5.60	0.00045	0	0	0	0	0	0	0	8	0	0	0	0	0	0	0	0
5.60	5.80	0.00037	0	0	0	0	0	0	0	3	0	0	0	0	0	0	0	0
5.80	6.00	0.00034	0	0	0	0	0	0	0	9	0	0	0	0	0	0	0	0
6.00	6.20	0.00024	0	0	0	0	0	0	0	4	0	0	0	0	0	0	0	0
6.20	6.40	0.00021	0	0	0	0	0	0	0	2	4	0	0	0	0	0	0	0
6.40	6.60	0.00014	0	0	0	0	0	0	0	2	3	0	0	0	0	0	0	0
6.60	6.80	0.00009	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
6.80	7.00	0.00009	0	0	0	0	0	0	0	0	9	0	0	0	0	0	0	0
7.00	7.20	0.00001	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
7.20	7.40	0.00001	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
7.40	7.60	0.00001	0	0	0	0	0	0	0	0	1	0	0	0	0	0	0	0
Parts per thousand for each wave period			5	174	360	223	147	71	18	2	0	0	0	0	0	0	0	0

Table 2.5 Distribution of wave height and direction for offshore point 3

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 = 159216
Based on HINDWAVE predictions for January 1974 - February 1992

H1	To H2	P(H>H1)	-10 10	10 30	30 50	50 70	70 90	90 110	110 130	130 150	150 170	170 190	190 210	210 230	230 250	250 270	270 290	290 310	310 330	330 350
0.00	0.20	0.97741	1237	1654	1078	1244	1506	2016	1224	916	724	534	590	666	1237	1840	1508	1591	1629	1610
0.20	0.40	0.74937	420	671	358	610	823	1895	3373	931	231	611	1134	1492	1717	1527	1830	1101	1148	1085
0.40	0.60	0.53980	89	159	62	73	126	871	2945	520	896	577	1060	2731	1972	1165	713	308	230	80
0.60	0.80	0.39403	10	0	1	4	9	215	2345	764	322	384	911	2508	1229	697	289	114	62	24
0.80	1.00	0.29516	0	0	0	0	3	82	939	301	465	744	1539	3155	769	217	58	24	9	0
1.00	1.20	0.21212	0	0	0	0	1	41	462	327	252	239	1363	2239	337	62	14	1	0	0
1.20	1.40	0.15875	0	0	0	0	0	11	315	242	232	513	1125	1678	155	10	9	1	0	0
1.40	1.60	0.11586	0	0	0	0	0	0	200	134	144	150	960	1515	81	3	0	0	0	0
1.60	1.80	0.08398	0	0	0	0	0	0	96	90	68	414	1046	1003	26	0	0	0	0	0
1.80	2.00	0.05656	0	0	0	0	0	0	77	65	182	101	320	336	17	0	0	0	0	0
2.00	2.20	0.04558	0	0	0	0	0	0	16	54	28	103	879	479	0	0	0	0	0	0
2.20	2.40	0.03000	0	0	0	0	0	0	65	45	130	177	291	305	0	0	0	0	0	0
2.40	2.60	0.01987	0	0	0	0	0	0	14	23	6	58	400	16	0	0	0	0	0	0
2.60	2.80	0.01470	0	0	0	0	0	0	30	19	52	78	151	122	0	0	0	0	0	0
2.80	3.00	0.01017	0	0	0	0	0	0	11	0	4	40	241	12	0	0	0	0	0	0
3.00	3.20	0.00710	0	0	0	0	0	0	9	19	9	55	90	66	0	0	0	0	0	0
3.20	3.40	0.00460	0	0	0	0	0	0	21	1	17	13	102	27	0	0	0	0	0	0
3.40	3.60	0.00279	0	0	0	0	0	0	1	8	1	2	4	6	0	0	0	0	0	0
3.60	3.80	0.00257	0	0	0	0	0	0	4	1	12	35	79	8	0	0	0	0	0	0
3.80	4.00	0.00117	0	0	0	0	0	0	2	2	0	1	3	1	0	0	0	0	0	0
4.00	4.20	0.00108	0	0	0	0	0	0	1	1	1	22	25	4	0	0	0	0	0	0
4.20	4.40	0.00054	0	0	0	0	0	0	0	3	2	1	3	3	0	0	0	0	0	0
4.40	4.60	0.00043	0	0	0	0	0	0	3	1	1	6	2	1	0	0	0	0	0	0
4.60	4.80	0.00029	0	0	0	0	0	0	0	0	1	0	1	4	0	0	0	0	0	0
4.80	5.00	0.00023	0	0	0	0	0	0	1	0	0	1	4	0	0	0	0	0	0	0
5.00	5.20	0.00018	0	0	0	0	0	0	0	0	3	4	3	1	0	0	0	0	0	0
5.20	5.40	0.00007	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
5.40	5.60	0.00007	0	0	0	0	0	0	0	0	1	0	5	0	0	0	0	0	0	0
5.60	5.80	0.00001	0	0	0	0	0	0	0	0	0	0	1	0	0	0	0	0	0	0
Parts per thousand for each direction			18	25	15	20	25	52	124	46	39	50	126	188	77	56	45	32	31	29

Significant wave heights for given exceedence levels

P(H>Hs)	All dir.	-10 10	10 30	30 50	50 70	70 90	90 110	110 130	130 150	150 170	170 190	190 210	210 230	230 250	250 270	270 290	290 310	310 330	330 350
0.50	0.45	0.14	0.15	0.14	0.15	0.16	0.25	0.49	0.53	0.60	0.87	1.12	0.90	0.47	0.31	0.27	0.19	0.18	0.17
0.20	1.05	0.28	0.30	0.26	0.30	0.31	0.44	0.78	1.07	1.30	1.66	1.85	1.42	0.77	0.58	0.45	0.36	0.34	0.31
0.10	1.50	0.36	0.37	0.35	0.36	0.37	0.56	1.04	1.41	1.87	2.19	2.30	1.70	0.96	0.72	0.58	0.48	0.40	0.37
0.05	1.92	0.42	0.44	0.39	0.39	0.42	0.68	1.36	1.84	2.27	2.63	2.71	2.05	1.14	0.81	0.70	0.59	0.52	0.39
0.02	2.40	0.54	0.00	0.51	0.50	0.54	0.87	1.82	2.33	2.71	3.15	3.15	2.33	1.36	0.96	0.79	0.73	0.63	0.52
0.01	2.81	0.58	0.00	0.55	0.56	0.58	1.00	2.32	2.69	3.22	3.72	3.54	2.71	1.52	1.06	0.92	0.79	0.73	0.59
Average	0.65	0.17	0.18	0.16	0.18	0.19	0.28	0.57	0.66	0.82	1.03	1.23	0.97	0.51	0.35	0.30	0.23	0.21	0.19

Table 2.6 Distribution of wave height and period for offshore point 3

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 = 159216
Based on HINDWAVE predictions for January 1974 - February 1992

H1 To H2		P(H>H1)	Zero-crossing wave period in seconds (Tz)															
			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	0.20	0.97741	1446	21359	0	0	0	0	0	0	0	0	0	0	0	0	0	0
0.20	0.40	0.74937	0	14887	6069	0	0	0	0	0	0	0	0	0	0	0	0	0
0.40	0.60	0.53980	0	675	13902	0	0	0	0	0	0	0	0	0	0	0	0	0
0.60	0.80	0.39403	0	0	7538	2350	0	0	0	0	0	0	0	0	0	0	0	0
0.80	1.00	0.29516	0	0	1636	6668	0	0	0	0	0	0	0	0	0	0	0	0
1.00	1.20	0.21212	0	0	113	5072	152	0	0	0	0	0	0	0	0	0	0	0
1.20	1.40	0.15875	0	0	5	3304	980	0	0	0	0	0	0	0	0	0	0	0
1.40	1.60	0.11586	0	0	0	477	2710	0	0	0	0	0	0	0	0	0	0	0
1.60	1.80	0.08398	0	0	0	18	2725	0	0	0	0	0	0	0	0	0	0	0
1.80	2.00	0.05656	0	0	0	0	1098	0	0	0	0	0	0	0	0	0	0	0
2.00	2.20	0.04558	0	0	0	0	1544	14	0	0	0	0	0	0	0	0	0	0
2.20	2.40	0.03000	0	0	0	0	215	797	0	0	0	0	0	0	0	0	0	0
2.40	2.60	0.01987	0	0	0	0	14	503	0	0	0	0	0	0	0	0	0	0
2.60	2.80	0.01470	0	0	0	0	1	452	0	0	0	0	0	0	0	0	0	0
2.80	3.00	0.01017	0	0	0	0	1	307	0	0	0	0	0	0	0	0	0	0
3.00	3.20	0.00710	0	0	0	0	0	239	11	0	0	0	0	0	0	0	0	0
3.20	3.40	0.00460	0	0	0	0	0	85	97	0	0	0	0	0	0	0	0	0
3.40	3.60	0.00279	0	0	0	0	0	6	16	0	0	0	0	0	0	0	0	0
3.60	3.80	0.00257	0	0	0	0	0	6	133	0	0	0	0	0	0	0	0	0
3.80	4.00	0.00117	0	0	0	0	0	1	8	0	0	0	0	0	0	0	0	0
4.00	4.20	0.00108	0	0	0	0	0	0	54	0	0	0	0	0	0	0	0	0
4.20	4.40	0.00054	0	0	0	0	0	0	11	0	0	0	0	0	0	0	0	0
4.40	4.60	0.00043	0	0	0	0	0	0	14	0	0	0	0	0	0	0	0	0
4.60	4.80	0.00029	0	0	0	0	0	0	2	4	0	0	0	0	0	0	0	0
4.80	5.00	0.00023	0	0	0	0	0	0	2	4	0	0	0	0	0	0	0	0
5.00	5.20	0.00018	0	0	0	0	0	0	0	11	0	0	0	0	0	0	0	0
5.20	5.40	0.00007	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
5.40	5.60	0.00007	0	0	0	0	0	0	0	6	0	0	0	0	0	0	0	0
5.60	5.80	0.00001	0	0	0	0	0	0	0	1	0	0	0	0	0	0	0	0
Parts per thousand for each wave period			15	378	299	183	97	25	4	0	0	0	0	0	0	0	0	0

Table 2.7 Distribution of wave height and direction for offshore point 4

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 = 159216
Based on HINDWAVE predictions for January 1974 - February 1992

H1	To H2	P(H>H1)	-10 10	10 30	30 50	50 70	70 90	90 110	110 130	130 150	150 170	170 190	190 210	210 230	230 250	250 270	270 290	290 310	310 330	330 350
			Wave direction in degrees North																	
0.00	0.20	0.97741	600	1148	783	673	673	799	790	678	623	496	545	561	611	659	644	685	766	758
0.20	0.40	0.85248	1419	1061	778	1063	1416	1850	2032	767	186	164	204	552	1180	1536	1279	1186	1018	1464
0.40	0.60	0.66092	785	361	862	674	664	1227	1965	462	772	676	788	635	556	1203	974	493	450	835
0.60	0.80	0.51708	305	268	164	146	253	583	1785	803	284	271	337	648	1748	1453	751	352	254	300
0.80	1.00	0.41004	100	136	52	40	103	169	769	328	406	407	372	1143	1474	997	410	133	52	68
1.00	1.20	0.33845	30	16	6	9	36	57	381	321	220	254	516	947	2419	1133	227	86	45	46
1.20	1.40	0.27096	6	0	1	10	18	89	263	249	208	266	263	842	653	820	128	33	11	22
1.40	1.60	0.23217	4	0	0	2	1	21	160	151	104	181	442	921	2818	658	46	13	4	2
1.60	1.80	0.17690	0	0	0	0	0	22	75	85	63	1	26	701	1717	177	27	6	0	0
1.80	2.00	0.14788	0	0	0	0	0	1	58	84	143	323	385	325	869	290	12	3	0	0
2.00	2.20	0.12295	0	0	0	0	0	0	18	41	38	1	91	1017	2186	156	7	0	0	0
2.20	2.40	0.08739	0	0	0	0	0	0	60	57	90	200	247	212	139	85	1	0	0	0
2.40	2.60	0.07647	0	0	0	0	0	0	21	35	13	1	43	603	1573	18	0	0	0	0
2.60	2.80	0.05341	0	0	0	0	0	0	4	18	32	6	121	418	334	52	0	0	0	0
2.80	3.00	0.04357	0	0	0	0	0	0	16	4	4	68	18	335	391	23	0	0	0	0
3.00	3.20	0.03497	0	0	0	0	0	0	21	14	3	5	9	393	710	2	0	0	0	0
3.20	3.40	0.02340	0	0	0	0	0	0	0	2	10	55	77	42	82	3	0	0	0	0
3.40	3.60	0.02068	0	0	0	0	0	0	4	8	0	4	4	202	553	8	0	0	0	0
3.60	3.80	0.01286	0	0	0	0	0	0	0	0	9	10	42	223	48	1	0	0	0	0
3.80	4.00	0.00953	0	0	0	0	0	0	0	6	0	13	11	71	261	0	0	0	0	0
4.00	4.20	0.00592	0	0	0	0	0	0	0	1	1	1	4	82	18	0	0	0	0	0
4.20	4.40	0.00487	0	0	0	0	0	0	0	6	1	15	28	9	103	0	0	0	0	0
4.40	4.60	0.00323	0	0	0	0	0	0	0	0	0	0	0	34	63	0	0	0	0	0
4.60	4.80	0.00227	0	0	0	0	0	0	0	1	1	5	6	36	59	0	0	0	0	0
4.80	5.00	0.00119	0	0	0	0	0	0	0	0	0	0	0	11	24	0	0	0	0	0
5.00	5.20	0.00084	0	0	0	0	0	0	0	0	3	0	1	3	3	0	0	0	0	0
5.20	5.40	0.00074	0	0	0	0	0	0	0	0	0	3	2	10	3	0	0	0	0	0
5.40	5.60	0.00057	0	0	0	0	0	0	0	0	1	1	0	6	11	0	0	0	0	0
5.60	5.80	0.00038	0	0	0	0	0	0	0	0	0	0	1	2	2	0	0	0	0	0
5.80	6.00	0.00033	0	0	0	0	0	0	0	0	0	0	1	3	1	0	0	0	0	0
6.00	6.20	0.00028	0	0	0	0	0	0	0	0	0	0	0	0	3	0	0	0	0	0
6.20	6.40	0.00025	0	0	0	0	0	0	0	0	0	0	0	0	4	0	0	0	0	0
6.40	6.60	0.00021	0	0	0	0	0	0	0	0	0	0	1	4	3	0	0	0	0	0
6.60	6.80	0.00014	0	0	0	0	0	0	0	0	0	0	0	0	4	0	0	0	0	0
6.80	7.00	0.00009	0	0	0	0	0	0	0	0	0	0	0	5	0	0	0	0	0	0
7.00	7.20	0.00004	0	0	0	0	0	0	0	0	0	0	0	0	4	0	0	0	0	0
7.20	7.40	0.00001	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
7.40	7.60	0.00001	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
7.60	7.80	0.00001	0	0	0	0	0	0	0	0	0	0	0	0	1	0	0	0	0	0
Parts per thousand for each direction			33	31	27	27	32	49	86	42	33	35	47	112	211	95	46	31	27	36

Significant wave heights for given exceedence levels

P(H>Hs)	All dir.	-10 10	10 30	30 50	50 70	70 90	90 110	110 130	130 150	150 170	170 190	190 210	210 230	230 250	250 270	270 290	290 310	310 330	330 350
		Wave direction in degrees North																	
0.50	0.63	0.34	0.26	0.33	0.31	0.32	0.37	0.53	0.63	0.60	0.83	1.00	1.41	1.50	0.76	0.46	0.33	0.30	0.33
0.20	1.52	0.54	0.49	0.53	0.50	0.53	0.59	0.83	1.15	1.26	1.81	1.88	2.48	2.40	1.30	0.78	0.61	0.53	0.53
0.10	2.13	0.67	0.68	0.59	0.58	0.67	0.75	1.12	1.52	1.83	2.24	2.32	3.01	2.92	1.56	0.99	0.78	0.68	0.65
0.05	2.67	0.78	0.80	0.71	0.70	0.80	0.93	1.41	1.96	2.21	2.81	2.75	3.54	3.44	1.91	1.19	0.98	0.78	0.77
0.02	3.42	0.95	0.93	0.81	0.84	0.98	1.28	1.90	2.46	2.58	3.33	3.62	3.94	3.91	2.20	1.40	1.19	1.03	0.99
0.01	3.77	1.04	0.98	0.92	0.97	1.12	1.39	2.33	2.79	2.80	3.82	3.96	4.41	4.35	2.52	1.60	1.35	1.15	1.15
Average	0.92	0.37	0.31	0.34	0.33	0.36	0.42	0.61	0.73	0.80	1.02	1.14	1.57	1.59	0.84	0.52	0.40	0.34	0.36

Table 2.8 Distribution of wave height and period for offshore point 4

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 = 159216
Based on HINDWAVE predictions for January 1974 - February 1992

H1	To H2	P(H>H1)	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
			Zero-crossing wave period in seconds (Tz)															
0.00	0.20	0.97741	483	12010	0	0	0	0	0	0	0	0	0	0	0	0	0	0
0.20	0.40	0.85248	0	11853	7303	0	0	0	0	0	0	0	0	0	0	0	0	0
0.40	0.60	0.66092	0	9	14375	0	0	0	0	0	0	0	0	0	0	0	0	0
0.60	0.80	0.51708	0	0	9030	1674	0	0	0	0	0	0	0	0	0	0	0	0
0.80	1.00	0.41004	0	0	1603	5556	0	0	0	0	0	0	0	0	0	0	0	0
1.00	1.20	0.33845	0	0	122	6626	0	0	0	0	0	0	0	0	0	0	0	0
1.20	1.40	0.27096	0	0	0	3424	455	0	0	0	0	0	0	0	0	0	0	0
1.40	1.60	0.23217	0	0	0	953	4574	0	0	0	0	0	0	0	0	0	0	0
1.60	1.80	0.17690	0	0	0	92	2810	0	0	0	0	0	0	0	0	0	0	0
1.80	2.00	0.14788	0	0	0	6	2487	0	0	0	0	0	0	0	0	0	0	0
2.00	2.20	0.12295	0	0	0	0	3556	0	0	0	0	0	0	0	0	0	0	0
2.20	2.40	0.08739	0	0	0	0	698	393	0	0	0	0	0	0	0	0	0	0
2.40	2.60	0.07647	0	0	0	0	39	2267	0	0	0	0	0	0	0	0	0	0
2.60	2.80	0.05341	0	0	0	0	1	984	0	0	0	0	0	0	0	0	0	0
2.80	3.00	0.04357	0	0	0	0	1	860	0	0	0	0	0	0	0	0	0	0
3.00	3.20	0.03497	0	0	0	0	0	1157	0	0	0	0	0	0	0	0	0	0
3.20	3.40	0.02340	0	0	0	0	0	271	0	0	0	0	0	0	0	0	0	0
3.40	3.60	0.02068	0	0	0	0	0	40	742	0	0	0	0	0	0	0	0	0
3.60	3.80	0.01286	0	0	0	0	0	17	316	0	0	0	0	0	0	0	0	0
3.80	4.00	0.00953	0	0	0	0	0	4	356	0	0	0	0	0	0	0	0	0
4.00	4.20	0.00592	0	0	0	0	0	0	106	0	0	0	0	0	0	0	0	0
4.20	4.40	0.00487	0	0	0	0	0	0	163	0	0	0	0	0	0	0	0	0
4.40	4.60	0.00323	0	0	0	0	0	0	97	0	0	0	0	0	0	0	0	0
4.60	4.80	0.00227	0	0	0	0	0	0	43	65	0	0	0	0	0	0	0	0
4.80	5.00	0.00119	0	0	0	0	0	0	4	30	0	0	0	0	0	0	0	0
5.00	5.20	0.00084	0	0	0	0	0	0	9	1	0	0	0	0	0	0	0	0
5.20	5.40	0.00074	0	0	0	0	0	0	0	18	0	0	0	0	0	0	0	0
5.40	5.60	0.00057	0	0	0	0	0	0	0	19	0	0	0	0	0	0	0	0
5.60	5.80	0.00038	0	0	0	0	0	0	0	5	0	0	0	0	0	0	0	0
5.80	6.00	0.00033	0	0	0	0	0	0	0	4	0	0	0	0	0	0	0	0
6.00	6.20	0.00028	0	0	0	0	0	0	0	3	0	0	0	0	0	0	0	0
6.20	6.40	0.00025	0	0	0	0	0	0	0	0	4	0	0	0	0	0	0	0
6.40	6.60	0.00021	0	0	0	0	0	0	0	3	4	0	0	0	0	0	0	0
6.60	6.80	0.00014	0	0	0	0	0	0	0	0	4	0	0	0	0	0	0	0
6.80	7.00	0.00009	0	0	0	0	0	0	0	0	5	0	0	0	0	0	0	0
7.00	7.20	0.00004	0	0	0	0	0	0	0	0	4	0	0	0	0	0	0	0
7.20	7.40	0.00001	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
7.40	7.60	0.00001	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
7.60	7.80	0.00001	0	0	0	0	0	0	0	0	1	0	0	0	0	0	0	0
Parts per thousand for each wave period			5	244	332	188	150	61	19	2	0	0	0	0	0	0	0	0

Table 2.9 Distribution of wave height and direction for offshore point 5

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 = 159216
Based on HINDWAVE predictions for January 1974 - February 1992

Wave direction in degrees North

H1	To H2	P(H>H1)	-10	10	30	50	70	90	110	130	150	170	190	210	230	250	270	290	310	330
			10	30	50	70	90	110	130	150	170	190	210	230	250	270	290	310	330	350
0.00	0.20	0.97741	1105	820	1000	1160	1251	1481	1458	950	755	545	582	597	658	805	1023	1571	1340	1234
0.20	0.40	0.79407	715	921	1153	603	898	1333	3287	1901	624	182	203	216	695	2026	2341	1119	901	691
0.40	0.60	0.59599	84	123	83	72	155	342	2431	1575	529	708	737	757	896	1671	1053	622	355	125
0.60	0.80	0.47280	12	0	4	10	29	117	651	687	354	278	315	574	1159	1588	911	110	60	30
0.80	1.00	0.40391	0	0	1	1	3	14	298	703	507	418	354	613	1207	2256	332	60	18	4
1.00	1.20	0.33603	0	0	0	0	0	7	190	608	236	255	449	739	1225	1173	209	18	8	0
1.20	1.40	0.28486	0	0	0	0	0	0	55	168	153	268	249	860	1972	1487	63	6	0	0
1.40	1.60	0.23204	0	0	0	0	0	0	35	117	138	185	407	471	2010	814	35	2	0	0
1.60	1.80	0.18991	0	0	0	0	0	0	19	131	104	18	6	768	1095	763	14	0	0	0
1.80	2.00	0.16072	0	0	0	0	0	0	14	43	65	305	381	340	2004	607	6	0	0	0
2.00	2.20	0.12307	0	0	0	0	0	0	10	89	52	1	85	379	790	165	1	0	0	0
2.20	2.40	0.10736	0	0	0	0	0	0	4	3	79	200	234	527	1497	339	0	0	0	0
2.40	2.60	0.07852	0	0	0	0	0	0	8	55	19	1	38	355	665	161	0	0	0	0
2.60	2.80	0.06551	0	0	0	0	0	0	9	6	30	8	124	418	1166	149	0	0	0	0
2.80	3.00	0.04640	0	0	0	0	0	0	6	18	9	67	19	21	313	100	0	0	0	0
3.00	3.20	0.04087	0	0	0	0	0	0	1	3	8	8	10	521	991	8	0	0	0	0
3.20	3.40	0.02539	0	0	0	0	0	0	0	8	3	54	80	19	234	33	0	0	0	0
3.40	3.60	0.02109	0	0	0	0	0	0	0	3	1	3	3	170	558	18	0	0	0	0
3.60	3.80	0.01352	0	0	0	0	0	0	0	2	9	25	43	176	106	6	0	0	0	0
3.80	4.00	0.00985	0	0	0	0	0	0	0	4	0	3	1	29	151	6	0	0	0	0
4.00	4.20	0.00792	0	0	0	0	0	0	0	4	1	10	7	85	188	0	0	0	0	0
4.20	4.40	0.00497	0	0	0	0	0	0	0	0	0	7	24	13	24	3	0	0	0	0
4.40	4.60	0.00426	0	0	0	0	0	0	0	0	1	1	0	19	171	0	0	0	0	0
4.60	4.80	0.00234	0	0	0	0	0	0	0	0	0	5	6	33	21	1	0	0	0	0
4.80	5.00	0.00168	0	0	0	0	0	0	0	0	0	0	0	5	77	0	0	0	0	0
5.00	5.20	0.00086	0	0	0	0	0	0	0	0	4	5	1	8	3	0	0	0	0	0
5.20	5.40	0.00065	0	0	0	0	0	0	0	0	0	0	0	3	4	0	0	0	0	0
5.40	5.60	0.00058	0	0	0	0	0	0	0	0	0	0	1	4	16	0	0	0	0	0
5.60	5.80	0.00038	0	0	0	0	0	0	0	0	0	0	1	1	3	0	0	0	0	0
5.80	6.00	0.00032	0	0	0	0	0	0	0	0	0	0	0	1	2	0	0	0	0	0
6.00	6.20	0.00030	0	0	0	0	0	0	0	0	0	0	0	0	4	0	0	0	0	0
6.20	6.40	0.00025	0	0	0	0	0	0	0	0	0	0	1	2	4	0	0	0	0	0
6.40	6.60	0.00018	0	0	0	0	0	0	0	0	0	0	0	1	3	0	0	0	0	0
6.60	6.80	0.00014	0	0	0	0	0	0	0	0	0	0	0	0	1	0	0	0	0	0
6.80	7.00	0.00014	0	0	0	0	0	0	0	0	0	0	0	5	4	0	0	0	0	0
7.00	7.20	0.00004	0	0	0	0	0	0	0	0	0	0	0	0	4	0	0	0	0	0
7.20	7.40	0.00001	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
7.40	7.60	0.00001	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
7.60	7.80	0.00001	0	0	0	0	0	0	0	0	0	0	0	0	1	0	0	0	0	0
Parts per thousand for each direction			20	19	23	19	24	34	87	72	38	36	45	89	204	145	61	36	27	21

Significant wave heights for given exceedence levels

Wave direction in degrees North

P(H>Hs)	All	-10	10	30	50	70	90	110	130	150	170	190	210	230	250	270	290	310	330
	dir.	10	30	50	70	90	110	130	150	170	190	210	230	250	270	290	310	330	350
0.50	0.56	0.17	0.22	0.22	0.16	0.18	0.22	0.36	0.48	0.56	0.81	0.97	1.38	1.59	0.87	0.36	0.23	0.20	0.16
0.20	1.55	0.32	0.34	0.34	0.30	0.34	0.37	0.56	0.95	1.13	1.59	1.89	2.46	2.59	1.47	0.68	0.43	0.38	0.32
0.10	2.25	0.37	0.39	0.38	0.36	0.39	0.48	0.73	1.18	1.61	2.23	2.33	3.08	3.11	1.85	0.83	0.55	0.49	0.38
0.05	2.76	0.40	0.45	0.40	0.40	0.49	0.58	0.94	1.61	2.10	2.82	2.76	3.53	3.52	2.26	1.02	0.63	0.57	0.48
0.02	3.43	0.54	0.00	0.50	0.52	0.58	0.72	1.19	2.11	2.48	3.35	3.43	4.00	4.13	2.65	1.20	0.85	0.70	0.59
0.01	3.79	0.58	0.00	0.56	0.58	0.65	0.78	1.51	2.51	2.78	3.76	3.79	4.29	4.53	2.86	1.38	0.97	0.80	0.69
Average	0.90	0.19	0.22	0.21	0.18	0.21	0.24	0.41	0.61	0.73	1.01	1.13	1.57	1.73	0.97	0.44	0.27	0.24	0.20

Table 2.10 Distribution of wave height and period for offshore point 5

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 = 159216
Based on HINDWAVE predictions for January 1974 - February 1992

Zero-crossing wave period in seconds (Tz)

H1	To H2	P(H>H1)	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	0.20	0.97741	799	17535	0	0	0	0	0	0	0	0	0	0	0	0	0	0
0.20	0.40	0.79407	0	12135	7673	0	0	0	0	0	0	0	0	0	0	0	0	0
0.40	0.60	0.59599	0	328	11990	0	0	0	0	0	0	0	0	0	0	0	0	0
0.60	0.80	0.47280	0	0	4782	2107	0	0	0	0	0	0	0	0	0	0	0	0
0.80	1.00	0.40391	0	0	252	6536	0	0	0	0	0	0	0	0	0	0	0	0
1.00	1.20	0.33603	0	0	17	5099	0	0	0	0	0	0	0	0	0	0	0	0
1.20	1.40	0.28486	0	0	0	4650	632	0	0	0	0	0	0	0	0	0	0	0
1.40	1.60	0.23204	0	0	0	710	3503	0	0	0	0	0	0	0	0	0	0	0
1.60	1.80	0.18991	0	0	0	27	2892	0	0	0	0	0	0	0	0	0	0	0
1.80	2.00	0.16072	0	0	0	0	3765	0	0	0	0	0	0	0	0	0	0	0
2.00	2.20	0.12307	0	0	0	0	1514	56	0	0	0	0	0	0	0	0	0	0
2.20	2.40	0.10736	0	0	0	0	1475	1409	0	0	0	0	0	0	0	0	0	0
2.40	2.60	0.07852	0	0	0	0	55	1247	0	0	0	0	0	0	0	0	0	0
2.60	2.80	0.06551	0	0	0	0	6	1905	0	0	0	0	0	0	0	0	0	0
2.80	3.00	0.04640	0	0	0	0	0	553	0	0	0	0	0	0	0	0	0	0
3.00	3.20	0.04087	0	0	0	0	0	1546	1	0	0	0	0	0	0	0	0	0
3.20	3.40	0.02539	0	0	0	0	0	423	7	0	0	0	0	0	0	0	0	0
3.40	3.60	0.02109	0	0	0	0	0	71	686	0	0	0	0	0	0	0	0	0
3.60	3.80	0.01352	0	0	0	0	0	9	358	0	0	0	0	0	0	0	0	0
3.80	4.00	0.00985	0	0	0	0	0	8	186	0	0	0	0	0	0	0	0	0
4.00	4.20	0.00792	0	0	0	0	0	0	295	0	0	0	0	0	0	0	0	0
4.20	4.40	0.00497	0	0	0	0	0	0	72	0	0	0	0	0	0	0	0	0
4.40	4.60	0.00426	0	0	0	0	0	0	192	0	0	0	0	0	0	0	0	0
4.60	4.80	0.00234	0	0	0	0	0	0	57	9	0	0	0	0	0	0	0	0
4.80	5.00	0.00168	0	0	0	0	0	0	3	80	0	0	0	0	0	0	0	0
5.00	5.20	0.00086	0	0	0	0	0	0	7	14	0	0	0	0	0	0	0	0
5.20	5.40	0.00065	0	0	0	0	0	0	0	7	0	0	0	0	0	0	0	0
5.40	5.60	0.00058	0	0	0	0	0	0	0	21	0	0	0	0	0	0	0	0
5.60	5.80	0.00038	0	0	0	0	0	0	0	6	0	0	0	0	0	0	0	0
5.80	6.00	0.00032	0	0	0	0	0	0	0	3	0	0	0	0	0	0	0	0
6.00	6.20	0.00030	0	0	0	0	0	0	0	4	0	0	0	0	0	0	0	0
6.20	6.40	0.00025	0	0	0	0	0	0	0	7	0	0	0	0	0	0	0	0
6.40	6.60	0.00018	0	0	0	0	0	0	0	1	3	0	0	0	0	0	0	0
6.60	6.80	0.00014	0	0	0	0	0	0	0	1	0	0	0	0	0	0	0	0
6.80	7.00	0.00014	0	0	0	0	0	0	0	0	9	0	0	0	0	0	0	0
7.00	7.20	0.00004	0	0	0	0	0	0	0	0	4	0	0	0	0	0	0	0
7.20	7.40	0.00001	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
7.40	7.60	0.00001	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
7.60	7.80	0.00001	0	0	0	0	0	0	0	0	1	0	0	0	0	0	0	0
Parts per thousand for each wave period			8	307	253	196	142	74	19	2	0	0	0	0	0	0	0	0

Table 2.11 Extreme wave conditions at offshore point 1

Return Period (Yrs)	Wave direction in degrees North																		
	-10 10	10 30	30 50	50 70	70 90	90 110	110 130	130 150	150 170	170 190	190 210	210 230	230 250	250 270	270 290	290 310	310 330	330 350	0 to 360
1	1.13	1.10	1.06	1.18	1.80	3.43	3.17	2.92	3.06	3.45	4.26	5.24	5.16	2.06	1.26	1.04	0.93	0.91	5.67
2	1.25	1.24	1.16	1.29	1.96	3.76	3.58	3.29	3.48	3.85	4.65	5.59	5.52	2.26	1.40	1.17	1.03	1.01	6.07
5	1.39	1.41	1.27	1.43	2.16	4.20	4.10	3.77	4.03	4.36	5.14	6.03	5.97	2.51	1.58	1.33	1.15	1.13	6.59
10	1.50	1.54	1.36	1.53	2.31	4.52	4.50	4.12	4.44	4.73	5.49	6.34	6.30	2.69	1.71	1.44	1.24	1.21	6.99
20	1.60	1.67	1.44	1.62	2.45	4.84	4.89	4.47	4.85	5.10	5.83	6.65	6.62	2.87	1.84	1.55	1.32	1.30	7.37
50	1.74	1.83	1.54	1.74	2.64	5.25	5.41	4.91	5.38	5.56	6.26	7.03	7.04	3.11	2.01	1.70	1.41	1.40	7.88
100	1.84	1.96	1.61	1.83	2.78	5.56	5.79	5.25	5.77	5.91	6.57	7.31	7.35	3.28	2.13	1.80	1.49	1.48	8.25
	Wave height in m																		

The corresponding mean period can be calculated from T_z 10.6 $\sqrt{H/g}$

Table 2.12 Extreme wave conditions at offshore point 2

Return Period (Yrs)	Wave direction in degrees North																	
	-10 10	10 30	30 50	50 70	70 90	90 110	110 130	130 150	150 170	170 190	190 210	210 230	230 250	250 270	270 290	290 310	310 330	330 350	0 to 360
1	1.37	1.29	1.25	1.41	1.93	3.16	3.45	3.02	3.27	3.76	4.24	5.38	5.15	2.44	1.62	1.36	1.18	1.14	5.69
2	1.53	1.42	1.36	1.54	2.10	3.49	3.86	3.40	3.72	4.18	4.63	5.75	5.48	2.62	1.77	1.52	1.31	1.26	6.09
5	1.74	1.59	1.49	1.70	2.32	3.92	4.39	3.89	4.31	4.71	5.12	6.22	5.90	2.84	1.97	1.72	1.48	1.42	6.60
10	1.90	1.71	1.58	1.82	2.48	4.23	4.78	4.25	4.75	5.10	5.48	6.55	6.20	3.01	2.11	1.87	1.59	1.54	6.99
20	2.06	1.83	1.67	1.93	2.63	4.55	5.17	4.61	5.19	5.48	5.82	6.87	6.49	3.16	2.25	2.02	1.70	1.66	7.36
50	2.26	1.98	1.79	2.07	2.83	4.95	5.67	5.07	5.76	5.97	6.26	7.28	6.87	3.36	2.42	2.21	1.85	1.80	7.86
100	2.41	2.09	1.87	2.18	2.98	5.26	6.05	5.41	6.18	6.32	6.59	7.58	7.14	3.50	2.55	2.34	1.95	1.91	8.23
	Wave height in m																		

The corresponding mean period can be calculated from T_z 10.4 $\sqrt{H/g}$

Table 2.13 Extreme wave conditions at offshore point 3

Return Period (Yrs)	Wave direction in degrees North																	
	-10 10	10 30	30 50	50 70	70 90	90 110	110 130	130 150	150 170	170 190	190 210	210 230	230 250	250 270	270 290	290 310	310 330	330 350	0 to 360
1					0.53	1.03	2.91	2.88	3.34	3.82	4.17	3.46	1.64	1.10	0.96	0.74	0.67		4.51
2					0.61	1.15	3.26	3.24	3.75	4.20	4.46	3.71	1.76	1.18	1.07	0.82	0.75		4.86
5					0.71	1.30	3.74	3.72	4.28	4.69	4.83	4.04	1.92	1.28	1.20	0.92	0.85		5.31
10					0.78	1.42	4.10	4.08	4.67	5.05	5.10	4.29	2.03	1.35	1.30	0.99	0.93		5.65
20					0.86	1.53	4.46	4.44	5.06	5.40	5.35	4.52	2.14	1.41	1.40	1.06	1.01		5.99
50					0.97	1.68	4.95	4.90	5.56	5.85	5.68	4.83	2.28	1.49	1.53	1.15	1.11		6.43
100					1.05	1.79	5.32	5.25	5.93	6.18	5.92	5.06	2.38	1.55	1.62	1.21	1.18		6.77
	Wave height in m																		

The corresponding mean period can be calculated from T_z 10.6 $\sqrt{H/g}$

Table 2.14 Extreme wave conditions at offshore point 4

Return Period (Yrs)	Wave direction in degrees North																		
	-10 10	10 30	30 50	50 70	70 90	90 110	110 130	130 150	150 170	170 190	190 210	210 230	230 250	250 270	270 290	290 310	310 330	330 350	0 to 360
1	1.03	0.96	0.86	0.94	1.02	1.37	2.60	2.98	3.13	3.70	4.09	5.23	5.46	2.81	1.62	1.30	1.06	1.06	5.81
2	1.14	1.05	0.92	1.06	1.12	1.52	2.89	3.35	3.53	4.11	4.48	5.60	5.81	3.02	1.77	1.46	1.18	1.17	6.24
5	1.27	1.16	1.00	1.20	1.25	1.70	3.27	3.82	4.04	4.62	4.98	6.06	6.24	3.28	1.96	1.66	1.35	1.32	6.79
10	1.37	1.24	1.05	1.31	1.35	1.84	3.56	4.18	4.43	4.99	5.34	6.39	6.56	3.47	2.09	1.81	1.47	1.42	7.20
20	1.47	1.32	1.11	1.41	1.44	1.98	3.84	4.53	4.80	5.35	5.69	6.71	6.86	3.65	2.22	1.96	1.59	1.52	7.61
50	1.60	1.41	1.17	1.55	1.56	2.15	4.22	4.98	5.29	5.82	6.14	7.12	7.25	3.88	2.39	2.16	1.75	1.65	8.14
100	1.69	1.48	1.22	1.65	1.65	2.28	4.50	5.32	5.65	6.16	6.47	7.42	7.54	4.05	2.51	2.31	1.86	1.75	8.54
	Wave height in m																		

The corresponding mean period can be calculated from T_z 10.5 $\sqrt{H/g}$

Table 2.15 Extreme wave conditions at offshore point 5

Return Period (Yrs)	Wave direction in degrees North																	
	-10 10	10 30	30 50	50 70	70 90	90 110	110 130	130 150	150 170	170 190	190 210	210 230	230 250	250 270	270 290	290 310	310 330	330 350	0 to 360
1			0.46	0.48	0.57	0.77	1.95	2.85	3.06	3.79	4.00	5.11	5.61	3.37	1.45	0.95	0.77	0.52	5.83
2			0.52	0.54	0.64	0.86	2.24	3.22	3.47	4.22	4.38	5.49	5.95	3.60	1.59	1.06	0.87	0.66	6.23
5			0.59	0.62	0.72	0.99	2.62	3.71	4.00	4.77	4.85	5.97	6.38	3.88	1.76	1.21	1.06	0.76	6.74
10			0.65	0.68	0.79	1.08	2.93	4.08	4.40	5.17	5.20	6.31	6.68	4.09	1.89	1.32	1.10	0.84	7.13
20			0.70	0.73	0.85	1.17	3.23	4.45	4.80	5.56	5.53	6.64	6.98	4.29	2.02	1.43	1.19	0.92	7.50
50			0.77	0.81	0.92	1.28	3.65	4.94	5.31	6.06	5.96	7.06	7.36	4.54	2.18	1.57	1.32	1.02	7.99
100			0.83	0.86	0.98	1.37	3.97	5.31	5.70	6.43	6.27	7.36	7.64	4.73	2.30	1.67	1.41	1.09	8.35
	Wave height in m																		

The corresponding mean period can be calculated from T_z $10.3\sqrt{H/g}$

Table 2.16 Distribution of wave height and direction at inshore point A

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 = 159192
Based on HINDWAVE predictions for January 1974 – February 1992

Wave direction in degrees North

H1	To H2	P(H>H1)	-10 10	10 30	30 50	50 70	70 90	90 110	110 130	130 150	150 170	170 190	190 210	210 230	230 250	250 270	270 290	290 310	310 330	330 350
0.00	0.20	0.97748	2004	1717	1429	1256	1129	1221	1965	346	364	727	1650	1288	1940	1481	2376	1469	1797	2088
0.20	0.40	0.71500	866	940	1364	1414	2165	1866	1295	329	1009	1885	2379	3312	1509	2256	2468	1747	1375	883
0.40	0.60	0.42436	69	113	269	506	1680	2725	928	746	660	2136	2863	2722	794	602	332	330	226	127
0.60	0.80	0.24607	1	12	52	85	379	2064	1302	431	729	2201	2293	514	293	111	18	19	11	4
0.80	1.00	0.14088	0	2	4	25	99	1071	815	546	621	1933	616	55	19	8	1	0	0	0
1.00	1.20	0.08274	0	0	0	1	11	517	591	166	500	1063	239	11	0	0	0	0	0	0
1.20	1.40	0.05174	0	0	0	0	12	295	408	346	433	478	26	1	0	0	0	0	0	0
1.40	1.60	0.03174	0	0	0	0	1	128	319	84	158	198	13	0	0	0	0	0	0	0
1.60	1.80	0.02273	0	0	0	0	1	91	194	205	212	106	0	0	0	0	0	0	0	0
1.80	2.00	0.01464	0	0	0	0	0	40	163	36	117	24	0	0	0	0	0	0	0	0
2.00	2.20	0.01084	0	0	0	0	0	21	81	115	50	27	0	0	0	0	0	0	0	0
2.20	2.40	0.00789	0	0	0	0	0	21	73	82	41	16	0	0	0	0	0	0	0	0
2.40	2.60	0.00556	0	0	0	0	0	0	67	37	31	6	0	0	0	0	0	0	0	0
2.60	2.80	0.00416	0	0	0	0	0	1	40	23	3	4	0	0	0	0	0	0	0	0
2.80	3.00	0.00345	0	0	0	0	0	0	54	47	23	1	0	0	0	0	0	0	0	0
3.00	3.20	0.00219	0	0	0	0	0	0	17	16	3	3	0	0	0	0	0	0	0	0
3.20	3.40	0.00181	0	0	0	0	0	0	23	5	4	5	0	0	0	0	0	0	0	0
3.40	3.60	0.00144	0	0	0	0	0	0	23	21	1	0	0	0	0	0	0	0	0	0
3.60	3.80	0.00099	0	0	0	0	0	0	4	6	0	0	0	0	0	0	0	0	0	0
3.80	4.00	0.00089	0	0	0	0	0	0	19	4	2	0	0	0	0	0	0	0	0	0
4.00	4.20	0.00065	0	0	0	0	0	0	4	10	0	0	0	0	0	0	0	0	0	0
4.20	4.40	0.00050	0	0	0	0	0	0	4	0	0	0	0	0	0	0	0	0	0	0
4.40	4.60	0.00046	0	0	0	0	0	0	21	0	0	0	0	0	0	0	0	0	0	0
4.60	4.80	0.00025	0	0	0	0	0	0	3	3	0	0	0	0	0	0	0	0	0	0
4.80	5.00	0.00020	0	0	0	0	0	0	0	1	0	0	0	0	0	0	0	0	0	0
5.00	5.20	0.00019	0	0	0	0	0	0	6	0	0	0	0	0	0	0	0	0	0	0
5.20	5.40	0.00013	0	0	0	0	0	0	0	5	0	0	0	0	0	0	0	0	0	0
5.40	5.60	0.00008	0	0	0	0	0	0	1	1	0	0	0	0	0	0	0	0	0	0
5.60	5.80	0.00006	0	0	0	0	0	0	1	1	0	0	0	0	0	0	0	0	0	0
5.80	6.00	0.00004	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
6.00	6.20	0.00004	0	0	0	0	0	0	3	0	0	0	0	0	0	0	0	0	0	0
6.20	6.40	0.00001	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
6.40	6.60	0.00001	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
6.60	6.80	0.00001	0	0	0	0	0	0	1	0	0	0	0	0	0	0	0	0	0	0
Parts per thousand for each direction			30	28	32	34	56	103	86	37	51	111	103	81	47	46	53	36	35	32

Significant wave heights for given exceedence levels

Wave direction in degrees North

P(H>Hs)	All dir.	-10 10	10 30	30 50	50 70	70 90	90 110	110 130	130 150	150 170	170 190	190 210	210 230	230 250	250 270	270 290	290 310	310 330	330 350
0.50	0.35	0.14	0.16	0.21	0.25	0.34	0.53	0.58	0.76	0.71	0.65	0.46	0.36	0.24	0.26	0.21	0.23	0.19	0.15
0.20	0.69	0.28	0.31	0.35	0.39	0.53	0.82	1.13	1.38	1.23	0.97	0.70	0.52	0.44	0.38	0.34	0.36	0.33	0.29
0.10	0.94	0.35	0.37	0.40	0.51	0.59	1.03	1.56	2.01	1.57	1.16	0.79	0.58	0.56	0.49	0.39	0.40	0.38	0.36
0.05	1.22	0.38	0.40	0.52	0.58	0.72	1.26	2.03	2.39	1.84	1.33	0.92	0.67	0.65	0.56	0.45	0.50	0.46	0.39
0.02	1.67	0.43	0.52	0.59	0.70	0.82	1.55	2.84	3.00	2.23	1.57	1.06	0.76	0.75	0.65	0.55	0.57	0.55	0.51
0.01	2.06	0.51	0.57	0.69	0.78	0.94	1.76	3.43	3.55	2.50	1.75	1.15	0.79	0.78	0.73	0.58	0.59	0.58	0.56
Average	0.45	0.16	0.18	0.23	0.26	0.35	0.58	0.73	0.94	0.80	0.67	0.46	0.36	0.27	0.27	0.22	0.23	0.20	0.17

Table 2.17 Distribution of wave height and period at inshore point A

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 = 159192
Based on HINDWAVE predictions for January 1974 - February 1992

H1 To H2		P(H>H1)	Zero-crossing wave period in seconds (Tz)															
			0.0	1.0	2.0	3.0	4.0	5.0	6.0	7.0	8.0	9.0	10.0	11.0	12.0	13.0	14.0	15.0
			1.0	2.0	3.0	4.0	5.0	6.0	7.0	8.0	9.0	10.0	11.0	12.0	13.0	14.0	15.0	16.0
0.00	0.20	0.90726	1724	17503	0	0	0	0	0	0	0	0	0	0	0	0	0	0
0.20	0.40	0.71500	0	22409	6655	0	0	0	0	0	0	0	0	0	0	0	0	0
0.40	0.60	0.42436	0	1881	15947	0	0	0	0	0	0	0	0	0	0	0	0	0
0.60	0.80	0.24607	0	0	8126	2393	0	0	0	0	0	0	0	0	0	0	0	0
0.80	1.00	0.14088	0	0	972	4843	0	0	0	0	0	0	0	0	0	0	0	0
1.00	1.20	0.08274	0	0	16	3084	0	0	0	0	0	0	0	0	0	0	0	0
1.20	1.40	0.05174	0	0	0	1797	203	0	0	0	0	0	0	0	0	0	0	0
1.40	1.60	0.03174	0	0	0	169	731	0	0	0	0	0	0	0	0	0	0	0
1.60	1.80	0.02273	0	0	0	40	770	0	0	0	0	0	0	0	0	0	0	0
1.80	2.00	0.01464	0	0	0	2	378	0	0	0	0	0	0	0	0	0	0	0
2.00	2.20	0.01084	0	0	0	0	259	36	0	0	0	0	0	0	0	0	0	0
2.20	2.40	0.00789	0	0	0	0	140	93	0	0	0	0	0	0	0	0	0	0
2.40	2.60	0.00556	0	0	0	0	4	136	0	0	0	0	0	0	0	0	0	0
2.60	2.80	0.00416	0	0	0	0	2	65	4	0	0	0	0	0	0	0	0	0
2.80	3.00	0.00345	0	0	0	0	0	125	1	0	0	0	0	0	0	0	0	0
3.00	3.20	0.00219	0	0	0	0	0	33	5	0	0	0	0	0	0	0	0	0
3.20	3.40	0.00181	0	0	0	0	0	17	20	0	0	0	0	0	0	0	0	0
3.40	3.60	0.00144	0	0	0	0	0	5	40	0	0	0	0	0	0	0	0	0
3.60	3.80	0.00099	0	0	0	0	0	4	6	0	0	0	0	0	0	0	0	0
3.80	4.00	0.00089	0	0	0	0	0	1	24	0	0	0	0	0	0	0	0	0
4.00	4.20	0.00065	0	0	0	0	0	0	14	0	0	0	0	0	0	0	0	0
4.20	4.40	0.00050	0	0	0	0	0	0	4	0	0	0	0	0	0	0	0	0
4.40	4.60	0.00046	0	0	0	0	0	0	21	0	0	0	0	0	0	0	0	0
4.60	4.80	0.00025	0	0	0	0	0	0	5	0	0	0	0	0	0	0	0	0
4.80	5.00	0.00020	0	0	0	0	0	0	0	1	0	0	0	0	0	0	0	0
5.00	5.20	0.00019	0	0	0	0	0	0	3	4	0	0	0	0	0	0	0	0
5.20	5.40	0.00013	0	0	0	0	0	0	0	5	0	0	0	0	0	0	0	0
5.40	5.60	0.00008	0	0	0	0	0	0	0	2	0	0	0	0	0	0	0	0
5.60	5.80	0.00006	0	0	0	0	0	0	0	2	0	0	0	0	0	0	0	0
5.80	6.00	0.00004	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
6.00	6.20	0.00004	0	0	0	0	0	0	0	3	0	0	0	0	0	0	0	0
6.20	6.40	0.00001	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
6.40	6.60	0.00001	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
6.60	6.80	0.00001	0	0	0	0	0	0	0	0	1	0	0	0	0	0	0	0
Parts per thousand for each wave period			19	461	350	136	27	6	2	0	0	0	0	0	0	0	0	0

Table 2.18 Distribution of wave height and direction at inshore point B

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 = 159192
Based on HINDWAVE predictions for January 1974 - February 1992

H1	To H2	P(H>H1)	-10 10	10 30	30 50	50 70	70 90	90 110	110 130	130 150	150 170	170 190	190 210	210 230	230 250	250 270	270 290	290 310	310 330	330 350
			Wave direction in degrees North																	
0.00	0.20	0.97751	2057	1773	1427	1271	1469	1843	563	661	363	1023	1542	1774	2325	2224	2723	1340	1819	2084
0.20	0.40	0.69468	926	965	1381	1811	3185	2932	1383	1115	1827	1375	2383	1834	1285	766	1197	825	852	969
0.40	0.60	0.42456	75	142	315	612	1700	2724	753	1229	3075	1277	560	534	286	47	27	14	28	124
0.60	0.80	0.28934	3	19	82	96	575	1702	1142	1405	1282	347	30	172	57	4	0	0	0	8
0.80	1.00	0.22009	0	3	9	25	122	865	768	2530	1595	30	3	15	6	0	0	0	0	0
1.00	1.20	0.16038	0	0	0	1	15	361	564	825	883	4	1	0	0	0	0	0	0	0
1.20	1.40	0.13384	0	0	0	0	12	232	371	2663	577	7	0	0	0	0	0	0	0	0
1.40	1.60	0.09524	0	0	0	0	1	141	227	1579	139	1	0	0	0	0	0	0	0	0
1.60	1.80	0.07436	0	0	0	0	1	83	172	1918	69	0	0	0	0	0	0	0	0	0
1.80	2.00	0.05193	0	0	0	0	0	43	97	936	27	0	0	0	0	0	0	0	0	0
2.00	2.20	0.04091	0	0	0	0	0	17	136	1370	3	0	0	0	0	0	0	0	0	0
2.20	2.40	0.02565	0	0	0	0	0	6	93	598	1	0	0	0	0	0	0	0	0	0
2.40	2.60	0.01866	0	0	0	0	0	1	73	577	1	0	0	0	0	0	0	0	0	0
2.60	2.80	0.01214	0	0	0	0	0	1	36	423	0	0	0	0	0	0	0	0	0	0
2.80	3.00	0.00754	0	0	0	0	0	2	21	116	0	0	0	0	0	0	0	0	0	0
3.00	3.20	0.00615	0	0	0	0	0	0	35	258	0	0	0	0	0	0	0	0	0	0
3.20	3.40	0.00323	0	0	0	0	0	0	8	30	0	0	0	0	0	0	0	0	0	0
3.40	3.60	0.00286	0	0	0	0	0	0	10	117	0	0	0	0	0	0	0	0	0	0
3.60	3.80	0.00159	0	0	0	0	0	0	28	27	0	0	0	0	0	0	0	0	0	0
3.80	4.00	0.00104	0	0	0	0	0	0	1	41	0	0	0	0	0	0	0	0	0	0
4.00	4.20	0.00062	0	0	0	0	0	0	8	10	0	0	0	0	0	0	0	0	0	0
4.20	4.40	0.00043	0	0	0	0	0	0	1	11	0	0	0	0	0	0	0	0	0	0
4.40	4.60	0.00032	0	0	0	0	0	0	3	5	0	0	0	0	0	0	0	0	0	0
4.60	4.80	0.00024	0	0	0	0	0	0	2	9	0	0	0	0	0	0	0	0	0	0
4.80	5.00	0.00014	0	0	0	0	0	0	3	5	0	0	0	0	0	0	0	0	0	0
5.00	5.20	0.00006	0	0	0	0	0	0	0	4	0	0	0	0	0	0	0	0	0	0
5.20	5.40	0.00002	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
5.40	5.60	0.00002	0	0	0	0	0	0	1	1	0	0	0	0	0	0	0	0	0	0
Parts per thousand for each direction			31	30	33	39	72	112	66	189	101	42	46	44	41	31	40	22	28	33

Significant wave heights for given exceedence levels

P(H>Hs)	All dir.	-10 10	10 30	30 50	50 70	70 90	90 110	110 130	130 150	150 170	170 190	190 210	210 230	230 250	250 270	270 290	290 310	310 330	330 350
		Wave direction in degrees North																	
0.50	0.34	0.15	0.16	0.22	0.27	0.32	0.44	0.68	1.29	0.57	0.34	0.26	0.24	0.17	0.13	0.14	0.16	0.14	0.15
0.20	0.87	0.28	0.31	0.36	0.39	0.51	0.74	1.20	1.96	0.96	0.53	0.37	0.38	0.33	0.25	0.27	0.30	0.28	0.29
0.10	1.38	0.35	0.37	0.45	0.51	0.60	0.95	1.67	2.31	1.16	0.60	0.45	0.50	0.39	0.33	0.34	0.35	0.34	0.36
0.05	1.84	0.38	0.42	0.55	0.58	0.73	1.18	2.18	2.65	1.31	0.70	0.53	0.59	0.50	0.37	0.37	0.38	0.37	0.39
0.02	2.36	0.44	0.55	0.66	0.69	0.81	1.50	2.72	3.11	1.45	0.78	0.58	0.71	0.59	0.40	0.39	0.39	0.39	0.51
0.01	2.69	0.52	0.59	0.74	0.77	0.93	1.70	3.18	3.47	1.60	0.80	0.60	0.77	0.68	0.48	0.40	0.40	0.40	0.56
Average	0.55	0.17	0.19	0.24	0.27	0.35	0.50	0.82	1.32	0.65	0.35	0.25	0.25	0.20	0.15	0.16	0.17	0.16	0.17

Table 2.19 Distribution of wave height and period at inshore point B

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 = 159192
Based on HINDWAVE predictions for January 1974 - February 1992

H1	To H2	P(H>H1)	Zero-crossing wave period in seconds (Tz)															
			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	0.20	0.90788	2169	19150	0	0	0	0	0	0	0	0	0	0	0	0	0	0
0.20	0.40	0.69468	0	17614	9398	0	0	0	0	0	0	0	0	0	0	0	0	0
0.40	0.60	0.42456	0	497	12472	553	0	0	0	0	0	0	0	0	0	0	0	0
0.60	0.80	0.28934	0	0	3951	2974	0	0	0	0	0	0	0	0	0	0	0	0
0.80	1.00	0.22009	0	0	150	5457	364	0	0	0	0	0	0	0	0	0	0	0
1.00	1.20	0.16038	0	0	2	1579	1074	0	0	0	0	0	0	0	0	0	0	0
1.20	1.40	0.13384	0	0	0	329	3528	3	0	0	0	0	0	0	0	0	0	0
1.40	1.60	0.09524	0	0	0	37	1273	778	0	0	0	0	0	0	0	0	0	0
1.60	1.80	0.07436	0	0	0	1	821	1420	0	0	0	0	0	0	0	0	0	0
1.80	2.00	0.05193	0	0	0	1	244	857	0	0	0	0	0	0	0	0	0	0
2.00	2.20	0.04091	0	0	0	0	109	1417	0	0	0	0	0	0	0	0	0	0
2.20	2.40	0.02565	0	0	0	0	6	631	62	0	0	0	0	0	0	0	0	0
2.40	2.60	0.01866	0	0	0	0	2	512	138	0	0	0	0	0	0	0	0	0
2.60	2.80	0.01214	0	0	0	0	0	315	145	0	0	0	0	0	0	0	0	0
2.80	3.00	0.00754	0	0	0	0	0	41	98	0	0	0	0	0	0	0	0	0
3.00	3.20	0.00615	0	0	0	0	0	91	200	1	0	0	0	0	0	0	0	0
3.20	3.40	0.00323	0	0	0	0	0	5	28	4	0	0	0	0	0	0	0	0
3.40	3.60	0.00286	0	0	0	0	0	6	117	4	0	0	0	0	0	0	0	0
3.60	3.80	0.00159	0	0	0	0	0	2	51	3	0	0	0	0	0	0	0	0
3.80	4.00	0.00104	0	0	0	0	0	0	39	3	0	0	0	0	0	0	0	0
4.00	4.20	0.00062	0	0	0	0	0	0	18	0	0	0	0	0	0	0	0	0
4.20	4.40	0.00043	0	0	0	0	0	0	8	4	0	0	0	0	0	0	0	0
4.40	4.60	0.00032	0	0	0	0	0	0	4	3	0	0	0	0	0	0	0	0
4.60	4.80	0.00024	0	0	0	0	0	0	2	9	0	0	0	0	0	0	0	0
4.80	5.00	0.00014	0	0	0	0	0	0	0	8	0	0	0	0	0	0	0	0
5.00	5.20	0.00006	0	0	0	0	0	0	0	4	0	0	0	0	0	0	0	0
5.20	5.40	0.00002	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
5.40	5.60	0.00002	0	0	0	0	0	0	0	2	0	0	0	0	0	0	0	0
Parts per thousand for each wave period			24	410	286	120	82	67	10	0	0	0	0	0	0	0	0	0

Table 2.20 Distribution of wave height and direction at inshore point C

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 = 159192
Based on HINDWAVE predictions for January 1974 - February 1992

H1	To H2	P(H>H1)	-10 10	10 30	30 50	50 70	70 90	90 110	110 130	130 150	150 170	170 190	190 210	210 230	230 250	250 270	270 290	290 310	310 330	330 350
0.00	0.20	0.97749	2052	1731	1414	1265	1450	1216	1584	482	393	1016	1209	2134	2495	2234	2821	1423	1837	2065
0.20	0.40	0.68927	900	942	1357	1836	3101	2136	2065	1142	1405	1819	1916	2931	2817	2131	2378	1792	1305	883
0.40	0.60	0.36072	73	118	274	673	1528	2145	1009	636	1406	1645	2118	1838	636	161	180	351	126	123
0.60	0.80	0.21031	1	11	51	106	576	1684	1252	618	1511	2006	1081	136	25	9	8	19	7	4
0.80	1.00	0.11928	0	2	4	19	165	756	694	575	1338	1249	291	52	3	0	0	0	0	0
1.00	1.20	0.06779	0	0	0	1	33	336	432	486	879	408	75	8	0	0	0	0	0	0
1.20	1.40	0.04121	0	0	0	0	10	197	209	320	695	199	6	0	0	0	0	0	0	0
1.40	1.60	0.02486	0	0	0	0	3	95	149	452	359	18	0	0	0	0	0	0	0	0
1.60	1.80	0.01411	0	0	0	0	0	67	97	95	137	11	0	0	0	0	0	0	0	0
1.80	2.00	0.01003	0	0	0	0	0	3	75	200	91	0	0	0	0	0	0	0	0	0
2.00	2.20	0.00634	0	0	0	0	0	6	107	44	52	0	0	0	0	0	0	0	0	0
2.20	2.40	0.00426	0	0	0	0	0	2	33	126	16	0	0	0	0	0	0	0	0	0
2.40	2.60	0.00249	0	0	0	0	0	3	34	34	3	0	0	0	0	0	0	0	0	0
2.60	2.80	0.00177	0	0	0	0	0	0	24	37	2	0	0	0	0	0	0	0	0	0
2.80	3.00	0.00114	0	0	0	0	0	0	23	26	0	0	0	0	0	0	0	0	0	0
3.00	3.20	0.00065	0	0	0	0	0	0	7	18	0	0	0	0	0	0	0	0	0	0
3.20	3.40	0.00040	0	0	0	0	0	0	7	8	0	0	0	0	0	0	0	0	0	0
3.40	3.60	0.00026	0	0	0	0	0	0	3	2	0	0	0	0	0	0	0	0	0	0
3.60	3.80	0.00021	0	0	0	0	0	0	1	8	0	0	0	0	0	0	0	0	0	0
3.80	4.00	0.00013	0	0	0	0	0	0	1	0	0	0	0	0	0	0	0	0	0	0
4.00	4.20	0.00013	0	0	0	0	0	0	3	4	0	0	0	0	0	0	0	0	0	0
4.20	4.40	0.00005	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
4.40	4.60	0.00005	0	0	0	0	0	0	0	3	0	0	0	0	0	0	0	0	0	0
4.60	4.80	0.00003	0	0	0	0	0	0	1	1	0	0	0	0	0	0	0	0	0	0
4.80	5.00	0.00001	0	0	0	0	0	0	0	1	0	0	0	0	0	0	0	0	0	0
Parts per thousand for each direction			31	29	32	40	70	88	80	54	85	86	68	73	61	46	55	37	34	31

Significant wave heights for given exceedence levels

P(H>Hs)	All dir.	-10 10	10 30	30 50	50 70	70 90	90 110	110 130	130 150	150 170	170 190	190 210	210 230	230 250	250 270	270 290	290 310	310 330	330 350
0.50	0.32	0.14	0.16	0.21	0.27	0.32	0.48	0.43	0.71	0.71	0.55	0.41	0.29	0.23	0.20	0.19	0.24	0.17	0.15
0.20	0.62	0.28	0.31	0.36	0.40	0.52	0.76	0.89	1.38	1.12	0.83	0.62	0.46	0.36	0.33	0.32	0.36	0.32	0.29
0.10	0.87	0.35	0.37	0.41	0.52	0.63	0.95	1.19	1.73	1.35	0.96	0.74	0.54	0.42	0.37	0.37	0.40	0.37	0.36
0.05	1.13	0.38	0.40	0.52	0.58	0.75	1.16	1.63	2.17	1.53	1.10	0.82	0.58	0.51	0.39	0.39	0.51	0.39	0.39
0.02	1.49	0.43	0.53	0.59	0.69	0.89	1.40	2.15	2.59	1.79	1.26	0.96	0.67	0.57	0.50	0.49	0.57	0.50	0.50
0.01	1.80	0.52	0.57	0.69	0.76	0.97	1.58	2.53	2.92	1.97	1.34	1.03	0.78	0.59	0.55	0.55	0.59	0.56	0.56
Average	0.41	0.17	0.18	0.23	0.28	0.35	0.52	0.58	0.85	0.75	0.55	0.42	0.30	0.23	0.20	0.20	0.24	0.19	0.17

Table 2.21 Distribution of wave height and period at inshore point C

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 = 159192
Based on HINDWAVE predictions for January 1974 - February 1992

H1	To H2	P(H>H1)	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
			Zero-crossing wave period in seconds (Tz)															
0.00	0.20	0.90560	2169	19464	0	0	0	0	0	0	0	0	0	0	0	0	0	0
0.20	0.40	0.68927	0	24072	8783	0	0	0	0	0	0	0	0	0	0	0	0	0
0.40	0.60	0.36072	0	2208	12699	133	0	0	0	0	0	0	0	0	0	0	0	0
0.60	0.80	0.21031	0	0	6239	2864	0	0	0	0	0	0	0	0	0	0	0	0
0.80	1.00	0.11928	0	0	508	4640	1	0	0	0	0	0	0	0	0	0	0	0
1.00	1.20	0.06779	0	0	4	2446	208	0	0	0	0	0	0	0	0	0	0	0
1.20	1.40	0.04121	0	0	0	878	757	0	0	0	0	0	0	0	0	0	0	0
1.40	1.60	0.02486	0	0	0	111	964	0	0	0	0	0	0	0	0	0	0	0
1.60	1.80	0.01411	0	0	0	13	394	0	0	0	0	0	0	0	0	0	0	0
1.80	2.00	0.01003	0	0	0	0	345	24	0	0	0	0	0	0	0	0	0	0
2.00	2.20	0.00634	0	0	0	0	68	141	0	0	0	0	0	0	0	0	0	0
2.20	2.40	0.00426	0	0	0	0	21	155	0	0	0	0	0	0	0	0	0	0
2.40	2.60	0.00249	0	0	0	0	2	71	0	0	0	0	0	0	0	0	0	0
2.60	2.80	0.00177	0	0	0	0	0	63	0	0	0	0	0	0	0	0	0	0
2.80	3.00	0.00114	0	0	0	0	0	40	9	0	0	0	0	0	0	0	0	0
3.00	3.20	0.00065	0	0	0	0	0	13	12	0	0	0	0	0	0	0	0	0
3.20	3.40	0.00040	0	0	0	0	0	4	10	0	0	0	0	0	0	0	0	0
3.40	3.60	0.00026	0	0	0	0	0	0	4	0	0	0	0	0	0	0	0	0
3.60	3.80	0.00021	0	0	0	0	0	0	8	0	0	0	0	0	0	0	0	0
3.80	4.00	0.00013	0	0	0	0	0	0	1	0	0	0	0	0	0	0	0	0
4.00	4.20	0.00013	0	0	0	0	0	0	8	0	0	0	0	0	0	0	0	0
4.20	4.40	0.00005	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
4.40	4.60	0.00005	0	0	0	0	0	0	1	1	0	0	0	0	0	0	0	0
4.60	4.80	0.00003	0	0	0	0	0	0	1	1	0	0	0	0	0	0	0	0
4.80	5.00	0.00001	0	0	0	0	0	0	0	1	0	0	0	0	0	0	0	0
Parts per thousand for each wave period			24	505	312	122	30	6	1	0	0	0	0	0	0	0	0	0

Table 2.22 Distribution of wave height and direction at inshore point D

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 = 159192
Based on HINDWAVE predictions for January 1974 - February 1992

H1	To H2	P(H>H1)	Wave direction in degrees North																	
			-10 10	10 30	30 50	50 70	70 90	90 110	110 130	130 150	150 170	170 190	190 210	210 230	230 250	250 270	270 290	290 310	310 330	330 350
0.00	0.20	0.97748	2061	1695	1518	1383	1381	1178	1583	398	448	858	1707	1374	2085	1297	2836	1362	1806	2060
0.20	0.40	0.70719	885	932	1360	1920	3096	2031	1665	898	1246	1775	2324	2529	1929	1912	2579	1743	1405	882
0.40	0.60	0.39609	70	112	272	703	1586	2062	844	305	574	2104	2525	1053	757	261	165	318	222	125
0.60	0.80	0.25551	1	11	52	113	757	1530	1168	841	1001	1647	1529	334	61	20	6	19	11	4
0.80	1.00	0.16446	0	2	4	19	132	657	526	468	643	2824	411	177	18	1	0	0	0	0
1.00	1.20	0.10563	0	0	0	1	52	217	376	299	739	1327	52	23	1	0	0	0	0	0
1.20	1.40	0.07475	0	0	0	0	12	119	161	149	286	1008	28	0	0	0	0	0	0	0
1.40	1.60	0.05711	0	0	0	0	1	36	95	175	512	1327	2	0	0	0	0	0	0	0
1.60	1.80	0.03563	0	0	0	0	0	28	73	67	99	169	4	0	0	0	0	0	0	0
1.80	2.00	0.03123	0	0	0	0	0	0	50	97	385	565	0	0	0	0	0	0	0	0
2.00	2.20	0.02026	0	0	0	0	0	1	24	35	40	422	0	0	0	0	0	0	0	0
2.20	2.40	0.01505	0	0	0	0	0	0	26	40	173	85	0	0	0	0	0	0	0	0
2.40	2.60	0.01182	0	0	0	0	0	0	18	12	138	298	0	0	0	0	0	0	0	0
2.60	2.80	0.00716	0	0	0	0	0	0	7	19	43	18	0	0	0	0	0	0	0	0
2.80	3.00	0.00629	0	0	0	0	0	0	14	8	92	111	0	0	0	0	0	0	0	0
3.00	3.20	0.00404	0	0	0	0	0	0	2	13	63	71	0	0	0	0	0	0	0	0
3.20	3.40	0.00255	0	0	0	0	0	0	0	3	40	14	0	0	0	0	0	0	0	0
3.40	3.60	0.00199	0	0	0	0	0	0	0	2	52	3	0	0	0	0	0	0	0	0
3.60	3.80	0.00143	0	0	0	0	0	0	0	2	19	23	0	0	0	0	0	0	0	0
3.80	4.00	0.00099	0	0	0	0	0	0	0	5	28	4	0	0	0	0	0	0	0	0
4.00	4.20	0.00062	0	0	0	0	0	0	0	1	19	4	0	0	0	0	0	0	0	0
4.20	4.40	0.00038	0	0	0	0	0	0	0	1	8	4	0	0	0	0	0	0	0	0
4.40	4.60	0.00025	0	0	0	0	0	0	0	0	3	1	0	0	0	0	0	0	0	0
4.60	4.80	0.00022	0	0	0	0	0	0	0	0	4	0	0	0	0	0	0	0	0	0
4.80	5.00	0.00018	0	0	0	0	0	0	0	0	16	0	0	0	0	0	0	0	0	0
5.00	5.20	0.00002	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
5.20	5.40	0.00002	0	0	0	0	0	0	0	0	2	0	0	0	0	0	0	0	0	0
Parts per thousand for each direction			31	28	33	42	72	80	68	39	68	150	88	56	50	36	57	35	35	31

Significant wave heights for given exceedence levels

P(H>Hs)	All dir.	Wave direction in degrees North																	
		-10 10	10 30	30 50	50 70	70 90	90 110	110 130	130 150	150 170	170 190	190 210	210 230	230 250	250 270	270 290	290 310	310 330	330 350
0.50	0.33	0.14	0.16	0.21	0.27	0.33	0.46	0.40	0.66	0.80	0.86	0.41	0.30	0.23	0.24	0.19	0.24	0.19	0.15
0.20	0.72	0.28	0.31	0.35	0.40	0.54	0.73	0.81	1.09	1.54	1.42	0.64	0.49	0.38	0.35	0.32	0.36	0.33	0.29
0.10	1.04	0.35	0.37	0.40	0.52	0.66	0.88	1.09	1.50	2.22	1.84	0.75	0.59	0.49	0.39	0.37	0.40	0.38	0.36
0.05	1.47	0.38	0.40	0.52	0.58	0.76	1.00	1.36	1.88	2.81	2.15	0.83	0.75	0.56	0.48	0.39	0.50	0.45	0.39
0.02	2.01	0.42	0.52	0.59	0.69	0.88	1.24	1.82	2.33	3.46	2.57	0.96	0.90	0.59	0.56	0.47	0.57	0.55	0.51
0.01	2.48	0.51	0.57	0.69	0.76	0.99	1.37	2.18	2.75	3.88	2.95	1.00	0.96	0.70	0.59	0.54	0.59	0.58	0.56
Average	0.48	0.16	0.18	0.22	0.27	0.36	0.49	0.53	0.74	1.02	0.94	0.42	0.33	0.25	0.24	0.20	0.24	0.20	0.17

Table 2.23 Distribution of wave height and period at inshore point D

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 = 159192
Based on HINDWAVE predictions for January 1974 - February 1992

H1	To H2	P(H>H1)	Zero-crossing wave period in seconds (Tz)															
			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	0.20	0.90231	1724	17788	0	0	0	0	0	0	0	0	0	0	0	0	0	0
0.20	0.40	0.70719	0	21470	9640	0	0	0	0	0	0	0	0	0	0	0	0	0
0.40	0.60	0.39609	0	1521	12488	50	0	0	0	0	0	0	0	0	0	0	0	0
0.60	0.80	0.25551	0	0	6911	2194	0	0	0	0	0	0	0	0	0	0	0	0
0.80	1.00	0.16446	0	0	358	5524	1	0	0	0	0	0	0	0	0	0	0	0
1.00	1.20	0.10563	0	0	6	2132	951	0	0	0	0	0	0	0	0	0	0	0
1.20	1.40	0.07475	0	0	0	857	906	0	0	0	0	0	0	0	0	0	0	0
1.40	1.60	0.05711	0	0	0	127	1863	158	0	0	0	0	0	0	0	0	0	0
1.60	1.80	0.03563	0	0	0	16	388	35	0	0	0	0	0	0	0	0	0	0
1.80	2.00	0.03123	0	0	0	0	541	556	0	0	0	0	0	0	0	0	0	0
2.00	2.20	0.02026	0	0	0	0	120	401	0	0	0	0	0	0	0	0	0	0
2.20	2.40	0.01505	0	0	0	0	35	256	32	0	0	0	0	0	0	0	0	0
2.40	2.60	0.01182	0	0	0	0	9	264	193	0	0	0	0	0	0	0	0	0
2.60	2.80	0.00716	0	0	0	0	1	79	8	0	0	0	0	0	0	0	0	0
2.80	3.00	0.00629	0	0	0	0	0	97	128	0	0	0	0	0	0	0	0	0
3.00	3.20	0.00404	0	0	0	0	0	32	112	4	0	0	0	0	0	0	0	0
3.20	3.40	0.00255	0	0	0	0	0	15	41	0	0	0	0	0	0	0	0	0
3.40	3.60	0.00199	0	0	0	0	0	0	55	1	0	0	0	0	0	0	0	0
3.60	3.80	0.00143	0	0	0	0	0	1	43	1	0	0	0	0	0	0	0	0
3.80	4.00	0.00099	0	0	0	0	0	0	28	9	0	0	0	0	0	0	0	0
4.00	4.20	0.00062	0	0	0	0	0	0	20	4	0	0	0	0	0	0	0	0
4.20	4.40	0.00038	0	0	0	0	0	0	8	5	0	0	0	0	0	0	0	0
4.40	4.60	0.00025	0	0	0	0	0	0	2	1	0	0	0	0	0	0	0	0
4.60	4.80	0.00022	0	0	0	0	0	0	1	4	0	0	0	0	0	0	0	0
4.80	5.00	0.00018	0	0	0	0	0	0	1	14	0	0	0	0	0	0	0	0
5.00	5.20	0.00002	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
5.20	5.40	0.00002	0	0	0	0	0	0	0	2	0	0	0	0	0	0	0	0
Parts per thousand for each wave period			19	452	326	121	53	21	7	1	0	0	0	0	0	0	0	0

Table 2.24 Distribution of wave height and direction at inshore point E

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 = 159192
Based on HINDWAVE predictions for January 1974 - February 1992

H1	To H2	P(H>H1)	-10 10	10 30	30 50	50 70	70 90	90 110	110 130	130 150	150 170	170 190	190 210	210 230	230 250	250 270	270 290	290 310	310 330	330 350
0.00	0.20	0.97749	1999	1689	1441	1407	1311	1156	1351	619	452	655	1721	1769	1354	1312	2425	1405	1768	2086
0.20	0.40	0.71827	867	933	1356	1942	3037	2045	1608	805	1081	1817	2559	1882	2080	1261	1135	1650	1217	870
0.40	0.60	0.43681	67	112	272	699	1476	2036	941	313	211	2076	2868	1074	1308	196	160	351	121	123
0.60	0.80	0.29276	1	10	52	109	332	1543	1249	729	1102	2999	3309	176	380	69	18	17	7	4
0.80	1.00	0.17169	0	2	5	17	28	542	666	242	475	2525	1663	131	38	7	0	0	0	0
1.00	1.20	0.10827	0	0	0	1	13	170	388	337	496	2736	380	19	3	0	0	0	0	0
1.20	1.40	0.06285	0	0	0	0	3	64	219	224	255	1427	89	1	0	0	0	0	0	0
1.40	1.60	0.04004	0	0	0	0	1	92	100	125	302	801	29	0	0	0	0	0	0	0
1.60	1.80	0.02555	0	0	0	0	0	21	66	154	289	379	4	0	0	0	0	0	0	0
1.80	2.00	0.01641	0	0	0	0	0	25	34	94	228	169	0	0	0	0	0	0	0	0
2.00	2.20	0.01091	0	0	0	0	0	0	23	62	183	86	0	0	0	0	0	0	0	0
2.20	2.40	0.00737	0	0	0	0	0	0	3	73	103	46	0	0	0	0	0	0	0	0
2.40	2.60	0.00512	0	0	0	0	0	0	18	35	46	26	0	0	0	0	0	0	0	0
2.60	2.80	0.00388	0	0	0	0	0	0	13	31	93	9	0	0	0	0	0	0	0	0
2.80	3.00	0.00242	0	0	0	0	0	0	2	29	34	1	0	0	0	0	0	0	0	0
3.00	3.20	0.00176	0	0	0	0	0	0	4	11	20	2	0	0	0	0	0	0	0	0
3.20	3.40	0.00138	0	0	0	0	0	0	4	13	41	0	0	0	0	0	0	0	0	0
3.40	3.60	0.00080	0	0	0	0	0	0	0	1	1	0	0	0	0	0	0	0	0	0
3.60	3.80	0.00079	0	0	0	0	0	0	0	22	18	0	0	0	0	0	0	0	0	0
3.80	4.00	0.00039	0	0	0	0	0	0	0	3	6	0	0	0	0	0	0	0	0	0
4.00	4.20	0.00030	0	0	0	0	0	0	0	4	5	0	0	0	0	0	0	0	0	0
4.20	4.40	0.00021	0	0	0	0	0	0	0	3	3	0	0	0	0	0	0	0	0	0
4.40	4.60	0.00015	0	0	0	0	0	0	0	1	0	0	0	0	0	0	0	0	0	0
4.60	4.80	0.00014	0	0	0	0	0	0	0	1	1	0	0	0	0	0	0	0	0	0
4.80	5.00	0.00012	0	0	0	0	0	0	0	1	2	0	0	0	0	0	0	0	0	0
5.00	5.20	0.00009	0	0	0	0	0	0	0	3	1	0	0	0	0	0	0	0	0	0
5.20	5.40	0.00005	0	0	0	0	0	0	0	0	3	0	0	0	0	0	0	0	0	0
5.40	5.60	0.00003	0	0	0	0	0	0	0	0	1	0	0	0	0	0	0	0	0	0
5.60	5.80	0.00002	0	0	0	0	0	0	0	1	0	0	0	0	0	0	0	0	0	0
5.80	6.00	0.00001	0	0	0	0	0	0	0	0	1	0	0	0	0	0	0	0	0	0
Parts per thousand for each direction			30	28	32	43	63	79	68	40	56	161	129	52	53	29	38	35	32	32

Significant wave heights for given exceedence levels

P(H>Hs)	All dir.	-10 10	10 30	30 50	50 70	70 90	90 110	110 130	130 150	150 170	170 190	190 210	210 230	230 250	250 270	270 290	290 310	310 330	330 350
0.50	0.36	0.14	0.16	0.21	0.27	0.31	0.45	0.47	0.65	0.77	0.81	0.53	0.27	0.31	0.21	0.15	0.23	0.17	0.14
0.20	0.75	0.28	0.31	0.35	0.40	0.48	0.71	0.85	1.28	1.58	1.18	0.77	0.47	0.50	0.35	0.30	0.36	0.32	0.28
0.10	1.04	0.35	0.37	0.41	0.51	0.57	0.85	1.10	1.78	2.00	1.39	0.91	0.56	0.58	0.40	0.36	0.41	0.37	0.36
0.05	1.31	0.38	0.40	0.52	0.58	0.64	0.99	1.33	2.28	2.39	1.58	0.98	0.68	0.68	0.53	0.40	0.51	0.39	0.39
0.02	1.72	0.43	0.52	0.59	0.68	0.75	1.34	1.69	2.88	2.95	1.82	1.13	0.87	0.77	0.65	0.53	0.57	0.51	0.50
0.01	2.05	0.51	0.57	0.70	0.75	0.79	1.53	1.99	3.38	3.33	2.02	1.20	0.95	0.79	0.74	0.57	0.59	0.56	0.56
Average	0.48	0.16	0.18	0.23	0.27	0.32	0.48	0.55	0.81	0.97	0.83	0.52	0.30	0.33	0.23	0.18	0.23	0.19	0.17

Table 2.25 Distribution of wave height and period at inshore point E

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 = 159192
Based on HINDWAVE predictions for January 1974 - February 1992

H1	To H2	P(H>H1)	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
			Zero-crossing wave period in seconds (Tz)															
0.00	0.20	0.90601	1742	17032	0	0	0	0	0	0	0	0	0	0	0	0	0	0
0.20	0.40	0.71827	0	18394	9752	0	0	0	0	0	0	0	0	0	0	0	0	0
0.40	0.60	0.43681	0	1202	13192	11	0	0	0	0	0	0	0	0	0	0	0	0
0.60	0.80	0.29276	0	0	6879	5228	0	0	0	0	0	0	0	0	0	0	0	0
0.80	1.00	0.17169	0	0	731	5611	0	0	0	0	0	0	0	0	0	0	0	0
1.00	1.20	0.10827	0	0	18	4507	16	0	0	0	0	0	0	0	0	0	0	0
1.20	1.40	0.06285	0	0	0	1756	525	0	0	0	0	0	0	0	0	0	0	0
1.40	1.60	0.04004	0	0	0	273	1177	0	0	0	0	0	0	0	0	0	0	0
1.60	1.80	0.02555	0	0	0	68	846	0	0	0	0	0	0	0	0	0	0	0
1.80	2.00	0.01641	0	0	0	4	518	27	0	0	0	0	0	0	0	0	0	0
2.00	2.20	0.01091	0	0	0	0	269	85	0	0	0	0	0	0	0	0	0	0
2.20	2.40	0.00737	0	0	0	0	91	134	0	0	0	0	0	0	0	0	0	0
2.40	2.60	0.00512	0	0	0	0	24	101	0	0	0	0	0	0	0	0	0	0
2.60	2.80	0.00388	0	0	0	0	4	142	0	0	0	0	0	0	0	0	0	0
2.80	3.00	0.00242	0	0	0	0	0	66	0	0	0	0	0	0	0	0	0	0
3.00	3.20	0.00176	0	0	0	0	0	28	10	0	0	0	0	0	0	0	0	0
3.20	3.40	0.00138	0	0	0	0	0	22	36	0	0	0	0	0	0	0	0	0
3.40	3.60	0.00080	0	0	0	0	0	1	1	0	0	0	0	0	0	0	0	0
3.60	3.80	0.00079	0	0	0	0	0	4	36	0	0	0	0	0	0	0	0	0
3.80	4.00	0.00039	0	0	0	0	0	0	9	0	0	0	0	0	0	0	0	0
4.00	4.20	0.00030	0	0	0	0	0	0	9	0	0	0	0	0	0	0	0	0
4.20	4.40	0.00021	0	0	0	0	0	0	6	0	0	0	0	0	0	0	0	0
4.40	4.60	0.00015	0	0	0	0	0	0	1	0	0	0	0	0	0	0	0	0
4.60	4.80	0.00014	0	0	0	0	0	0	1	1	0	0	0	0	0	0	0	0
4.80	5.00	0.00012	0	0	0	0	0	0	1	2	0	0	0	0	0	0	0	0
5.00	5.20	0.00009	0	0	0	0	0	0	0	4	0	0	0	0	0	0	0	0
5.20	5.40	0.00005	0	0	0	0	0	0	0	3	0	0	0	0	0	0	0	0
5.40	5.60	0.00003	0	0	0	0	0	0	0	1	0	0	0	0	0	0	0	0
5.60	5.80	0.00002	0	0	0	0	0	0	0	1	0	0	0	0	0	0	0	0
5.80	6.00	0.00001	0	0	0	0	0	0	0	1	0	0	0	0	0	0	0	0
Parts per thousand for each wave period			19	404	337	193	38	7	1	0	0	0	0	0	0	0	0	0

Table 2.26 Distribution of wave height and direction at inshore point F

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 = 159192
Based on HINDWAVE predictions for January 1974 - February 1992

Wave direction in degrees North

H1	To H2	P(H>H1)	-10 10	10 30	30 50	50 70	70 90	90 110	110 130	130 150	150 170	170 190	190 210	210 230	230 250	250 270	270 290	290 310	310 330	330 350
0.00	0.20	0.96568	526	835	702	222	101	1443	6419	1205	624	780	1172	900	2596	7835	3506	724	1030	646
0.20	0.40	0.65302	0	0	0	0	0	90	4904	2670	668	994	1031	2144	3664	4702	298	0	3	0
0.40	0.60	0.44135	0	0	0	0	0	0	1964	1342	762	517	1212	2738	4351	2197	4	0	0	0
0.60	0.80	0.29047	0	0	0	0	0	0	672	1967	544	778	1389	2949	2433	426	0	0	0	0
0.80	1.00	0.17891	0	0	0	0	0	0	223	964	766	714	1178	2216	719	18	0	0	0	0
1.00	1.20	0.11092	0	0	0	0	0	0	35	744	258	579	1307	1221	136	2	0	0	0	0
1.20	1.40	0.06809	0	0	0	0	0	0	5	255	351	513	639	533	21	0	0	0	0	0
1.40	1.60	0.04492	0	0	0	0	0	0	0	215	255	467	661	209	0	0	0	0	0	0
1.60	1.80	0.02684	0	0	0	0	0	0	0	155	178	205	279	81	0	0	0	0	0	0
1.80	2.00	0.01785	0	0	0	0	0	0	0	26	126	169	287	13	0	0	0	0	0	0
2.00	2.20	0.01164	0	0	0	0	0	0	0	109	89	153	73	4	0	0	0	0	0	0
2.20	2.40	0.00737	0	0	0	0	0	0	0	24	17	84	60	0	0	0	0	0	0	0
2.40	2.60	0.00552	0	0	0	0	0	0	0	48	77	66	52	0	0	0	0	0	0	0
2.60	2.80	0.00310	0	0	0	0	0	0	0	11	6	44	15	0	0	0	0	0	0	0
2.80	3.00	0.00234	0	0	0	0	0	0	0	13	38	26	9	0	0	0	0	0	0	0
3.00	3.20	0.00147	0	0	0	0	0	0	0	2	1	14	7	0	0	0	0	0	0	0
3.20	3.40	0.00124	0	0	0	0	0	0	0	23	23	20	6	0	0	0	0	0	0	0
3.40	3.60	0.00052	0	0	0	0	0	0	0	5	0	1	1	0	0	0	0	0	0	0
3.60	3.80	0.00044	0	0	0	0	0	0	0	3	5	5	2	0	0	0	0	0	0	0
3.80	4.00	0.00029	0	0	0	0	0	0	0	2	1	3	5	0	0	0	0	0	0	0
4.00	4.20	0.00019	0	0	0	0	0	0	0	1	3	2	0	0	0	0	0	0	0	0
4.20	4.40	0.00014	0	0	0	0	0	0	0	4	1	1	0	0	0	0	0	0	0	0
4.40	4.60	0.00008	0	0	0	0	0	0	0	3	0	0	0	0	0	0	0	0	0	0
4.60	4.80	0.00005	0	0	0	0	0	0	0	0	3	0	0	0	0	0	0	0	0	0
4.80	5.00	0.00002	0	0	0	0	0	0	0	1	0	0	0	0	0	0	0	0	0	0
5.00	5.20	0.00001	0	0	0	0	0	0	0	0	1	0	0	0	0	0	0	0	0	0
Parts per thousand for each direction			5	9	7	2	1	16	147	101	50	64	97	135	144	157	39	7	11	7

Significant wave heights for given exceedence levels

Wave direction in degrees North

P(H>Hs)	All dir.	-10 10	10 30	30 50	50 70	70 90	90 110	110 130	130 150	150 170	170 190	190 210	210 230	230 250	250 270	270 290	290 310	310 330	330 350
0.50	0.34	0.00	0.00	0.00	0.00	0.00	0.10	0.22	0.53	0.69	0.77	0.76	0.63	0.42	0.19	0.10	0.00	0.10	0.00
0.20	0.76	0.00	0.00	0.00	0.00	0.00	0.17	0.40	0.92	1.30	1.40	1.25	0.94	0.64	0.38	0.17	0.00	0.16	0.00
0.10	1.05	0.00	0.00	0.00	0.00	0.00	0.19	0.55	1.17	1.68	1.75	1.55	1.12	0.75	0.50	0.19	0.00	0.18	0.00
0.05	1.36	0.00	0.00	0.00	0.00	0.00	0.22	0.66	1.53	2.03	2.13	1.82	1.26	0.84	0.57	0.27	0.00	0.19	0.00
0.02	1.75	0.00	0.00	0.00	0.00	0.00	0.00	0.79	2.08	2.55	2.57	2.10	1.44	0.96	0.66	0.35	0.00	0.20	0.00
0.01	2.08	0.00	0.00	0.00	0.00	0.00	0.00	0.90	2.46	2.93	2.86	2.40	1.56	1.02	0.74	0.38	0.00	0.20	0.00
Average	0.47	0.10	0.10	0.10	0.10	0.10	0.11	0.26	0.62	0.82	0.87	0.81	0.64	0.42	0.23	0.11	0.10	0.10	0.10

Table 2.27 Distribution of wave height and period at inshore point F

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 = 159192
Based on HINDWAVE predictions for January 1974 – February 1992

H1	To H2	P(H>H1)	Zero-crossing wave period in seconds (Tz)															
			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	0.20	0.86178	4435	14666	1775	0	0	0	0	0	0	0	0	0	0	0	0	0
0.20	0.40	0.65302	0	6919	13856	392	0	0	0	0	0	0	0	0	0	0	0	0
0.40	0.60	0.44135	0	4	13329	1755	0	0	0	0	0	0	0	0	0	0	0	0
0.60	0.80	0.29047	0	0	6183	4948	25	0	0	0	0	0	0	0	0	0	0	0
0.80	1.00	0.17891	0	0	396	6324	79	0	0	0	0	0	0	0	0	0	0	0
1.00	1.20	0.11092	0	0	2	3777	504	0	0	0	0	0	0	0	0	0	0	0
1.20	1.40	0.06809	0	0	0	1680	637	0	0	0	0	0	0	0	0	0	0	0
1.40	1.60	0.04492	0	0	0	200	1586	22	0	0	0	0	0	0	0	0	0	0
1.60	1.80	0.02684	0	0	0	21	812	67	0	0	0	0	0	0	0	0	0	0
1.80	2.00	0.01785	0	0	0	0	531	90	0	0	0	0	0	0	0	0	0	0
2.00	2.20	0.01164	0	0	0	0	200	226	0	0	0	0	0	0	0	0	0	0
2.20	2.40	0.00737	0	0	0	0	40	138	8	0	0	0	0	0	0	0	0	0
2.40	2.60	0.00552	0	0	0	0	13	221	8	0	0	0	0	0	0	0	0	0
2.60	2.80	0.00310	0	0	0	0	1	67	9	0	0	0	0	0	0	0	0	0
2.80	3.00	0.00234	0	0	0	0	0	76	11	0	0	0	0	0	0	0	0	0
3.00	3.20	0.00147	0	0	0	0	0	16	7	0	0	0	0	0	0	0	0	0
3.20	3.40	0.00124	0	0	0	0	0	16	56	0	0	0	0	0	0	0	0	0
3.40	3.60	0.00052	0	0	0	0	0	1	6	0	0	0	0	0	0	0	0	0
3.60	3.80	0.00044	0	0	0	0	0	0	15	0	0	0	0	0	0	0	0	0
3.80	4.00	0.00029	0	0	0	0	0	0	8	3	0	0	0	0	0	0	0	0
4.00	4.20	0.00019	0	0	0	0	0	0	5	0	0	0	0	0	0	0	0	0
4.20	4.40	0.00014	0	0	0	0	0	0	5	1	0	0	0	0	0	0	0	0
4.40	4.60	0.00008	0	0	0	0	0	0	0	3	0	0	0	0	0	0	0	0
4.60	4.80	0.00005	0	0	0	0	0	0	0	3	0	0	0	0	0	0	0	0
4.80	5.00	0.00002	0	0	0	0	0	0	0	1	0	0	0	0	0	0	0	0
5.00	5.20	0.00001	0	0	0	0	0	0	0	1	0	0	0	0	0	0	0	0
Parts per thousand for each wave period			51	251	412	222	51	11	2	0	0	0	0	0	0	0	0	0

Table 2.28 Distribution of wave height and direction at inshore point G

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 = 159192
Based on HINDWAVE predictions for January 1974 - February 1992

H1	To H2	P(H>H1)	-10 10	10 30	30 50	50 70	70 90	90 110	110 130	130 150	150 170	170 190	190 210	210 230	230 250	250 270	270 290	290 310	310 330	330 350
0.00	0.20	0.96537	526	841	700	214	502	244	5036	4723	649	545	1714	1316	3604	7710	3539	725	1030	646
0.20	0.40	0.62274	0	0	0	0	4	3	628	6953	1110	1395	1236	2197	3729	3319	297	0	3	0
0.40	0.60	0.41402	0	0	0	0	0	0	3	2033	1030	273	1092	3729	2998	693	4	0	0	0
0.60	0.80	0.29547	0	0	0	0	0	0	0	376	957	1145	1297	2473	1317	107	0	0	0	0
0.80	1.00	0.21875	0	0	0	0	0	0	0	18	781	447	1466	2452	173	1	0	0	0	0
1.00	1.20	0.16538	0	0	0	0	0	0	0	6	432	527	1603	1571	57	0	0	0	0	0
1.20	1.40	0.12343	0	0	0	0	0	0	0	0	146	332	1621	714	17	0	0	0	0	0
1.40	1.60	0.09514	0	0	0	0	0	0	0	0	103	237	1433	800	1	0	0	0	0	0
1.60	1.80	0.06940	0	0	0	0	0	0	0	0	133	302	1656	254	0	0	0	0	0	0
1.80	2.00	0.04595	0	0	0	0	0	0	0	0	24	55	1034	26	0	0	0	0	0	0
2.00	2.20	0.03456	0	0	0	0	0	0	0	0	52	267	1012	23	0	0	0	0	0	0
2.20	2.40	0.02102	0	0	0	0	0	0	0	0	7	12	550	3	0	0	0	0	0	0
2.40	2.60	0.01531	0	0	0	0	0	0	0	0	26	132	388	0	0	0	0	0	0	0
2.60	2.80	0.00986	0	0	0	0	0	0	0	0	5	12	278	0	0	0	0	0	0	0
2.80	3.00	0.00691	0	0	0	0	0	0	0	0	3	76	166	0	0	0	0	0	0	0
3.00	3.20	0.00446	0	0	0	0	0	0	0	0	3	16	84	0	0	0	0	0	0	0
3.20	3.40	0.00344	0	0	0	0	0	0	0	0	1	26	76	0	0	0	0	0	0	0
3.40	3.60	0.00240	0	0	0	0	0	0	0	0	2	24	43	0	0	0	0	0	0	0
3.60	3.80	0.00170	0	0	0	0	0	0	0	0	3	2	28	0	0	0	0	0	0	0
3.80	4.00	0.00138	0	0	0	0	0	0	0	0	1	25	36	0	0	0	0	0	0	0
4.00	4.20	0.00076	0	0	0	0	0	0	0	0	0	4	9	0	0	0	0	0	0	0
4.20	4.40	0.00063	0	0	0	0	0	0	0	0	0	1	16	0	0	0	0	0	0	0
4.40	4.60	0.00045	0	0	0	0	0	0	0	0	0	6	4	0	0	0	0	0	0	0
4.60	4.80	0.00035	0	0	0	0	0	0	0	0	0	0	4	0	0	0	0	0	0	0
4.80	5.00	0.00030	0	0	0	0	0	0	0	0	0	8	3	0	0	0	0	0	0	0
5.00	5.20	0.00020	0	0	0	0	0	0	0	0	0	1	6	0	0	0	0	0	0	0
5.20	5.40	0.00013	0	0	0	0	0	0	0	0	0	3	1	0	0	0	0	0	0	0
5.40	5.60	0.00009	0	0	0	0	0	0	0	0	0	0	1	0	0	0	0	0	0	0
5.60	5.80	0.00009	0	0	0	0	0	0	0	0	0	0	1	0	0	0	0	0	0	0
5.80	6.00	0.00008	0	0	0	0	0	0	0	0	0	1	2	0	0	0	0	0	0	0
6.00	6.20	0.00006	0	0	0	0	0	0	0	0	0	0	1	0	0	0	0	0	0	0
6.20	6.40	0.00005	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
6.40	6.60	0.00005	0	0	0	0	0	0	0	0	0	0	5	0	0	0	0	0	0	0
Parts per thousand for each direction			5	9	7	2	5	3	59	146	57	61	175	161	123	123	40	8	11	7

Wave direction in degrees North

Significant wave heights for given exceedence levels

P(H>Hs)	All dir.	-10 10	10 30	30 50	50 70	70 90	90 110	110 130	130 150	150 170	170 190	190 210	210 230	230 250	250 270	270 290	290 310	310 330	330 350
0.50	0.32	0.00	0.00	0.00	0.00	0.10	0.10	0.11	0.26	0.57	0.71	1.17	0.62	0.31	0.15	0.10	0.00	0.10	0.00
0.20	0.87	0.00	0.00	0.00	0.00	0.16	0.16	0.18	0.39	0.95	1.40	1.85	1.02	0.54	0.30	0.17	0.00	0.16	0.00
0.10	1.37	0.00	0.00	0.00	0.00	0.18	0.18	0.21	0.50	1.17	2.01	2.19	1.26	0.65	0.37	0.19	0.00	0.18	0.00
0.05	1.77	0.00	0.00	0.00	0.00	0.19	0.19	0.31	0.57	1.55	2.45	2.54	1.48	0.74	0.45	0.27	0.00	0.19	0.00
0.02	2.24	0.00	0.00	0.00	0.00	0.20	0.20	0.36	0.66	1.90	2.99	2.96	1.60	0.80	0.56	0.35	0.00	0.20	0.00
0.01	2.59	0.00	0.00	0.00	0.00	0.20	0.24	0.38	0.73	2.17	3.51	3.36	1.71	0.94	0.60	0.38	0.00	0.20	0.00
Average	0.53	0.10	0.10	0.10	0.10	0.10	0.10	0.12	0.26	0.64	0.89	1.19	0.68	0.34	0.18	0.11	0.10	0.10	0.10

Table 2.29 Distribution of wave height and period at inshore point G

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 = 159192
Based on HINDWAVE predictions for January 1974 - February 1992

H1	To H2	P(H>H1)	Zero-crossing wave period in seconds (Tz)															
			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	0.20	0.83399	4439	14850	1836	0	0	0	0	0	0	0	0	0	0	0	0	0
0.20	0.40	0.62274	0	5837	14680	356	0	0	0	0	0	0	0	0	0	0	0	0
0.40	0.60	0.41402	0	5	10901	949	0	0	0	0	0	0	0	0	0	0	0	0
0.60	0.80	0.29547	0	0	2823	4849	0	0	0	0	0	0	0	0	0	0	0	0
0.80	1.00	0.21875	0	0	57	5186	94	0	0	0	0	0	0	0	0	0	0	0
1.00	1.20	0.16538	0	0	0	3385	810	0	0	0	0	0	0	0	0	0	0	0
1.20	1.40	0.12343	0	0	0	867	1954	9	0	0	0	0	0	0	0	0	0	0
1.40	1.60	0.09514	0	0	0	67	2491	16	0	0	0	0	0	0	0	0	0	0
1.60	1.80	0.06940	0	0	0	8	2241	95	0	0	0	0	0	0	0	0	0	0
1.80	2.00	0.04595	0	0	0	0	482	653	4	0	0	0	0	0	0	0	0	0
2.00	2.20	0.03456	0	0	0	0	457	893	4	0	0	0	0	0	0	0	0	0
2.20	2.40	0.02102	0	0	0	0	46	525	0	0	0	0	0	0	0	0	0	0
2.40	2.60	0.01531	0	0	0	0	16	509	21	0	0	0	0	0	0	0	0	0
2.60	2.80	0.00986	0	0	0	0	1	290	4	0	0	0	0	0	0	0	0	0
2.80	3.00	0.00691	0	0	0	0	1	163	82	0	0	0	0	0	0	0	0	0
3.00	3.20	0.00446	0	0	0	0	0	59	43	0	0	0	0	0	0	0	0	0
3.20	3.40	0.00344	0	0	0	0	0	14	90	0	0	0	0	0	0	0	0	0
3.40	3.60	0.00240	0	0	0	0	0	12	58	0	0	0	0	0	0	0	0	0
3.60	3.80	0.00170	0	0	0	0	0	2	31	0	0	0	0	0	0	0	0	0
3.80	4.00	0.00138	0	0	0	0	0	0	57	5	0	0	0	0	0	0	0	0
4.00	4.20	0.00076	0	0	0	0	0	0	13	1	0	0	0	0	0	0	0	0
4.20	4.40	0.00063	0	0	0	0	0	0	4	13	0	0	0	0	0	0	0	0
4.40	4.60	0.00045	0	0	0	0	0	0	6	4	0	0	0	0	0	0	0	0
4.60	4.80	0.00035	0	0	0	0	0	0	1	4	0	0	0	0	0	0	0	0
4.80	5.00	0.00030	0	0	0	0	0	0	0	10	0	0	0	0	0	0	0	0
5.00	5.20	0.00020	0	0	0	0	0	0	0	6	1	0	0	0	0	0	0	0
5.20	5.40	0.00013	0	0	0	0	0	0	0	4	0	0	0	0	0	0	0	0
5.40	5.60	0.00009	0	0	0	0	0	0	0	1	0	0	0	0	0	0	0	0
5.60	5.80	0.00009	0	0	0	0	0	0	0	0	1	0	0	0	0	0	0	0
5.80	6.00	0.00008	0	0	0	0	0	0	0	1	1	0	0	0	0	0	0	0
6.00	6.20	0.00006	0	0	0	0	0	0	0	0	1	0	0	0	0	0	0	0
6.20	6.40	0.00005	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
6.40	6.60	0.00005	0	0	0	0	0	0	0	0	5	0	0	0	0	0	0	0
Parts per thousand for each wave period			53	248	363	188	103	39	5	1	0	0	0	0	0	0	0	0

Table 2.30 Distribution of wave height and direction at inshore point H

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 = 159192
Based on HINDWAVE predictions for January 1974 - February 1992

H1	To H2	P(H>H1)	-10 10	10 30	30 50	50 70	70 90	90 110	110 130	130 150	150 170	170 190	190 210	210 230	230 250	250 270	270 290	290 310	310 330	330 350
			Wave direction in degrees North																	
0.00	0.20	0.96537	526	837	704	222	374	614	5268	4056	546	417	1041	3331	3417	5445	3515	725	1030	646
0.20	0.40	0.63824	0	0	0	0	4	10	731	6087	901	552	1031	2649	4246	2423	296	0	3	0
0.40	0.60	0.44889	0	0	0	0	0	0	9	3140	454	762	999	3770	2660	595	4	0	0	0
0.60	0.80	0.32497	0	0	0	0	0	0	1	659	1386	545	1109	3209	1533	68	0	0	0	0
0.80	1.00	0.23987	0	0	0	0	0	0	0	67	744	879	1403	2873	332	9	0	0	0	0
1.00	1.20	0.17681	0	0	0	0	0	0	0	11	393	284	1261	2678	33	0	0	0	0	0
1.20	1.40	0.13021	0	0	0	0	0	0	0	1	127	477	1518	1180	5	0	0	0	0	0
1.40	1.60	0.09713	0	0	0	0	0	0	0	0	195	87	616	1129	3	0	0	0	0	0
1.60	1.80	0.07683	0	0	0	0	0	0	0	0	70	389	1844	527	0	0	0	0	0	0
1.80	2.00	0.04852	0	0	0	0	0	0	0	0	32	64	692	195	0	0	0	0	0	0
2.00	2.20	0.03868	0	0	0	0	0	0	0	0	26	269	985	134	0	0	0	0	0	0
2.20	2.40	0.02453	0	0	0	0	0	0	0	0	34	24	324	66	0	0	0	0	0	0
2.40	2.60	0.02004	0	0	0	0	0	0	0	0	5	101	640	33	0	0	0	0	0	0
2.60	2.80	0.01226	0	0	0	0	0	0	0	0	3	13	224	2	0	0	0	0	0	0
2.80	3.00	0.00984	0	0	0	0	0	0	0	0	3	6	325	1	0	0	0	0	0	0
3.00	3.20	0.00648	0	0	0	0	0	0	0	0	1	63	133	1	0	0	0	0	0	0
3.20	3.40	0.00449	0	0	0	0	0	0	0	0	1	7	134	0	0	0	0	0	0	0
3.40	3.60	0.00307	0	0	0	0	0	0	0	0	3	12	73	0	0	0	0	0	0	0
3.60	3.80	0.00219	0	0	0	0	0	0	0	0	1	21	57	0	0	0	0	0	0	0
3.80	4.00	0.00139	0	0	0	0	0	0	0	0	0	2	47	0	0	0	0	0	0	0
4.00	4.20	0.00090	0	0	0	0	0	0	0	0	0	19	15	0	0	0	0	0	0	0
4.20	4.40	0.00057	0	0	0	0	0	0	0	0	0	1	18	0	0	0	0	0	0	0
4.40	4.60	0.00037	0	0	0	0	0	0	0	0	0	1	5	0	0	0	0	0	0	0
4.60	4.80	0.00031	0	0	0	0	0	0	0	0	0	6	5	0	0	0	0	0	0	0
4.80	5.00	0.00020	0	0	0	0	0	0	0	0	0	0	1	0	0	0	0	0	0	0
5.00	5.20	0.00019	0	0	0	0	0	0	0	0	0	8	8	0	0	0	0	0	0	0
5.20	5.40	0.00004	0	0	0	0	0	0	0	0	0	0	3	0	0	0	0	0	0	0
5.40	5.60	0.00002	0	0	0	0	0	0	0	0	0	1	0	0	0	0	0	0	0	0
5.60	5.80	0.00001	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
5.80	6.00	0.00001	0	0	0	0	0	0	0	0	0	0	1	0	0	0	0	0	0	0
Parts per thousand for each direction			5	9	7	2	4	6	62	145	51	52	150	226	127	88	40	8	11	7

Significant wave heights for given exceedence levels

P(H>Hs)	All dir.	-10 10	10 30	30 50	50 70	70 90	90 110	110 130	130 150	150 170	170 190	190 210	210 230	230 250	250 270	270 290	290 310	310 330	330 350
		Wave direction in degrees North																	
0.50	0.35	0.00	0.00	0.00	0.00	0.10	0.10	0.11	0.29	0.67	0.83	1.22	0.65	0.32	0.15	0.10	0.00	0.10	0.00
0.20	0.93	0.00	0.00	0.00	0.00	0.16	0.16	0.18	0.46	0.97	1.53	2.00	1.11	0.55	0.31	0.17	0.00	0.16	0.00
0.10	1.38	0.00	0.00	0.00	0.00	0.18	0.18	0.23	0.55	1.20	2.03	2.46	1.37	0.68	0.38	0.19	0.00	0.18	0.00
0.05	1.79	0.00	0.00	0.00	0.00	0.19	0.19	0.32	0.60	1.52	2.40	2.85	1.57	0.77	0.48	0.27	0.00	0.19	0.00
0.02	2.40	0.00	0.00	0.00	0.00	0.20	0.20	0.37	0.74	1.85	3.12	3.30	1.79	0.87	0.57	0.35	0.00	0.20	0.00
0.01	2.79	0.00	0.00	0.00	0.00	0.20	0.27	0.39	0.78	2.20	3.67	3.63	2.02	0.95	0.60	0.38	0.00	0.20	0.00
Average	0.56	0.10	0.10	0.10	0.10	0.10	0.10	0.12	0.30	0.68	0.97	1.29	0.70	0.34	0.18	0.11	0.10	0.10	0.10

Table 2.31 Distribution of wave height and period at inshore point H

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 = 159192
Based on HINDWAVE predictions for January 1974 - February 1992

H1	To H2	P(H>H1)	Zero-crossing wave period in seconds (Tz)															
			0.0	1.0	2.0	3.0	4.0	5.0	6.0	7.0	8.0	9.0	10.0	11.0	12.0	13.0	14.0	15.0
			1.0	2.0	3.0	4.0	5.0	6.0	7.0	8.0	9.0	10.0	11.0	12.0	13.0	14.0	15.0	16.0
0.00	0.20	0.85803	4414	15620	1946	0	0	0	0	0	0	0	0	0	0	0	0	0
0.20	0.40	0.63824	0	6332	12602	1	0	0	0	0	0	0	0	0	0	0	0	0
0.40	0.60	0.44889	0	5	10828	1559	0	0	0	0	0	0	0	0	0	0	0	0
0.60	0.80	0.32497	0	0	4612	3898	0	0	0	0	0	0	0	0	0	0	0	0
0.80	1.00	0.23987	0	0	229	6007	70	0	0	0	0	0	0	0	0	0	0	0
1.00	1.20	0.17681	0	0	3	4035	622	0	0	0	0	0	0	0	0	0	0	0
1.20	1.40	0.13021	0	0	0	1886	1423	0	0	0	0	0	0	0	0	0	0	0
1.40	1.60	0.09713	0	0	0	389	1635	7	0	0	0	0	0	0	0	0	0	0
1.60	1.80	0.07683	0	0	0	21	2796	14	0	0	0	0	0	0	0	0	0	0
1.80	2.00	0.04852	0	0	0	1	553	430	0	0	0	0	0	0	0	0	0	0
2.00	2.20	0.03868	0	0	0	0	783	633	0	0	0	0	0	0	0	0	0	0
2.20	2.40	0.02453	0	0	0	0	62	386	0	0	0	0	0	0	0	0	0	0
2.40	2.60	0.02004	0	0	0	0	30	746	3	0	0	0	0	0	0	0	0	0
2.60	2.80	0.01226	0	0	0	0	4	237	1	0	0	0	0	0	0	0	0	0
2.80	3.00	0.00984	0	0	0	0	1	333	2	0	0	0	0	0	0	0	0	0
3.00	3.20	0.00648	0	0	0	0	0	94	106	0	0	0	0	0	0	0	0	0
3.20	3.40	0.00449	0	0	0	0	0	70	72	0	0	0	0	0	0	0	0	0
3.40	3.60	0.00307	0	0	0	0	0	30	58	0	0	0	0	0	0	0	0	0
3.60	3.80	0.00219	0	0	0	0	0	5	74	0	0	0	0	0	0	0	0	0
3.80	4.00	0.00139	0	0	0	0	0	2	47	0	0	0	0	0	0	0	0	0
4.00	4.20	0.00090	0	0	0	0	0	1	33	0	0	0	0	0	0	0	0	0
4.20	4.40	0.00057	0	0	0	0	0	0	18	1	0	0	0	0	0	0	0	0
4.40	4.60	0.00037	0	0	0	0	0	0	6	0	0	0	0	0	0	0	0	0
4.60	4.80	0.00031	0	0	0	0	0	0	9	2	0	0	0	0	0	0	0	0
4.80	5.00	0.00020	0	0	0	0	0	0	1	0	0	0	0	0	0	0	0	0
5.00	5.20	0.00019	0	0	0	0	0	0	1	14	0	0	0	0	0	0	0	0
5.20	5.40	0.00004	0	0	0	0	0	0	1	2	0	0	0	0	0	0	0	0
5.40	5.60	0.00002	0	0	0	0	0	0	0	1	0	0	0	0	0	0	0	0
5.60	5.80	0.00001	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
5.80	6.00	0.00001	0	0	0	0	0	0	0	1	0	0	0	0	0	0	0	0
Parts per thousand for each wave period			51	256	352	207	93	35	5	0	0	0	0	0	0	0	0	0

Table 2.32 Extreme wave conditions at inshore point A

H_s is in metres, derived assuming 3-hourly events.
The values quoted are for unbroken waves. Wave breaking effects would limit all the values marked with an asterisk (*).

Return Period (Yrs)	Wave direction in degrees North							
	90 110	110 130	130 150	150 170	170 190	190 210	210 230	0 360
1	1.94	4.00*	3.65*	2.65	2.28	1.25	0.91	4.24*
2	2.09	4.50*	4.11*	2.89	2.49	1.31	0.96	4.71*
5	2.28	5.16*	4.70*	3.20	2.75	1.40	1.03	5.34*
10	2.41	5.66*	5.15*	3.42*	2.95	1.45	1.08	5.84*
20	2.55	6.16*	5.58*	3.63*	3.14	1.51	1.13	6.34*
50	2.72	6.82*	6.15*	3.90*	3.39*	1.58	1.19	7.02*
100	2.84	7.32*	6.58*	4.11*	3.58*	1.63	1.24	7.55*

The corresponding mean period can be calculated from $T_z = 10.7 \sqrt{H/g}$

Table 2.33 Extreme wave conditions at inshore point B

H_s is in metres, derived assuming 3-hourly events.
The values quoted are for unbroken waves. Wave breaking effects would limit all the values marked with an asterisk (*).

Return Period (Yrs)	Wave direction in degrees North							
	50 70	70 90	90 110	110 130	130 150	150 170	170 190	0 360
1	0.74	1.03	1.95*	3.47*	4.17*	1.78*	0.94	4.38*
2	0.80	1.12*	2.13*	3.86*	4.41*	1.89*	1.02	4.73*
5	0.89	1.24*	2.36*	4.37*	4.71*	2.03*	1.11*	5.21*
10	0.94	1.32*	2.52*	4.76*	4.93*	2.13*	1.19*	5.56*
20	1.00	1.40*	2.69*	5.13*	4.14*	2.22*	1.26*	5.91*
50	1.08	1.51*	2.91*	5.63*	4.41*	2.34*	1.34*	6.38*
100	1.13*	1.59*	3.07*	6.00*	5.61*	2.43*	1.41*	6.73*

The corresponding mean period can be calculated from $T_z = 11.3 \sqrt{H/g}$

Table 2.34 Extreme wave conditions at inshore point C

H_s is in metres, derived assuming 3-hourly events.
The values quoted are for unbroken waves. Wave breaking effects would limit all the values marked with an asterisk (*).

Return Period (Yrs)	Wave direction in degrees North																		
	-10 10	10 30	30 50	50 70	70 90	90 110	110 130	130 150	150 170	170 190	190 210	210 230	230 250	250 270	270 290	290 310	310 330	330 350	0 to 360
1		0.52		0.73	1.06	1.75*	2.79*	3.21*	2.10*	1.43*	1.06	0.84	0.62						3.34
2		0.59		0.78	1.15*	1.90*	3.12*	3.55*	2.22*	1.51*	1.12*	0.91	0.67						3.67
5		0.67		0.85	1.27*	2.08*	3.55*	3.99*	2.38*	1.60*	1.19*	0.98	0.73						4.11
10		0.74		0.90	1.36*	2.21*	3.87*	4.32*	2.49*	1.66*	1.24*	1.04	0.77						4.46
20		0.80		0.95	1.44*	2.34*	4.20*	4.64*	2.60*	1.72*	1.28*	1.10*	0.81						4.80
50		0.89		1.05	21,55	2.51*	4.62*	5.05*	2.74*	1.80*	1.34*	1.17*	0.87						5.27
100		0.95		1.06	1.63*	2.63*	4.94*	5.36*	2.84*	1.85*	1.38*	1.22*	0.90						5.63
	Wave height in m																		

The corresponding mean period can be calculated from T_z 11.0 $\sqrt{H/g}$.

Table 2.35 Extreme wave conditions at inshore point D

H_s is in metres, derived assuming 3-hourly events.
The values quoted are for unbroken waves. Wave breaking effects would limit all the values marked with an asterisk (*).

Return Period (Yrs)	Wave direction in degrees North																		
	-10 10	10 30	30 50	50 70	70 90	90 110	110 130	130 150	150 170	170 190	190 210	210 230	230 250	250 270	270 290	290 310	310 330	330 350	0 to 360
1					1.06	1.48	2.34	2.77	4.16	3.43	1.22								4.32
2					1.14	1.58	2.61	3.10	4.58	3.68	1.30								4.75
5					1.25	1.72	2.96	3.53	5.11	4.01	1.41								5.34
10					1.33	1.81	3.22	3.85	5.50	4.25	1.48								5.78
20					1.40	1.90	3.48	4.17	5.88	4.48	1.55								6.24
50					1.50	2.02	3.82	4.58	6.38	4.77	1.64								6.84
100					1.57	2.11	4.07	4.89	6.75	4.99	1.71								7.30
	Wave height in m																		

The corresponding mean period can be calculated from T_z 11.0 $\sqrt{H/g}$

Table 2.36 Extreme wave conditions at inshore point E

H_s is in metres, derived assuming 3-hourly events.
The values quoted are for unbroken waves. Wave breaking effects would limit all the values marked with an asterisk (*).

Return Period (Yrs)	Wave direction in degrees North																	
	-10 10	10 30	30 50	50 70	70 90	90 110	110 130	130 150	150 170	170 190	190 210	210 230	230 250	250 270	270 290	290 310	310 330	330 350	0 to 360
1					0.91	1.64*	2.32*	3.38*	3.75*	2.37*	1.37*	0.91	0.83						3.94
2					0.99	1.78*	2.58*	3.81*	4.15*	2.50*	1.44*	0.98	0.88						4.33
5					1.10*	1.97*	2.91*	4.37*	4.67*	2.67*	1.52*	1.06*	0.94						4.85
10					1.18*	2.10*	3.17*	4.79*	5.08*	2.79*	1.57*	1.13*	0.99						5.25
20					1.25*	2.23*	3.42*	5.20*	5.47*	2.90*	1.62*	1.19*	1.03						5.65
50					1.35*	2.40*	3.74*	5.74*	5.93*	3.03*	1.69*	1.26*	1.08						6.19
100					1.42*	2.53*	4.99*	6.14*	6.29*	3.15*	1.74*	1.32*	1.12*						6.60
	Wave height in m																		

The corresponding mean period can be calculated from T_z 10.5 $\sqrt{H/g}$

Table 2.37 Extreme wave conditions at inshore point F

H_s is in metres, derived assuming 3-hourly events.
The values quoted are for unbroken waves. Wave breaking effects would limit all the values marked with an asterisk (*).

Return Period (Yrs)	Wave direction in degrees North																		
	-10 10	10 30	30 50	50 70	70 90	90 110	110 130	130 150	150 170	170 190	190 210	210 230	230 250	250 270	270 290	290 310	310 330	330 350	0 to 360
1							1.02	3.03*	3.19*	3.07*	2.85*	1.72*	1.14*	0.80					3.65
2							1.09	3.39*	3.54*	3.33*	3.07*	1.81*	1.19*	0.85					3.97
5							1.19*	3.87*	4.00*	3.65*	3.34*	1.92*	1.26*	0.90					4.39
10							1.26*	4.23*	4.33*	3.88*	3.54*	1.99*	1.31*	0.94					4.71
20							1.33*	4.59*	4.66*	4.11*	3.73*	2.06*	1.35*	0.98					5.03
50							1.41*	5.07*	5.08*	4.39*	3.97*	2.15*	1.40*	1.03					5.45
100							1.48*	5.44*	5.40*	4.60*	4.15*	2.22*	1.44*	1.06					5.77
	Wave height in m																		

The corresponding mean period can be calculated from T_z 11.0 $\sqrt{H/g}$

Table 2.38 Extreme wave conditions at inshore point G

H_s is in metres, derived assuming 3-hourly events.
The values quoted are for unbroken waves. Wave breaking effects would limit all the values marked with an asterisk (*).

Return Period (Yrs)	Wave direction in degrees North																		
	-10 10	10 30	30 50	50 70	70 90	90 110	110 130	130 150	150 170	170 190	190 210	210 230	230 250	250 270	270 290	290 310	310 330	330 350	0 to 360
1								0.82	2.44*	3.79*	4.56*	1.93*	1.08	0.65					4.71
2								0.87	2.71*	4.22*	4.88*	2.02*	1.14*	0.69					5.15
5								0.95	3.06*	4.78*	5.29*	2.14*	1.23*	0.73					5.70
10								1.00	3.31*	5.20*	5.59*	2.23*	1.29*	0.77					6.13
20								1.06	3.57*	5.62*	5.88*	2.30*	1.34*	0.80					6.56
50								1.12*	3.90*	6.15*	6.25*	2.40*	1.42*	0.84					7.13
100								1.17*	4.14*	6.56*	6.52*	2.48*	1.47*	0.86					7.55
	Wave height in m																		

The corresponding mean period can be calculated from T_z 10.7 $\sqrt{H/g}$

Table 2.39 Extreme wave conditions at inshore point H

H_s is in metres, derived assuming 3-hourly events.
The values quoted are for unbroken waves. Wave breaking effects would limit all the values marked with an asterisk (*).

Return Period (Yrs)	Wave direction in degrees North																	
	-10 10	10 30	30 50	50 70	70 90	90 110	110 130	130 150	150 170	170 190	190 210	210 230	230 250	250 270	270 290	290 310	310 330	330 350	0 to 360
1								0.91	2.32*	3.79*	4.27*	2.36*	1.09						4.40
2								0.97	2.55*	4.20*	4.53*	2.49*	1.16*						4.74
5								1.04	2.84*	4.73*	4.85*	2.64*	1.25*						5.18
10								1.10*	3.06*	5.13*	5.07*	2.75*	1.31*						5.50
20								1.15*	3.26*	5.51*	5.29*	2.86*	1.37*						5.83
50								1.21*	3.53*	6.01*	5.57*	2.99*	1.44*						6.25
100								1.26*	3.73*	6.38*	5.77*	3.08*	1.50*						6.57
	Wave height in m																		

The corresponding mean period can be calculated from T_z 10.7 $\sqrt{H/g}$

Table 2.40 Wind conditions representative of Poole Harbour

Direction bound (degs N)	Average wind speed (m/s)	10% exceedence	1% exceedence	1:1 return period	1:100 return period	Percentage of data in sector
-15 to 15	4.29	8.24	11.74	15.15	20.75	0.1
15 to 45	4.65	8.31	10.78	13.63	17.65	9.1
45 to 75	3.95	7.06	9.23	11.02	14.60	8.7
75 to 105	3.88	7.30	9.31	11.40	15.49	5.0
105 to 135	5.48	9.91	13.30	15.80	21.40	4.0
135 to 165	5.38	9.65	13.80	17.65	23.90	3.9
165 to 195	6.90	12.01	18.16	23.28	31.33	8.0
195 to 225	7.85	13.19	18.39	23.50	29.92	10.3
225 to 255	7.12	11.20	17.29	22.24	28.66	16.0
255 to 285	5.96	10.60	15.80	20.95	28.31	11.6
285 to 315	4.10	9.06	13.70	17.03	22.46	9.1
315 to 345	3.94	8.25	12.02	16.07	23.29	6.2

Table 2.41 Extreme wave conditions in Poole Harbour

Return period	Extreme significant wave height H_sm								
	A	B	C	D	E	F	G	H	I
1	0.50	0.63	0.35	0.65	0.66	0.62	0.61	0.71	0.62
2	0.53	0.67	0.37	0.69	0.71	0.66	0.66	0.77	0.66
5	0.56	0.71	0.40	0.73	0.76	0.71	0.72	0.86	0.71
10	0.59	0.75	0.42	0.77	0.80	0.74	0.78	0.92	0.75
20	0.61	0.78	0.44	0.80	0.85	0.77	0.83	0.98	0.79
50	0.64	0.83	0.47	0.84	0.90	0.81	0.89	1.06	0.84
100	0.67	0.86	0.49	0.87	0.94	0.84	0.94	1.12	0.87
	Extreme zero-crossing wave period (s)								
10	2.0	2.25	1.75	2.25	2.25	2.25	2.25	3.0	2.25
100	2.6	3.0	2.2	3.0	3.1	2.9	3.1	3.4	3.0

Table 2.42 Tidal harmonic constituents in Poole Harbour

Tidal constituent	Frequency (degrees per hour)	Amplitude (metres)	Phase (degrees)
ZO	0.000	1.47	0.
SA	0.041	0.09	183.
SSA	0.082	0.03	59.
MM	0.544	0.02	216.
MSF	1.016	0.02	233.
MF	1.098	0.01	204.
2Q1	12.854	0.01	286.
SIG1	12.927	0.00	14.
Q1	13.399	0.01	339.
RO1	13.472	0.00	294.
O1	13.943	0.04	352.
MP1	14.025	0.01	191.
M1	14.492	0.00	144.
CHI1	14.570	0.00	81.
PI1	14.918	0.00	300.
P1	14.959	0.03	115.
S1	15.000	0.01	39.
K1	15.041	0.09	113.
PSI1	15.082	0.00	74.
PHI1	15.123	0.00	268.
TH1	15.513	0.00	126.
J1	15.585	0.00	171.
SO1	16.057	0.01	292.
OO1	16.139	0.01	252.
OQ1	27.342	0.01	345.
MNS2	27.424	0.02	167.
2N2	27.895	0.01	205.
MU2	27.968	0.07	200.
N2	28.440	0.11	254.
NU2	28.513	0.02	292.
OP2	28.902	0.01	44.
MA2	28.943	0.01	50.
M2	28.984	0.44	279.
MB2	29.025	0.00	310.
MKS2	29.066	0.01	216.
LAM2	29.456	0.01	86.
L2	29.528	0.01	101.
T2	29.959	0.19	311.
S2	30.000	0.00	300.
R2	30.041	0.05	292.

MO3	42.927	0.00	281.
M3	43.476	0.01	188.
SO3	43.943	0.00	71.
MK3	44.025	0.01	133.
SK3	45.041	0.00	234.
MN4	57.424	0.07	16.
M4	57.968	0.20	40.
SN4	58.440	0.01	128.
MS4	58.984	0.13	99.
MK4	59.066	0.03	95.
S4	60.000	0.01	204.
SK4	60.082	0.01	197.
2MN6	86.408	0.04	69.
M6	86.952	0.07	94.
MSN6	87.424	0.02	113.
2MS6	87.968	0.07	137.
2MK6	88.050	0.02	137.
2SM6	88.984	0.02	190.
MSK6	89.066	0.01	186.

Table 2.43 Distribution of high water levels in Poole Harbour

Water level range (mCD)		Percentage of data in range	Cumulative percentage of data
1.0	2.0	63.3	63.3
2.0	2.1	14.3	77.6
2.1	2.2	9.2	86.8
2.2	2.3	6.3	93.1
2.3	2.4	2.8	95.9
2.4	2.5	1.9	97.8
2.5	2.6	1.3	99.1
2.6	2.7	0.6	99.7
2.7	2.8	0.1	99.8
2.8	2.9	0.2	100.0

Table 2.44 Extreme water levels in Poole Harbour

Return period (years)	Extreme water level (mCD)			Average extreme water level (mOD)
	Weibull	Gumbel	Average	
1	2.78	2.77	2.78	1.38
2	2.86	2.83	2.85	1.45
5	2.96	2.90	2.93	1.53
10	3.05	2.96	3.01	1.61
20	3.13	3.02	3.08	1.68
50	3.25	3.09	3.17	1.77
100	3.34	3.15	3.25	1.85
150	3.39	3.18	3.29	1.89
200	3.42	3.21	3.32	1.92

Table 2.45 Extreme combinations of H_s and water level at offshore point 2

Water level (mOD)	Significant wave height (m)	Mean wave period (s)
1.22	8.2	9.5
1.31	7.9	9.3
1.38	7.4	9.0
1.45	7.0	8.8
1.53	6.6	8.5
1.61	6.1	8.2
1.68	5.7	7.9
1.77	5.3	7.6
1.85	4.8	7.3

Table 2.46 Extreme combinations of H_s and water level at inshore points A and B

Water level (mOD)	Significant wave height (m)	
	Point A	Point B
1.22	7.5	6.7
1.31	7.0	6.4
1.38	6.3	5.9
1.45	5.8	5.6
1.53	5.3	5.2
1.61	4.7	4.7
1.68	4.2	4.4
1.77	3.8	4.0
1.85	3.3	3.6

Note that the values quoted are for unbroken waves.

The corresponding mean wave period can be calculated from $T_z = 11.0 \sqrt{H/g}$

Table 2.47 Extreme combinations of H_s and water level at inshore points C, D and E

Water level (mOD)	Significant wave height (m)		
	Point C	Point D	Point E
1.25	5.6	7.3	6.6
1.34	5.3	6.8	6.2
1.41	4.8	6.2	5.7
1.48	4.5	5.8	5.2
1.56	4.1	5.3	4.8
1.64	3.7	4.7	4.3
1.71	3.3	4.3	3.9
1.80	3.0	3.9	3.5
1.88	2.7	3.3	2.9

Note that the values quoted are for unbroken waves.

The corresponding mean wave periods can be calculated from $T_z = 10.8 \sqrt{H/g}$.

Table 2.48 Extreme combinations of H_s and water level at inshore point F

Water level (mOD)	Significant wave height (m)
1.27	5.8
1.36	5.4
1.43	5.0
1.50	4.7
1.58	4.4
1.66	4.0
1.73	3.7
1.82	3.3
1.90	2.9

Note that the values quoted are for unbroken waves.

The corresponding mean wave periods can be calculated from $T_z = 11.0 \sqrt{H/g}$.

Table 2.49 Extreme combinations of H_s and water level at inshore points G and H

Wave level (mOD)	Significant wave height	
	Point G	Point H
1.29	7.5	6.6
1.38	7.1	6.2
1.45	6.6	5.8
1.52	6.1	5.5
1.60	5.7	5.2
1.68	5.1	4.7
1.75	4.7	4.4
1.84	4.3	4.1
1.92	3.7	3.6

Note that the values quoted are for unbroken waves.

The corresponding mean wave periods can be calculated from $T_z = 10.7 \sqrt{H/g}$.

Table 2.50 Extreme combinations of H_s and water level at Poole Harbour points A to I

Water level (mOD)	Significant wave height (m)								
	Point A	Point B	Point C	Point D	Point E	Point F	Point G	Point H	Point I
1.22	0.67	0.86	0.49	0.87	0.94	0.84	0.94	1.12	0.89
1.31	0.64	0.83	0.47	0.84	0.90	0.81	0.89	1.06	0.84
1.38	0.61	0.78	0.44	0.80	0.85	0.77	0.83	0.98	0.79
1.45	0.59	0.75	0.42	0.77	0.80	0.74	0.78	0.92	0.75
1.53	0.56	0.71	0.40	0.73	0.76	0.71	0.72	0.86	0.71
1.61	0.53	0.67	0.37	0.69	0.71	0.66	0.66	0.77	0.66
1.68	0.50	0.63	0.35	0.65	0.66	0.62	0.61	0.71	0.60
1.77	0.47	0.59	0.34	0.61	0.61	0.58	0.56	0.64	0.55
1.85	0.44	0.55	0.30	0.57	0.56	0.53	0.50	0.56	0.49

For calculation of corresponding mean wave periods, see Table 3.10

Table 3.1 Analysis of movement of High Water Mark and cliff top

Distance (m) see Figure 3.9 for location	Movement of HW contour			Movement of cliff line		
	1901-25	1925-55	1955-93	1901-25	1925-55	1955-93
1649				-4	-9	-4
1594	-18	2	6	-1	-8	-3
1516	-16	0	8	0	-7	-6
1399	-15	14	8	0	0	-8
1357	-13	-6	7	-8	-5	-4
1250	-10	-6	0	-3	0	0
1155	-7	-5	-1	-1	-8	0
1111	-7	-5	-3	0	0	0
1040				-16	-7	0
970				-28	-7	0
910	-14	2	-3	0	0	0
883				0	0	0
812	-4	0	5	0	0	0
707				-8	0	0
680				-16	0	0
650				-24	0	0
622	-12	2	9	-7	-1	0
550				-3	0	0
488	-3	3	8	-3	-1	0
445	-2	5	4	-2	-5	0
345	6	7	3	0	-10	0
261	17	4	1			
154	22	2	4			
102	9	-6	6			
0	11	-13	4			
-100	17	0	-5			
-200	17	0	2			
-300	29	0	-4			
-400	15	2	-5			
-500	34	0	-8			
-600	30	0	-8			
-707	51	0	-8			
-807	45	11	-9			
-907	32	16	-11			
-1007	20	15	-13			
-1107	2	17	-8			
-1207	-1	10	-24			
-1307	-5	-25	4			
-1414	-12	-5	-1			
-1514	2	-3	-3			

Table 3.2 Potential annual drift rates in Poole Bay

Point Ref	Beach normal (°N)	Av. inshore wave direction (°N)	Right m^3/yr	Left m^3/yr	Gross m^3/yr	Nett m^3/yr
B	129	136.32	-152094.6	612332.1	764426.8	460237.5
C	131	137.26	-118055.8	239141.9	357197.7	121086.0
D	152	162.06	-138461.0	553732.1	692193.2	415271.1
E	163	163	-239890.1	263911.8	503801.9	24021.7
F	173	179.08	-227524.0	407484.5	635008.4	179960.5
G	190	197.29	-166442.8	595413.4	761856.1	428970.6
H	194	199.66	-196411.5	563684.3	760095.8	367272.8

Results for data JULY 1974 to JUNE 1991

Table 3.3 Average annual rates of cliff erosion (from May, 1969)

	Survey Dates	Cliff-top erosion (m)	Rate (m/year)
Hamworthy	1886 - 1952	18	0.3
Whitecliff	1886 - 1952	36	0.5
Lilliput	1886 - 1952	43	0.6
West Brownsea	1886 - 1952	24	0.4
Arne	1886 - 1952	24	0.4

Table 3.4 Potential annual drift rates in Poole Harbour

Point	Av. inshore wave direction (°N)	Right m^3/year	Left m^3/year	Gross m^3/year	Nett m^3/year	Beach normal (°N)
A	205.3	-2294.1	3294.2	5588.2	1000.1	203.0
B	192.47	-2933.9	6568.8	9502.7	3634.9	187.0
D	192.94	-2302.5	9126.6	1142.1	6824.1	184.0
F	224.46	-3896.5	5847.1	9743.6	1950.6	222.0
G	218.48	-4760.4	624.8	5385.2	-4135.5	232.0

Table 3.5 "Area" changes in Poole Harbour (May, 1969)

	Area reclaimed (acres)		
	Human influence	Natural agencies	Total
6000 B.P. to A.D. 1807			
Wareham & Jeysworth	1043	48	1101
Lytchett Bay		250	250
Southern Shores		511	511
Brownsea & Islands		25	25
Holes Bay		122	122
Poole	48		48
TOTAL	1091	966	2057
Cliff erosion		-70	-70
Net changes	1091	896	1987
1807 to 1966			
Wareham & Keysworth		75	75
Lytchett Bay	42	45	87
Southern Shores		175	175
Brownsea & Islands	(70)		(70)
Holes Bay	68	30	98
Poole	100		100
Parkstone & eastern shores	18		18
TOTAL	228	315	543
Cliff erosion		-32	-32
Net changes	228	228	511

Net changes 6000 BP to 1966 = about 2,500 acres

Table 3.6 Volume of dredging, Poole Harbour 1984-1993 (from Fahy, 1993)

Year	Volume of capital dredging (m^3)	Volume of maintenance dredging (m^3)	Total volume (m^3)
1984/85	23 735	48 808	72 543
1985/86	302 518	56 178	358 696
1986/87	65 639	46 086	111 722
1987/88	nil	61 943	61 943
1988/89	739 422	56 745	796 167
1989/90	542 000	73 493	615 493
1990/91	nil	88 845	88 845
1991/92	780 000	56 799	810 869
1992/93	nil	66 687	66 687

Table 4.1 Analysis of overtopping discharge 1:10 year storm

Unit and subsection	Location	Cross-section Figure	Seawall type	Crest level (mOD)	Slope	Beach level at toe (mOD)	Land use at Seawall	Subsequent land use	Wave Climate	Near-shore wave height H_s (m)	Water level (mOD)	Discharge (l/s/m)
1/4	Branksome	5.3	Concrete	3.20	Stepped	1.1	Promenade	Beach huts at 3m	PB.D	3.3	1.64	16
1/6	Canford Cliffs	5.3	Concrete	3.20	Vertical	1.1	Promenade	Beach huts at 4m	PB.D	3.3	1.64	21
1/8	Flag head	5.3	Concrete	3.14	Stepped	1.4	Promenade	Beach huts at 4m	PB.D	3.3	1.64	0.05
1/10	Sandbanks	5.4	Concrete	3.50	Sloping	2.4	Garden	Houses/Hotels at >3m	PB.C	2.7	1.64	0
1/11	Sandbanks	5.4	Concrete	2.55	Steep	2.6	Promenade	Beach huts at 4m	PB.C	2.7	1.64	0
1/14	Sandbanks	5.4	Steel	2.9	Vertical	1.2	Garden	Houses at 20m	PB.B	3.6	1.61	[1]
3/3	Sandbanks	5.5	Masonry	3.00	Vertical	0.5	Private garden	Houses at >20m	PH.I	0.49	1.61	0.003
4/2	Banks Road	5.5	Concrete	1.59	Vertical with step	0.0	Footpath	Road at 9m (2.0mOD)	PH.I	0.49	1.61	[2]
4/5	Shore Road	5.5	Concrete	2.03	Vertical	0.0	Footpath	Road at 9m (1.5mOD)	PH.H	0.56	1.61	2.0
5/2	Evening Hill	5.6	Gabion	1.50	Steep	0.6	Footpath	Grass slope	PH.K	0.50	1.61	[2]
5/4	Lilliput	5.6	Masonry	2.22	Steep	0.6	Private garden (sloping)	House at >20m	PH.K	0.50	1.61	0.7
6/3	Parkstone	5.6	Masonry	2.00	Steep	0.6	Private garden (sloping)	House at 30m	PH.F	0.53	1.61	3.4
7/4	Parkstone Bay	5.7	Rock	2.00	1:2	0.3	Footpath	Recreation ground	PH.E	0.56	1.61	6.1
8/2	Town Quay	5.7	SSP	1.90	Vertical	-5.0	Quay	Road at >10m	PH.D	0.57	1.61	[1]
9/2	Holes Bay	5.7	Rock	2.05	1:2	-0.6	Footpath	Road at >10m	PH.L	0.30	1.61	0.1
13/2	Hamworthy Park	5.8	Concrete	1.30	Vertical	1.0	Footpath	Beach huts at 5m	PH.B	0.55	1.61	[2]
13/4	Lake	5.8	Concrete	2.57	Vertical	1.2	Footpath	House at >15m	PH.A	0.44	1.61	0

Notes:
- PB - Poole Bay wave climate; PH- Poole Harbour wave climate;
- 1 - Outside the range of existing models but evidently high overtopping;
- 2 - Crest below still water level, ie weir flow over wall

Table 4.2 Analysis of overtopping discharge for 1:50 year storm

Unit and subsection	Location	Cross-section Figure	Seawall type	Crest level (mOD)	Slope	Beach level at toe (mOD)	Land use at Seawall	Subsequent land use	Wave Climate	Near-shore wave height H_s (m)	Water level (mOD)	Discharge (l/s/m)
1/4	Branksome	5.3	Concrete	3.20	Stepped	1.1	Promenade	Beach huts at 3m	PB.D	3.3	1.80	62
1/6	Canford Cliffs	5.3	Concrete	3.20	Vertical	1.1	Promenade	Beach huts at 4m	PB.D	3.3	1.80	27
1/8	Flag head	5.3	Concrete	3.14	Stepped	1.4	Promenade	Beach huts at 4m	PB.D	3.3	1.80	3.8
1/10	Sandbanks	5.4	Concrete	3.50	Sloping	2.4	Garden	Houses/Hotels at >3m	PB.C	2.7	1.80	0
1/11	Sandbanks	5.4	Concrete	2.55	Steep	2.6	Promenade	Beach huts at 4m	PB.C	2.7	1.80	0
1/14	Sandbanks	5.4	Steel	2.9	Vertical	1.2	Garden	Houses at 20m	PB.B	3.6	1.77	[1]
3/3	Sandbanks	5.5	Masonry	3.00	Vertical	0.5	Private garden	Houses at >20m	PH.I	0.49	1.77	0.004
4/2	Banks Road	5.5	Concrete	1.59	Vertical with step	0.0	Footpath	Road at 9m (2.0mOD)	PH.I	0.49	1.77	[2]
4/5	Sandbanks	5.5	Concrete	2.03	Vertical	0.0	Footpath	Road at 9m (1.5mOD)	PH.H	0.56	1.77	3.7
5/2	Evening Hill	5.6	Gabion	1.50	Steep	0.6	Footpath	Grass slope	PH.K	0.50	1.77	[2]
5/4	Lilliput	5.6	Masonry	2.22	Steep	0.6	Private garden (sloping)	House at >20m	PH.K	0.50	1.77	1.6
6/3	Parkstone	5.6	Masonry	2.00	Steep	0.6	Private garden (sloping)	House at 30m	PH.F	0.53	1.77	8.5
7/4	Parkstone Bay	5.7	Rock	2.00	1:2	0.3	Footpath	Recreation ground	PH.E	0.56	1.77	23
8/2	Town Quay	5.7	SSP	1.90	Vertical	-5.0	Quay	Road at >10m	PH.D	0.57	1.77	[1]
9/2	Holes Bay	5.7	Rock	2.05	1:2	-0.6	Footpath	Road at >10m	PH.L	0.30	1.77	1.1
13/2	Hamworthy Park	5.8	Concrete	1.30	Vertical	1.0	Footpath	Beach huts at 5m	PH.B	0.55	1.77	[2]
13/4	Lake	5.8	Concrete	2.57	Vertical	1.2	Footpath	House at >15m	PH.A	0.44	1.77	0.03

Notes:
PB - Poole Bay wave climate; PH- Poole Harbour wave climate;
1 - Outside the range of existing models but evidently high overtopping;
2 - Crest below still water level, ie weir flow over wall

Table 4.3 Analysis of overtopping discharge for 1:50 year storm after 50 years of sea level rise

Unit and subsection	Location	Cross-section Figure	Seawall type	Crest level (mOD)	Slope	Beach level at toe (mOD)	Land use at Seawall	Subsequent land use	Wave Climate	Near-shore wave height H_s (m)	Water level (mOD)	Discharge (l/s/m)
1/4	Branksome	5.3	Concrete	3.20	Stepped	1.1	Promenade	Beach huts at 3m	PB.D	3.3	2.08	267
1/6	Canford Cliffs	5.3	Concrete	3.20	Vertical	1.1	Promenade	Beach huts at 4m	PB.D	3.3	2.08	80
1/8	Flag head	5.3	Concrete	3.14	Stepped	1.4	Promenade	Beach huts at 4m	PB.D	3.3	2.08	74
1/10	Sandbanks	5.4	Concrete	3.50	Sloping	2.4	Garden	Houses/Hotels at >3m	PB.C	2.7	2.08	0
1/11	Sandbanks	5.4	Concrete	2.55	Steep	2.6	Promenade	Beach huts at 4m	PB.C	2.7	2.08	0
1/14	Sandbanks	5.4	Steel	2.9	Vertical	1.2	Garden	Houses at 20m	PB.B	3.6	2.05	[1]
3/3	Sandbanks	5.5	Masonry	3.00	Vertical	0.5	Private garden	Houses at >20m	PH.I	0.49	2.05	0.015
4/2	Banks Road	5.5	Concrete	1.59	Vertical with step	0.0	Footpath	Road at 9m (2.0mOD)	PH.I	0.49	2.05	[2]
4/5	Sandbanks	5.5	Concrete	2.03	Vertical	0.0	Footpath	Road at 9m (1.5mOD)	PH.H	0.56	2.05	[2]
5/2	Evening Hill	5.6	Gabion	1.50	Steep	0.6	Footpath	Grass slope	PH.K	0.50	2.05	[2]
5/4	Lilliput	5.6	Masonry	2.22	Steep	0.6	Private garden (sloping)	House at >20m	PH.K	0.50	2.05	9.4
6/3	Parkstone	5.6	Masonry	2.00	Steep	0.6	Private garden (sloping)	House at 30m	PH.F	0.53	2.05	[2]
7/4	Parkstone Bay	5.7	Rock	2.00	1:2	0.3	Footpath	Recreation ground	PH.E	0.56	2.05	[2]
8/2	Town Quay	5.7	SSP	1.90	Vertical	-5.0	Quay	Road at >10m	PH.D	0.57	2.05	[2]
9/2	Holes Bay	5.7	Rock	2.05	1:2	-0.6	Footpath	Road at >10m	PH.L	0.30	2.05	61
13/2	Hamworthy Park	5.8	Concrete	1.30	Vertical	1.0	Footpath	Beach huts at 5m	PH.B	0.55	2.05	[2]
13/4	Lake	5.8	Concrete	2.57	Vertical	1.2	Footpath	House at >15m	PH.A	0.44	2.05	0.9

Notes:
PB - Poole Bay wave climate; PH - Poole Harbour wave climate;
1 - Outside the range of existing models but evidently high overtopping;
2 - Crest below still water level, ie weir flow over wall

Table 4.4 Management Unit 1 Poole Bay

Sub-section	Length (m)	Land use		Seawall				Foreshore	
		Type	Level (mOD)	Type	Material	Condition	Level (mOD)	Type	Level at seawall toe (mOD)
1	190	Promenade Eroding Cliffs	3.2	Stepped	Concrete	Good	3.20	Sand	0.7
2	465	Promenade Beach huts Cliffs	3.2	Stepped	Concrete	Fair	3.20	Sand	0.7
3	555	Promenade Beach huts Cliffs	3.2	Stepped	Concrete	Fair	3.20	Sand	1.0
4	245	Promenade Eroding Cliffs	3.2	Stepped	Concrete	Good	3.20	Sand	1.1
5	125	Promenade Beach huts Cliffs	3.7	Vertical	Concrete	Good	3.20	Sand	1.5
6	460	Promenade Beach huts Cliffs	3.0	Vertical with small curve	Concrete	Fair	3.20	Sand	1.4
7	160	Promenade Vegetated Cliffs	3.2	Vertical with small curve	Concrete	Good	3.71	Sand	0.8
8	220	Promenade Beach hut Cliffs	3.2	Stepped	Concrete	Fair	3.14	Sand	1.4

See Drawing 1

Notes:

1. Lengths to nearest 5m

2. Seawall levels from MAFF (1993) Coast Protection Survey. Foreshore levels from PBC beach monitoring (early 1994 readings)

Table 4.4 Continued Management Unit 1 Poole Bay

Sub-section	Length (m)	Land use		Seawall				Foreshore	
		Type	Level (mOD)	Type	Material	Condition	Level (MOD)	Type	Level at seawall toe (mOD)
9	230	Promenade Beach hut Cliffs	3.2	Stepped	Concrete	Fair	3.14	Sand	-
10	640	Houses and Hotels	3.5	Various sloping	Concrete	Poor	3.50	Sand	2.4
11	450	Promenade Commercial	2.6	Vertical	Concrete	Fair	2.55	Sand	2.6
12	155	Houses	5+	Vertical	Masonry	Fair	2.99	Sand	-
13	170	Houses	5.5	Dune	Vegetated sand	Poor	-	Sand	1.2
14	110	Houses	2.9	Sheet pile	Steel	Fair	3.0	Sand	1.2
15	220	Houses	3.8	Near vertical	Masonry Steel Concrete Some rock	Fair	3.8	Sand	0.9
16	75	Haven Hotel	3.0	Bermed with recurve	Concrete Steel Rock	Fair	3.7	Sand	1.2

See Drawing 1

Notes:

1. Lengths to nearest 5m
2. Seawall levels from MAFF (1993) Coast Protection Survey. Foreshore levels from PBC beach monitoring (early 1994 readings)

Table 4.5 Management Unit 2 Haven Hotel to North Haven Point

Sub-section	Length (m)	Land use		Seawall				Foreshore	
		Type	Level (mOD)	Type	Material	Condition	Level (MOD)	Type	Level at seawall toe (mOD)
1	80	Hotel	3.0	Bermed wall	Rock Steel Concrete	Good	3.7	Sand	-1.5
2	20	Slipway	2.7	Slipway	Concrete	Good	-	-	-
3	40	Wharf	2.7	Sheet pile	Steel	Good	2.70	Sand	-1.2
4	370	Gardens Houses	2.5+	Vertical wall	Concrete Steel	Poor to Good	Varies	Sand	-1.2

See Drawing 1

Notes:

1. Lengths to nearest 5m

2. Seawall levels from MAFF (1993) Coast Protection Survey.

Table 4.6 Management Unit 3 North Haven Hotel Point to Whitley Lake

Sub-section	Length (m)	Land use		Seawall				Foreshore	
		Type	Level (mOD)	Type	Material	Condition	Level (MOD)	Type	Level at seawall toe (mOD)
1	395	Private housing	2+	Vertical wall	Concrete Masonry Steel	Fair to Good	3.0	Sand Patches of shingle	-1.0
2	70	Marinas	-	Sheet pile jetties	Steel	Good	-	-	-
3	385	Private housing and marinas	2+	Vertical wall	Concrete Masonry Steel	Fair to Good	3.0	Sandy mud, scattered boulders	-1.0

See Drawing 1

Notes:

1. Lengths to nearest 5m

2. Seawall levels from MAFF (1993) Coast Protection Survey.

Table 4.7 Management Unit 4 Whitley Lake

Sub-section	Length (m)	Land use		Seawall				Foreshore	
		Type	Level (mOD)	Type	Material	Condition	Level (MOD)	Type	Level at seawall toe (mOD)
1	30	Road	-	Vertical wall	Masonry	Fair	-	Sand with gravel upper. Muddy sand lower	-0.5
2	895	Road	1.8	Vertical wall with step	Concrete	Poor	1.59	Muddy sand	0.0
3	220	Road	-	Vertical	Concrete	Poor	-	Muddy sand	-
4	190	Road	1.5	Vertical	Concrete	Poor	2.03	Vegetated sand upper. Muddy sand lower	Varies
5	615	Road	1.5	Vertical with round crest	Concrete	Good	2.03	Sand and gravel upper. Muddy sand lower	0.0

See Drawing 1

Notes:

1. Lengths to nearest 5m

2. Seawall levels from MAFF (1993) Coast Protection Survey.

Table 4.8 Management Unit 5 Dorset Sailing Club to Salterns Marina

Sub-section	Length (m)	Land use		Seawall				Foreshore	
		Type	Level (mOD)	Type	Material	Condition	Level (MOD)	Type	Level at seawall toe (mOD)
1	60	Road	2.5+	Wall	Concrete	Poor	2.03	Sand & shingle upper. Muddy lower	Upper beach above HW
2	255	Footpath Vegtated Slope	2.5+	Gabion baskets	Stone	Bad	1.50	Muddy sand with boulders	-0.5
3	75	Gardens and houses	3.0+	Wall Toe protected by sewer pipe	Concrete	Bad	2.22	Muddy sand with boulders	-
4	350	Gardens and houses	3.0+	Wall	Various (concrete, masonry, SSP)	Fair to Poor	Various (2.2)	Sand some mud	-
5	80	Gardens and houses	2.0+	Wall behind sand bank	Blockwork	Poor	-	Sand upper. Muddy sand lower	Upper beach above HW

See Drawing 2

Notes:

1. Lengths to nearest 5m
2. Levels from MAFF (1993) Coast Protection Survey.

Table 4.9 Management Unit 6 Salterns Marina to Parkstone Yatch Club

Sub-section	Length (m)	Land use		Seawall				Foreshore	
		Type	Level (mOD)	Type	Material	Condition	Level (MOD)	Type	Level at seawall toe (mOD)
1	85	Marina	-	Sheet pile	Steel	Good	-	Sand	-
2	390	Blue Lagoon	-	Vertical wall	Concrete Steel Gabions	Bad	1.5	Sand and shingle	0.0
3	450	Private gardens and houses	2.5	Vertical wall	Concrete Steel Masonry	Poor to Good	2.0	Sand and shingle	0.5
4	185	Marina	1.5	Vertical Wall	Concrete	Fair	-	Sand and shingle	-

See Drawing 2

Notes:

1. Lengths to nearest 5m

2. Levels from MAFF (1993) Coast Protection Survey.

Table 4.10 Management Unit 7 Parkstone Bay to Baiter Park

Sub-section	Length (m)	Land use		Seawall				Foreshore	
		Type	Level (mOD)	Type	Material	Condition	Level (MOD)	Type	Level at seawall toe (mOD)
1	230	Marina	1.5	Sheet pile	Steel	Good	-	Muddy sand	-
2	120	Boat yard	-	Earth bank	Vegetated	Fair	-	Muddy sand	-
3	(230)	Recreation ground	2.0	Sloping wall	Concrete	Poor	2.5 (2.3 in NRA survey)	Sand with debris	0.5
4	(1210)	Recreation ground	2.0	Sloping revetment	Rock	Fair	2.0	Sand with debris	0.3
5	100	Footpath Car park Recreation ground	1.5	Earth bank	Shingle in front Tarmac footpath on crest	Fair	1.3 (1.4 in NRA survey)	Sand and shingle above mud to gravel	-
6	180	Footpath Car park Recreation ground	1.7	Stepped wall	Concrete	Good	-	Sand and shingle above mud to gravel	-
7	160	Footpath Recreation ground	1.5	Sloping revetment	Rock	Good	1.3	Sand and shingle above mud to gravel	0.5
8	155	Footpath Road Housing	1.7	Sloping revetment	Interlocking blocks	Good	1.2	Sand and shingle above mud to gravel	0.5
9	65	Boat yard	-	Apron slab	Concrete	Fair	-	Sand and shingle above mud to gravel	-

See Drawing 2

Notes:

1. Lengths to nearest 5m
2. Levels from MAFF (1993) Coast Protection Survey.

Table 4.11 Management Unit 8 Town Quays

Sub-section	Length (m)	Land use		Seawall				Foreshore	
		Type	Level (mOD)	Type	Material	Condition	Level (MOD)	Type	Level at seawall toe (mOD)
1	140	Quay Boatyard	1.5	Vertical wall	Masonry	Fair	1.42	Sand	0.0
2	770	Quay Car park Commerical	1.9	Vertical wall	Masonry Steel Concrete	Fair to Good	1.85	Sand	-3.5 to -5.0
3	740	Commerical Factories	1.5	Jetties Vertical wall	Masonry Steel Concrete	Poor to Fair	-1.45	Mud Silt	-0.5

See Drawing 2

Notes:

1. Lengths to nearest 5m
2. Levels from MAFF (1993) Coast Protection Survey.

Table 4.12 Management Unit 9 Eastern Holes Bay

Sub-section	Length (m)	Land use		Seawall				Foreshore	
		Type	Level (mOD)	Type	Material	Condition	Level (MOD)	Type	Level at seawall toe (mOD)
1	390	Industrial	1.7	Revetment	Rock	Good	1.68	Mud to shingle	0.7
2	1520	Footpath Grass Road	2.1	Revetment	Rock	Good	2.05	Mud to shingle debris	-0.7

See Drawing 3

Notes:

1. Lengths to nearest 5m
2. Levels from MAFF (1993) Coast Protection Survey.

Table 4.13 Management Unit 10 Northern Holes Bay

Sub-section	Length (m)	Land use		Seawall				Foreshore	
		Type	Level (mOD)	Type	Material	Condition	Level (MOD)	Type	Level at seawall toe (mOD)
1	980	Road	2.0+	Embankment	Rubble & earth	Fair	-	Salting	-
2	1530	Country park	-	Salting	-	-	1.0	Salting	-

See Drawing 3

Notes:

1. Lengths to nearest 5m
2. Levels from MAFF (1993) Coast Protection Survey.

Table 4.14 Management Unit 11 Western Holes Bay

Sub-section	Length (m)	Land use		Seawall				Foreshore	
		Type	Level (mOD)	Type	Material	Condition	Level (MOD)	Type	Level at seawall toe (mOD)
1	810	Housing	-	Salting	-	-	-	Salting	-
2	370	Marina	-	Sheet pile	Steel	Good	-	-	Below MLW
3	1380	Housing, School, Industrial	-	Earth bank	Vegetated	Fair	-	Salting	-

See Drawing 3

Notes:

1. Lengths to nearest 5m

2. Levels from MAFF (1993) Coast Protection Survey.

Table 4.15 Management Unit 12 Hamworthy Quays

Sub-section	Length (m)	Land use		Seawall				Foreshore	
		Type	Level (mOD)	Type	Material	Condition	Level (MOD)	Type	Level at seawall toe (mOD)
1	305	Disused Industrial	2.0	Vertical wall	Concrete	Fair	2.43	Sand	Below LW
2	55	Industrial	-	Revetment	Rock	Fair	-	-	-
3	105	Wharves	-	Vertical wall	Concrete	Fair	2.5	Sand	Below LW
4	705	Wharves	2.0	Vertical wall	Concrete steel	Good	2.5	Sand	Below LW
5	375	Port	2.5	Revetment	Rock	Good	2.9	Sand	Below LW
6	335	Port	2.5	Wall	Concrete	Good	2.5	Sand	Below LW
7	3750	Marina	-	Revetment Breakwater	Rock	Good	2.9	Sand	Below LW

See Drawing 2

Notes:

1. Lengths to nearest 5m
2. Levels from MAFF (1993) Coast Protection Survey.

Table 4.16 Management Unit 13 Lower Hamworthy to Rockley Viaduct

Sub-section	Length (m)	Land use		Seawall				Foreshore	
		Type	Level (mOD)	Type	Material	Condition	Level (MOD)	Type	Level at seawall toe (mOD)
1	230	Industrial Railway line	2.0	Earth bank	Grassed	Poor	2.0	Sand / shingle upper. Silt to shingle lower	0.5
2	175	Recreation ground	2.0	Near-vertical wall	Concrete	Poor	1.3 (from PBC)	Sand (nourished) upper. Silt to shingle lower	1.0
3	440		1.5	Near-vertical wall	Concrete	Fair	1.3 (from PBC)	Silt to shingle	-
4	625	Housing	2.0	Walls	Concrete Steel Rock	Poor to Good	2.6	Sand upper. Silt to shingle lower	1.0
5	275	Marina and Royal Marines	-	Marina slipways	-	Good	-	-	-
6	170	Domestic Car park	2.5	Earth bank	Vegetated	Fair	2.1	Sand upper. Silt to shingle lower	1.0
7	40	Car park	2.5	Gabions	Rock	Poor	2.1	Sand upper. Silt to shingle lower	1.0
8	600	Nature reserve	4.5	Cliff	Unprotected	Fair eroding	-	Sand with gravel. Silt to shingle lower	-

See Drawing 4

Notes:

1. Lengths to nearest 5m
2. Levels from MAFF (1993) Coast Protection Survey.

Table 4.17 Management Unit 14 Lytchett Bay

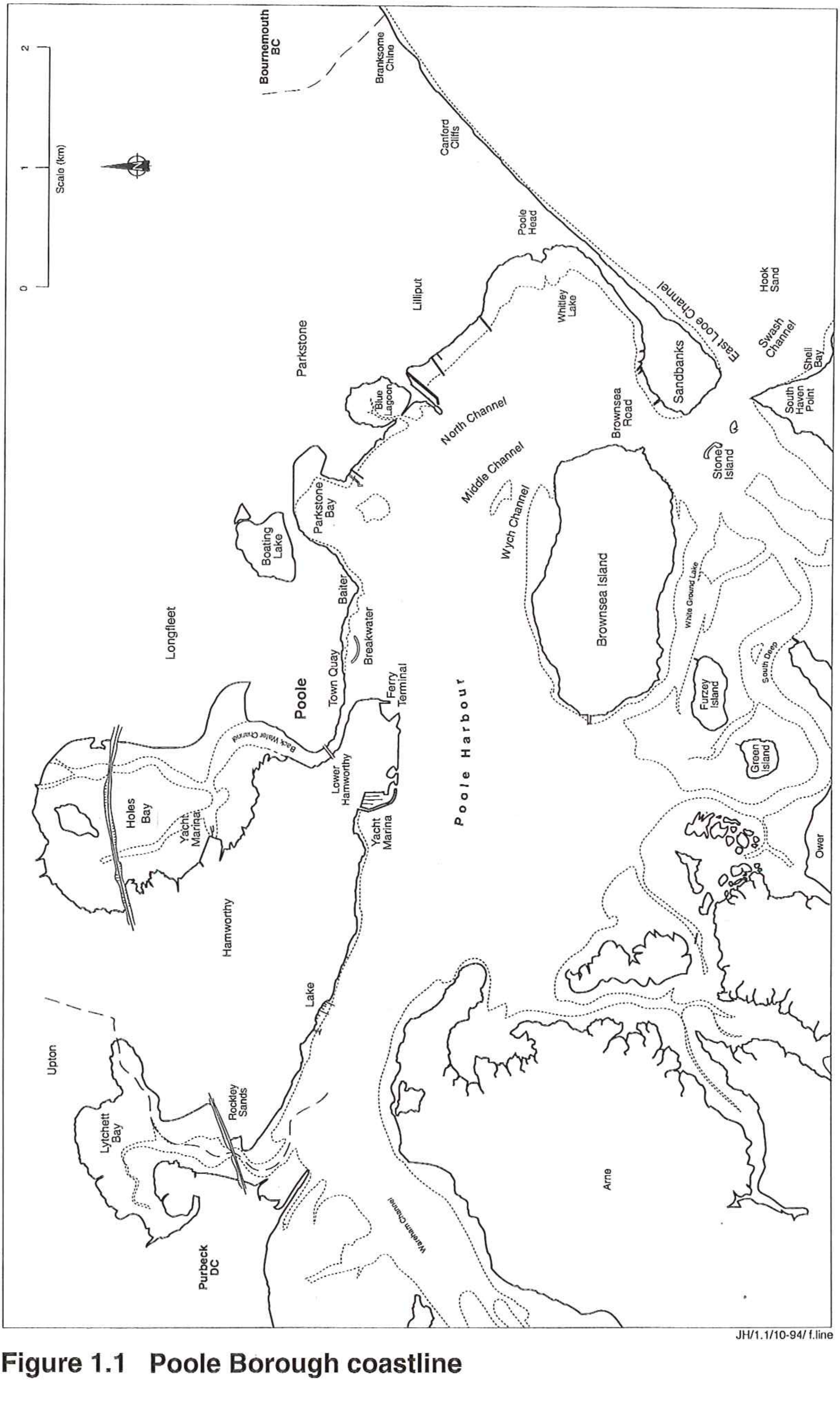
Sub-section	Length (m)	Land use		Seawall				Foreshore	
		Type	Level (mOD)	Type	Material	Condition	Level (MOD)	Type	Level at seawall toe (mOD)
1	310	Railway embankment	2.5	Embankment	Rubble	Fair	-	Sand and silt upper. Mud lower	-
2	495	Open ground houses	-	None	-	-	-	Sand and silt upper. Mud lower	-
3	925	Open ground Plaing fields and Houses	-	Embankment in reclaimed areas	Grassed earth	Good	-	Saltmarsh	-

See Drawing 4

Notes:

1. Lengths to nearest 5m
2. Levels from MAFF (1993) Coast Protection Survey.

Figures



JH/1.1/10-94/ f.line

Figure 1.1 Poole Borough coastline

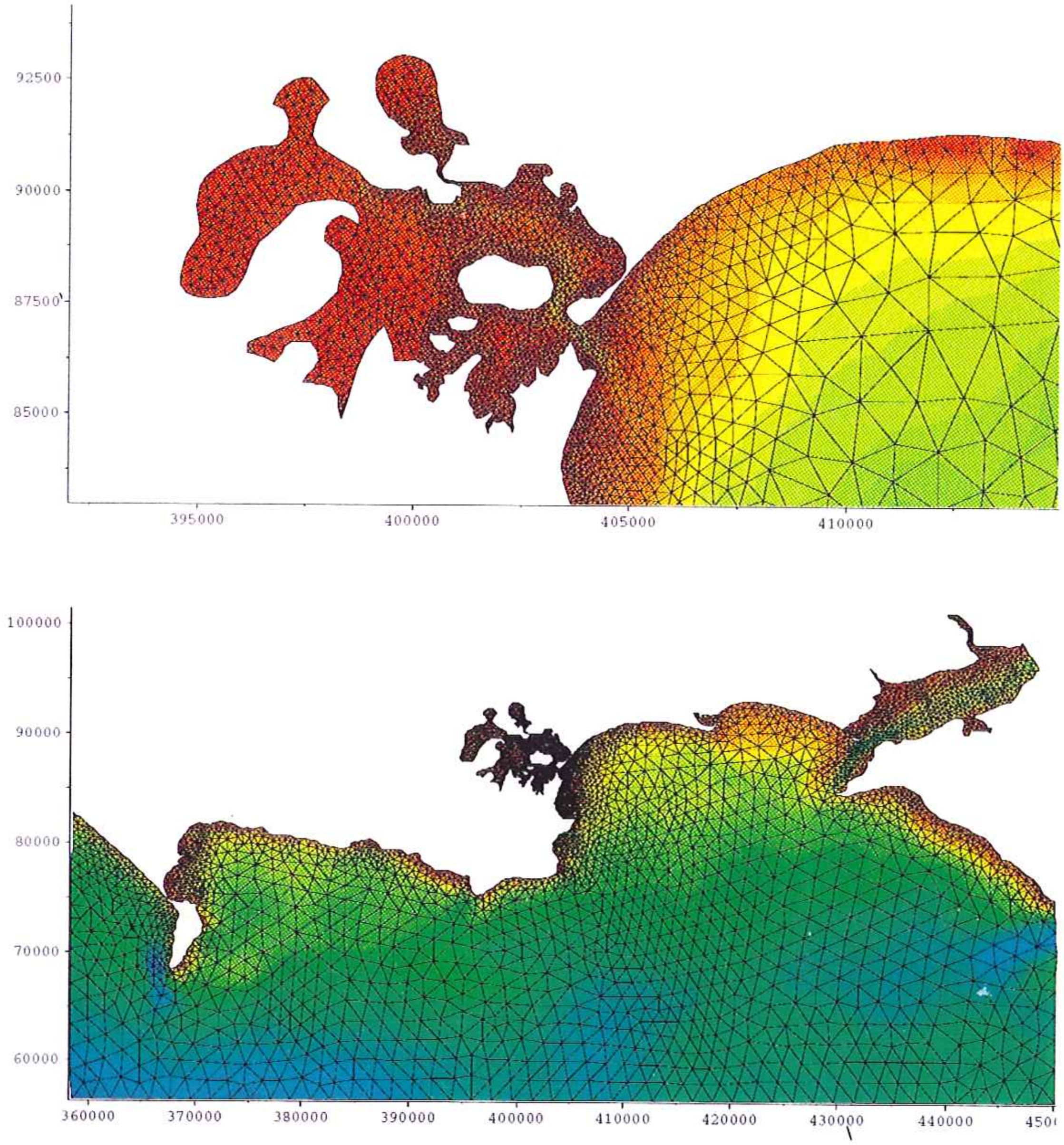


Figure 2.1 TELEMAC model mesh

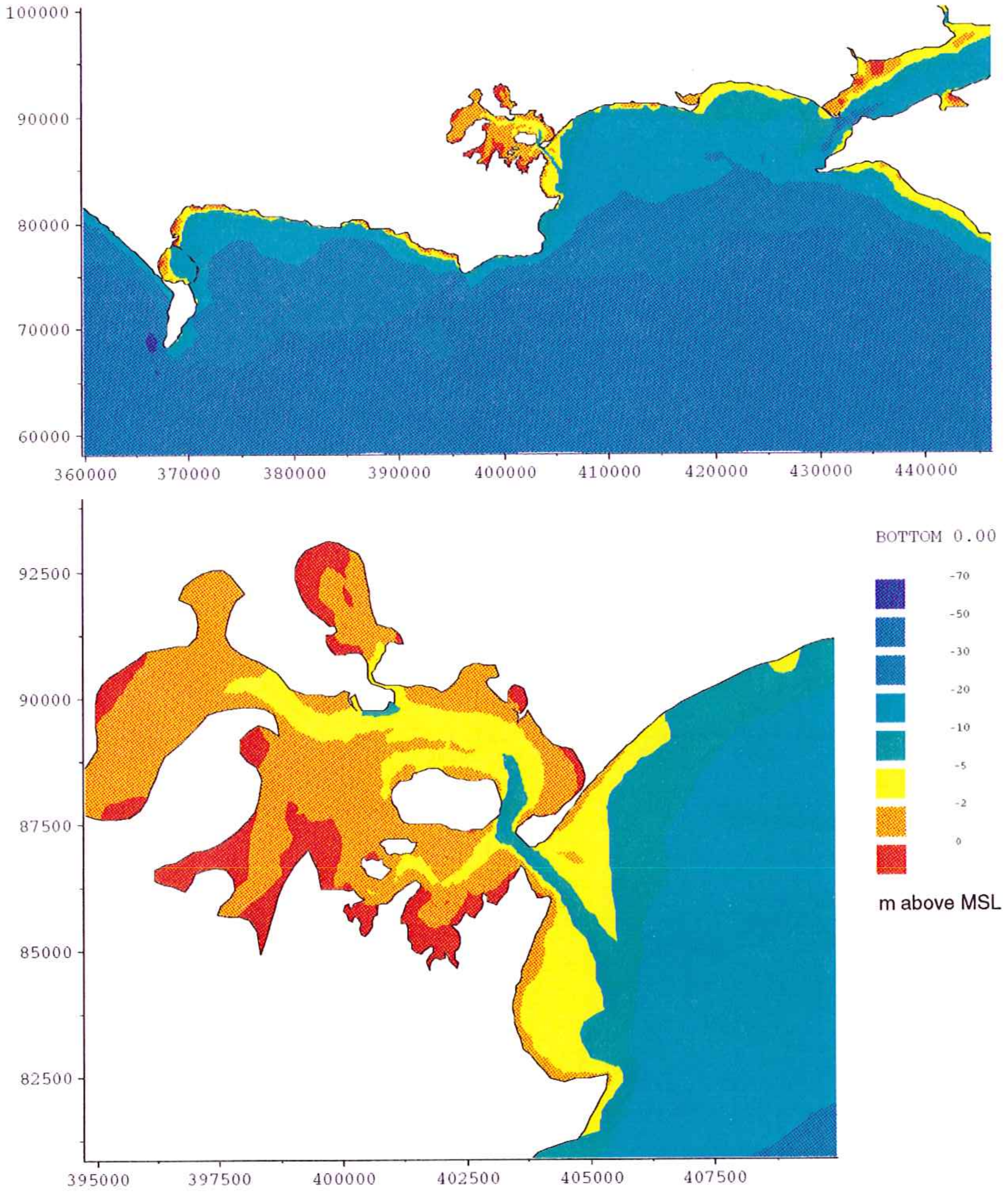


Figure 2.2 TELEMAC model bathymetry

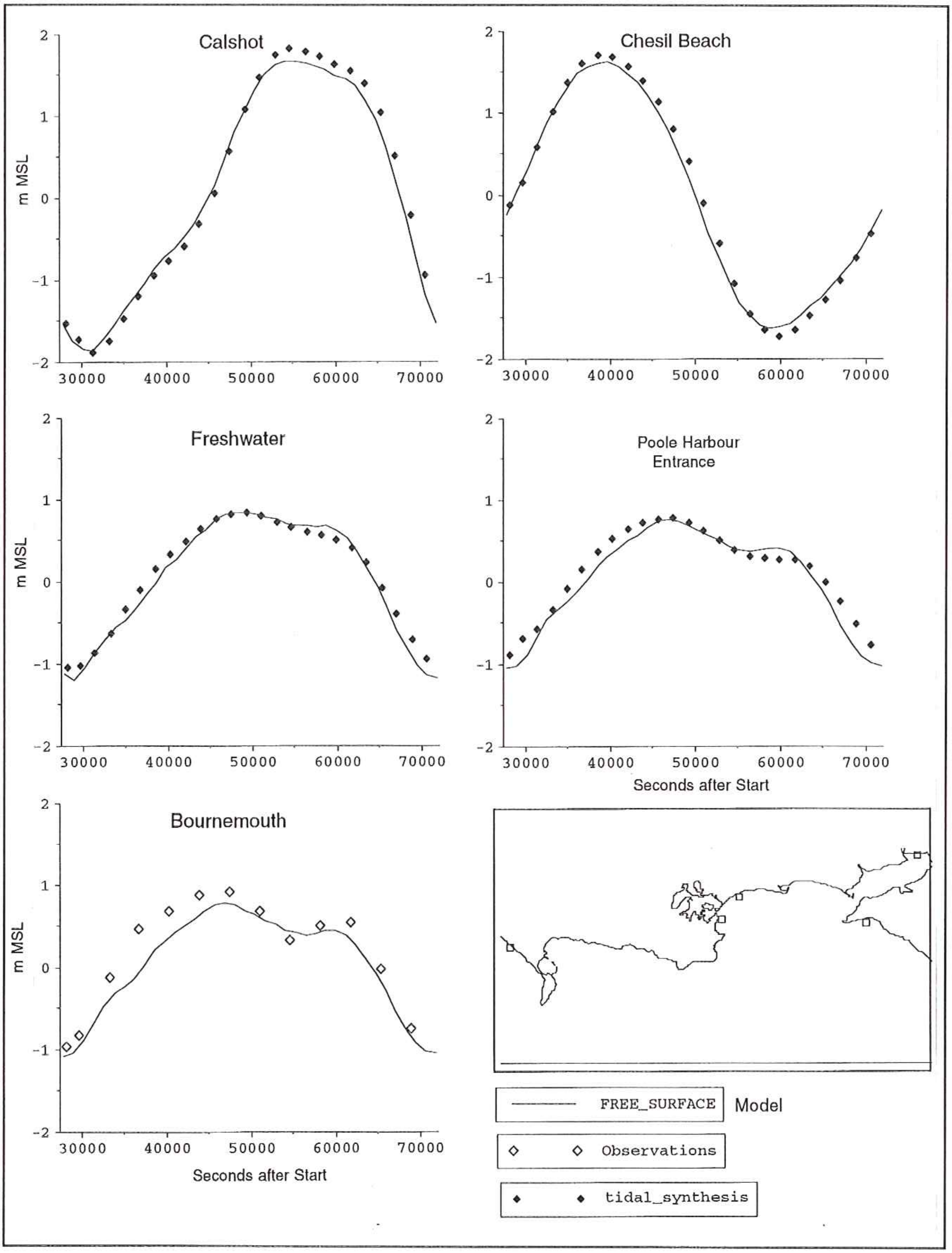


Figure 2.3 Tidal elevation calibration (spring tide)

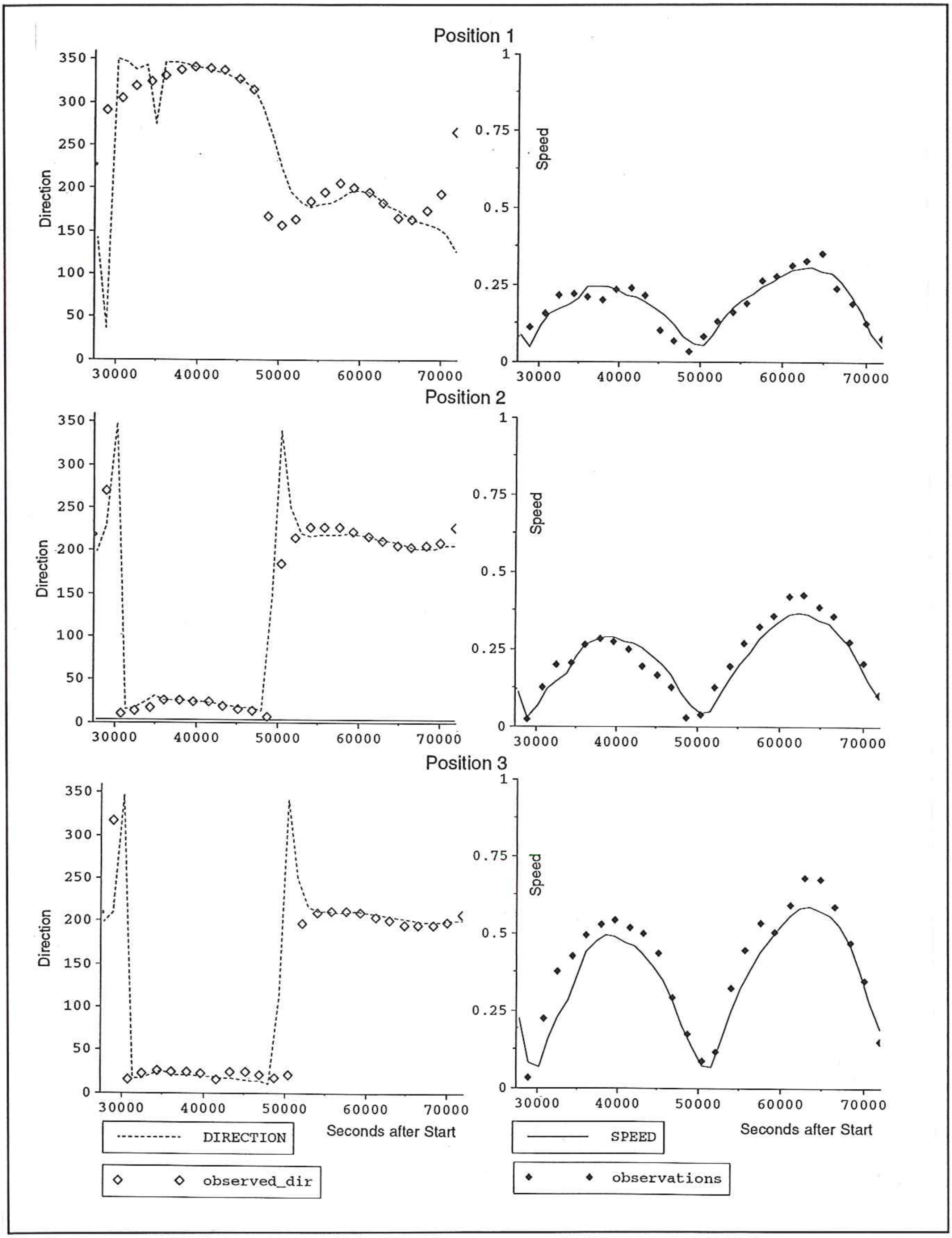


Figure 2.4 Tidal current calibration (spring tide) for observation sites in Poole Bay

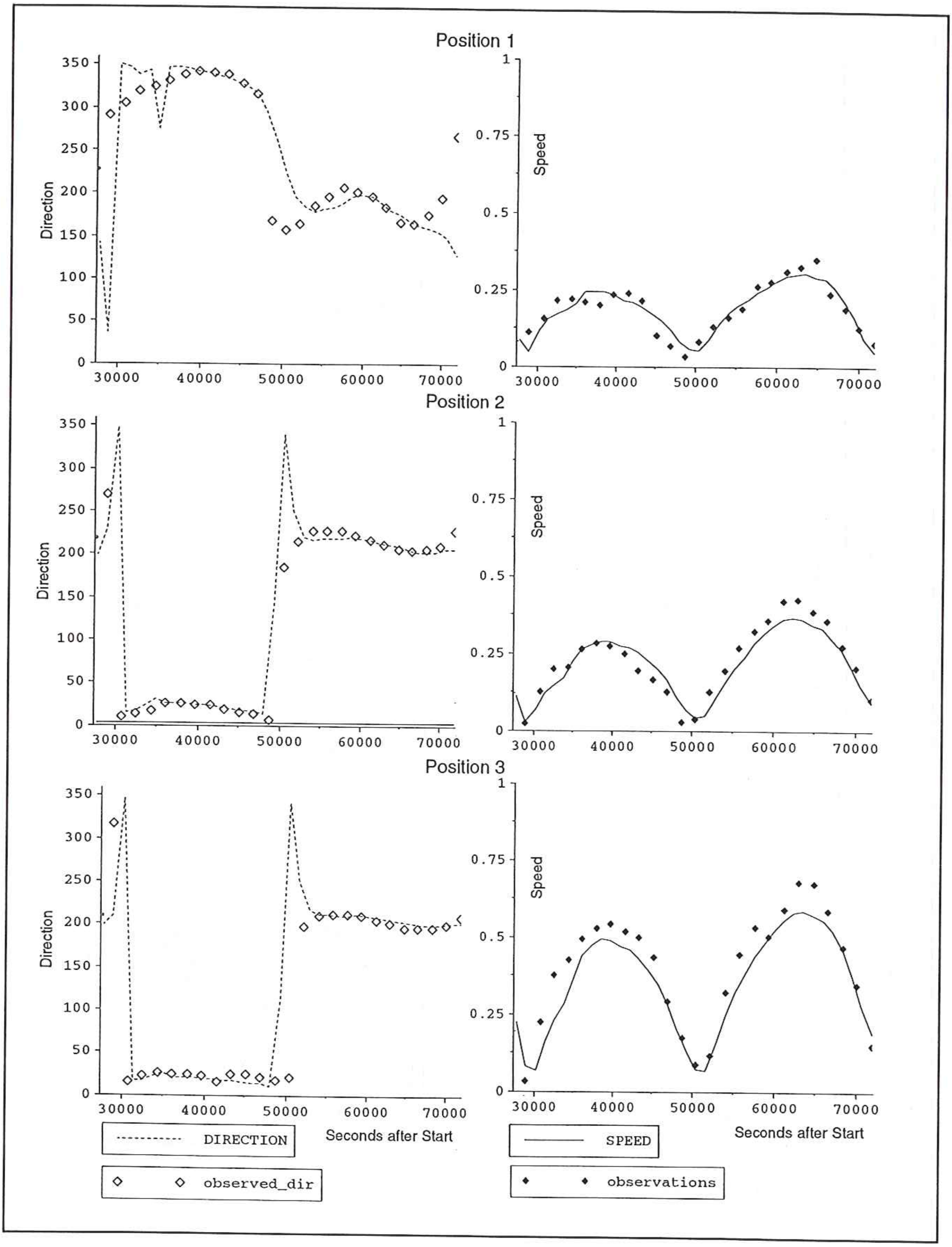


Figure 2.5 Tidal current calibration (neap tide) for observation sites in Poole Bay

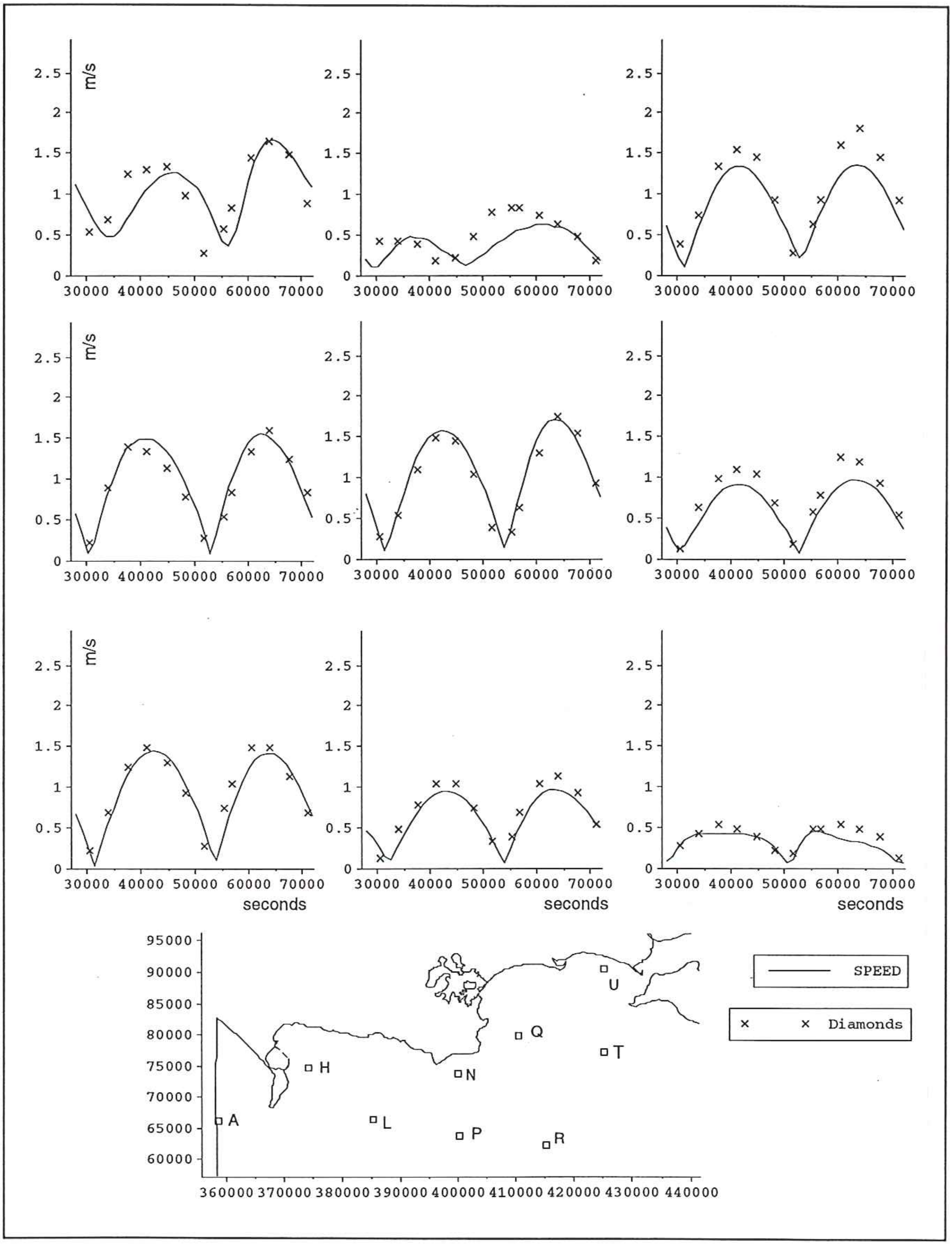


Figure 2.6 Comparison of model predictions with Admiralty diamonds

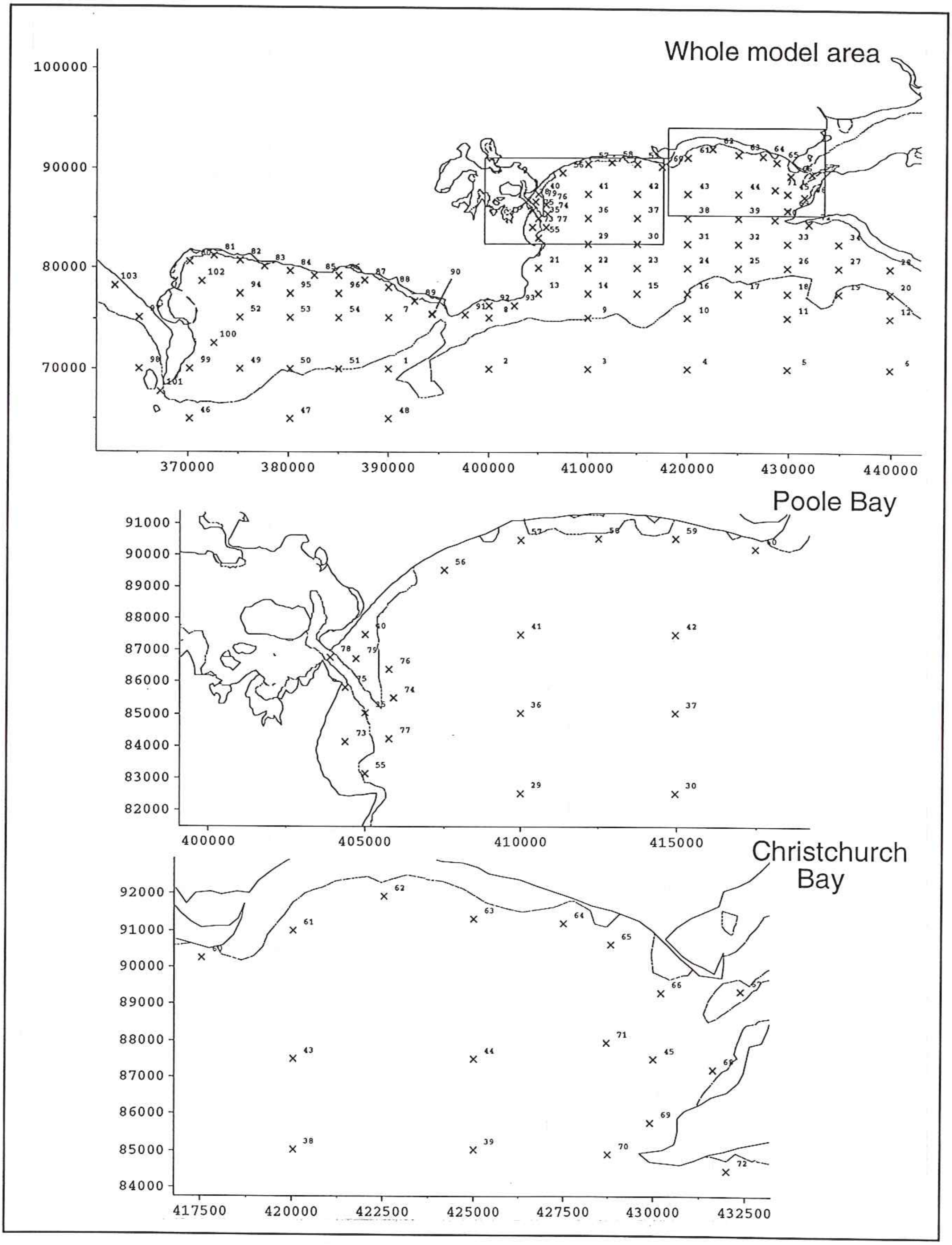


Figure 2.7 Model prediction points

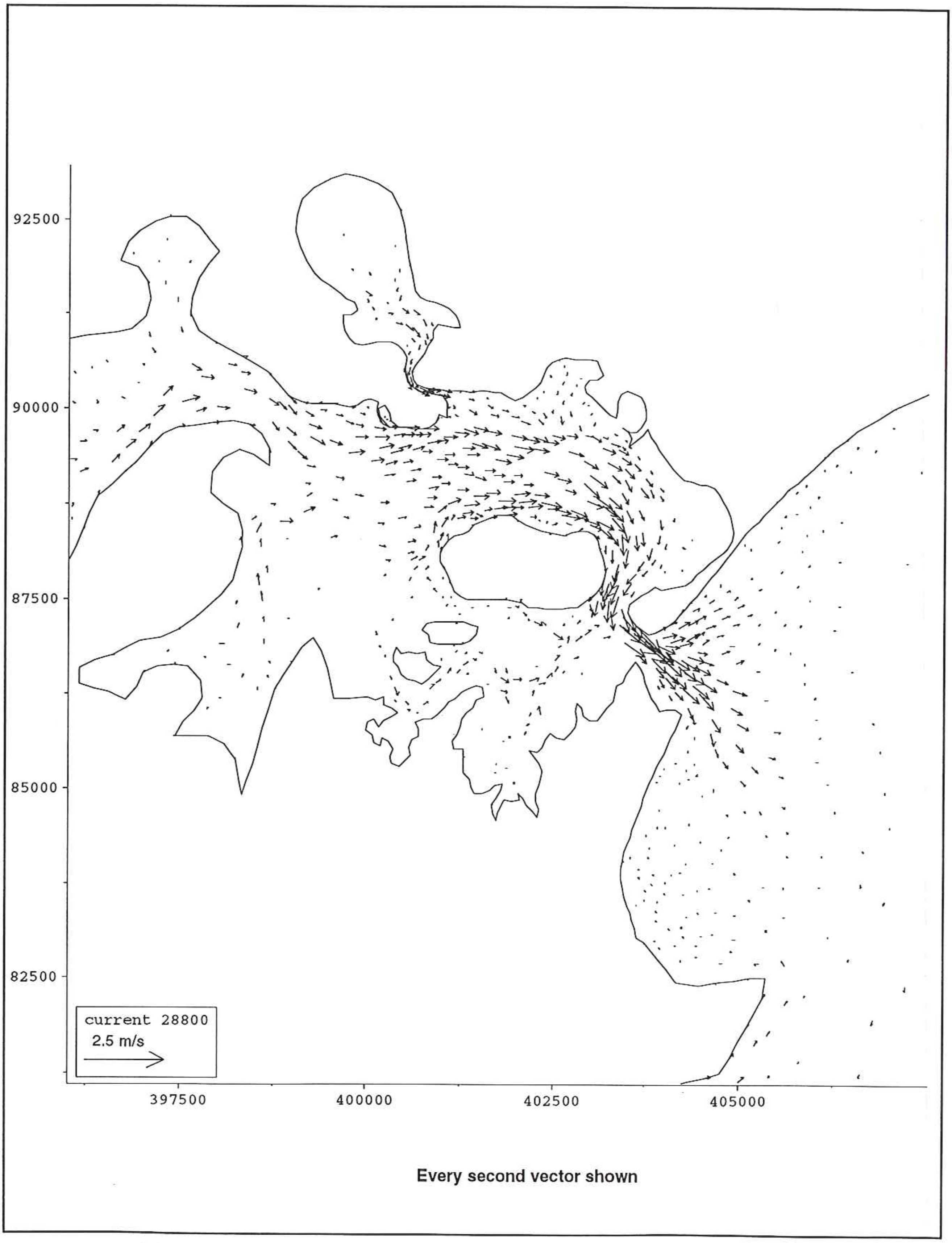


Figure 2.8 Tidal vectors at Low Water

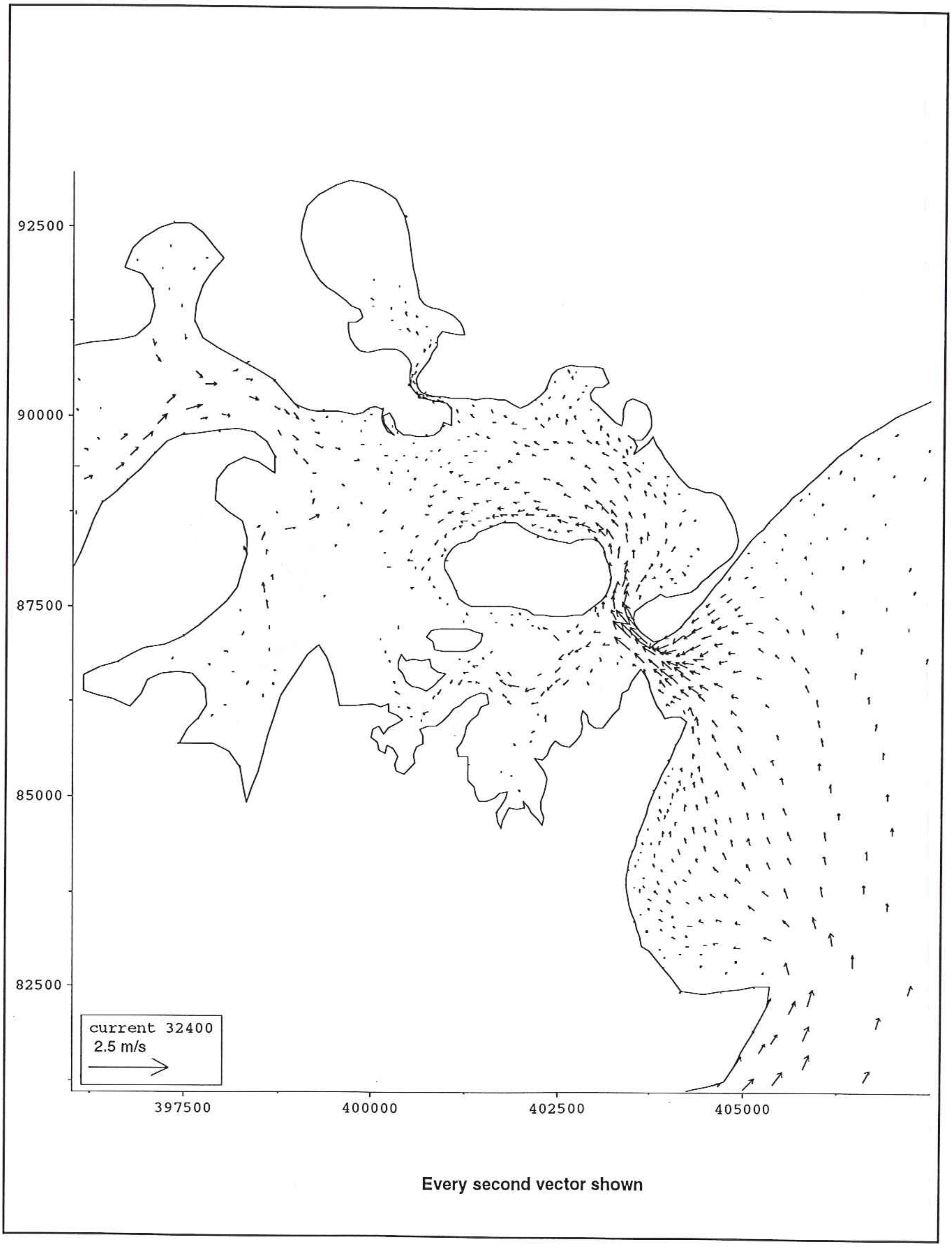


Figure 2.9 Tidal vector at 1 hour after Low Water

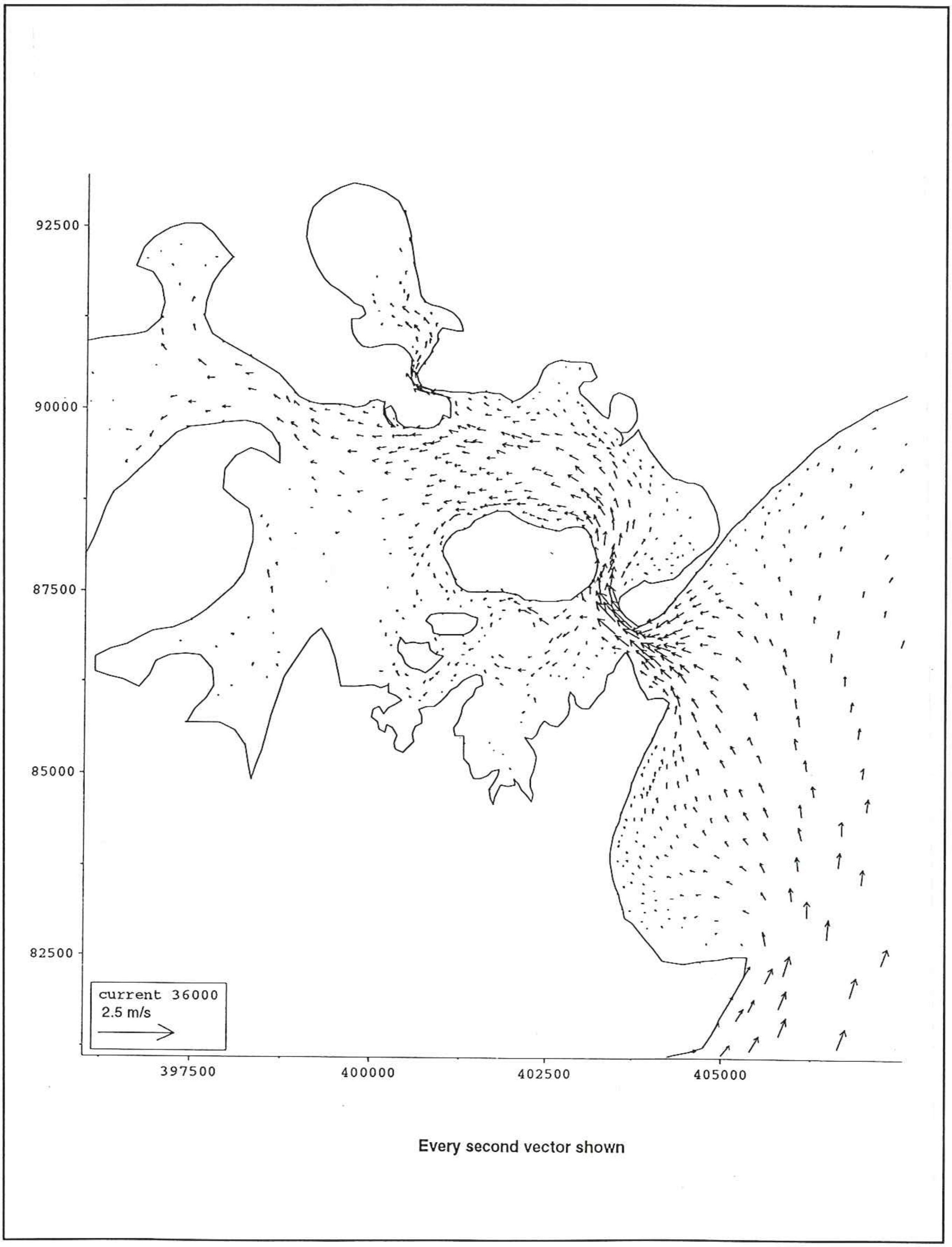


Figure 2.10 Tidal vectors at 2 hours after Low Water

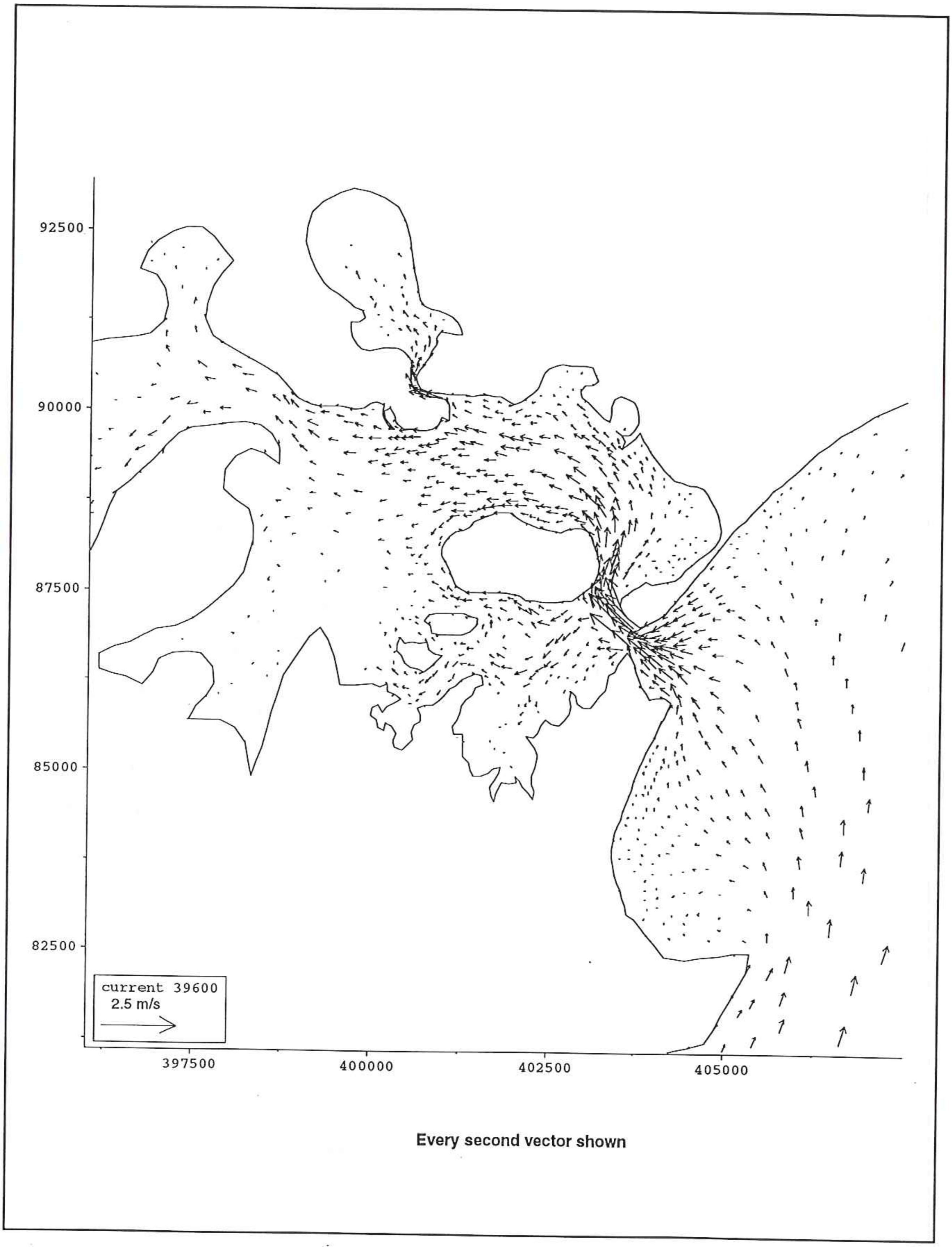
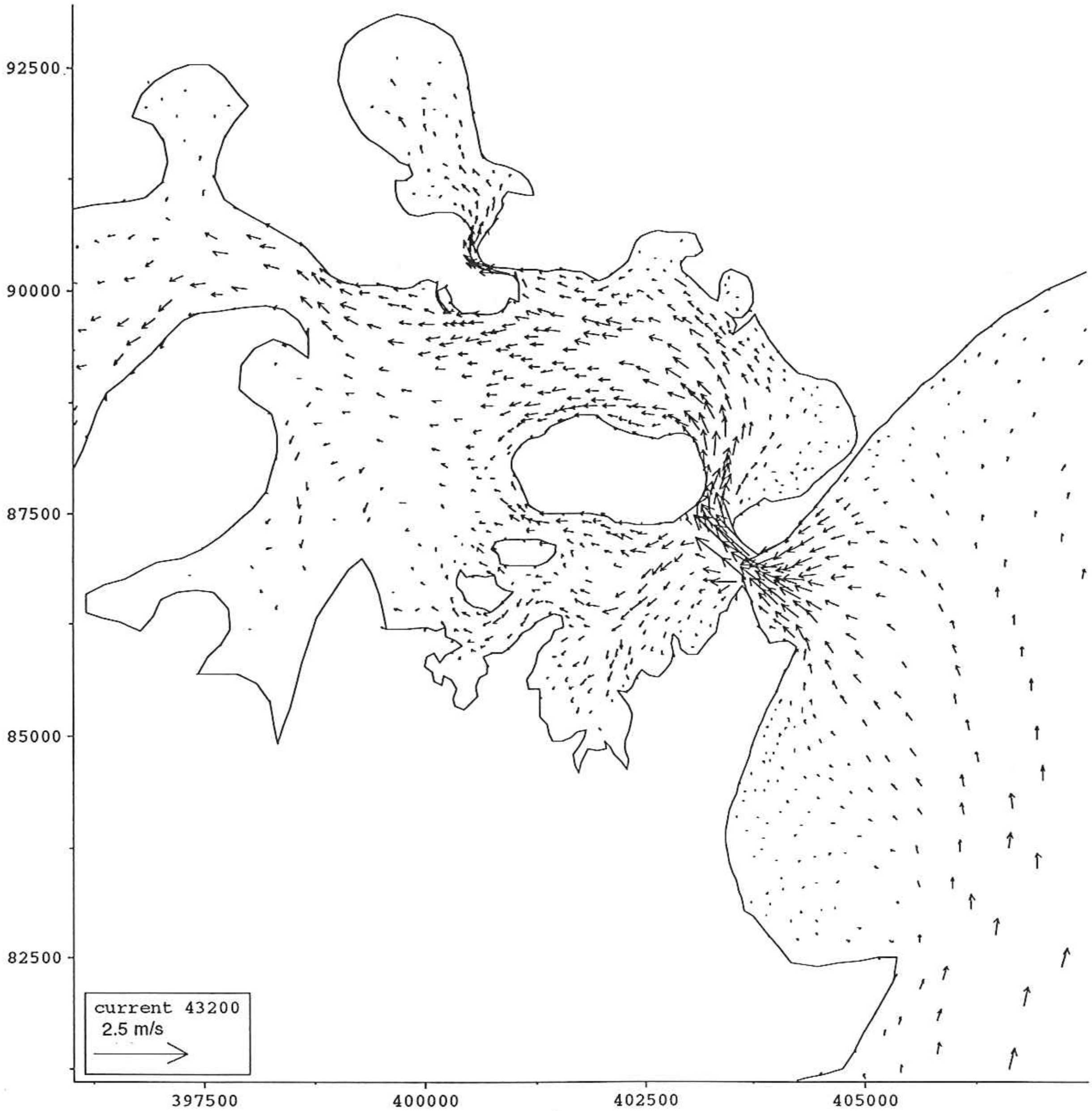


Figure 2.11 Tidal vectors at 3 hours after Low Water



Every second vector shown

Figure 2.12 Tidal vectors at 4 hours after Low Water

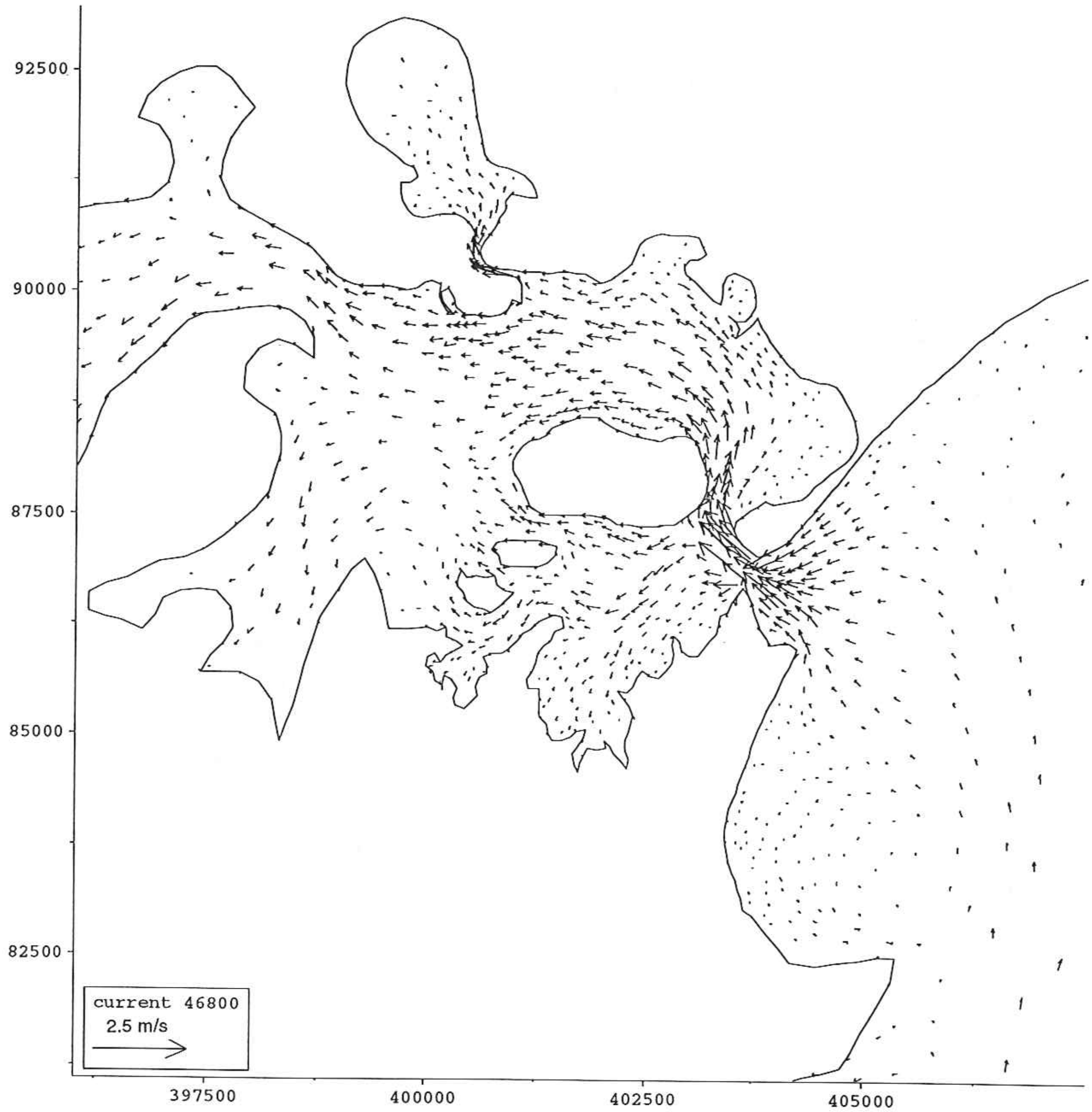


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Figure 2.13 Tidal vectors at 5 hours after Low Water

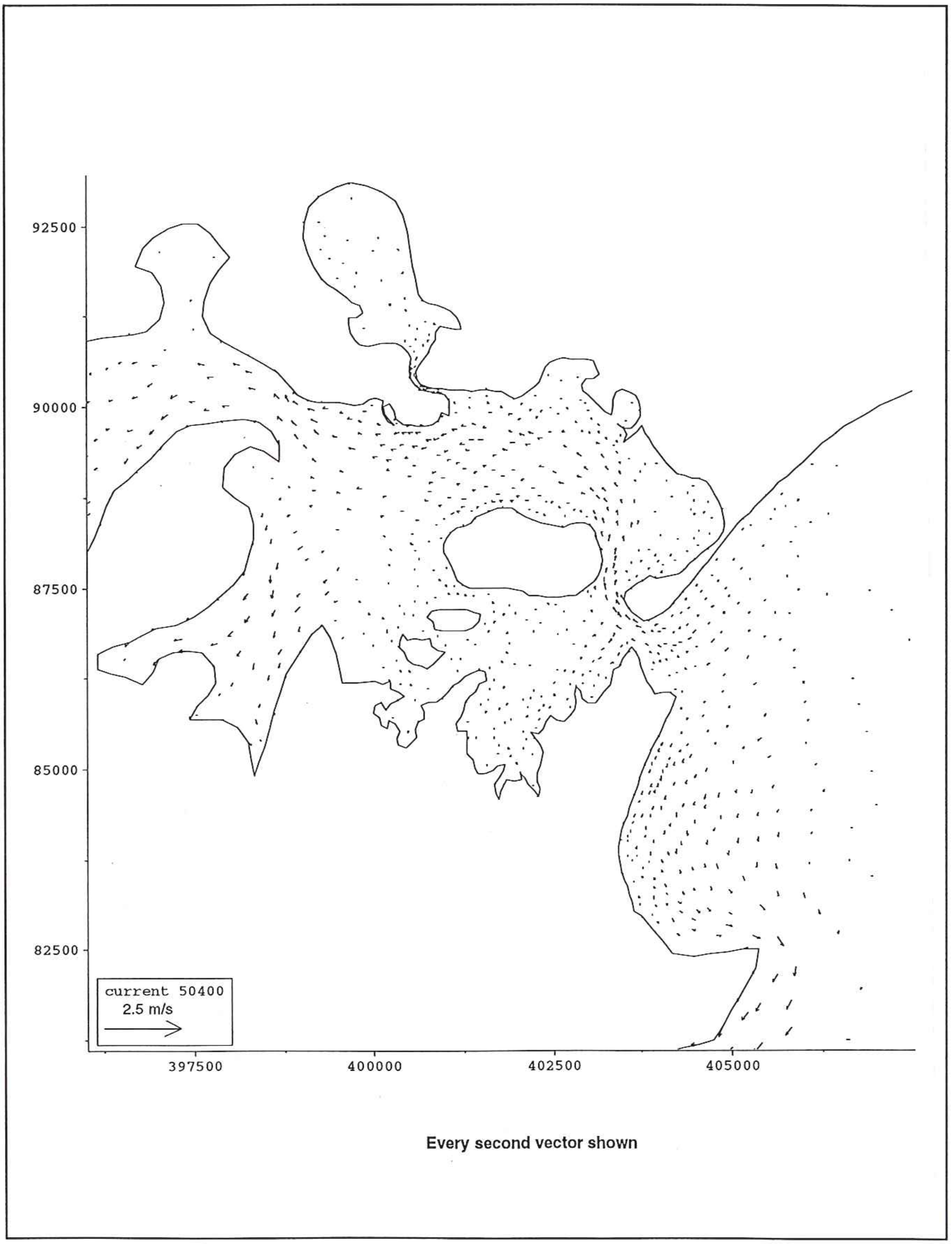


Figure 2.14 Tidal vectors at 6 hours after Low Water

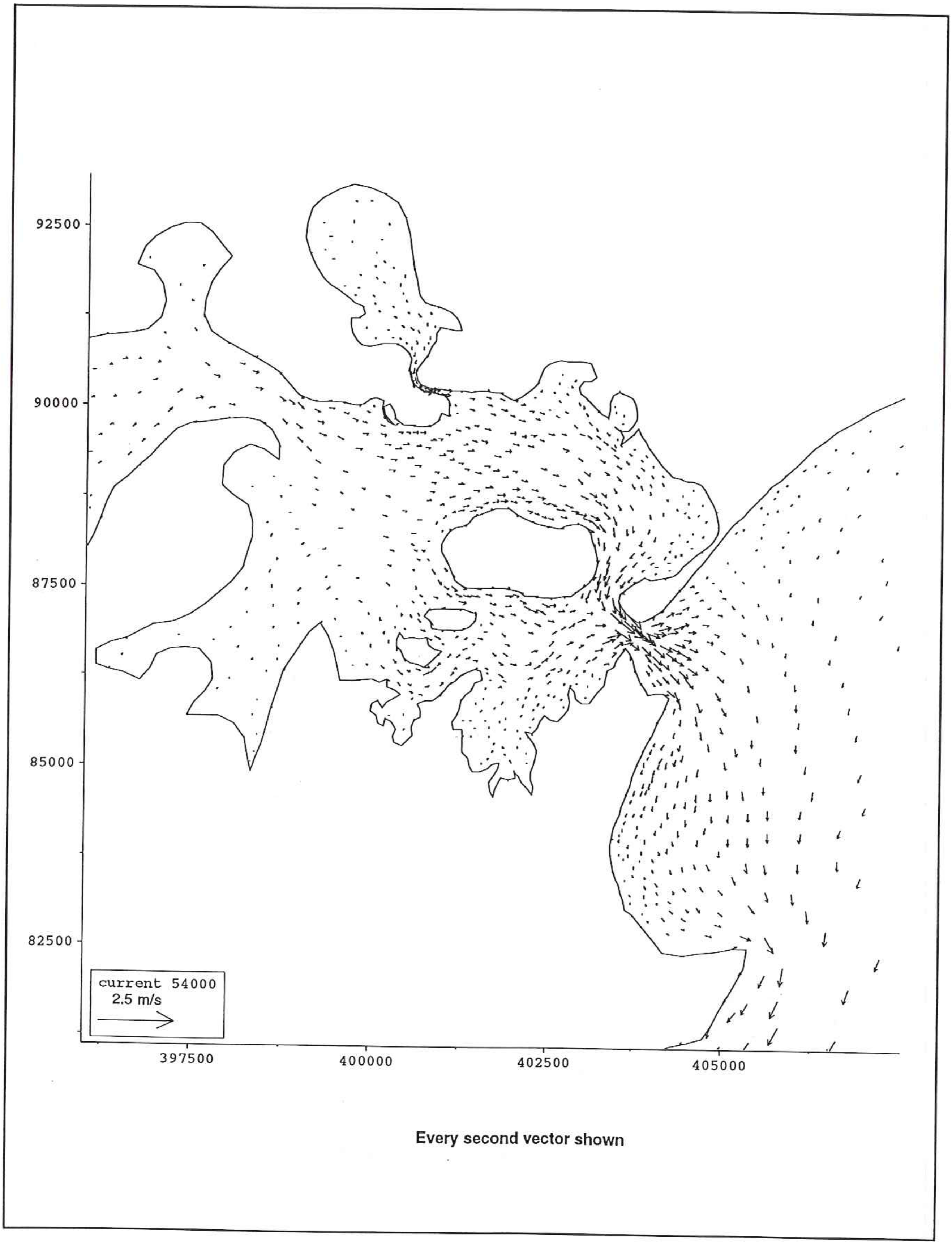


Figure 2.15 Tidal vectors at 7 hours after Low Water

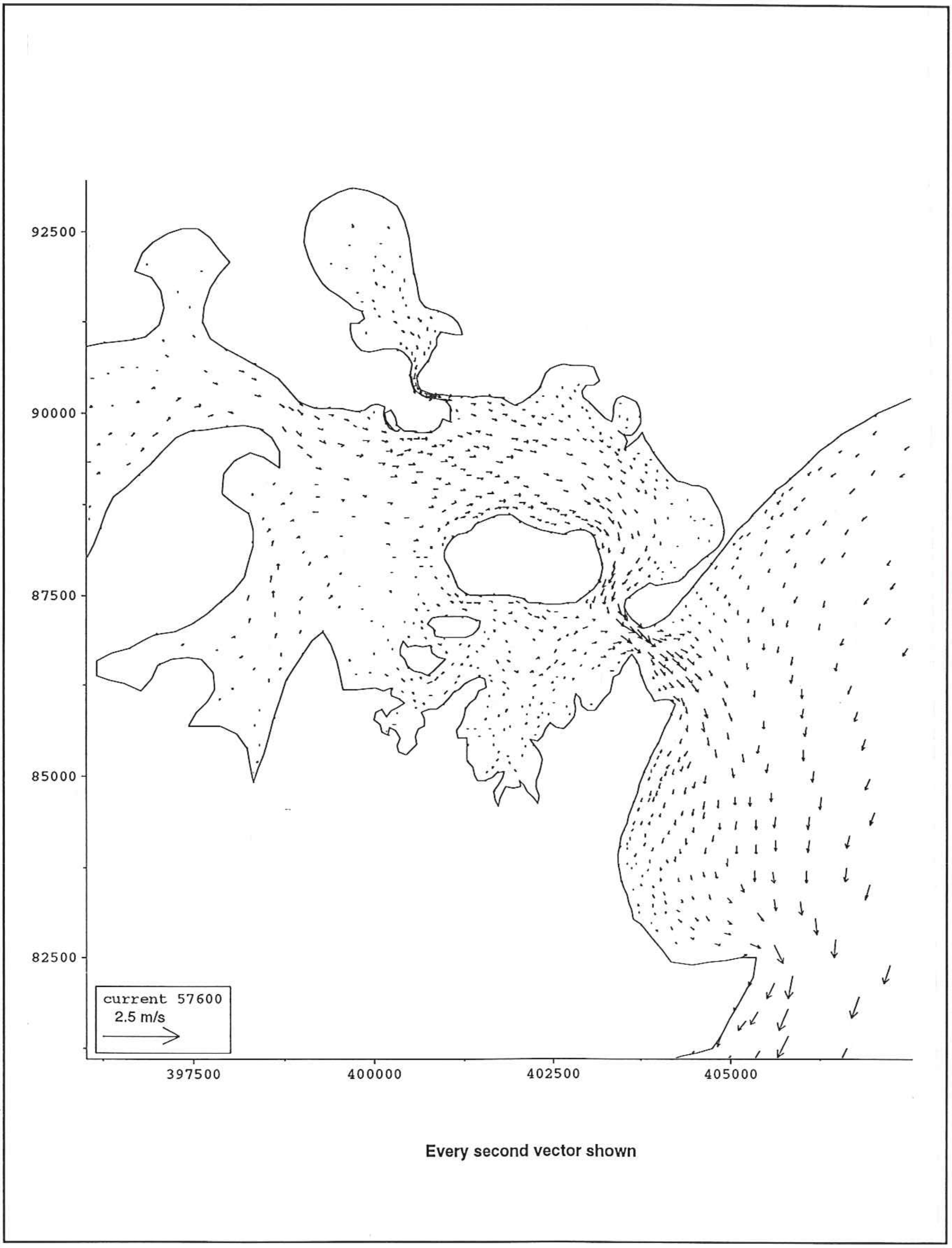


Figure 2.16 Tidal vectors at 8 hours after Low Water

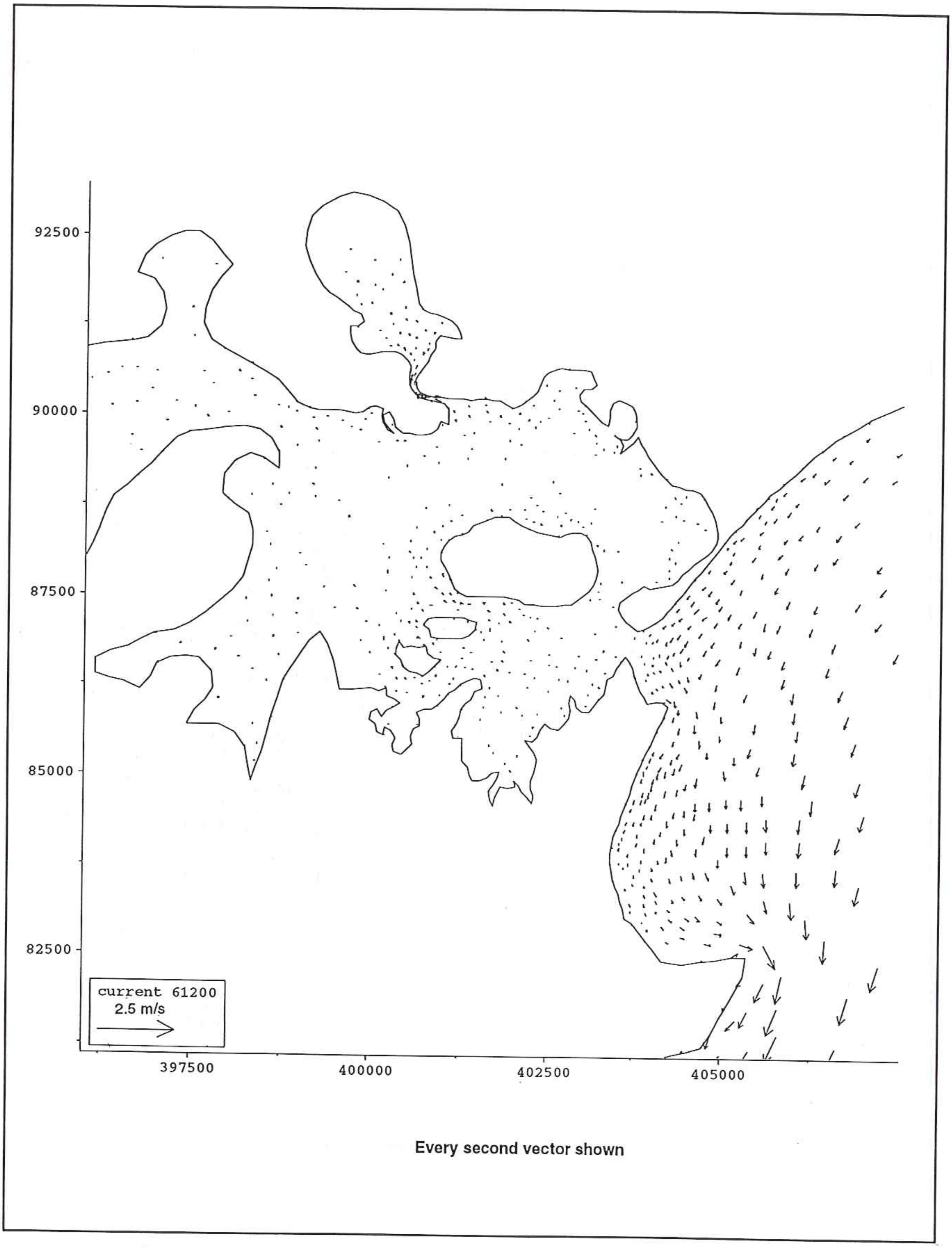
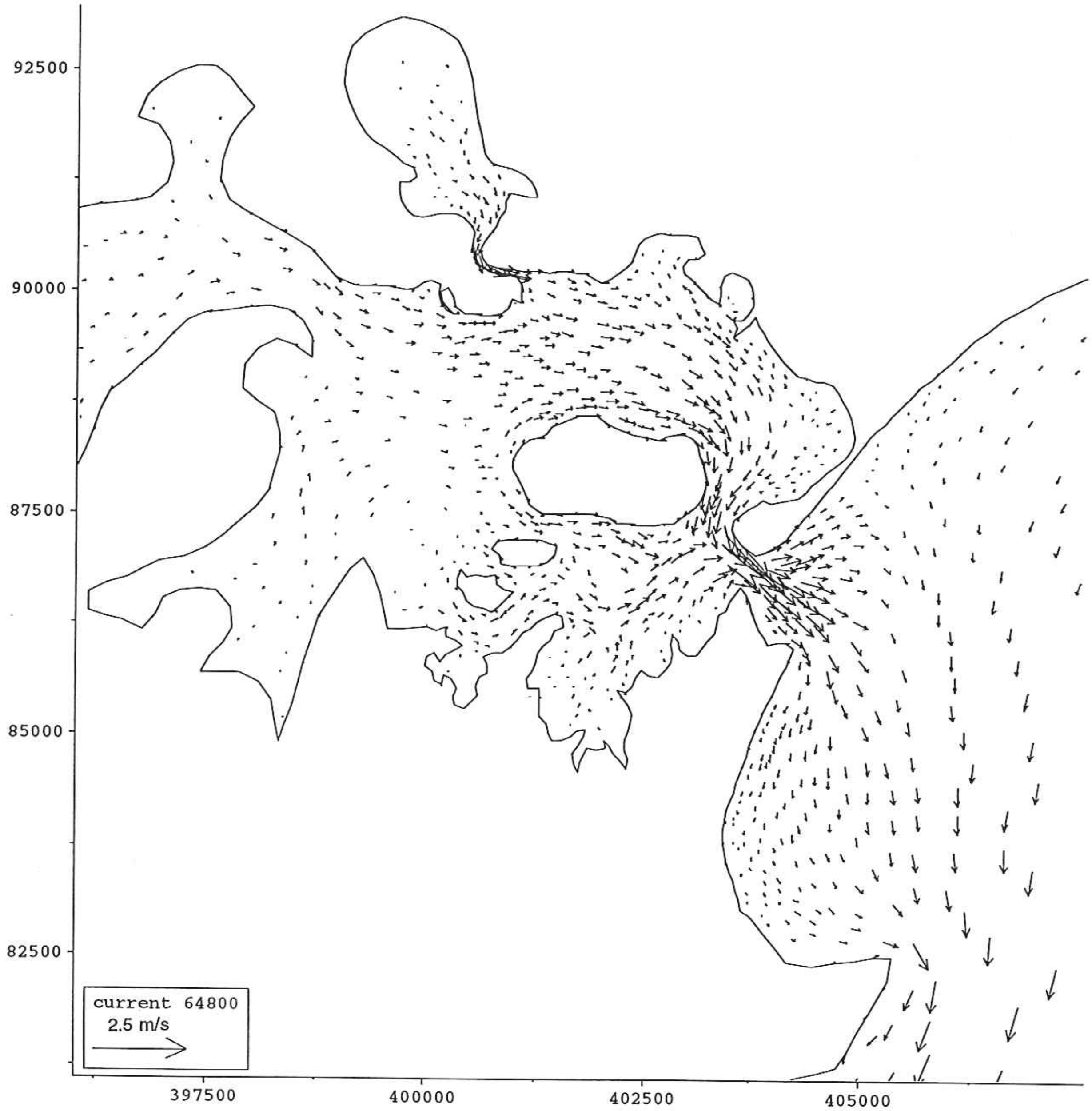
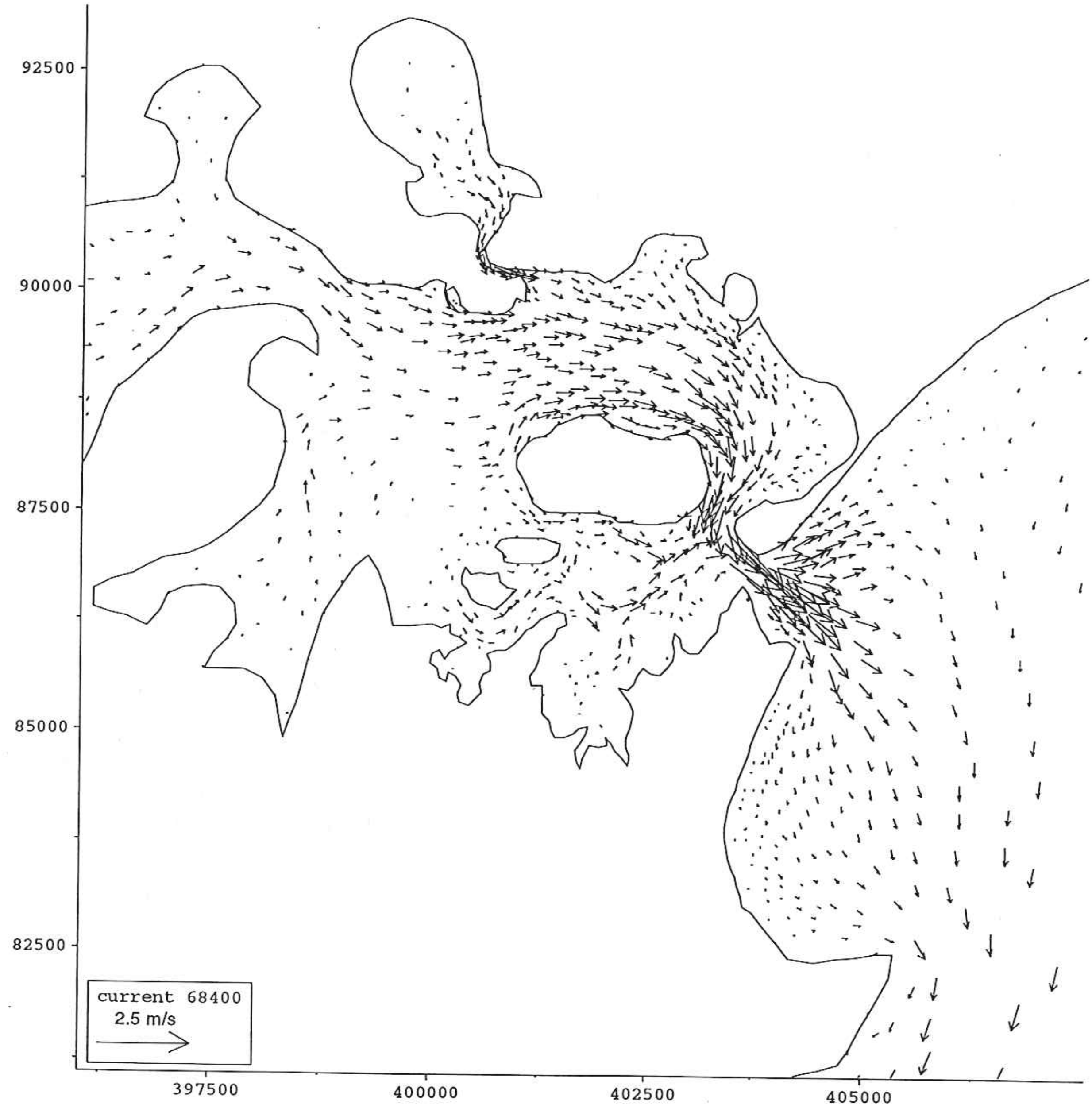


Figure 2.17 Tidal vectors at 9 hours after Low Water



Every second vector shown

Figure 2.18 Tidal vectors at 10 hours after Low Water



Every second vector shown

Figure 2.19 Tidal vectors at 11 hours after Low Water

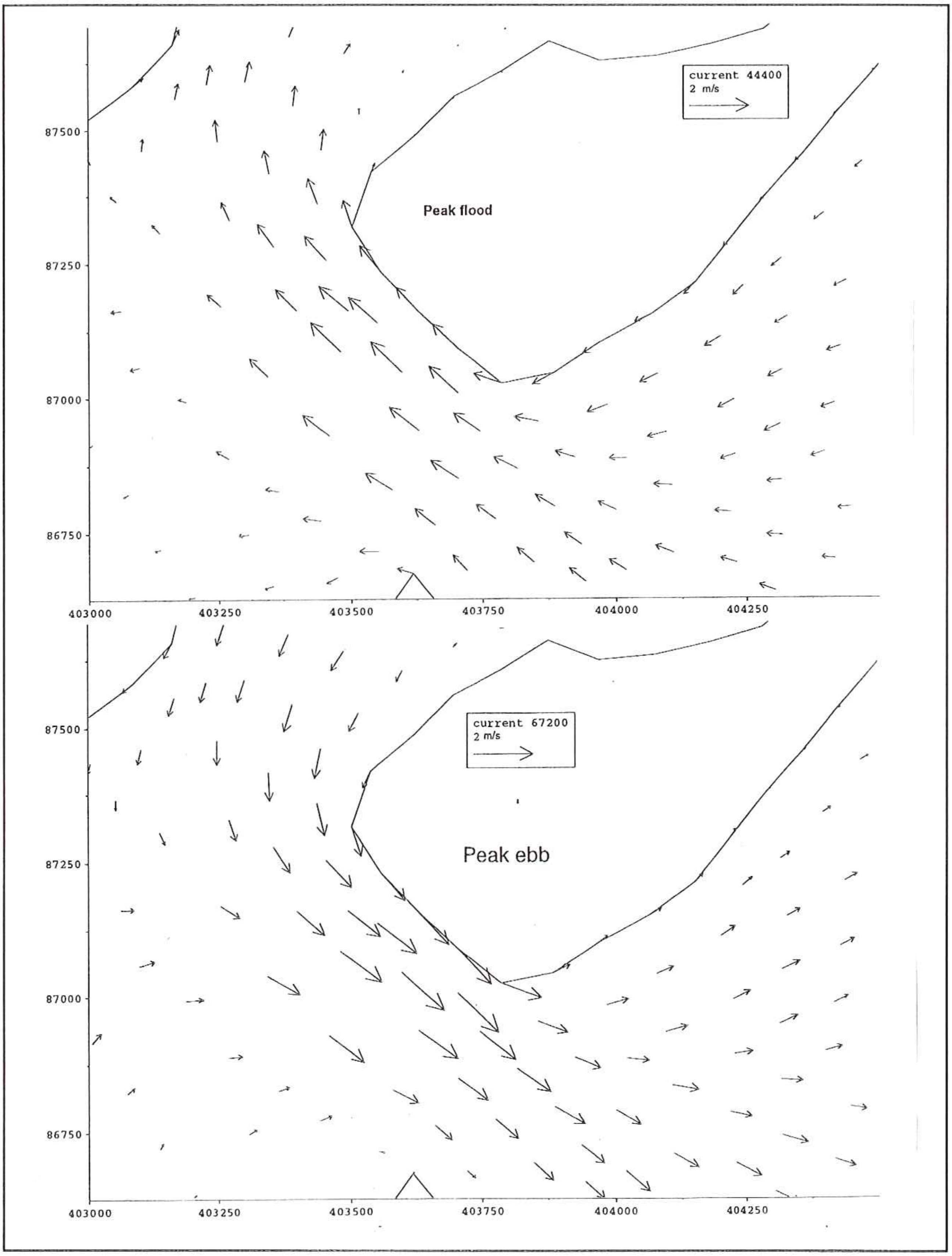


Figure 2.20 Peak flood and ebb tidal current vectors near Poole Harbour entrance

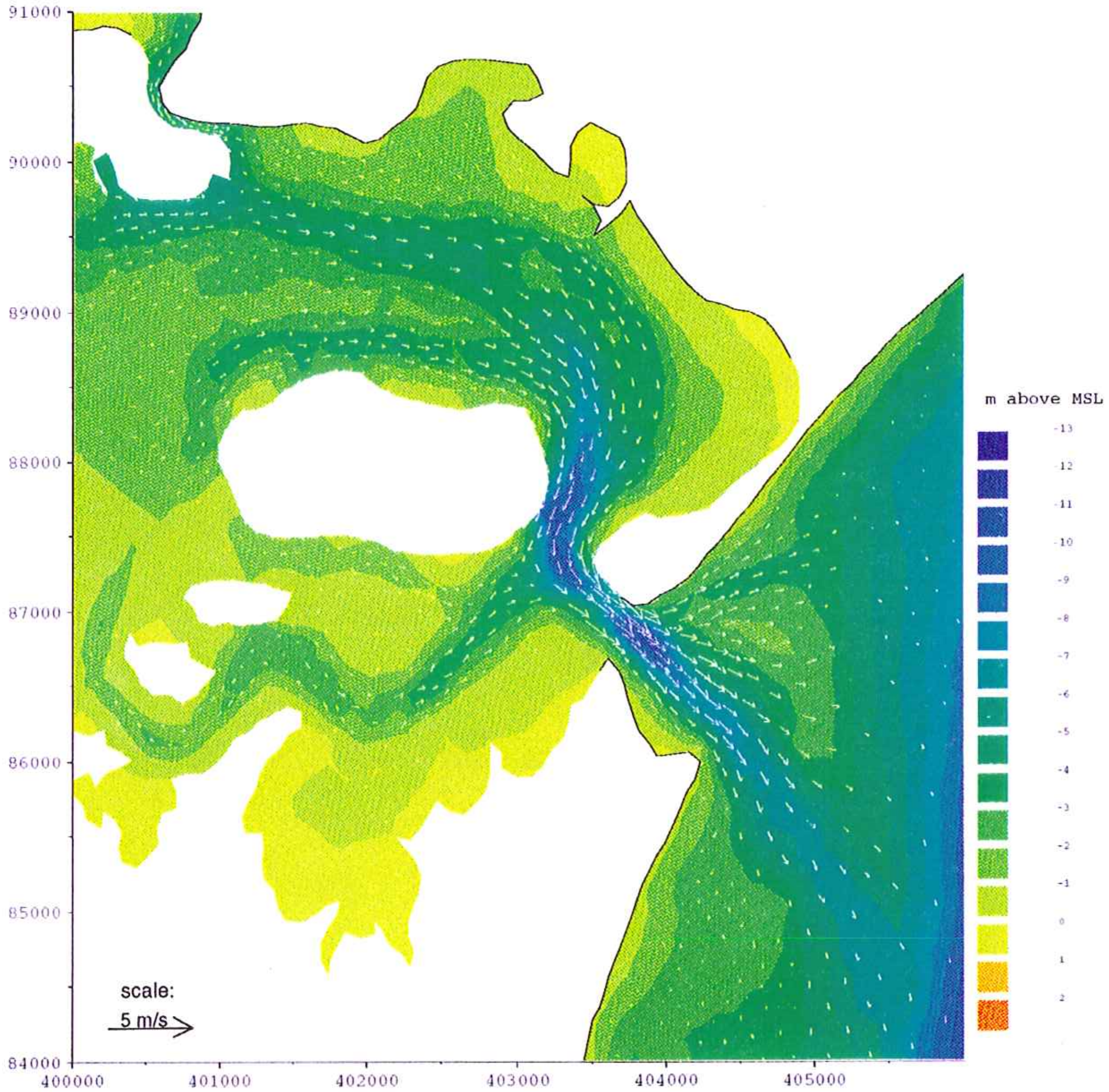


Figure 2.21 Peak ebb tidal current vectors

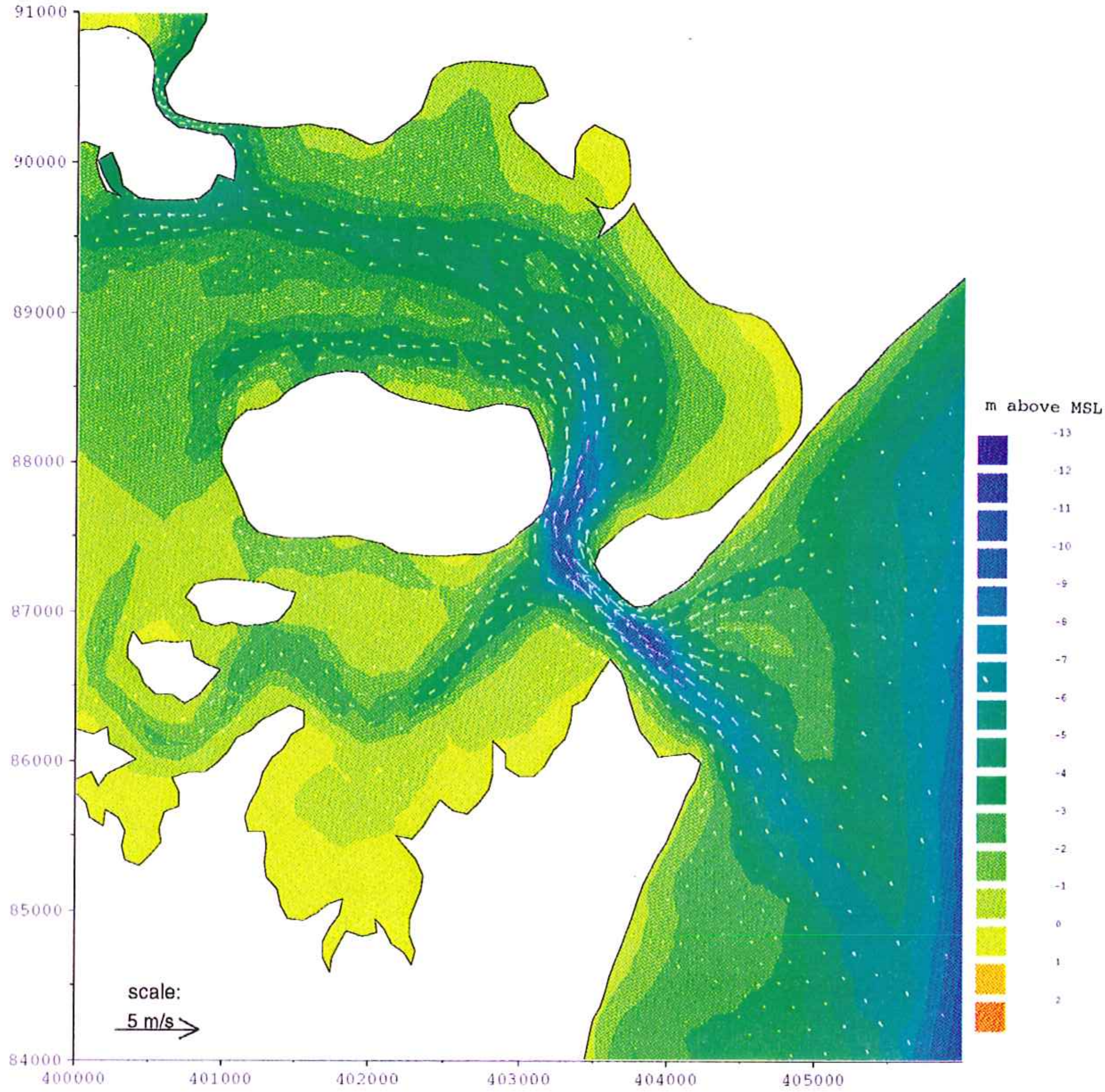


Figure 2.22 Peak flood tidal current vectors

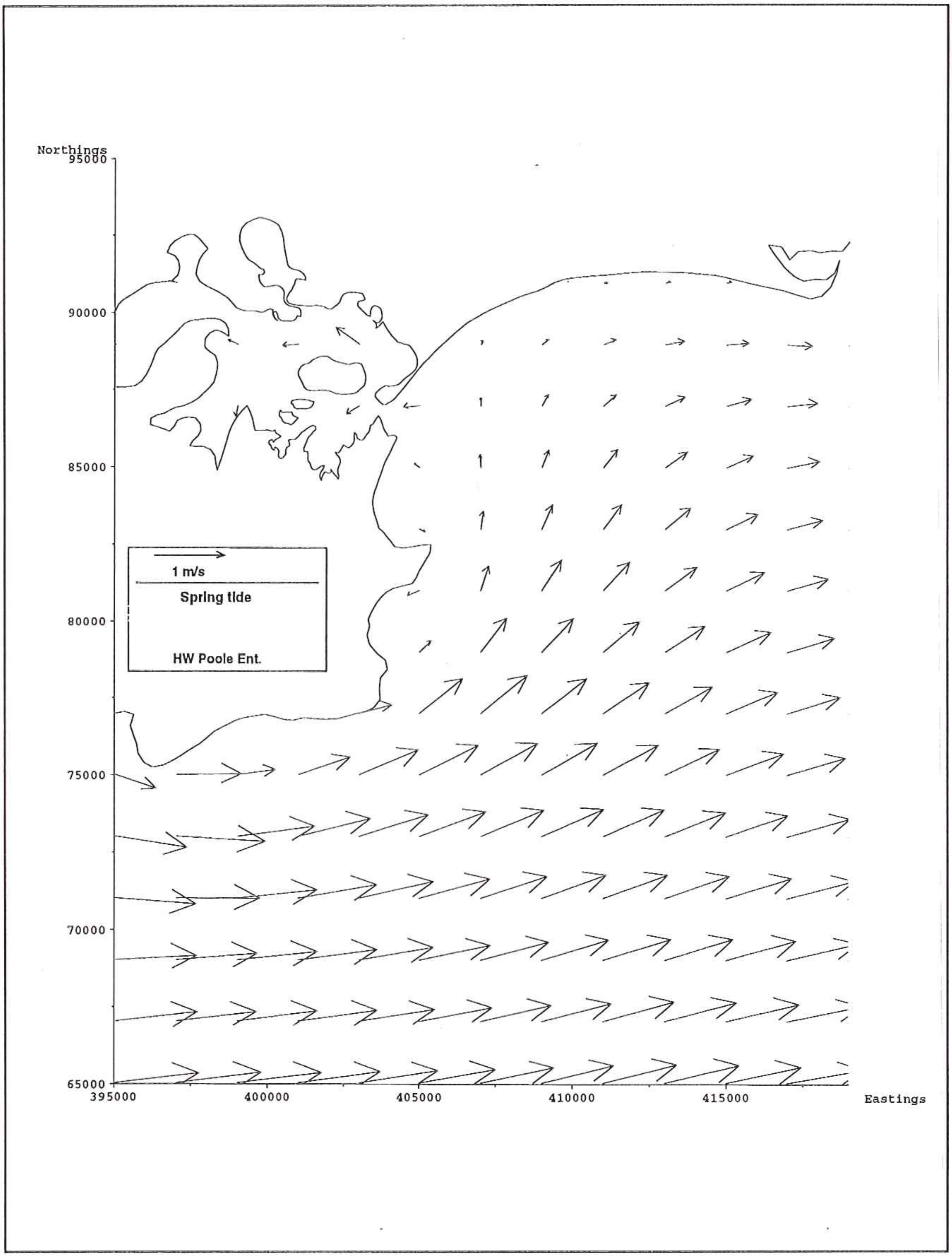


Figure 2.23 Spring tide current vectors at High Water (Poole Harbour entrance)

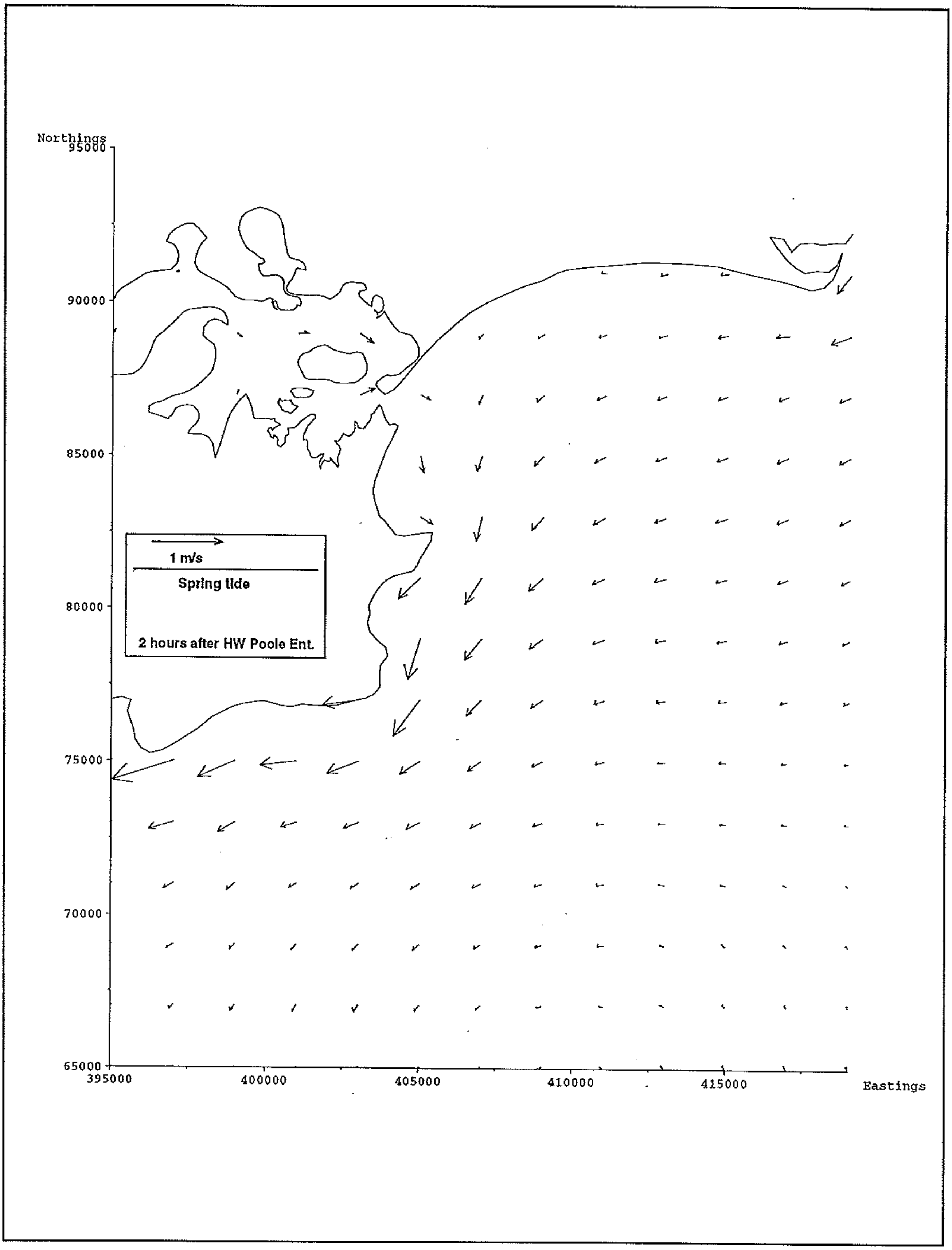


Figure 2.24 Spring tide current vectors 2 hours after High Water (Poole Harbour entrance)

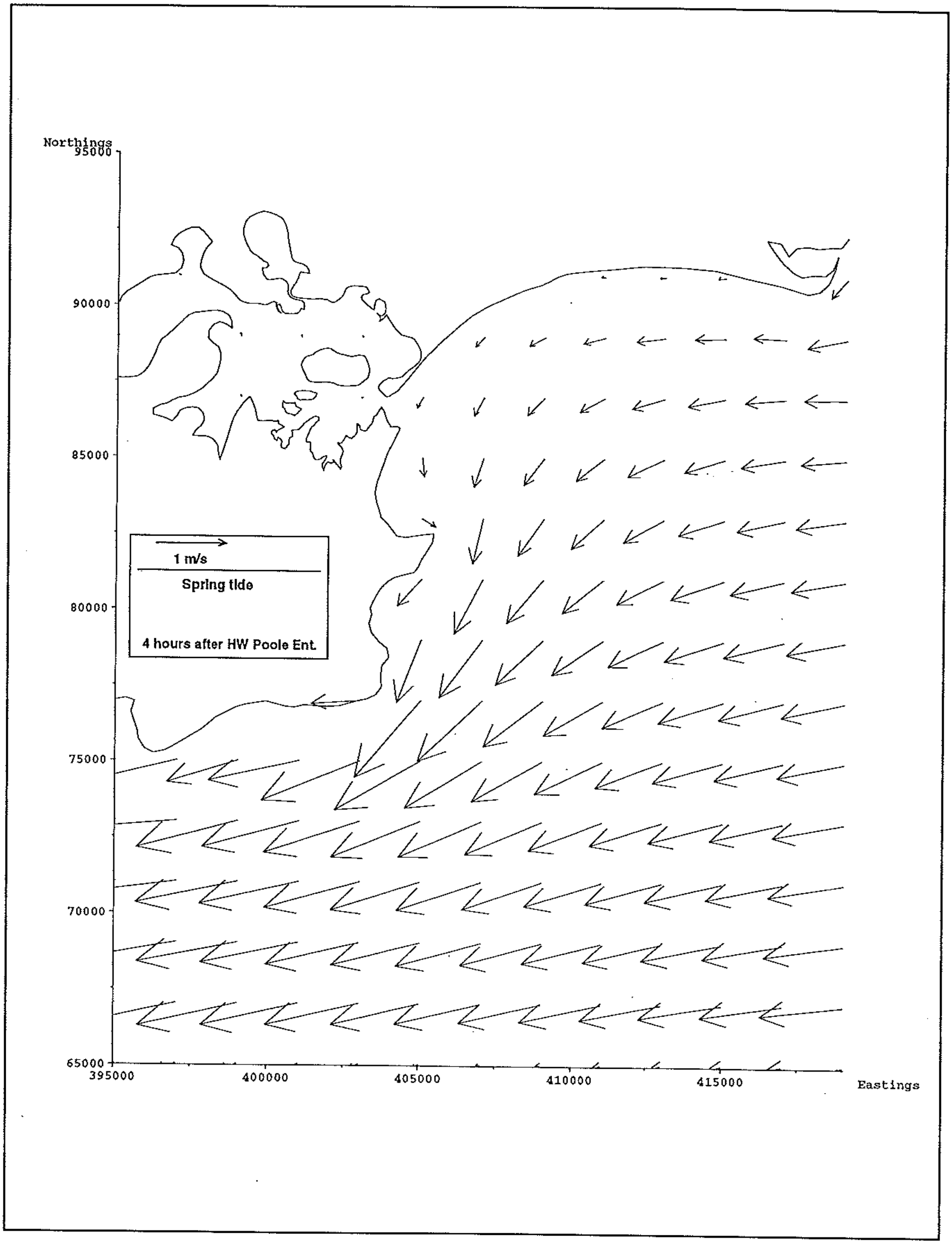


Figure 2.25 Spring tide current vectors 4 hours after High Water (Poole Harbour entrance)

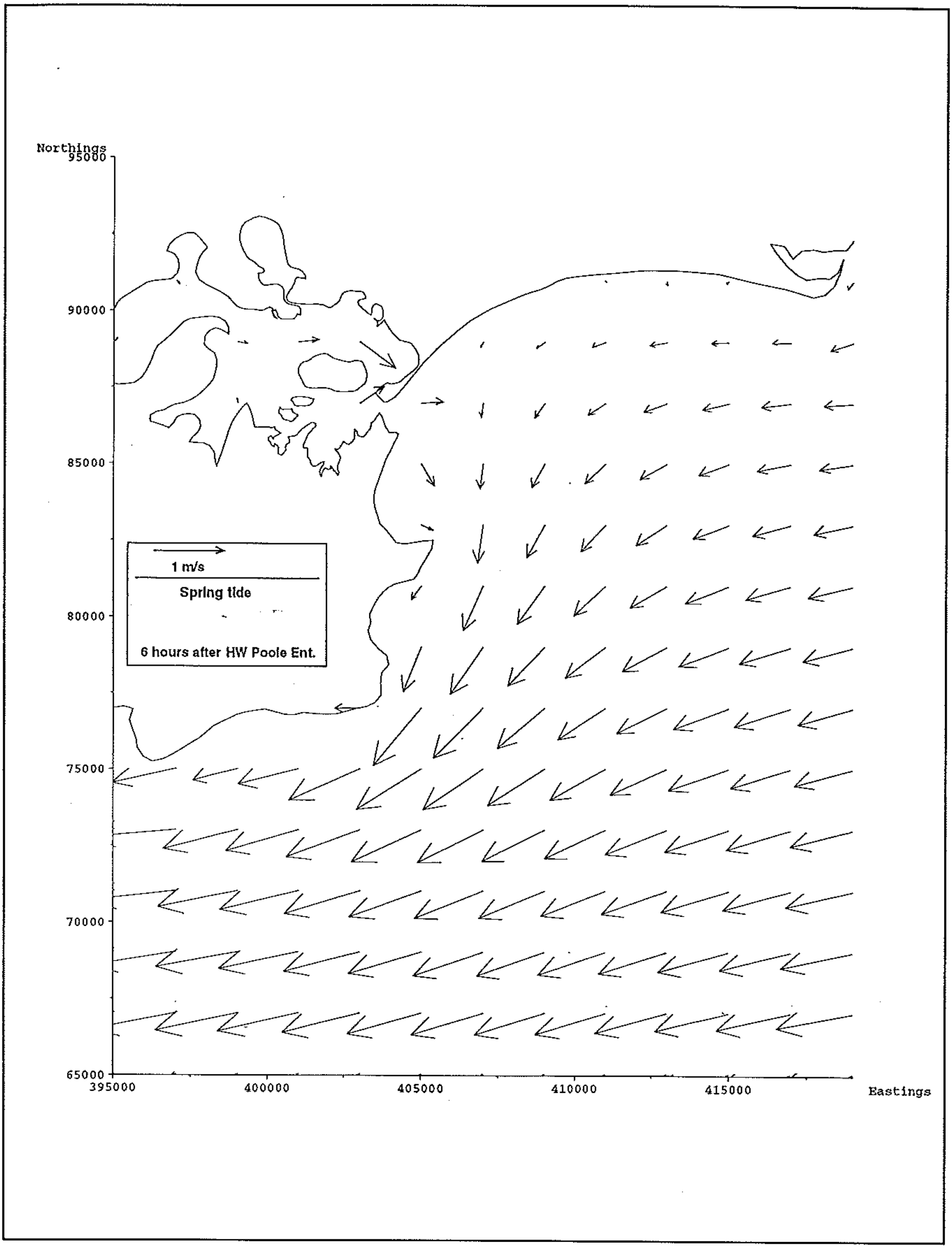


Figure 2.26 Spring tide current vectors 6 hours after High Water (Poole Harbour entrance)

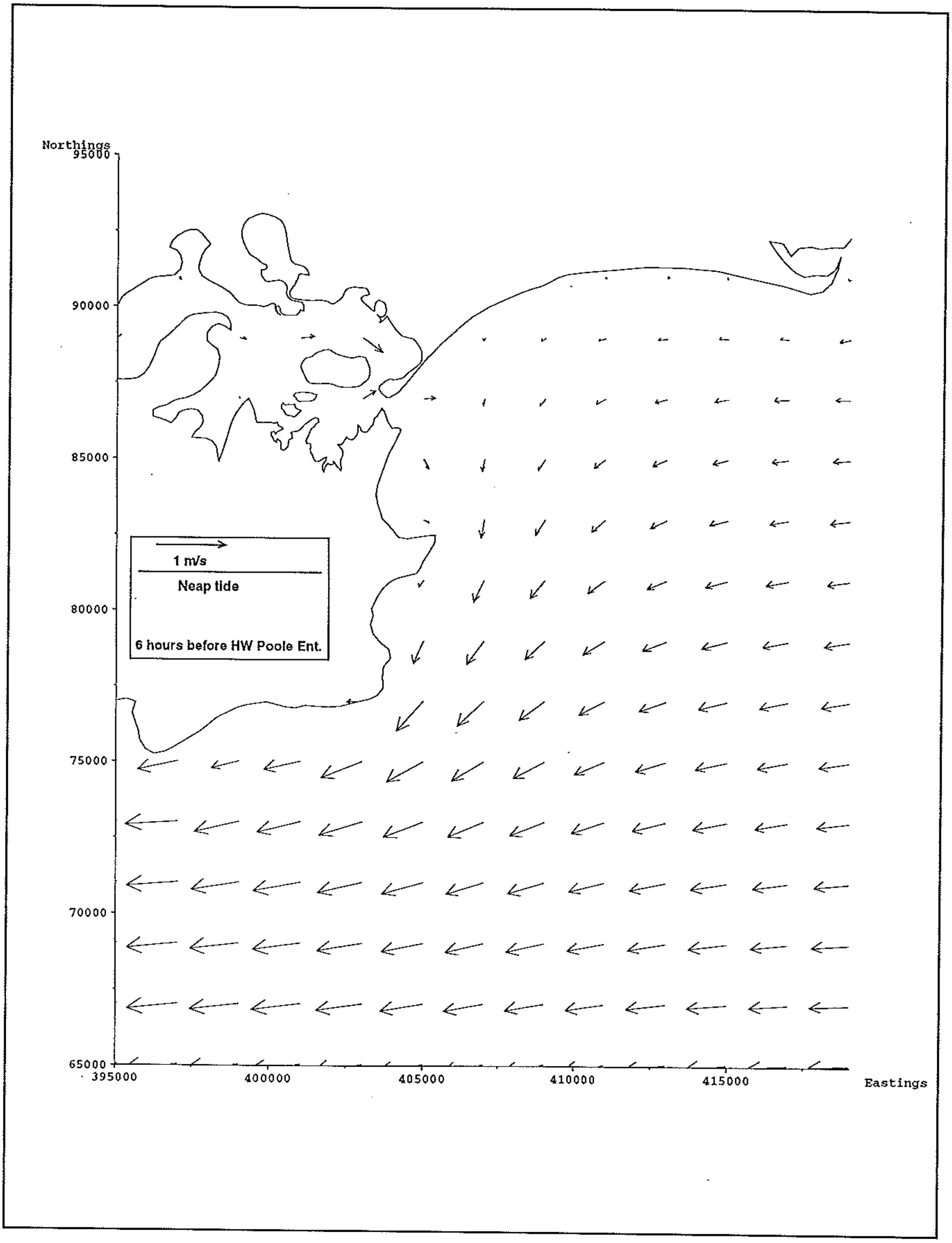


Figure 2.27 Neap tide current vectors 6 hours before High Water (Poole Harbour entrance)

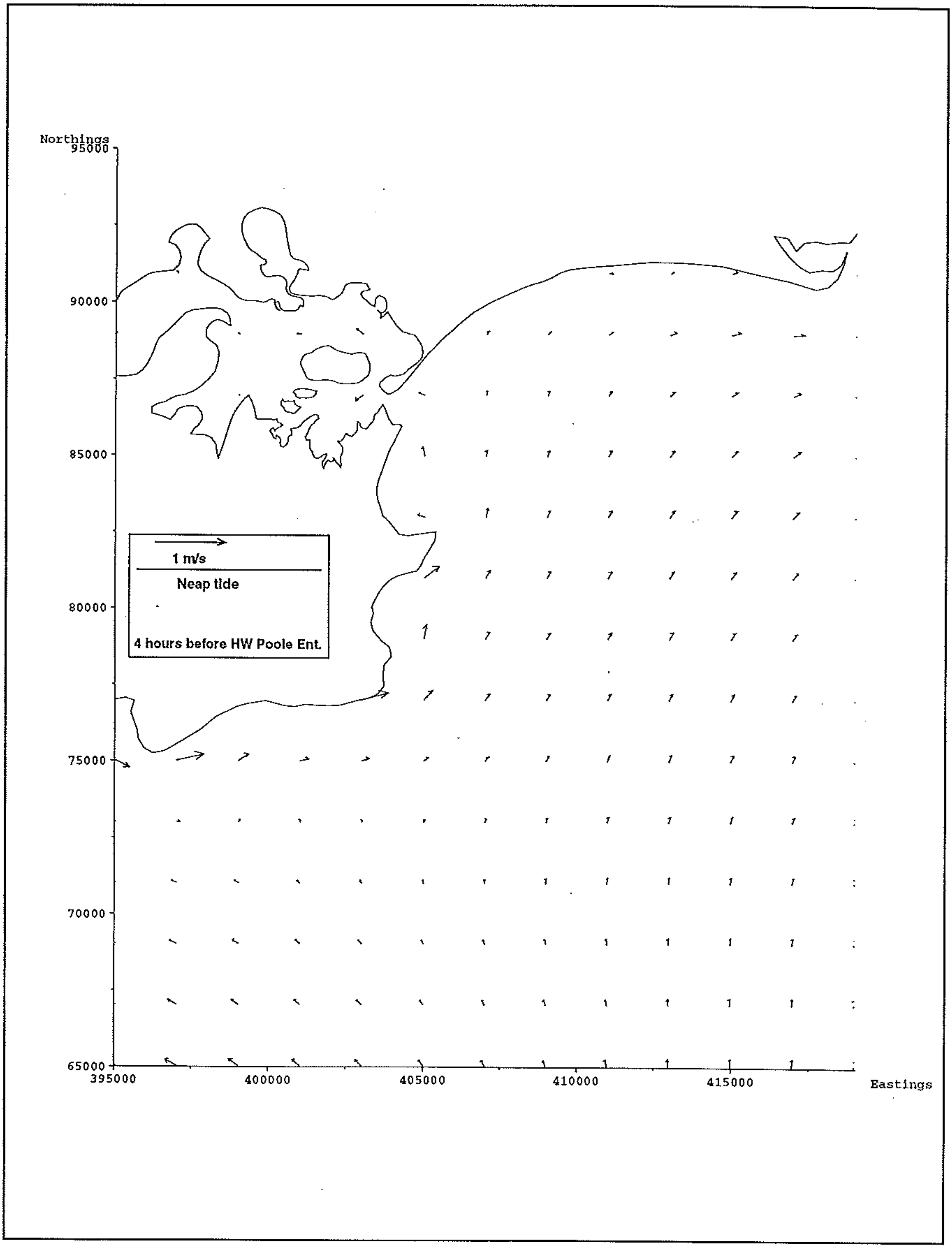


Figure 2.28 Neap tide current vectors 4 hours before High Water (Poole Harbour entrance)

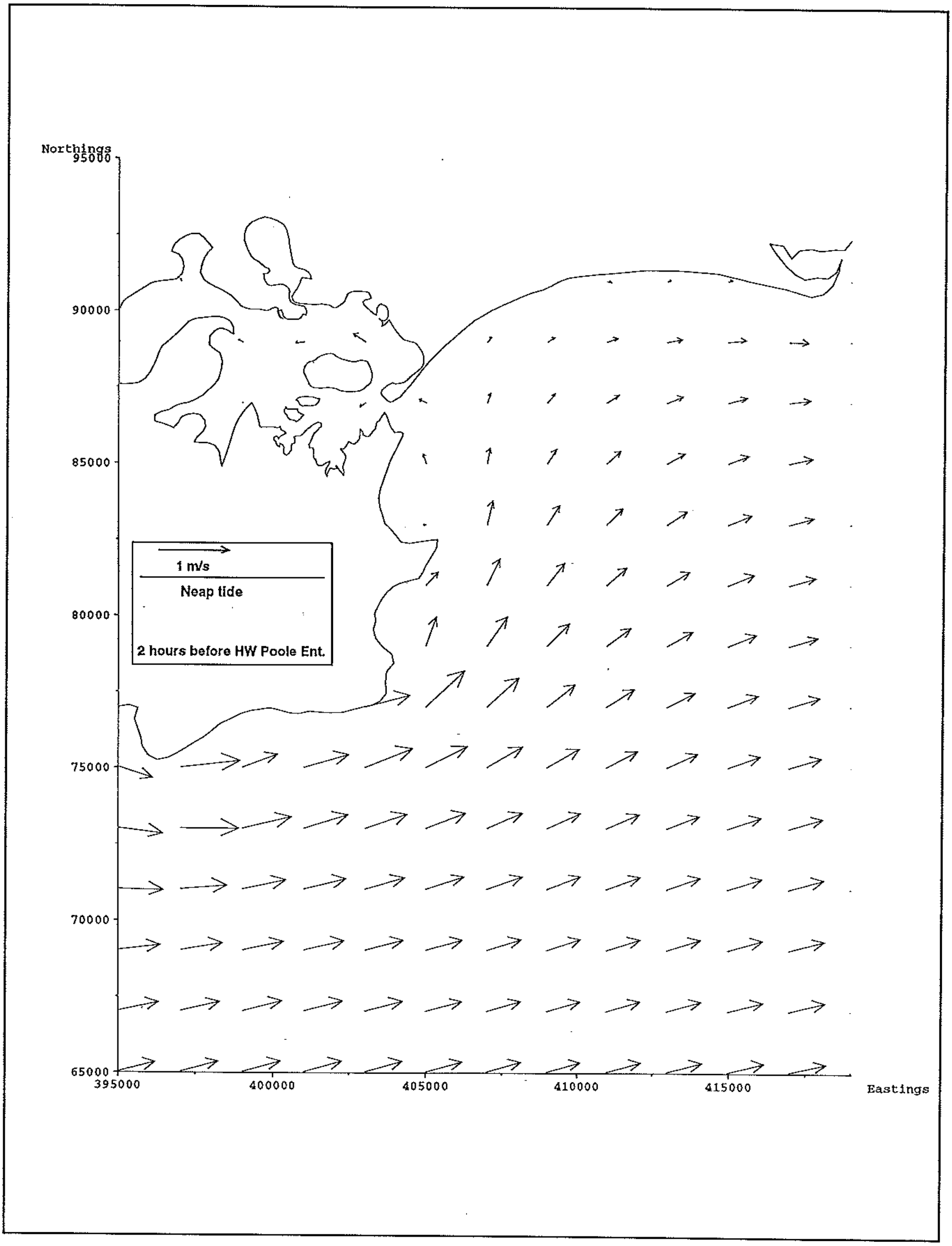
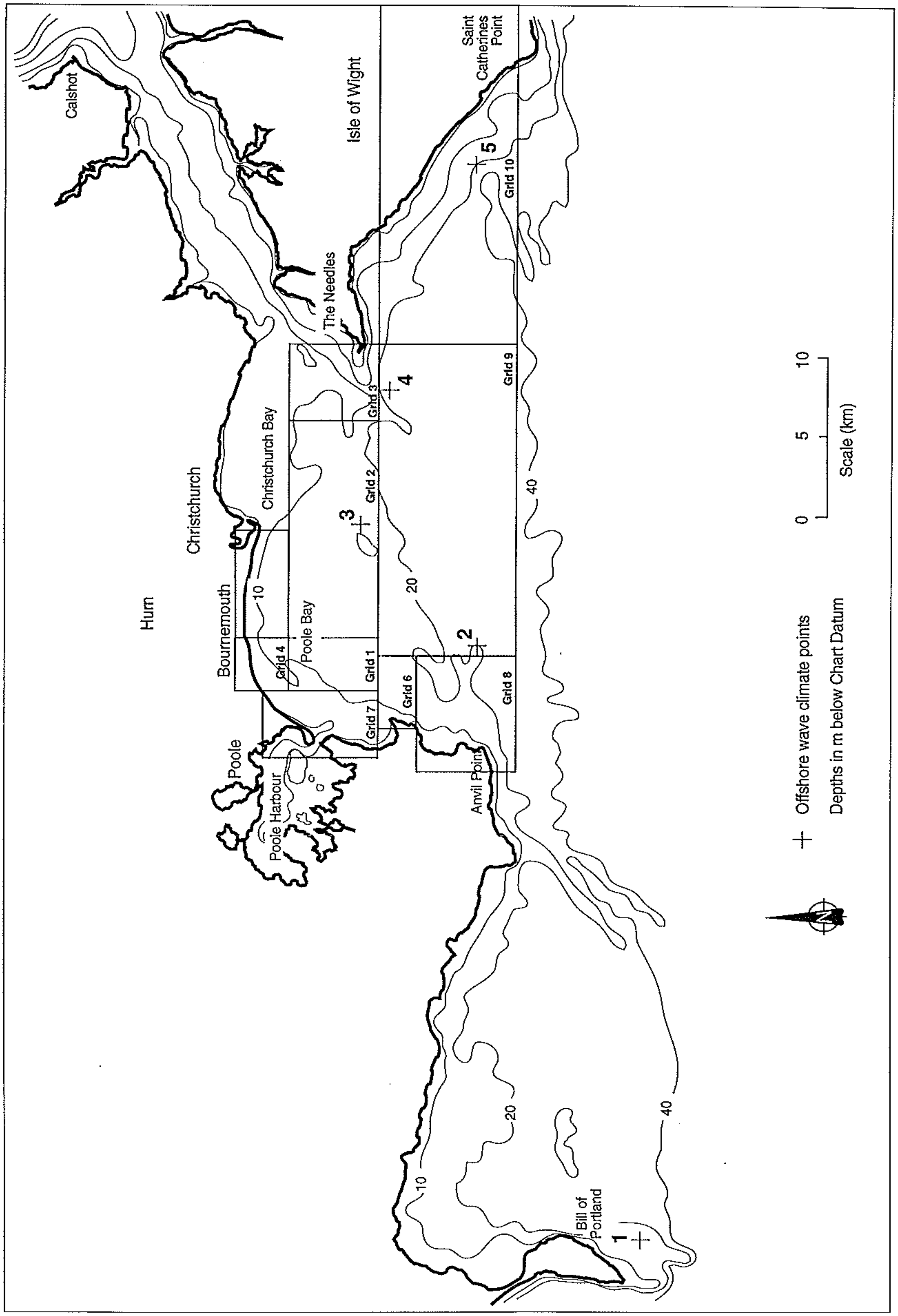


Figure 2.29 Neap tide current vectors 2 hours before High Water (Poole Harbour entrance)



JH/2.30/10-94/ f.line

Figure 2.30 Area of regional wave modelling

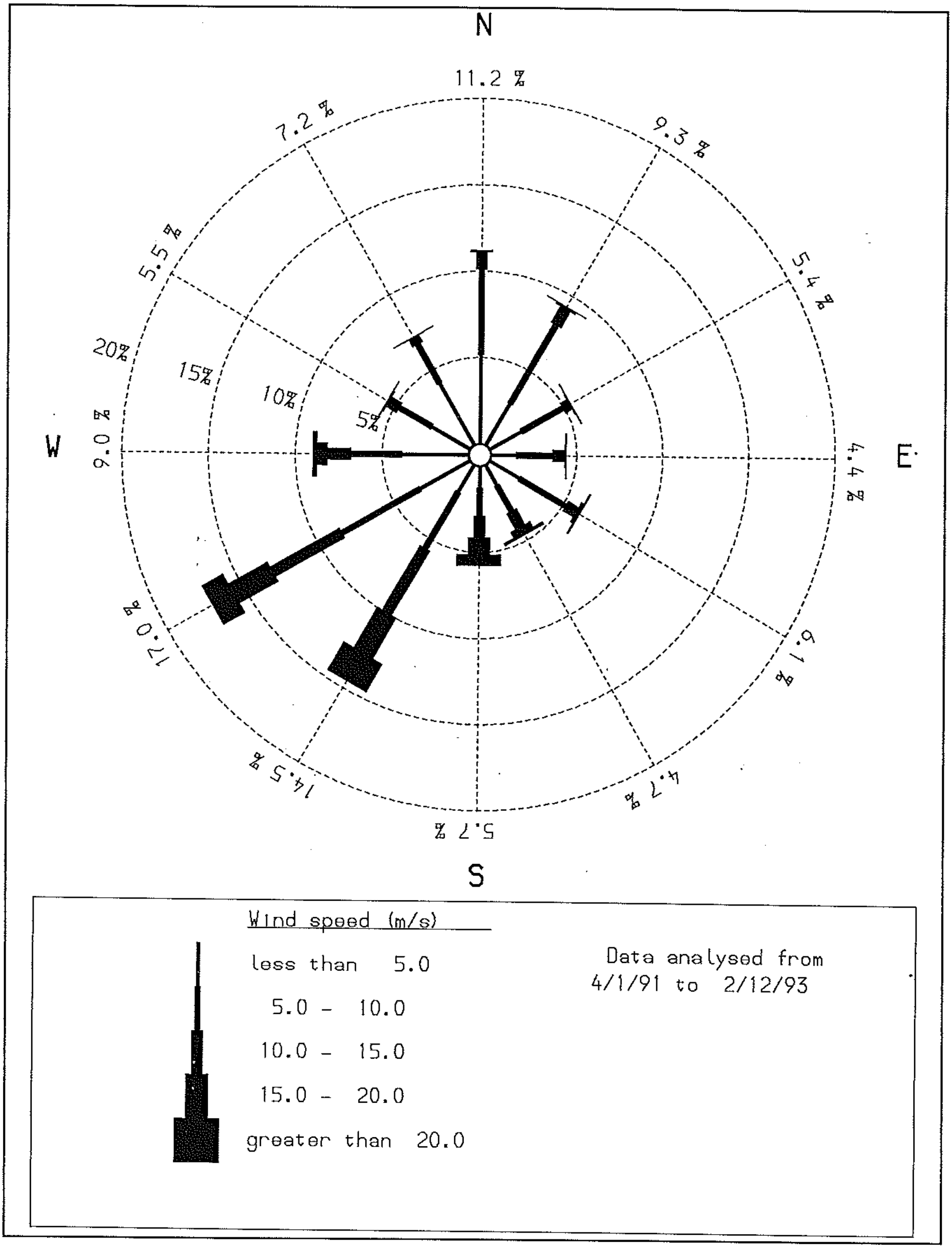


Figure 2.31 Wind rose for Poole Harbour

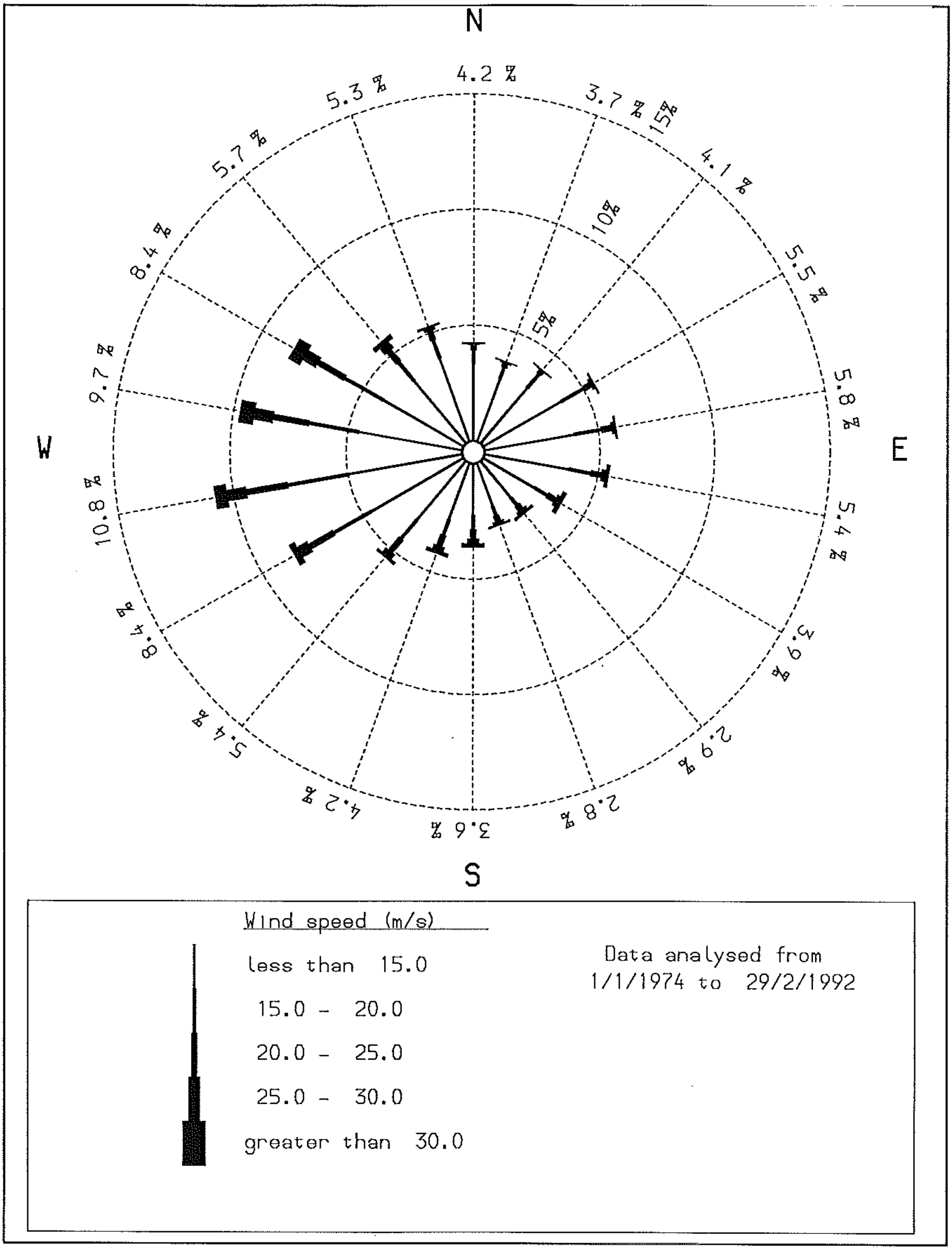


Figure 2.32 Wind rose for Portland

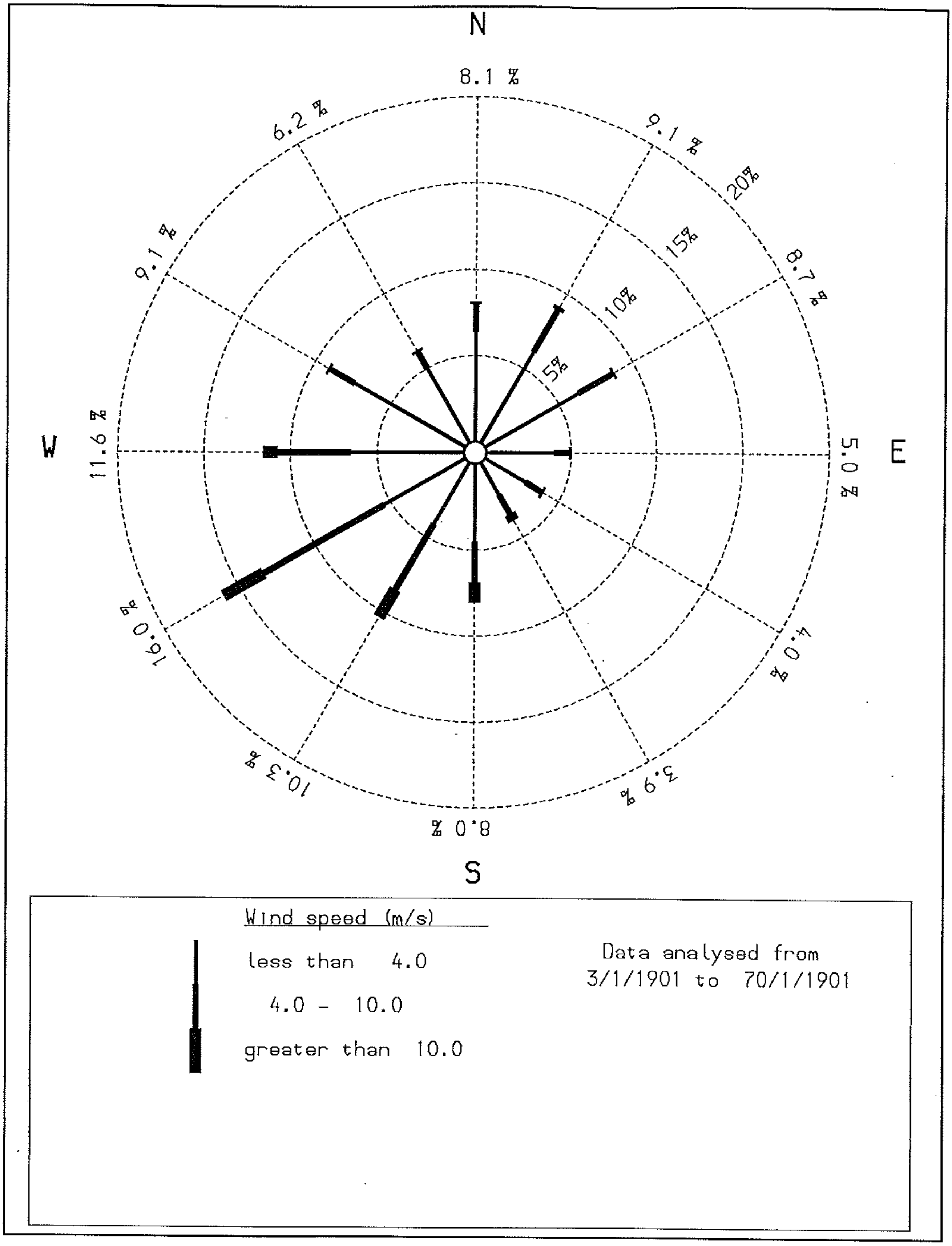


Figure 2.33 Wind rose for Hurn

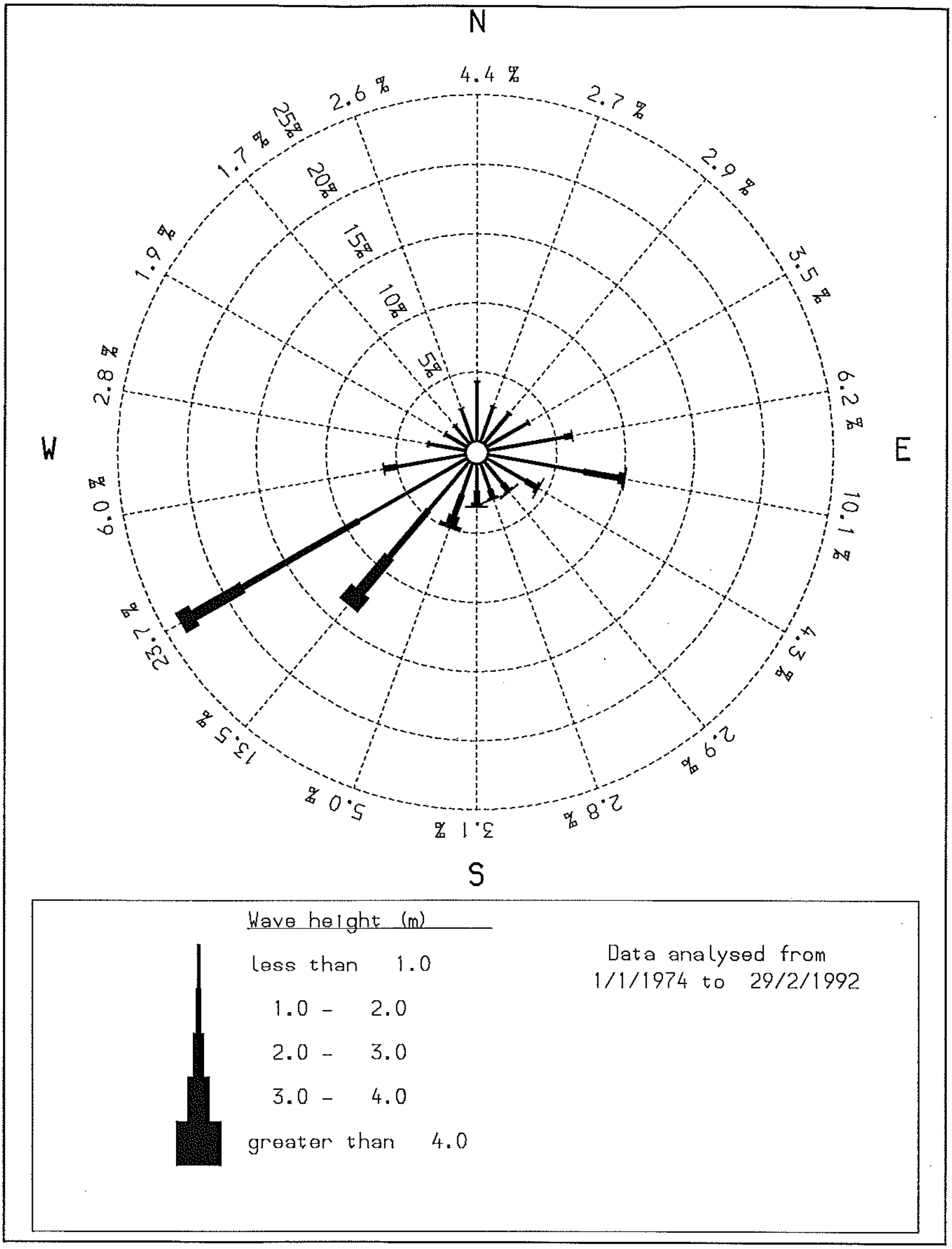


Figure 2.34 Wave rose for offshore point 1

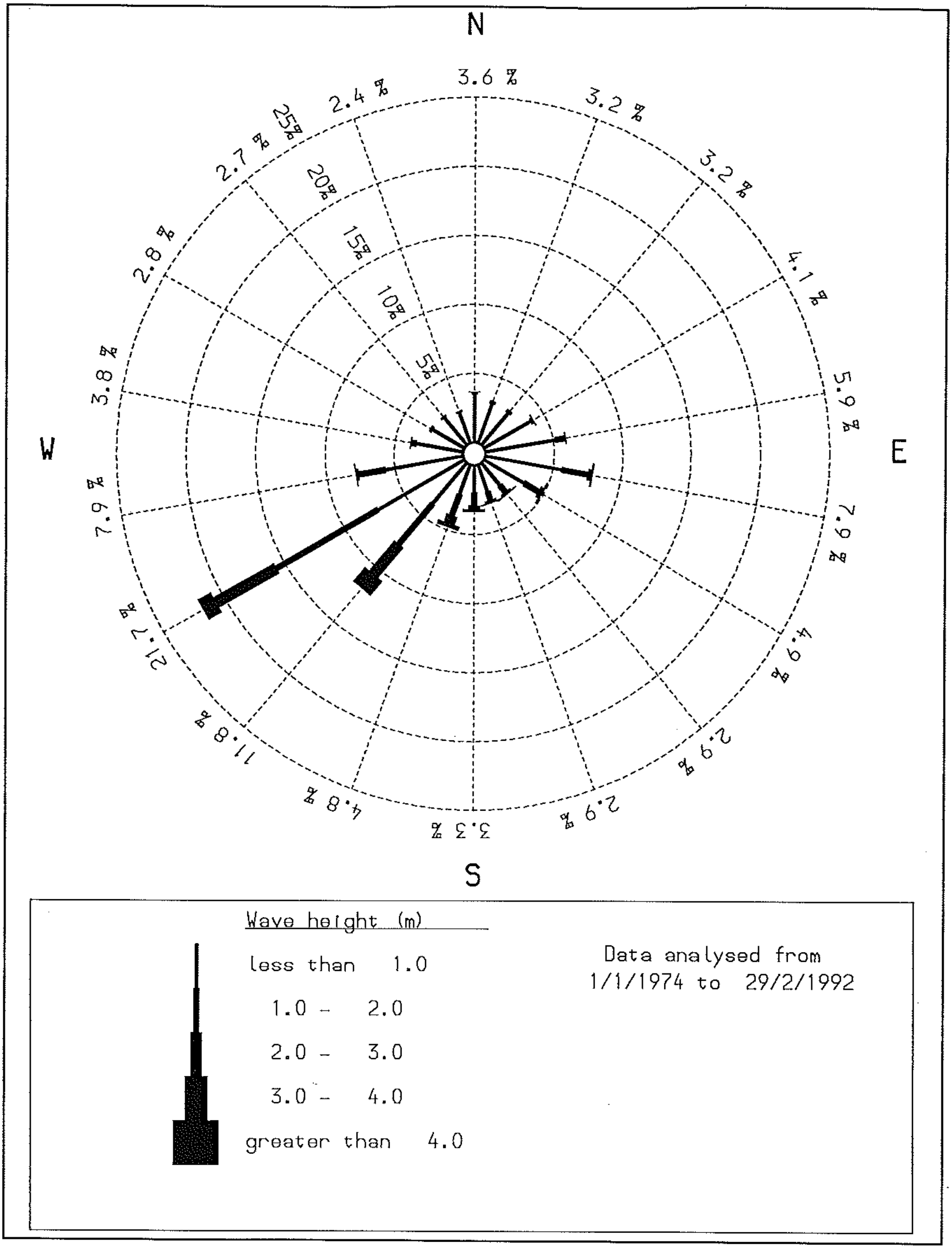


Figure 2.35 Wave rose for offshore point 2

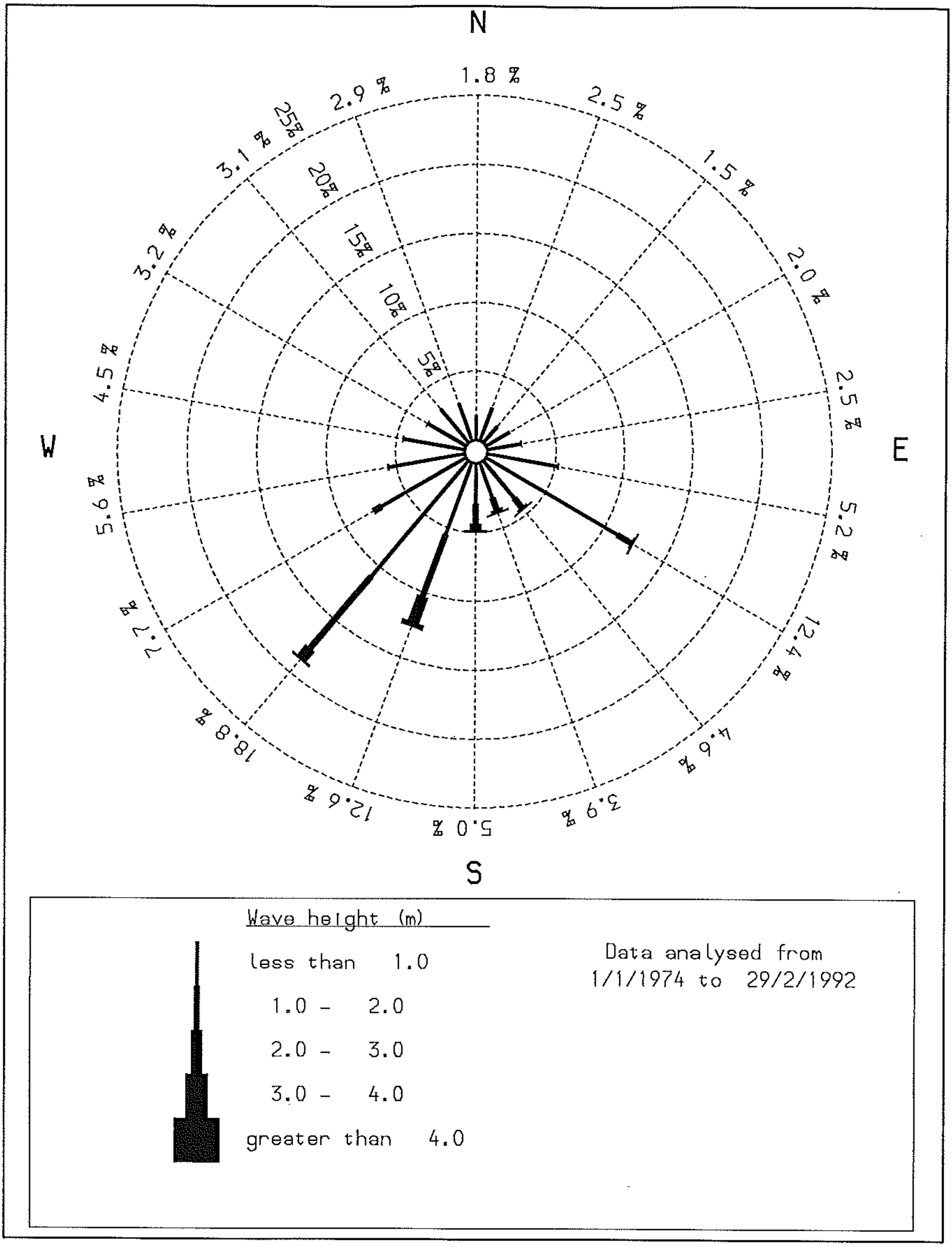


Figure 2.36 Wave rose for offshore point 3

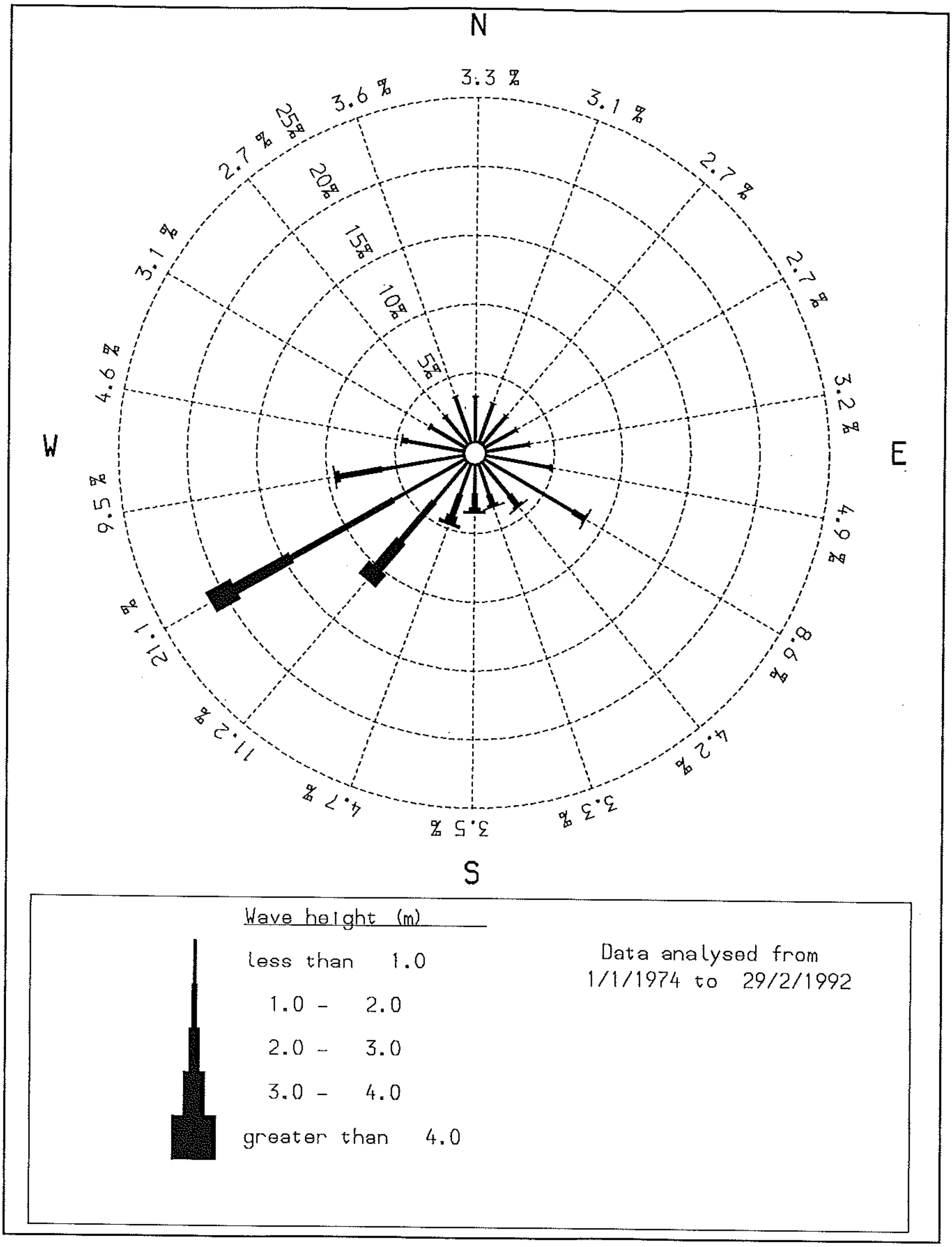


Figure 2.37 Wave rose for offshore point 4

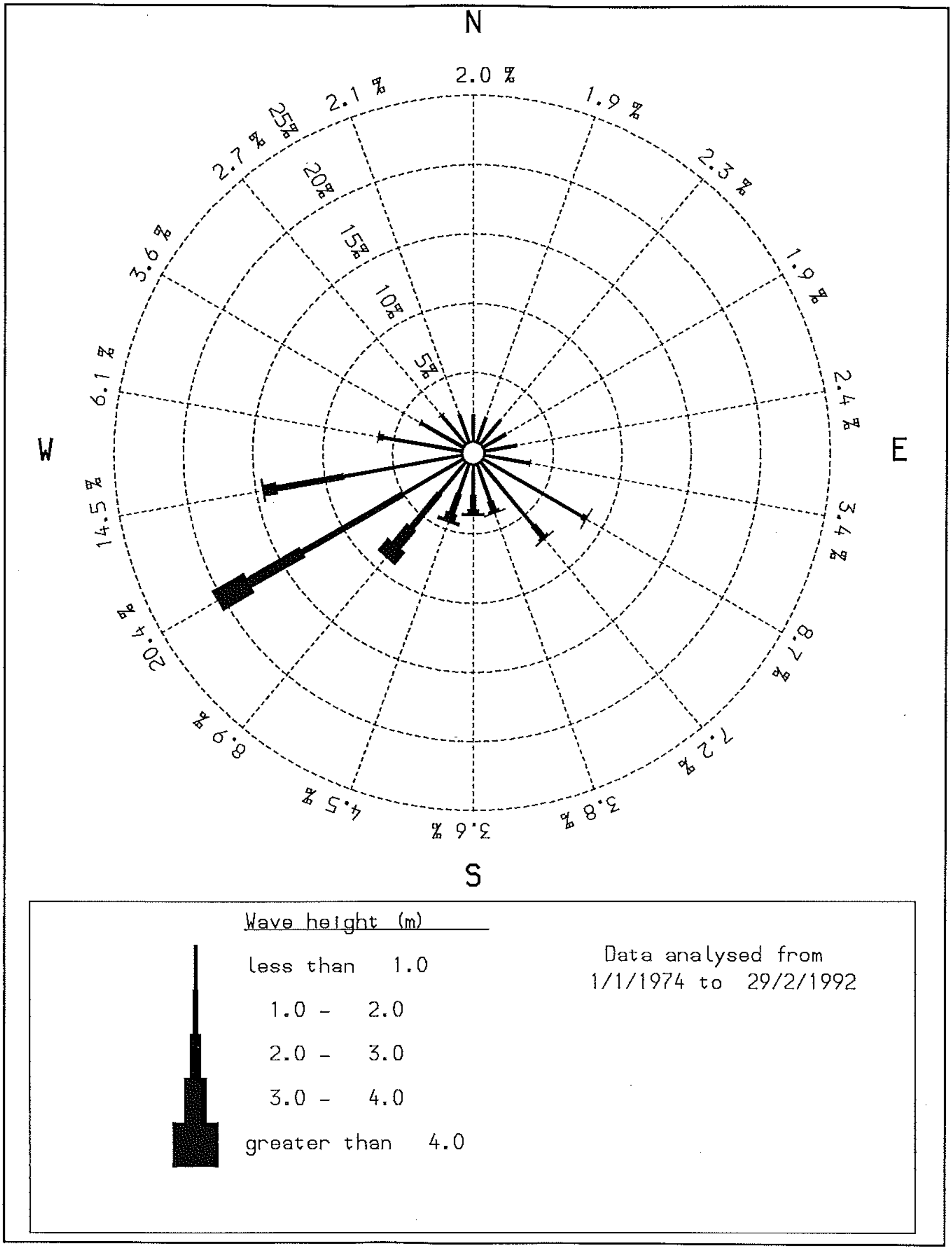


Figure 2.38 Wave rose for offshore point 5

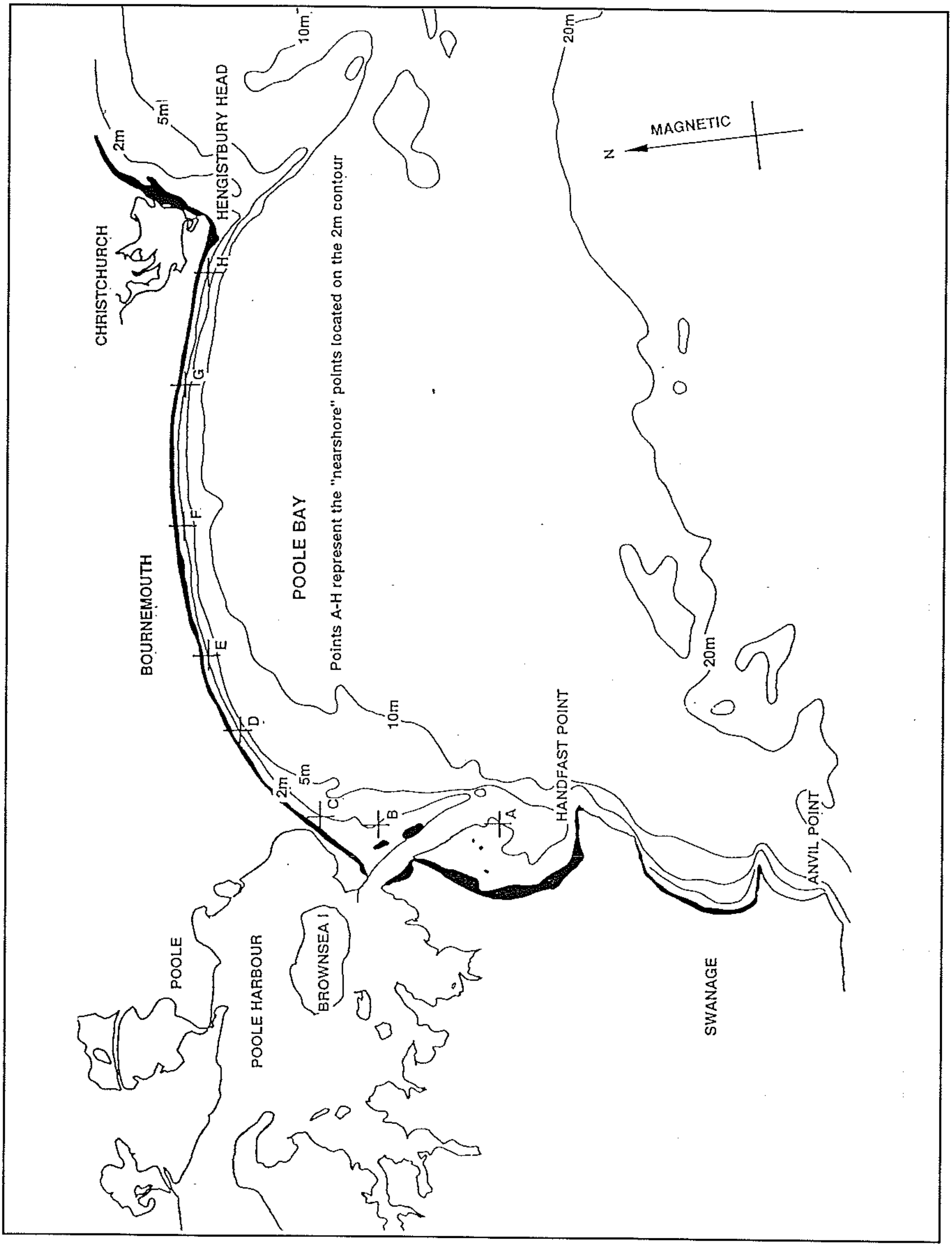


Figure 2.39 Location of points for nearshore wave climate assessment

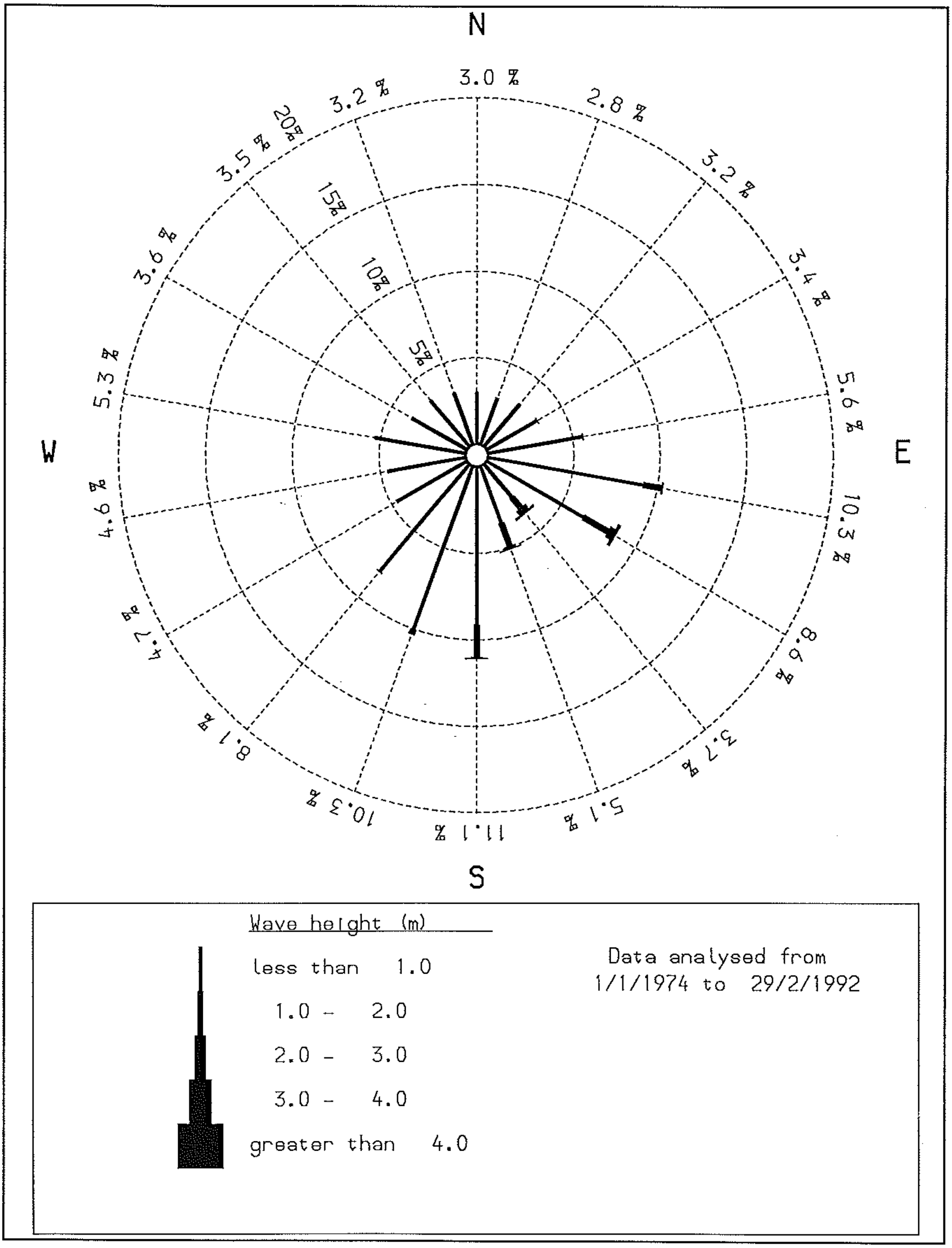


Figure 2.40 Wave rose for inshore point A

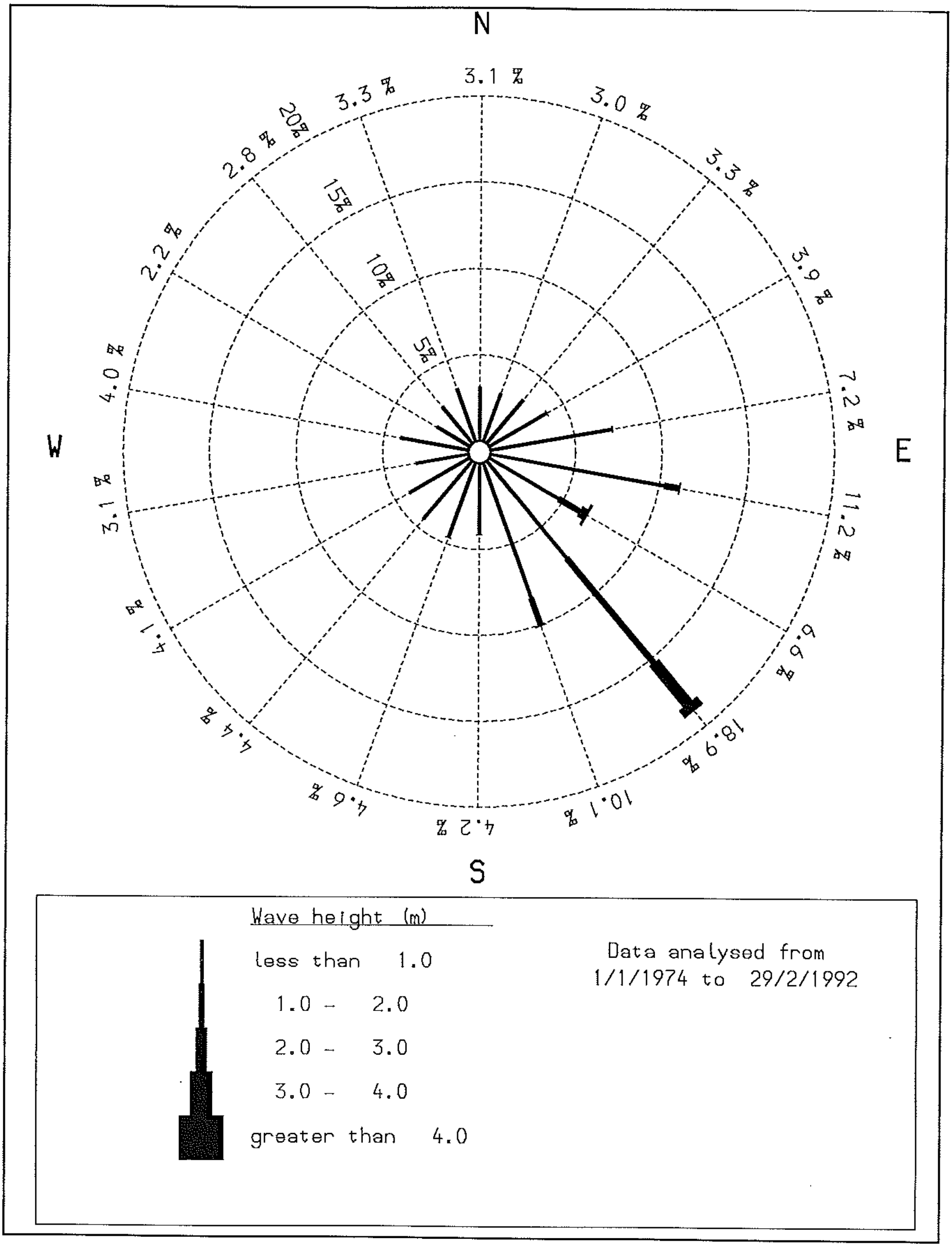


Figure 2.41 Wave rose for inshore point B

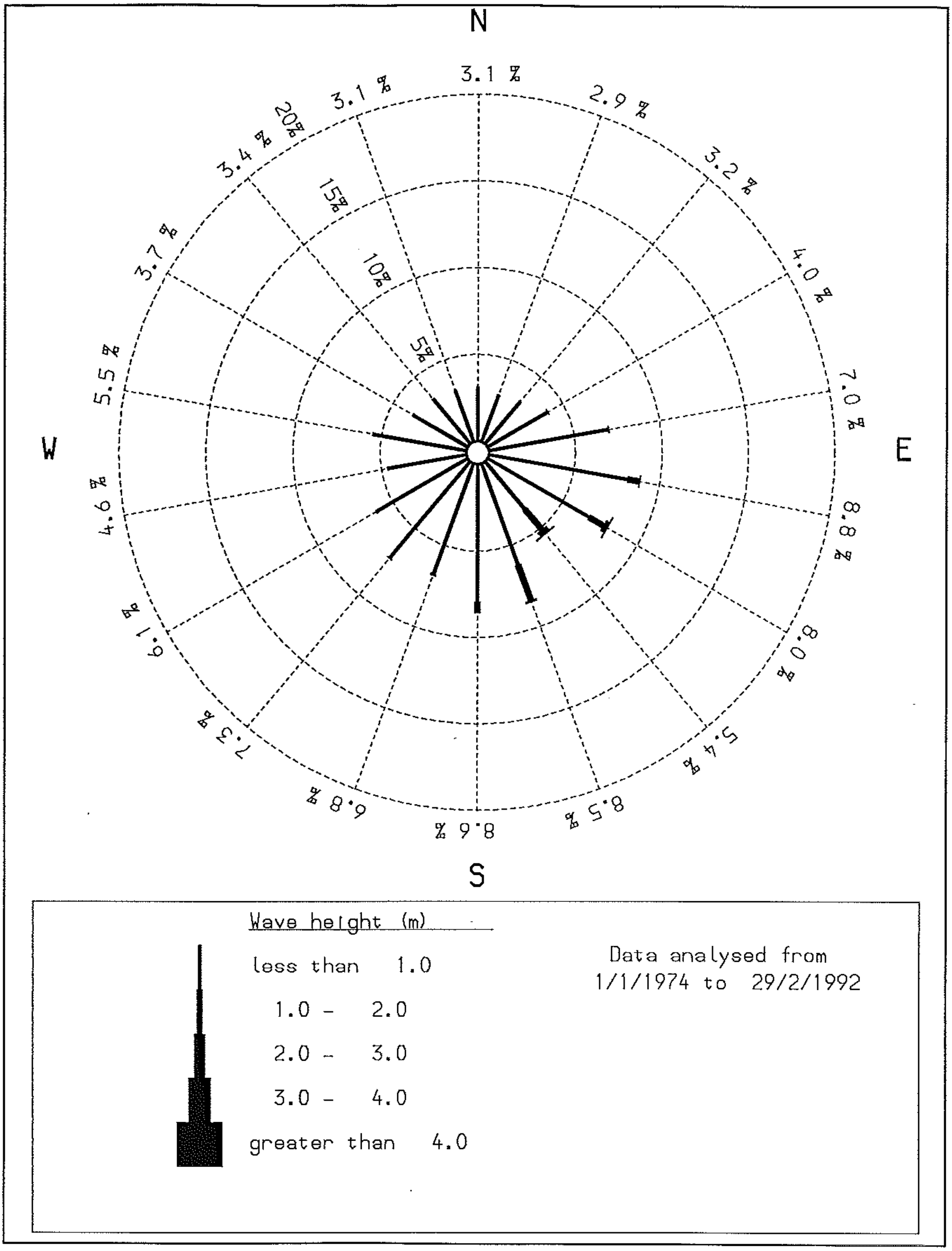


Figure 2.42 Wave rose for inshore point C

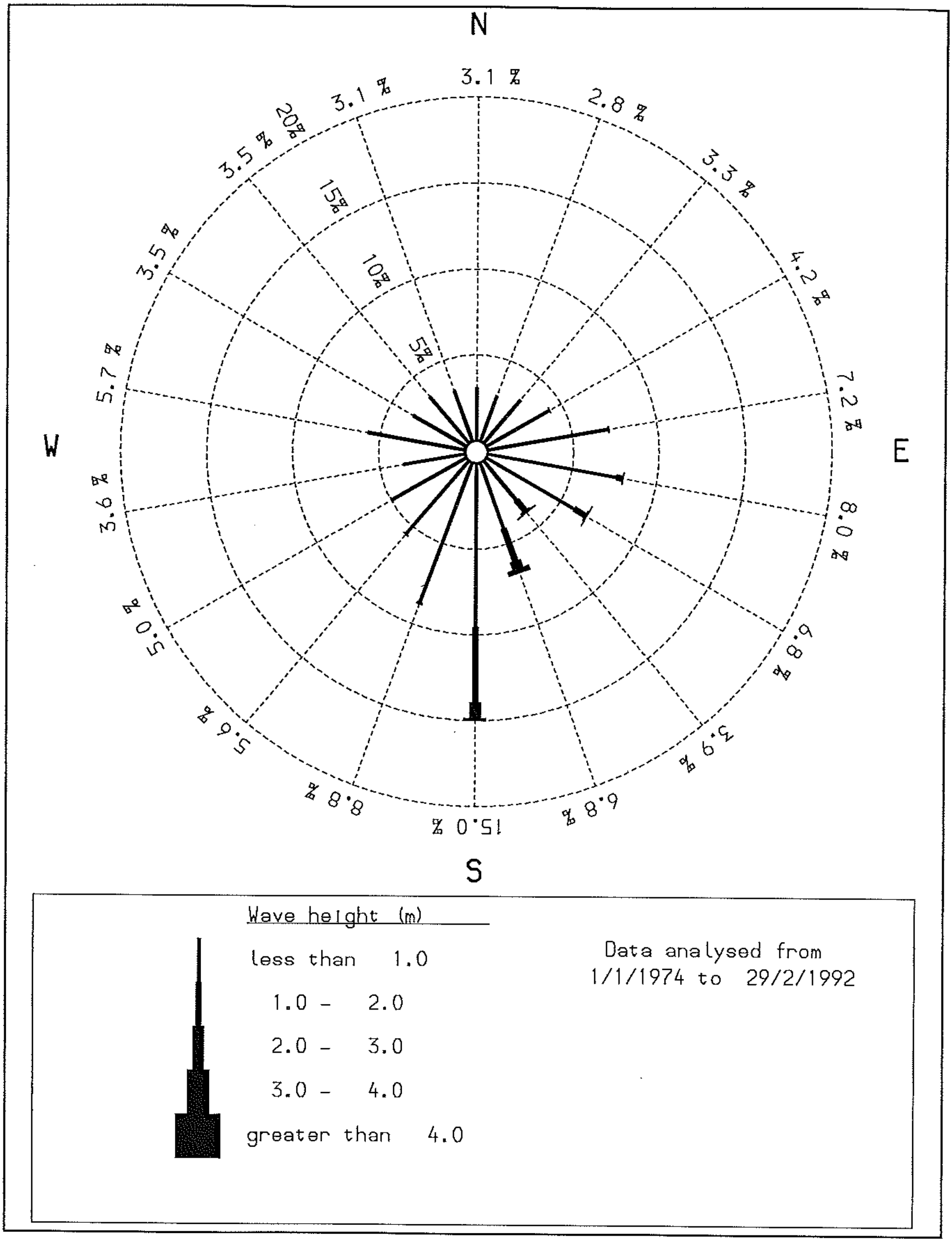


Figure 2.43 Wave rose for inshore point D

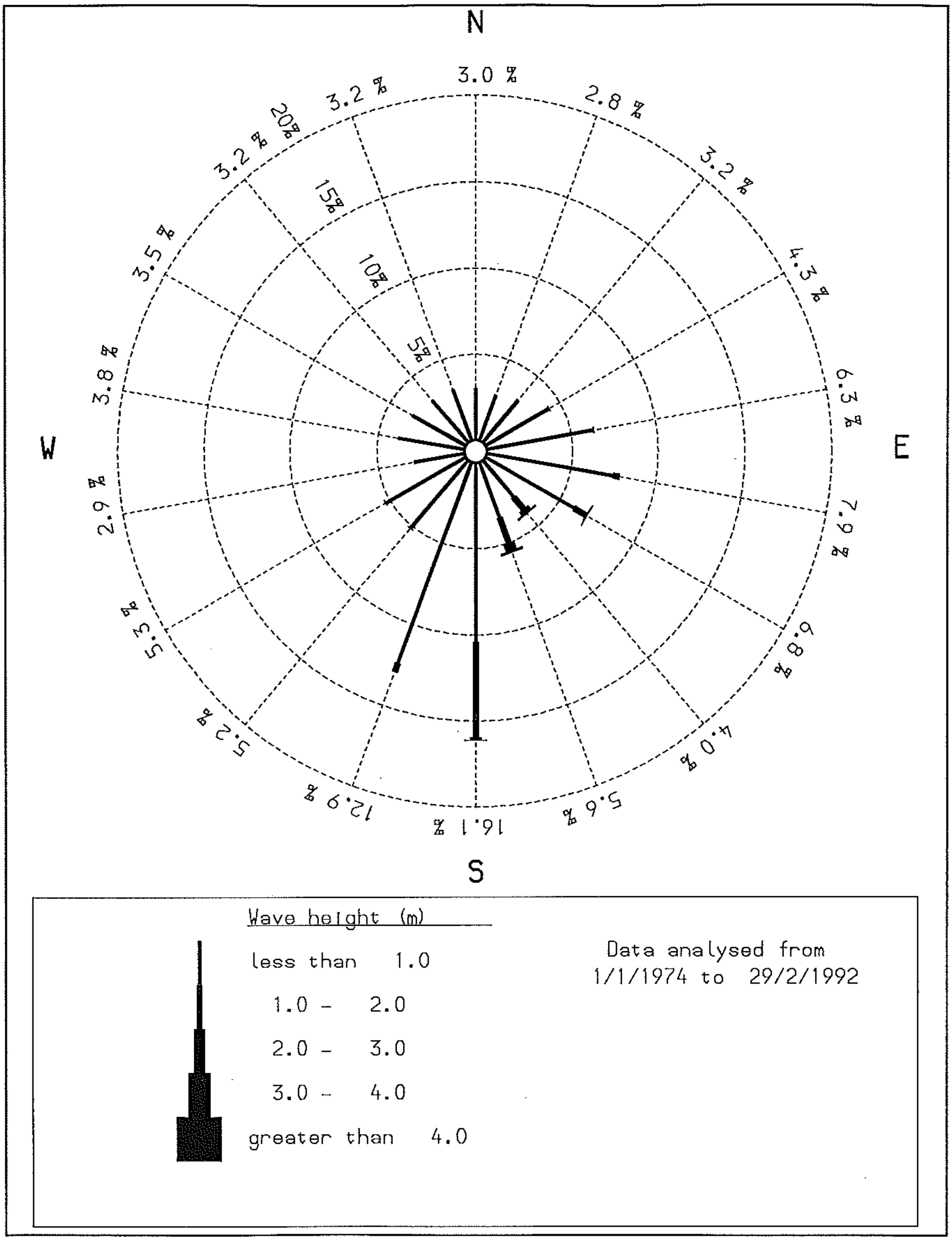


Figure 2.44 Wave rose for inshore point E

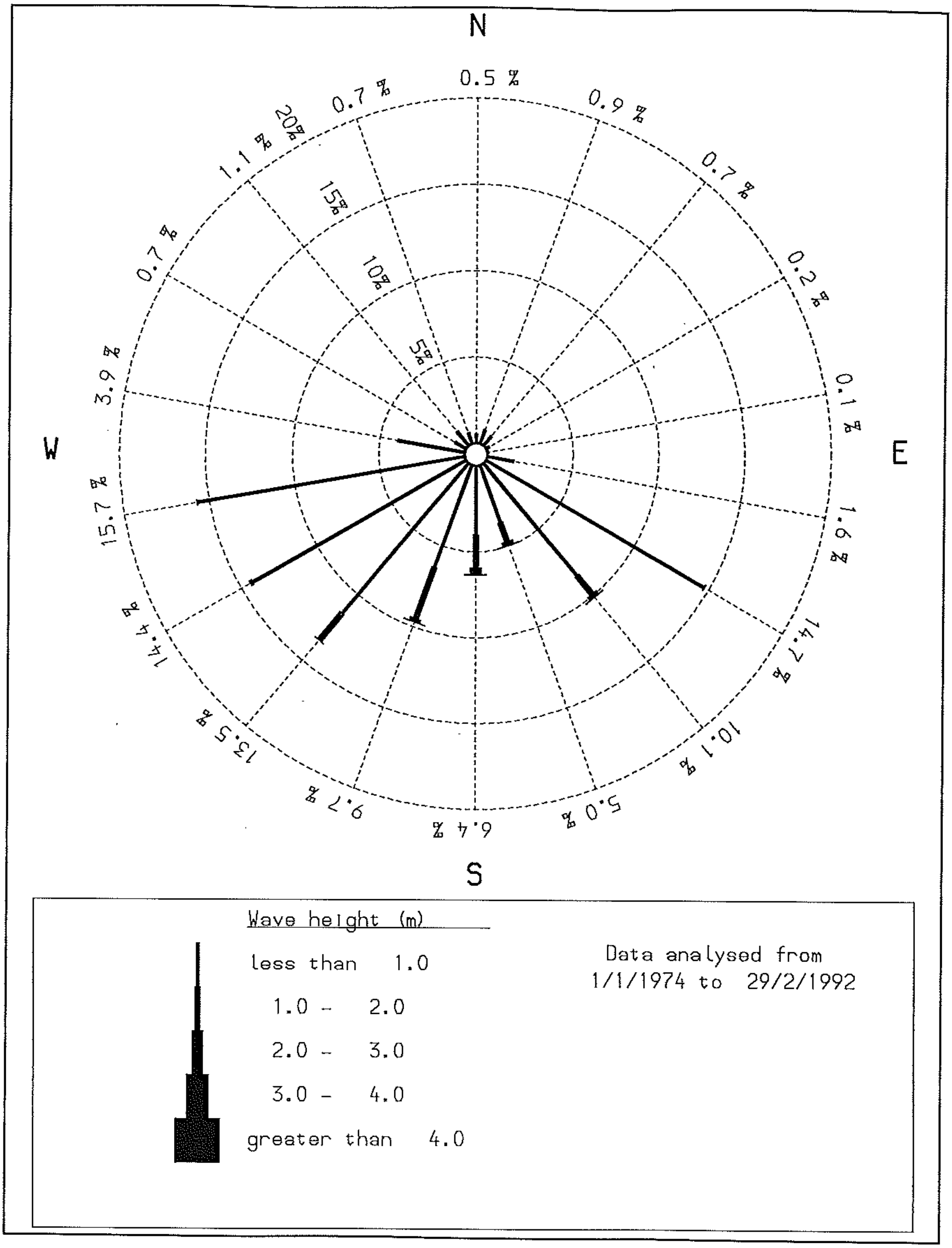


Figure 2.45 Wave rose for inshore point F

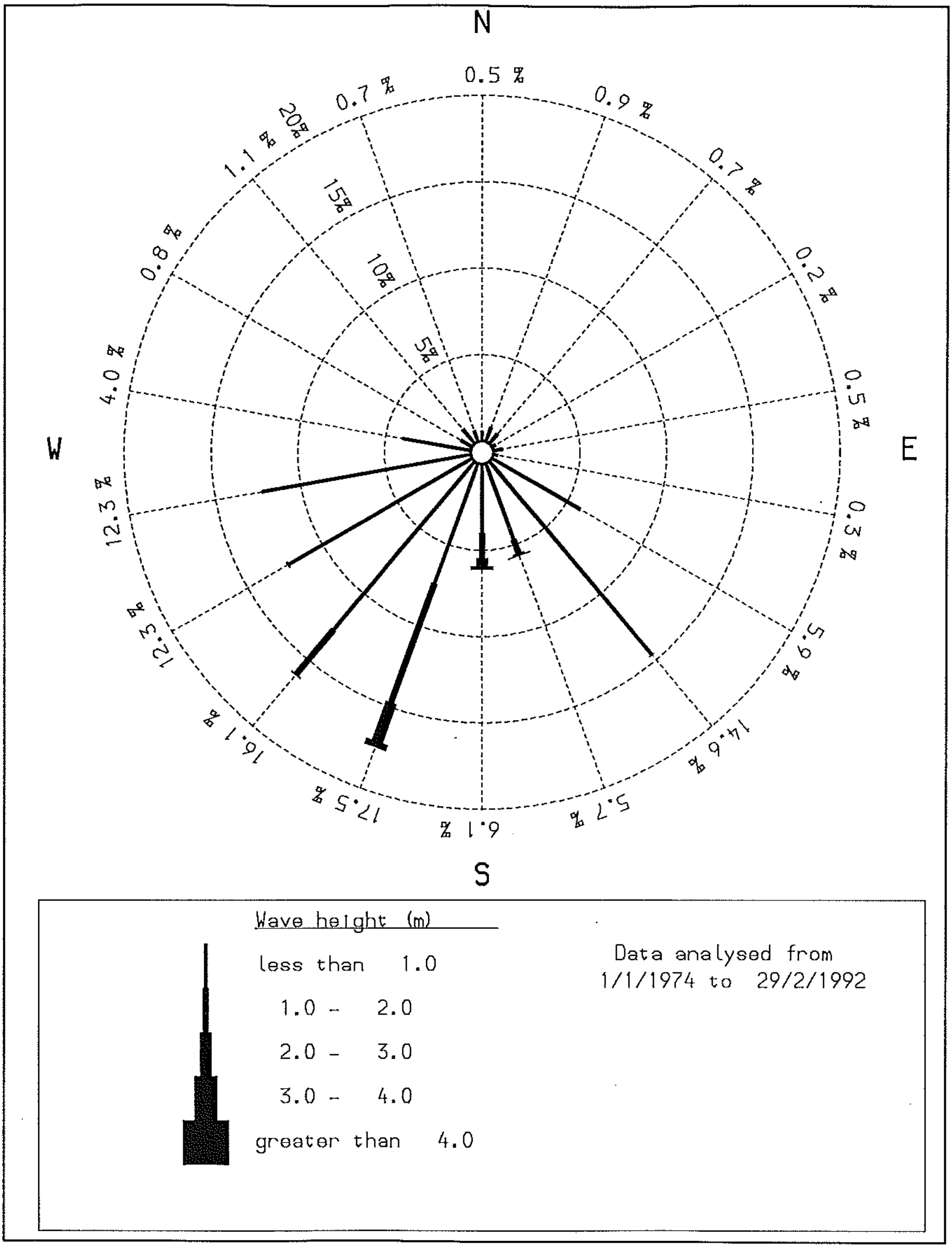


Figure 2.46 Wave rose for inshore point G

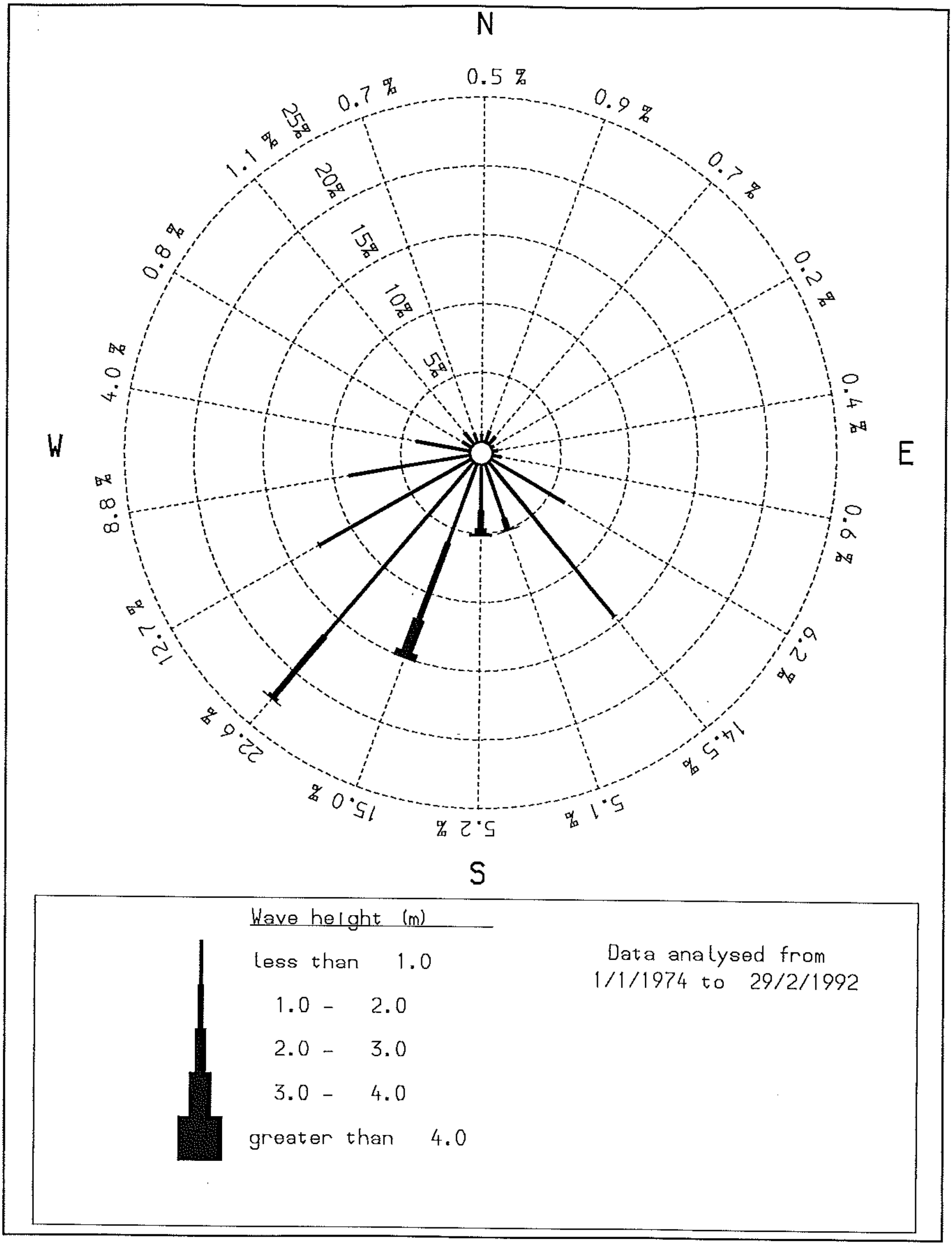


Figure 2.47 Wave rose for inshore point H

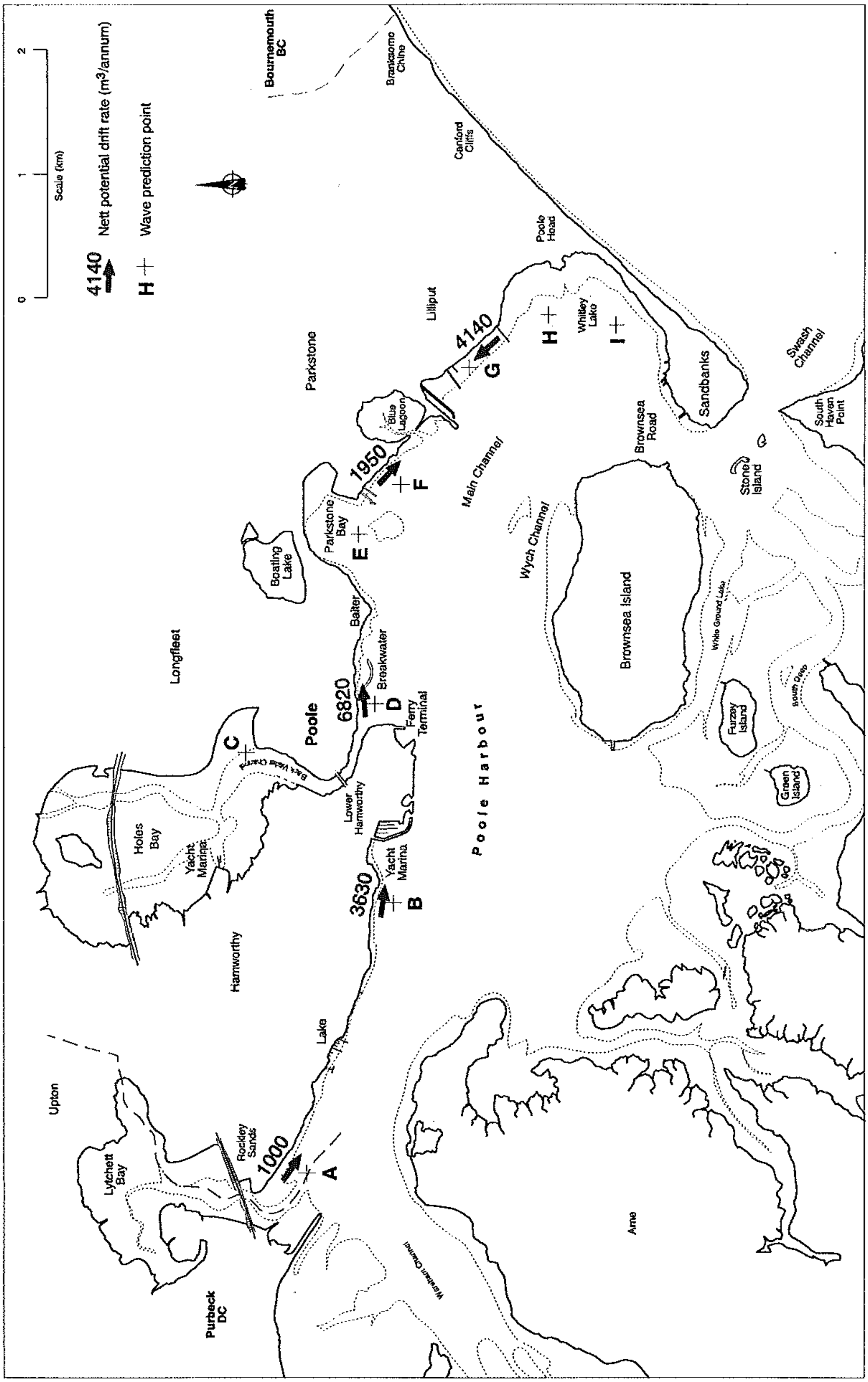


Figure 2.48 Poole Harbour wave prediction points and potential drift rates

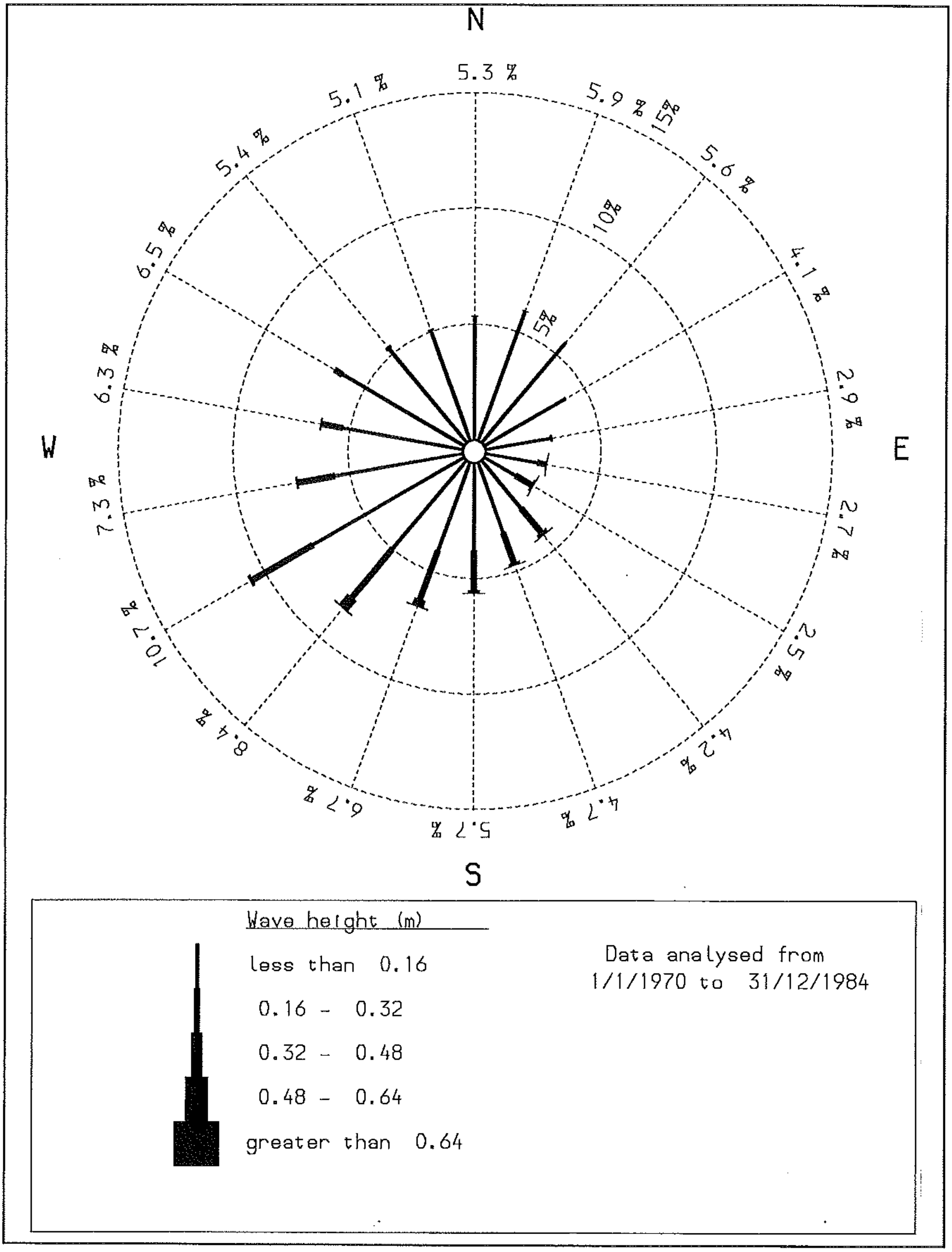


Figure 2.49 Wave rose for Harbour point A

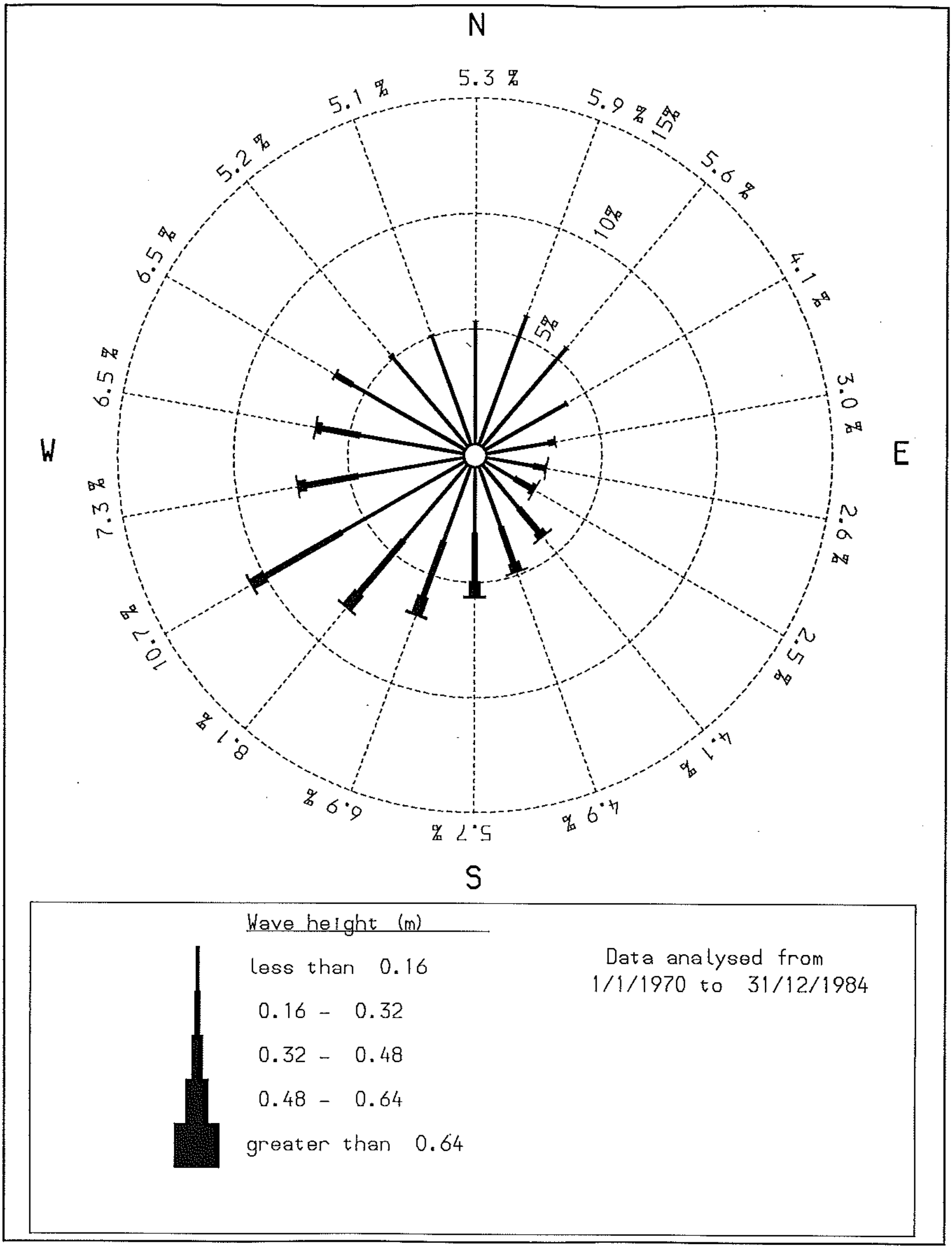


Figure 2.50 Wave rose for Harbour point B

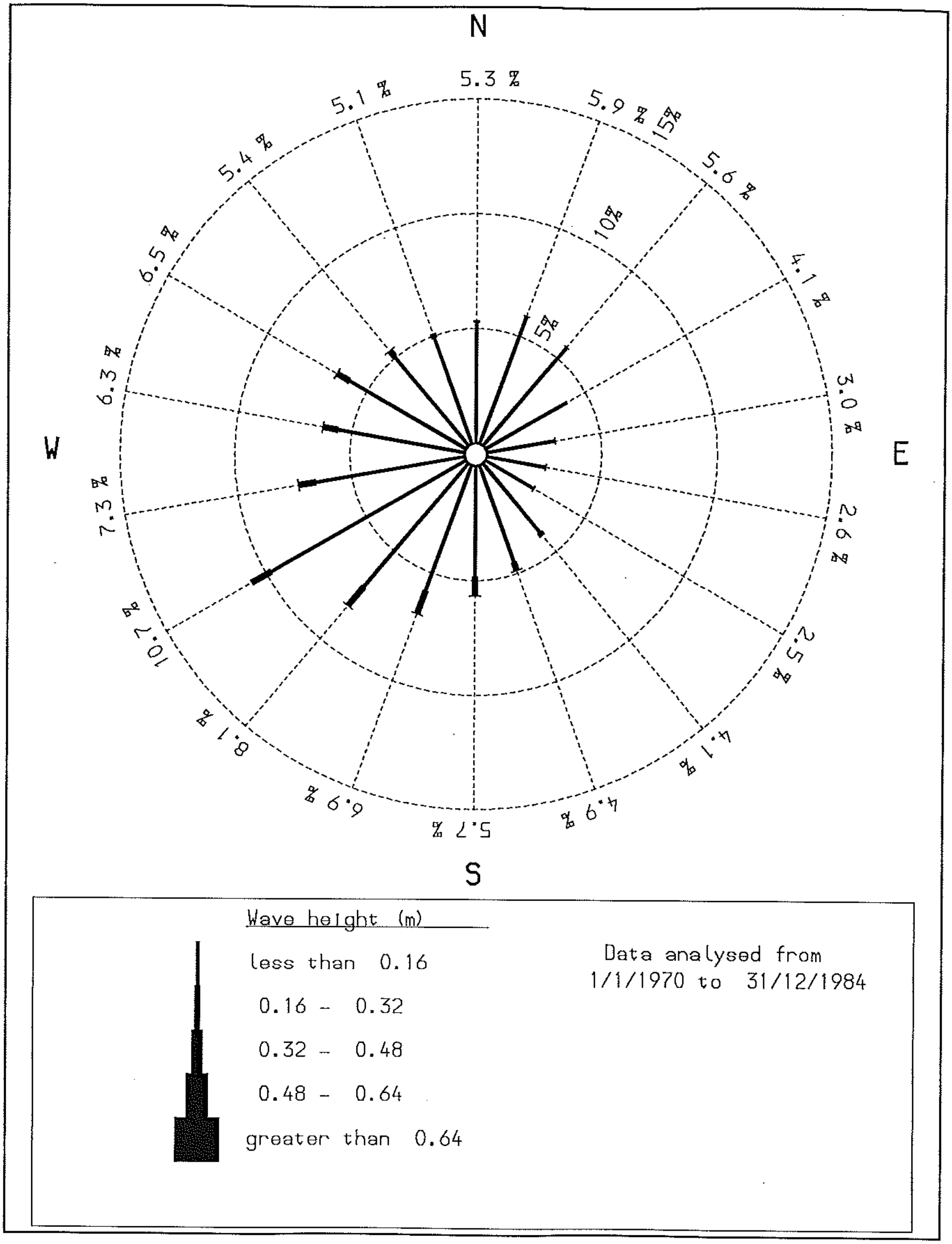


Figure 2.51 Wave rose for Harbour point C

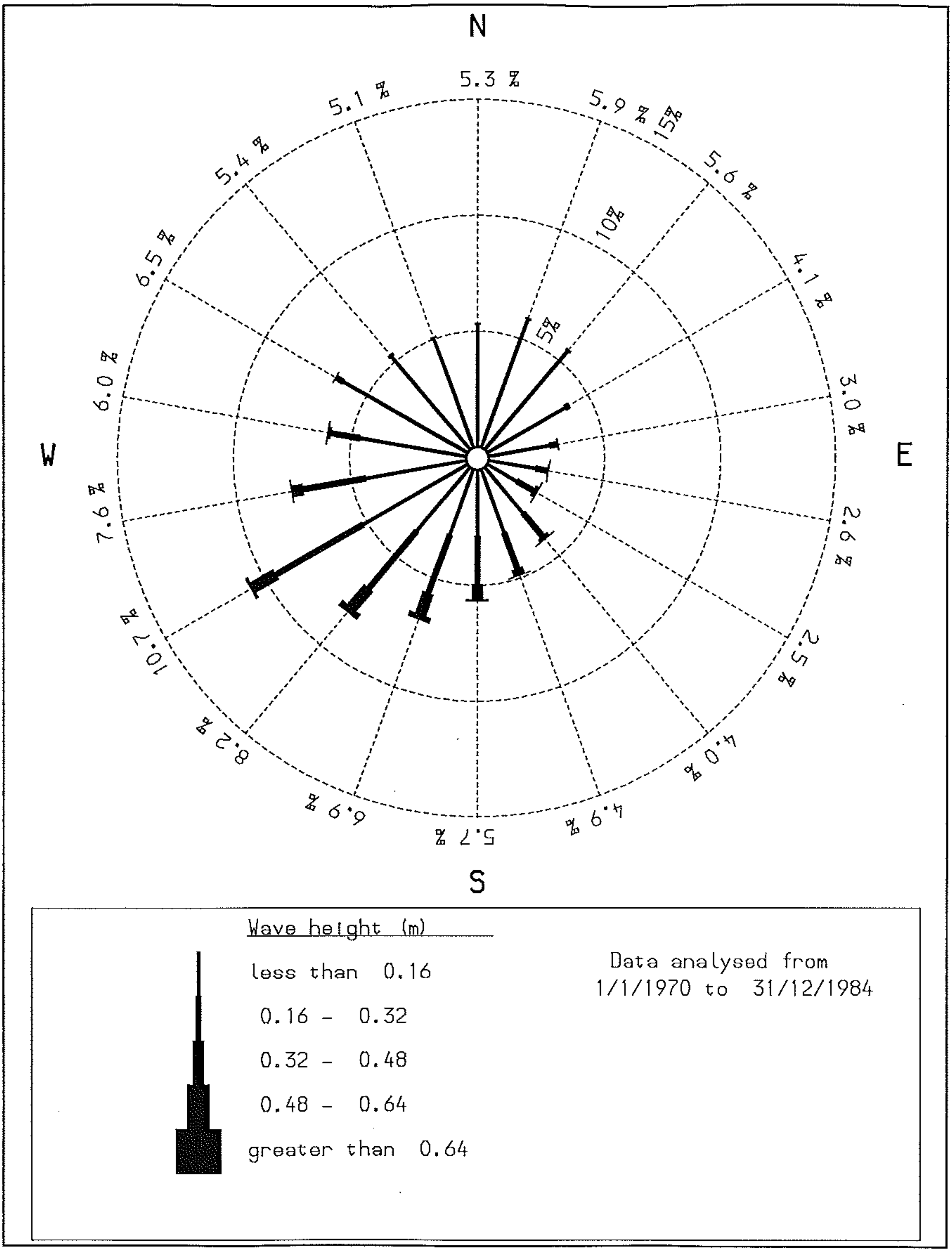


Figure 2.52 Wave rose for Harbour point D

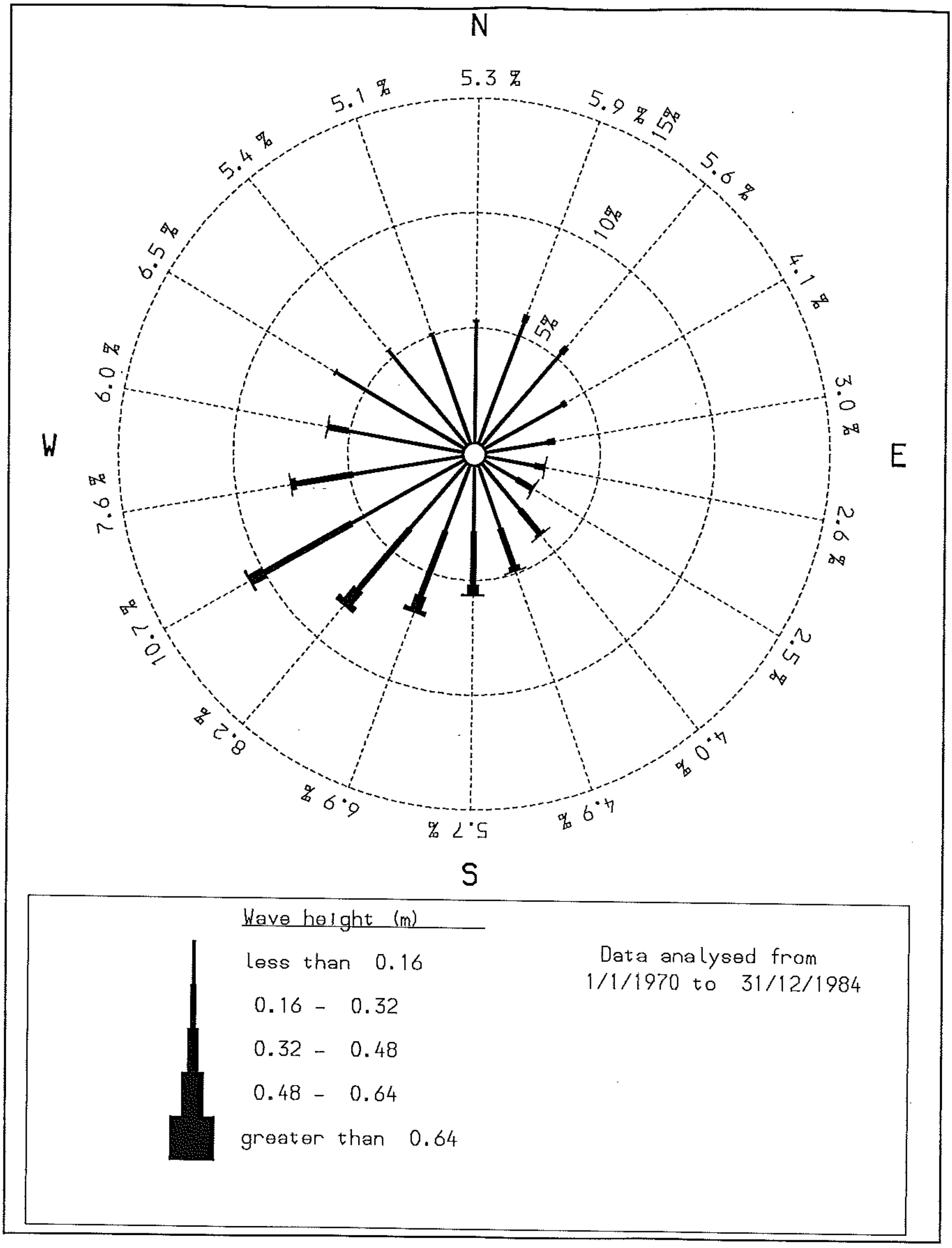


Figure 2.53 Wave rose for Harbour point E

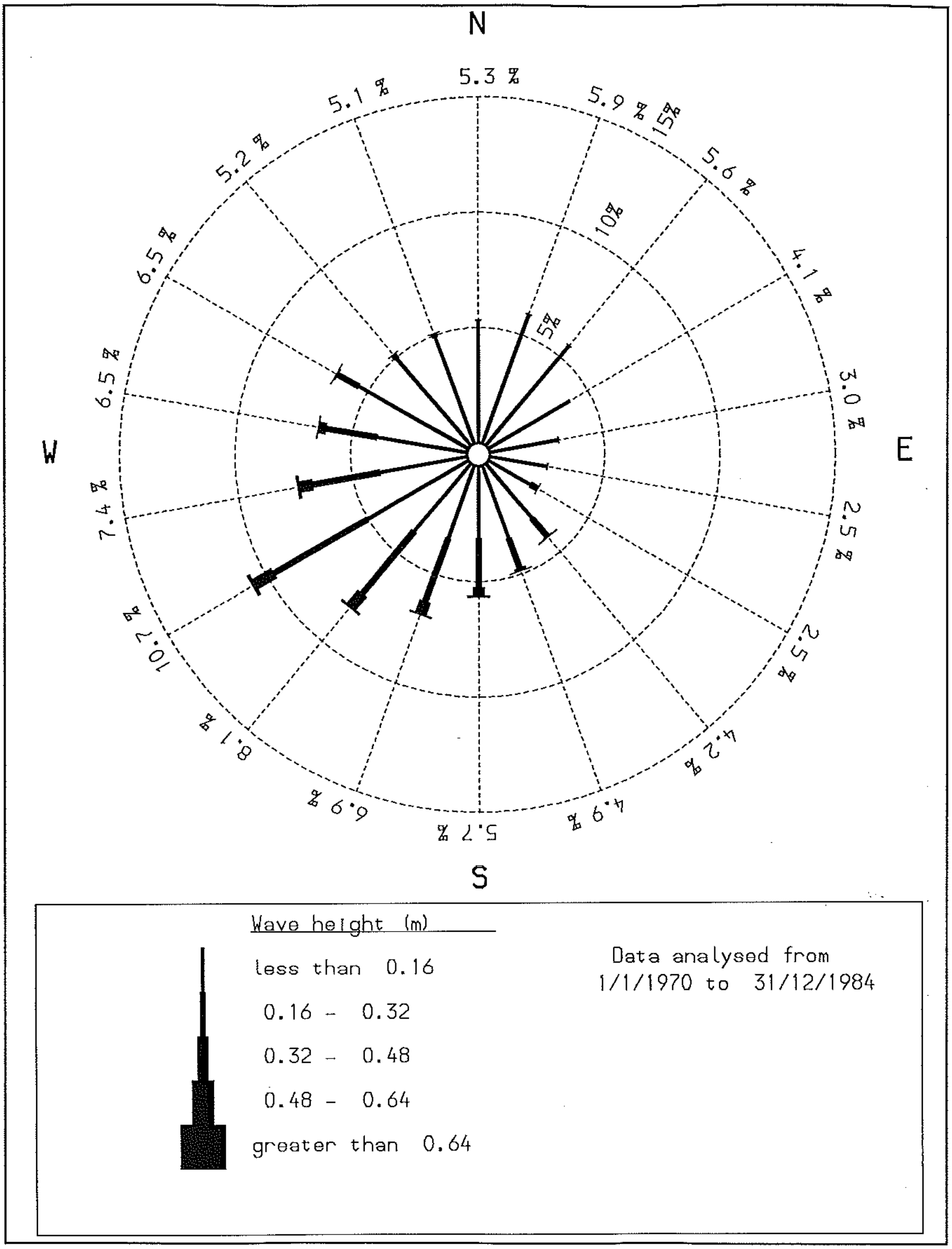


Figure 2.54 Wave rose for Harbour point F

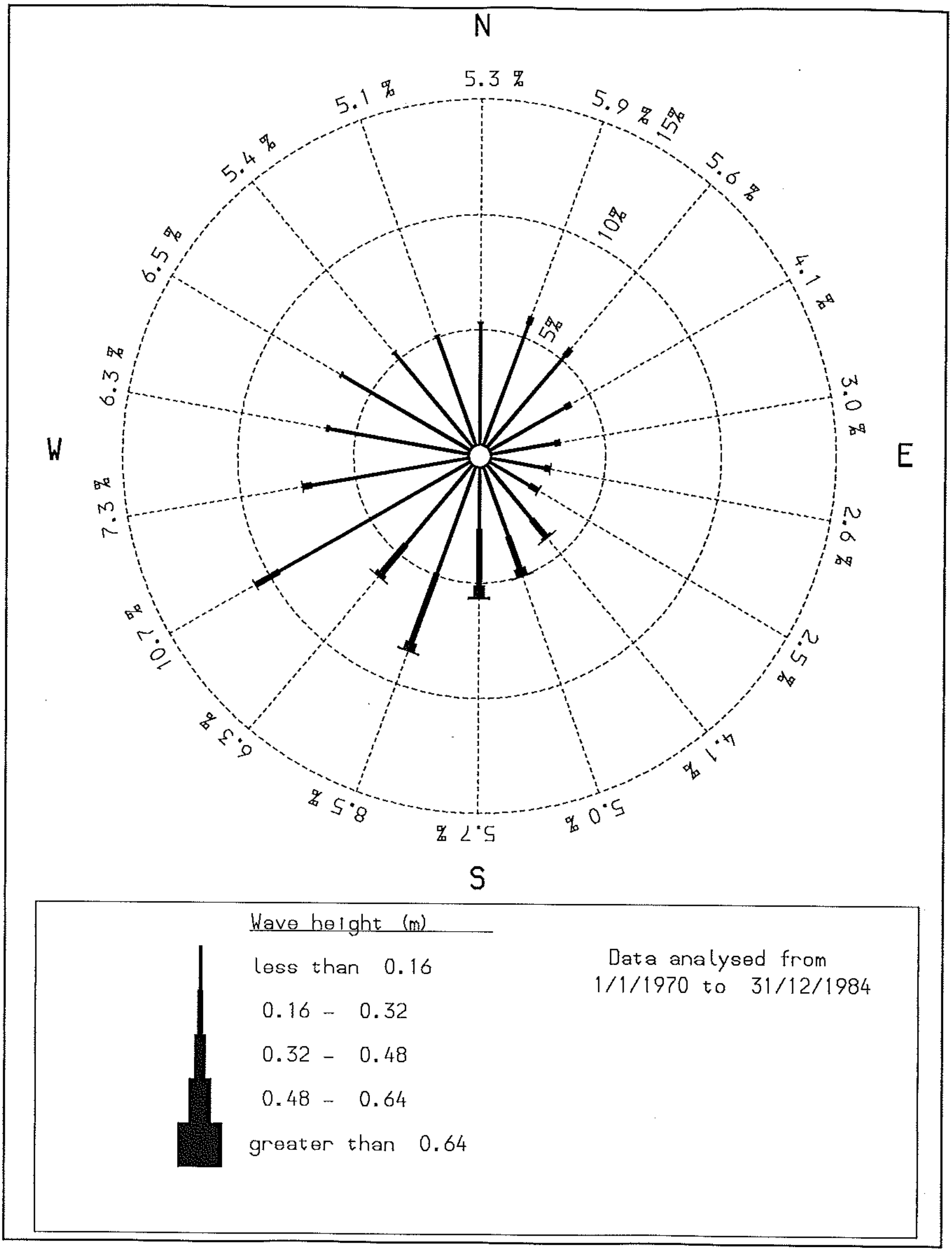


Figure 2.55 Wave rose for Harbour point G

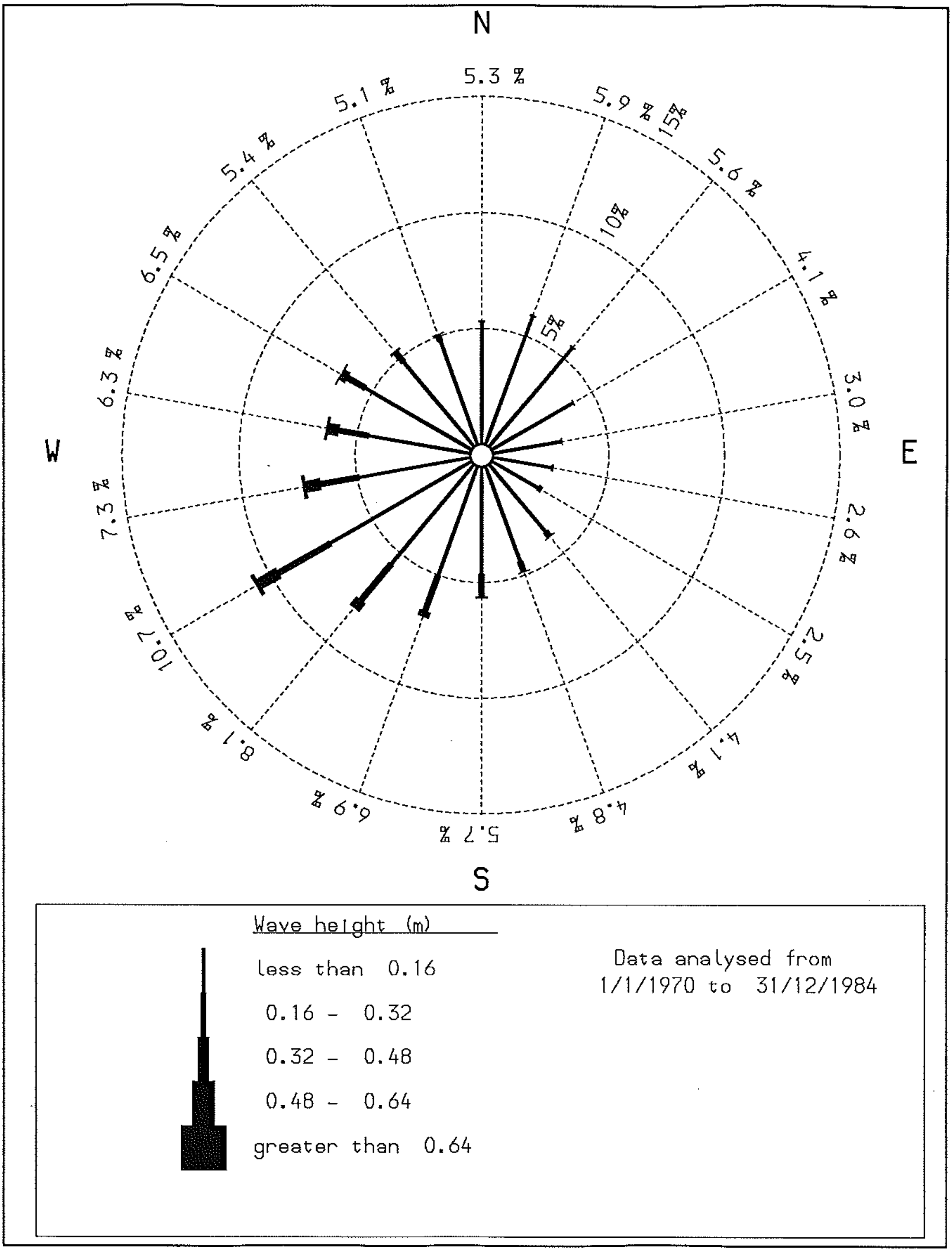


Figure 2.56 Wave rose for Harbour point H

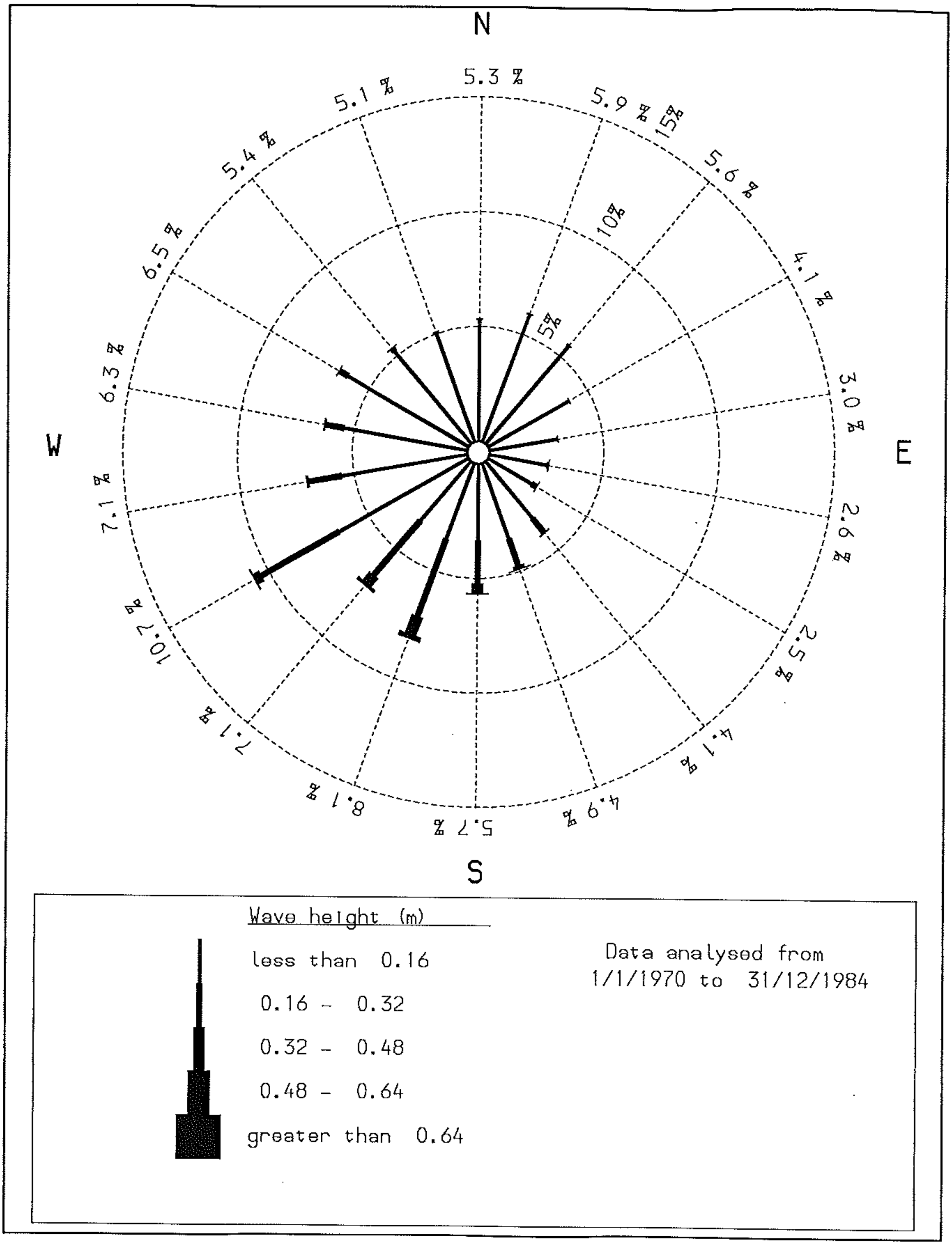


Figure 2.57 Wave rose for Harbour point I

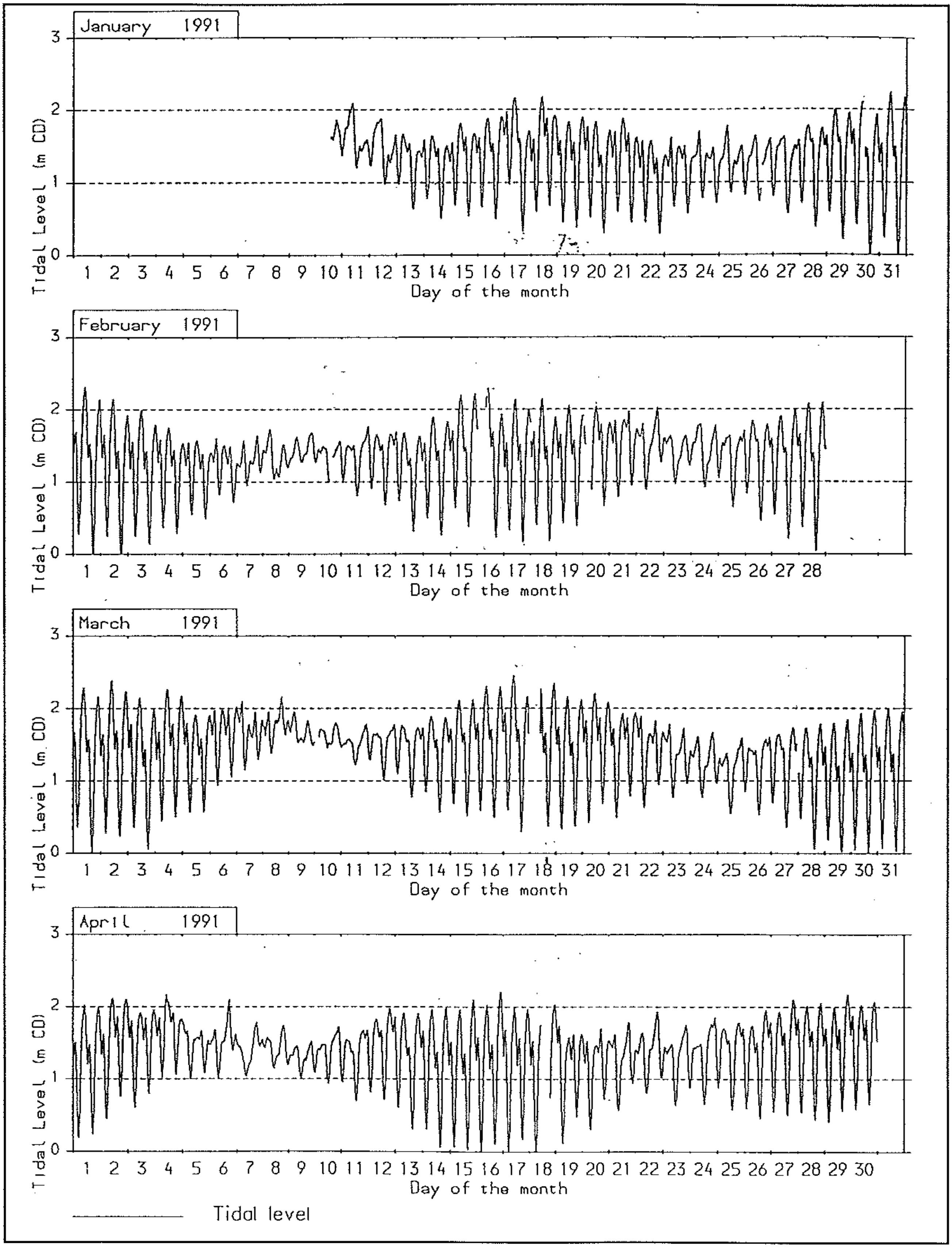


Figure 2.58 Measured tide levels in Poole Harbour. January - April 1991

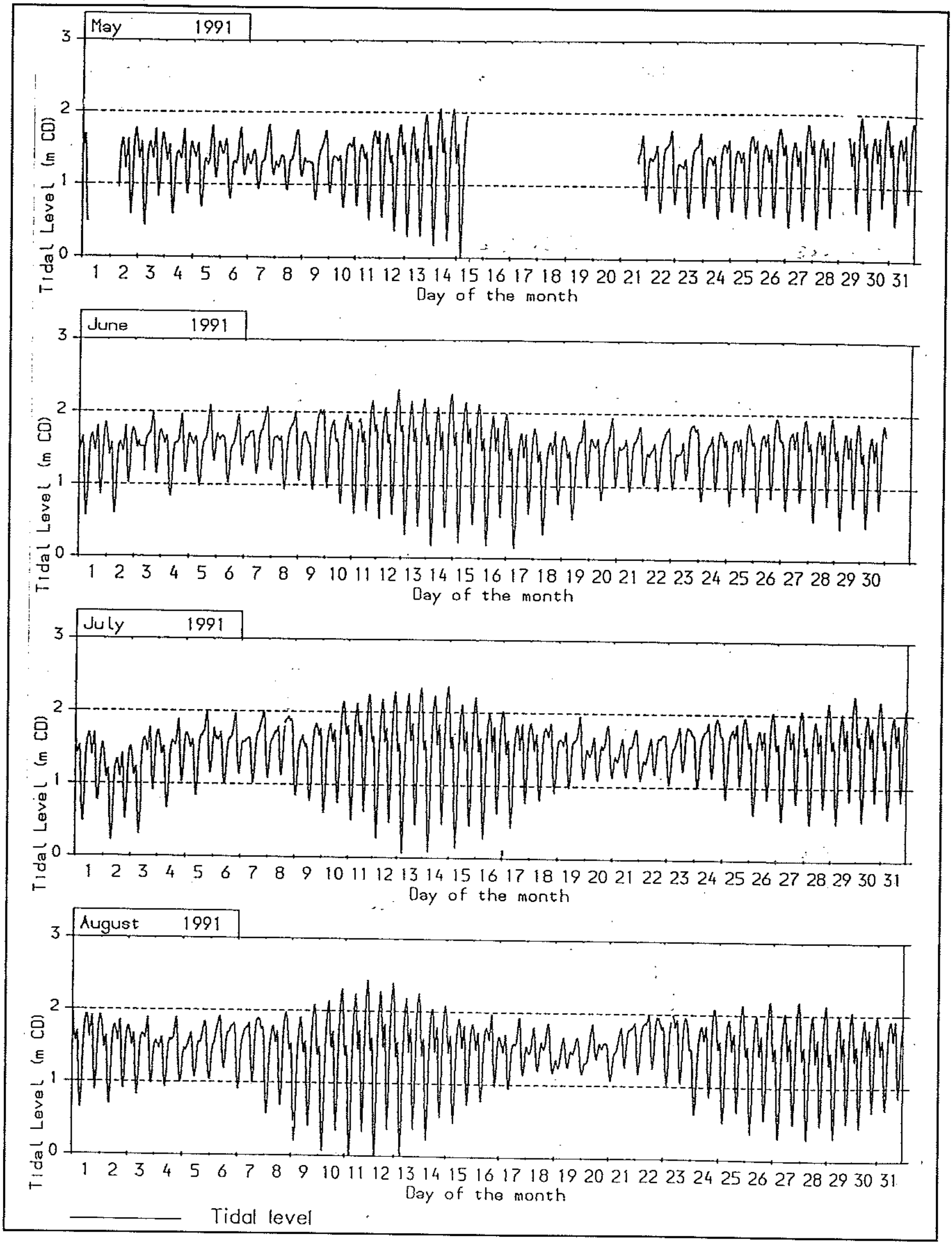


Figure 2.59 Measured tide levels in Poole Harbour. May - August 1991

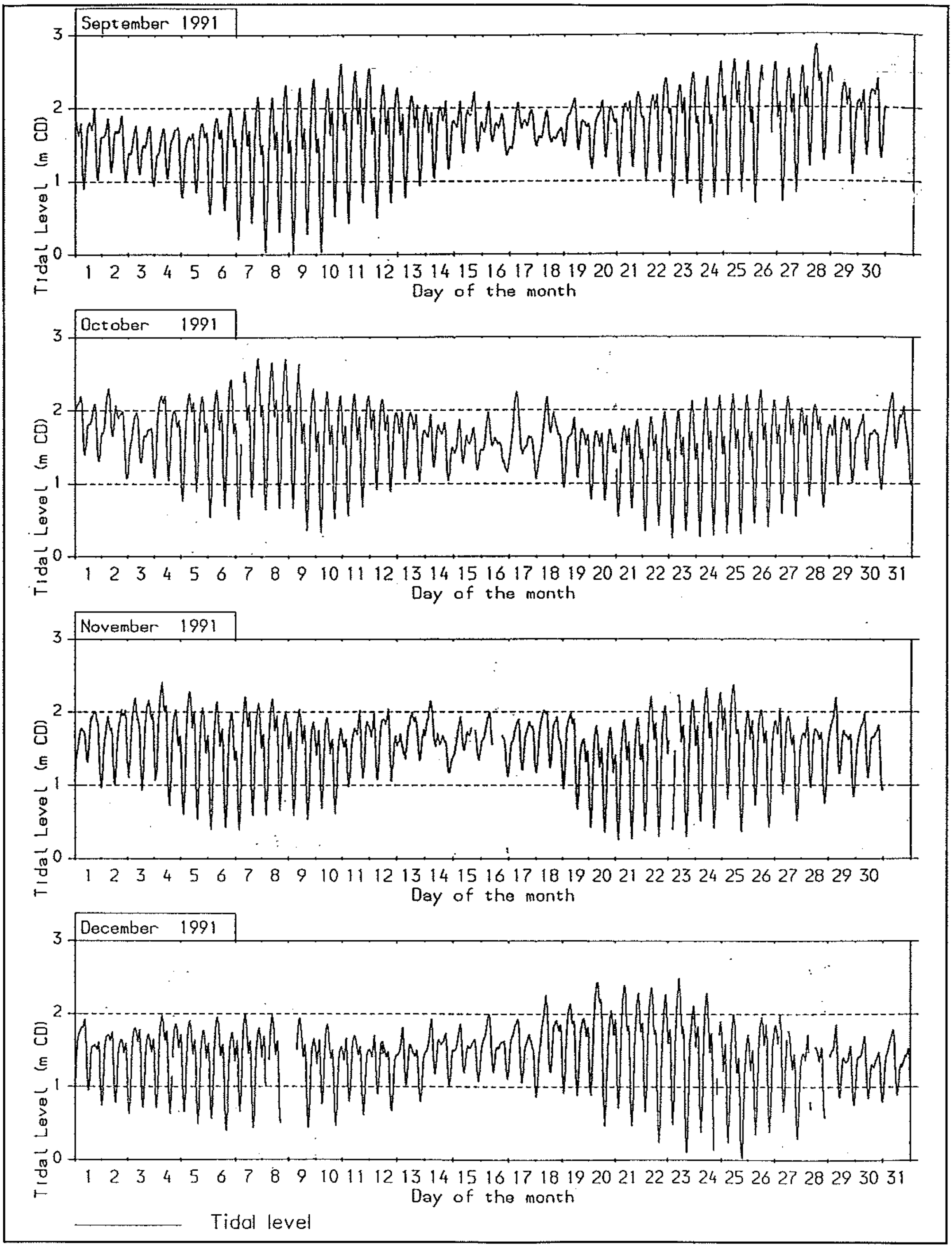


Figure 2.60 Measured tide levels in Poole Harbour. September - December 1991

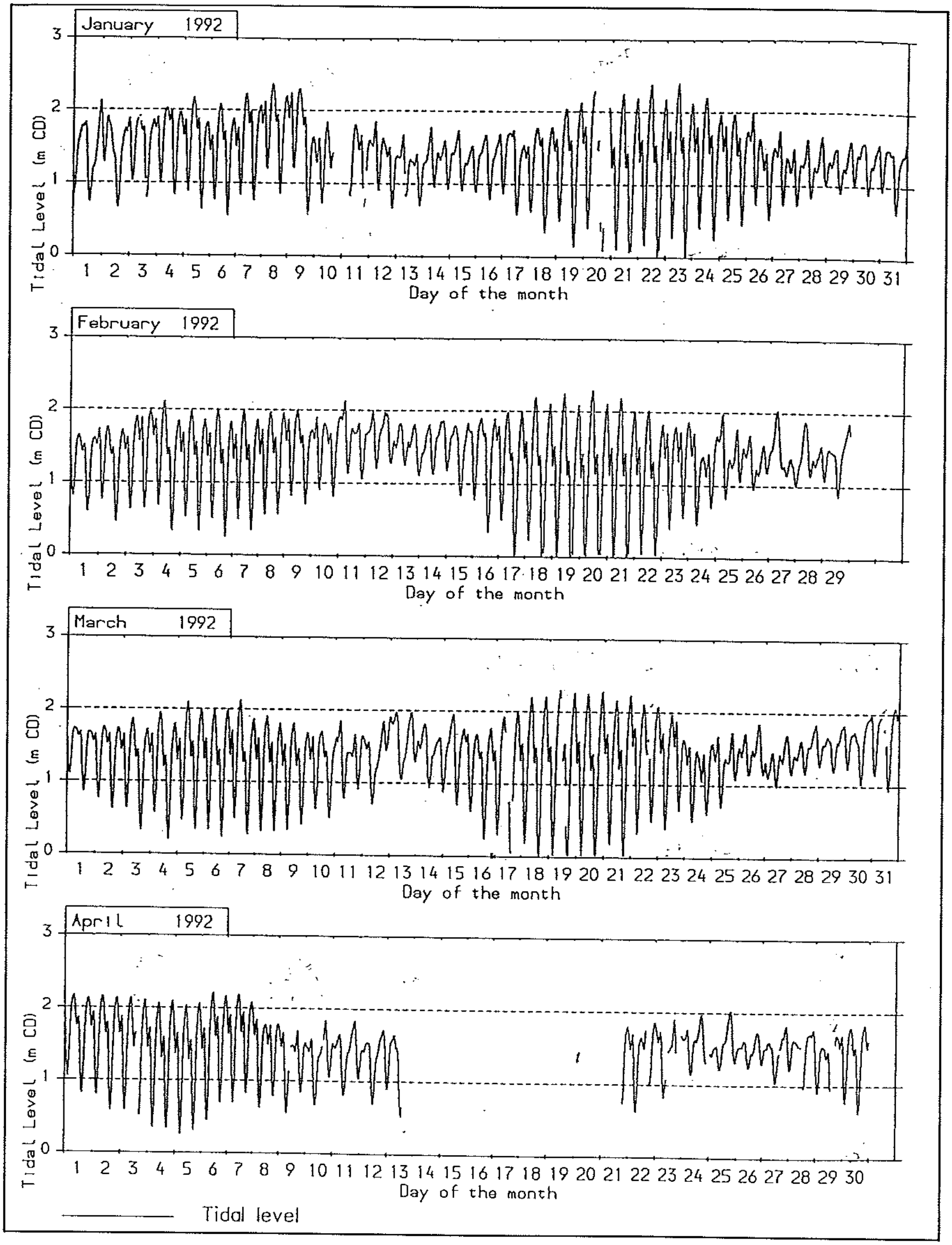


Figure 2.61 Measured tide levels in Poole Harbour. January - April 1992

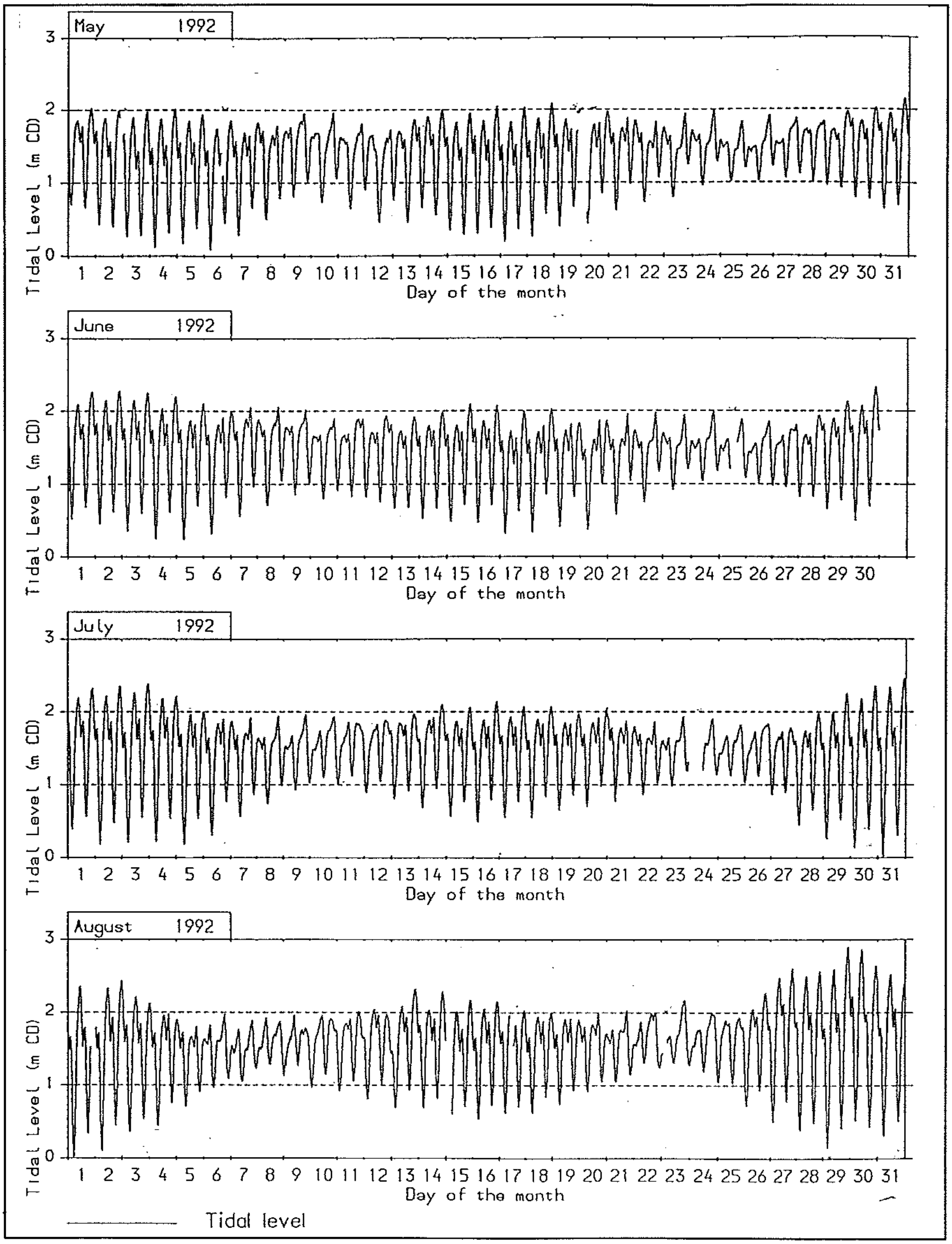


Figure 2.62 Measured tide levels in Poole Harbour. May - August 1992

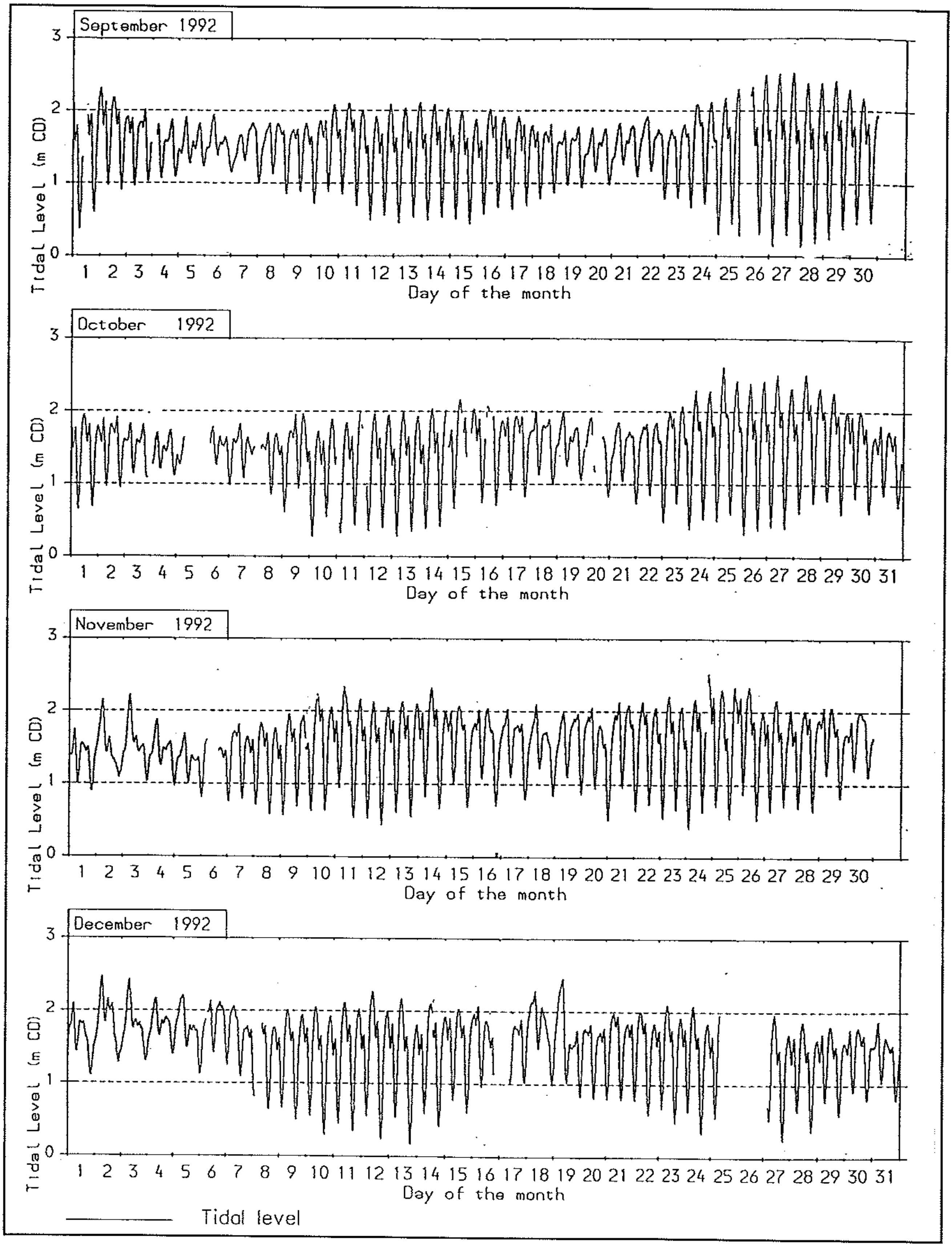


Figure 2.63 Measured tide levels in Poole Harbour. September - December 1992

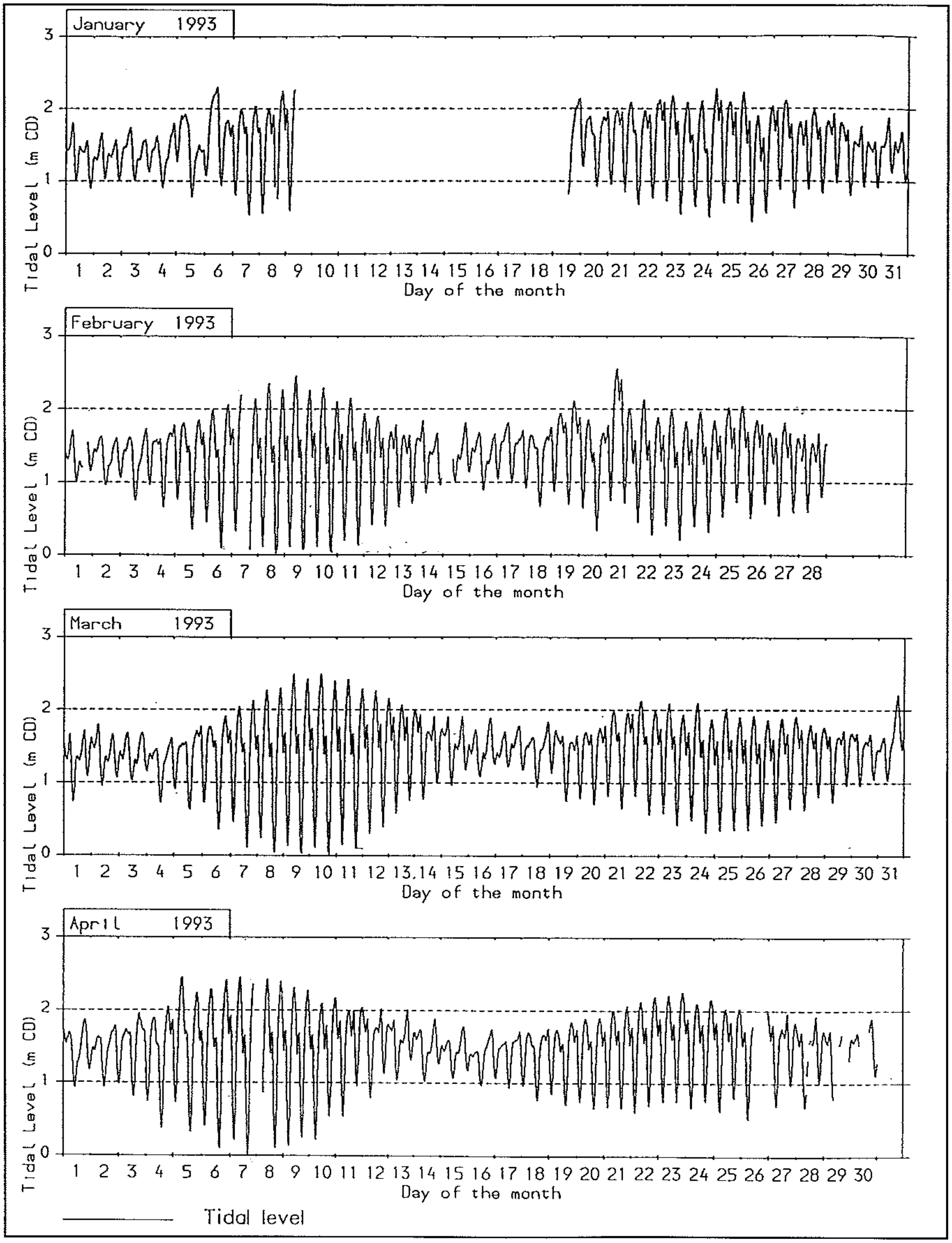


Figure 2.64 Measured tide levels in Poole Harbour. January - April 1993

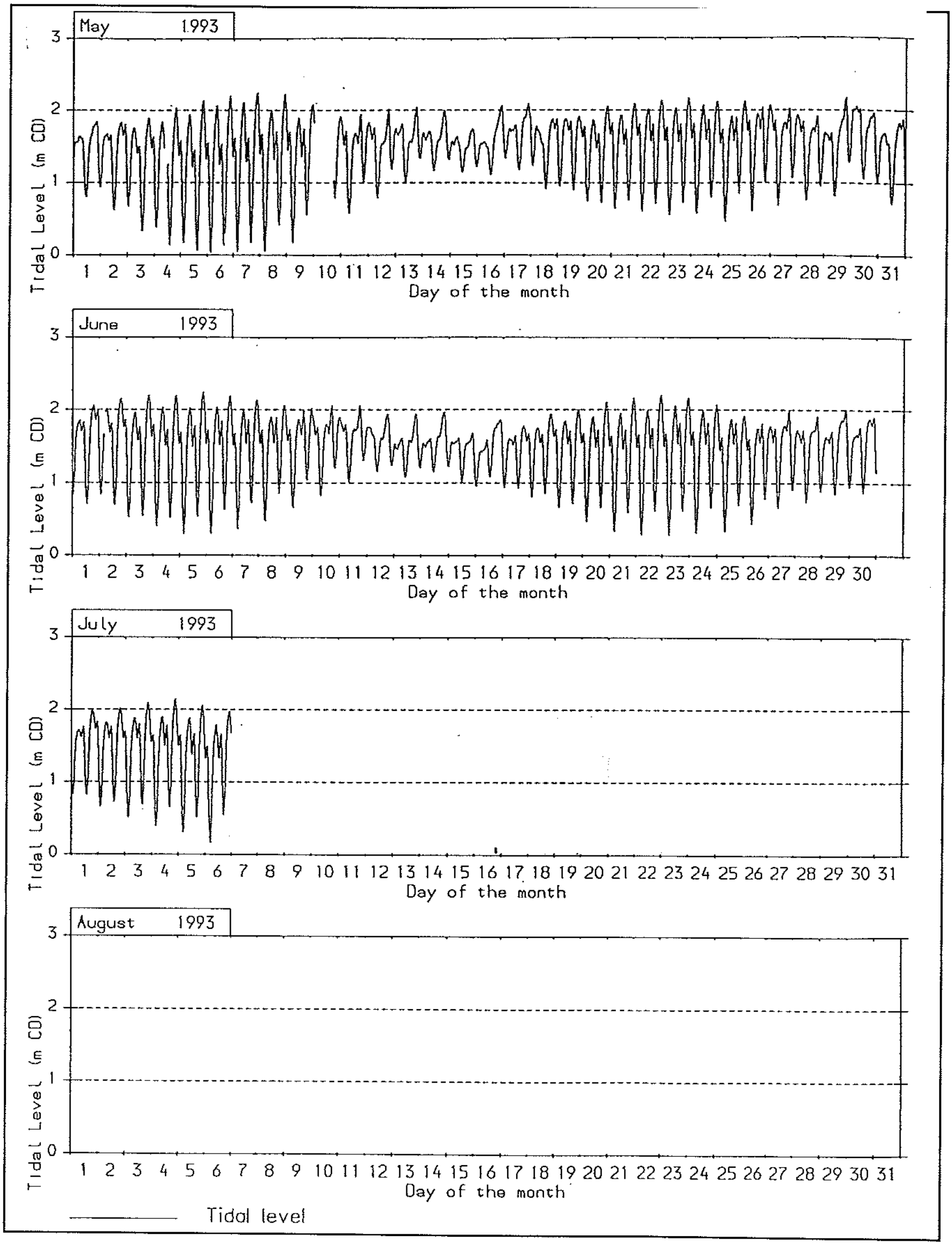


Figure 2.65 Measured tide levels in Poole Harbour. May - August 1993

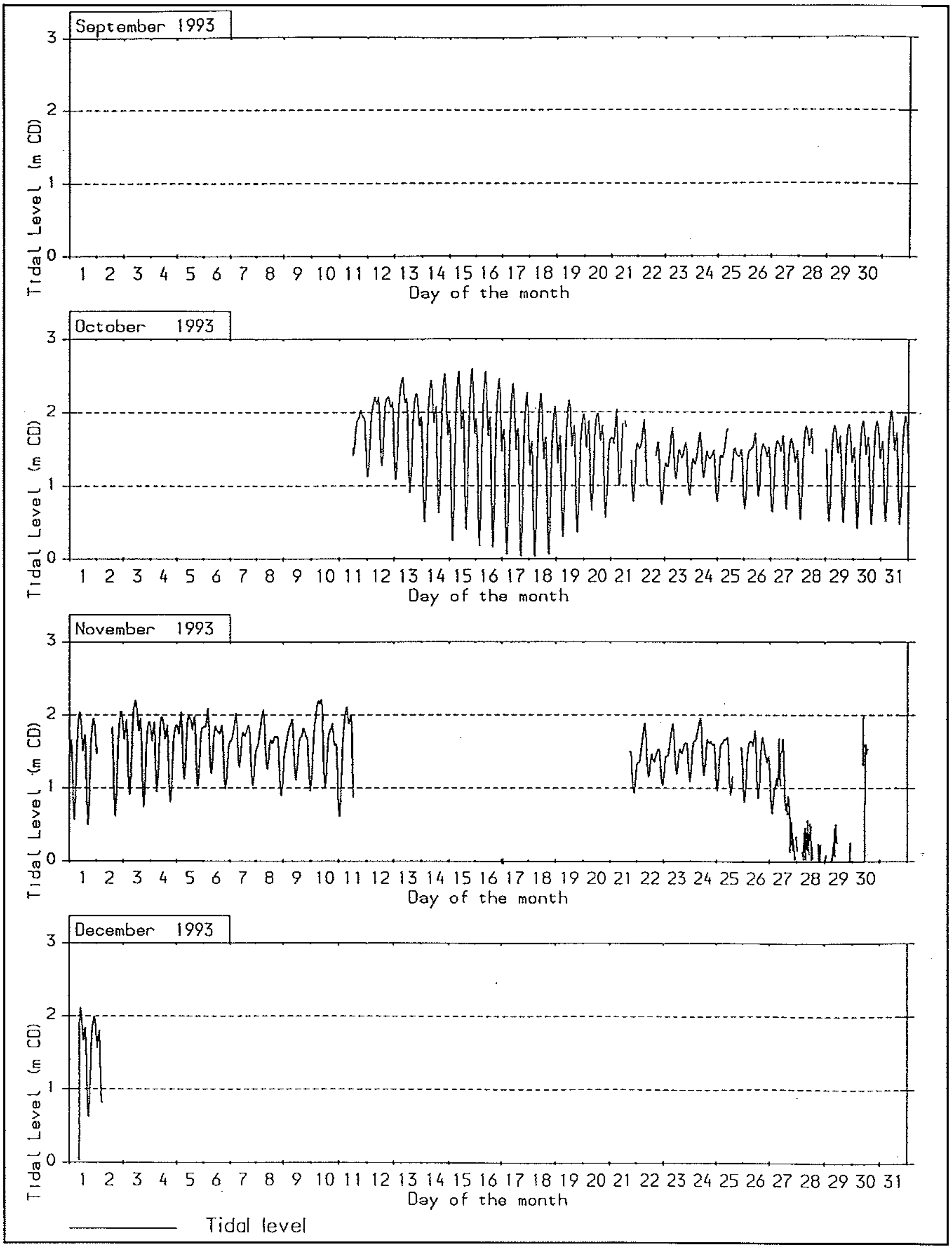


Figure 2.66 Measured tide levels in Poole Harbour. September - December 1993

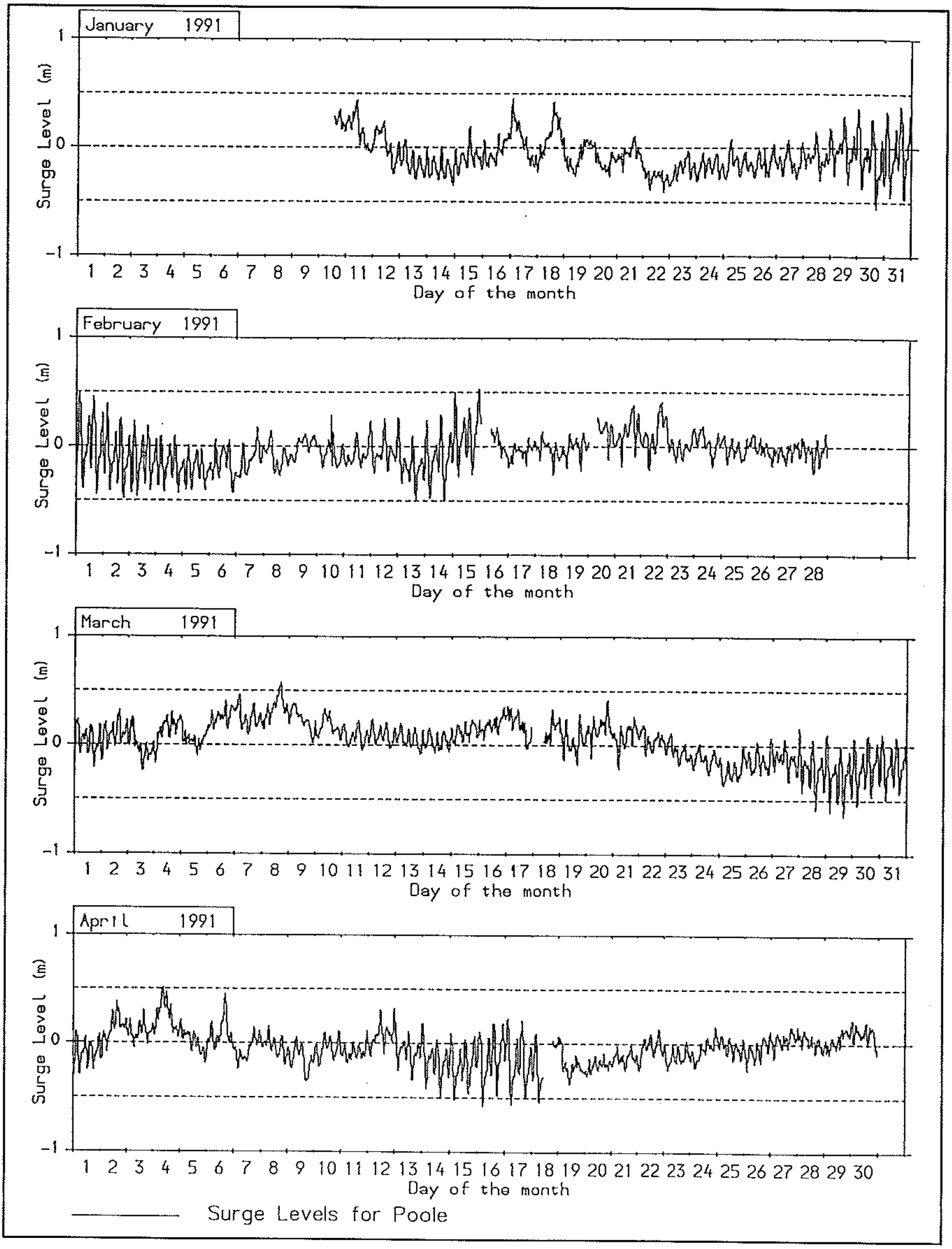


Figure 2.67 Surge levels in Poole Harbour. January - April 1991

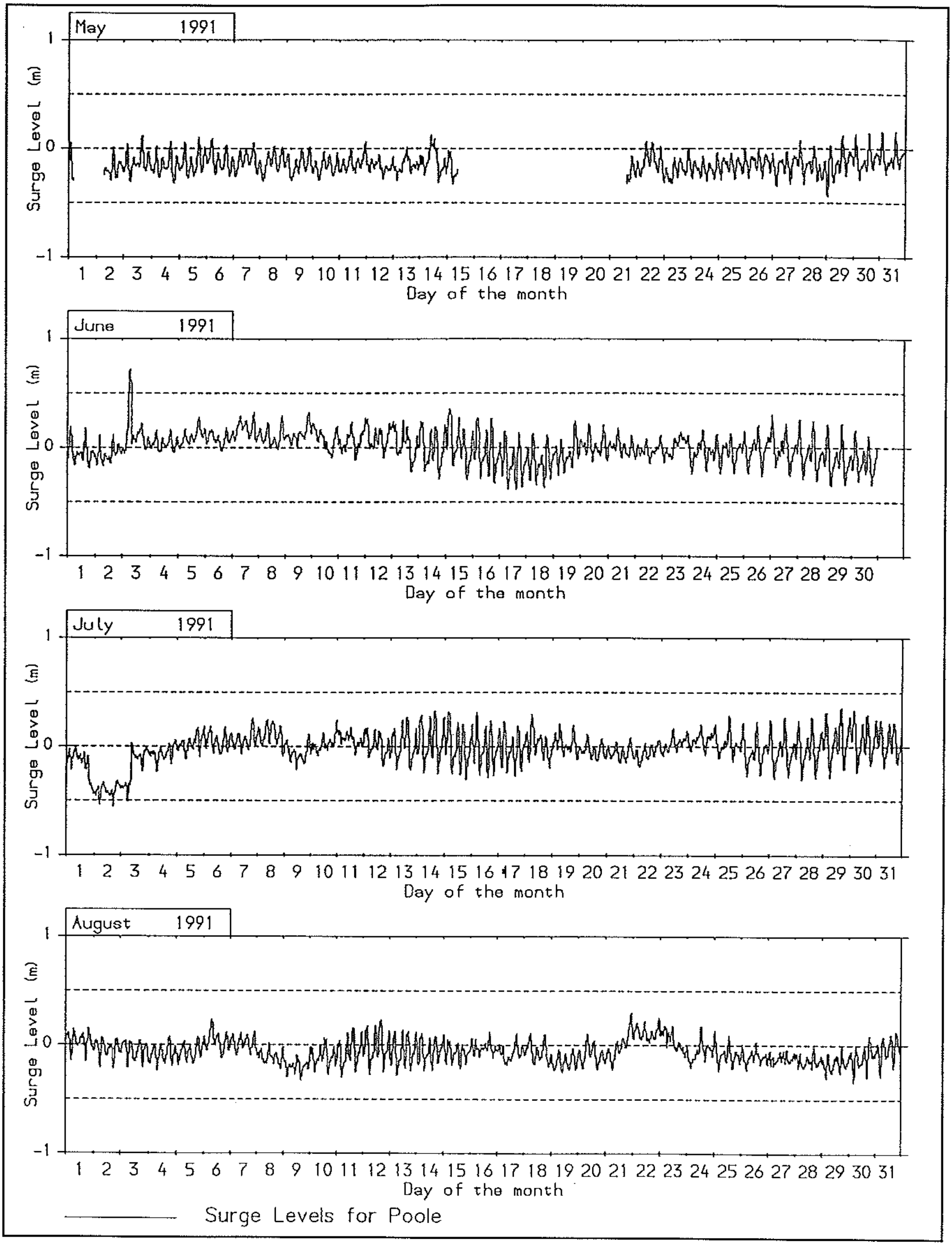


Figure 2.68 Surge levels in Poole Harbour. May - August 1991

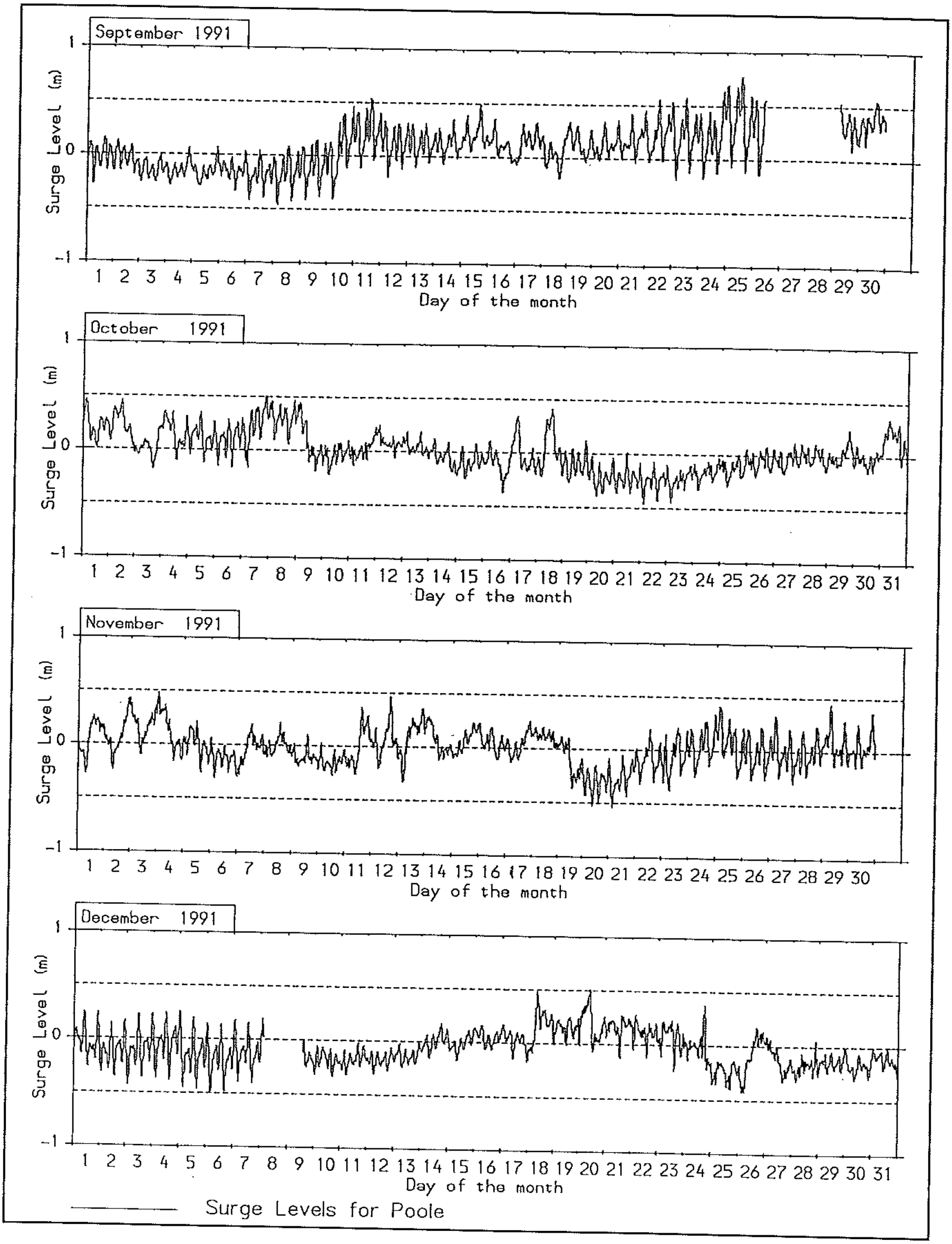


Figure 2.69 Surge levels in Poole Harbour. September - December 1991

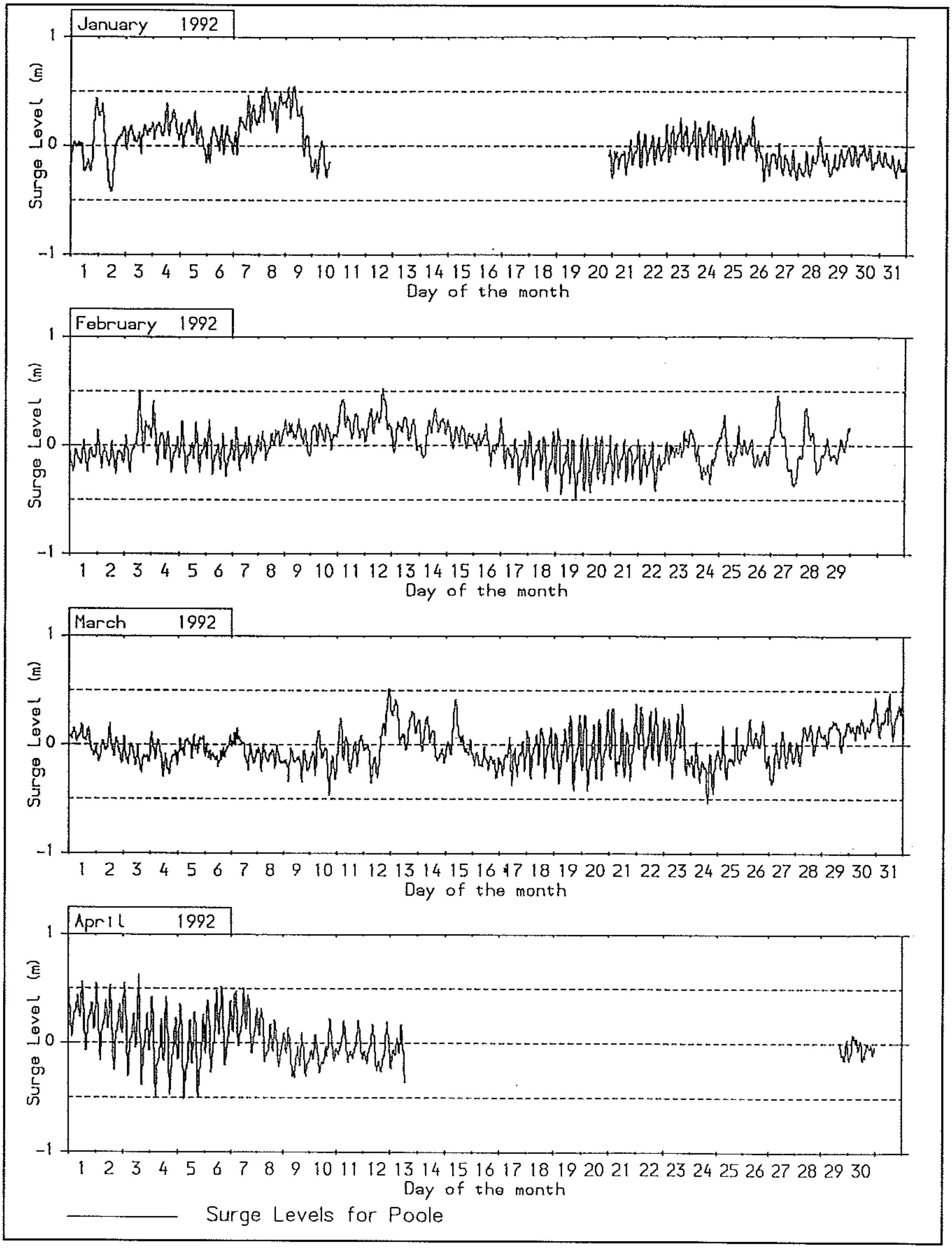


Figure 2.70 Surge levels in Poole Harbour. January - April 1992

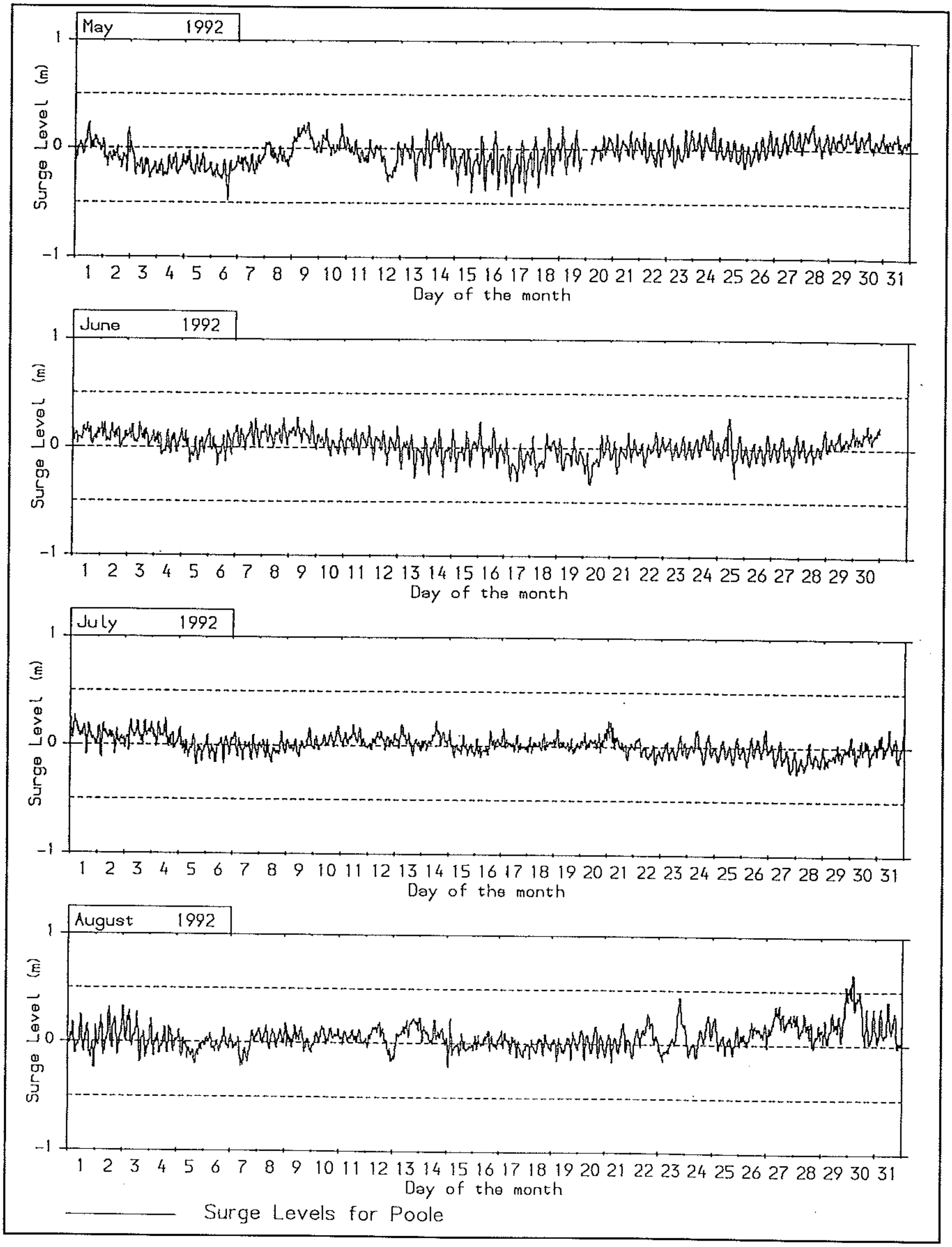


Figure 2.71 Surge levels in Poole Harbour. May - August 1992

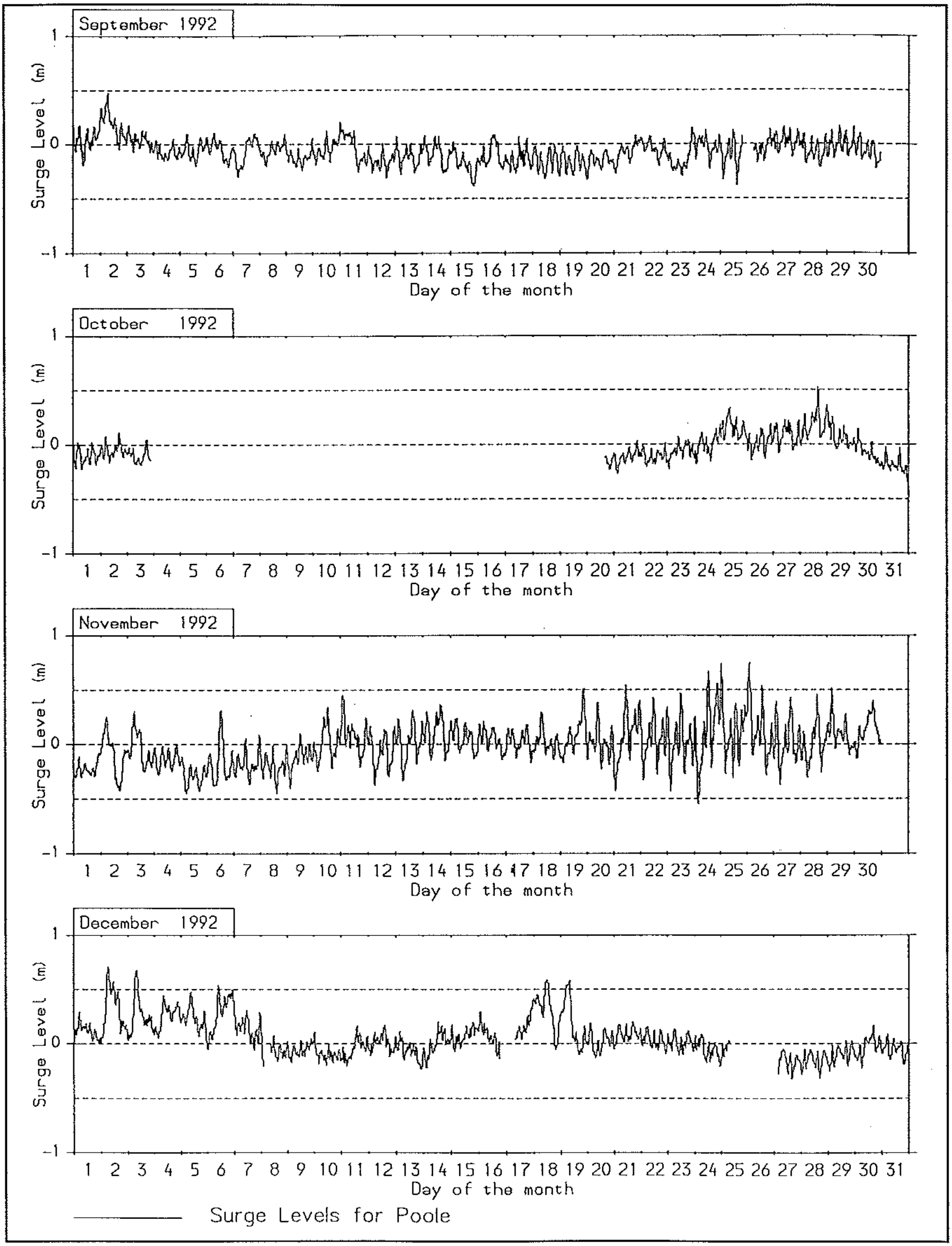


Figure 2.72 Surge levels in Poole Harbour. September - December 1992

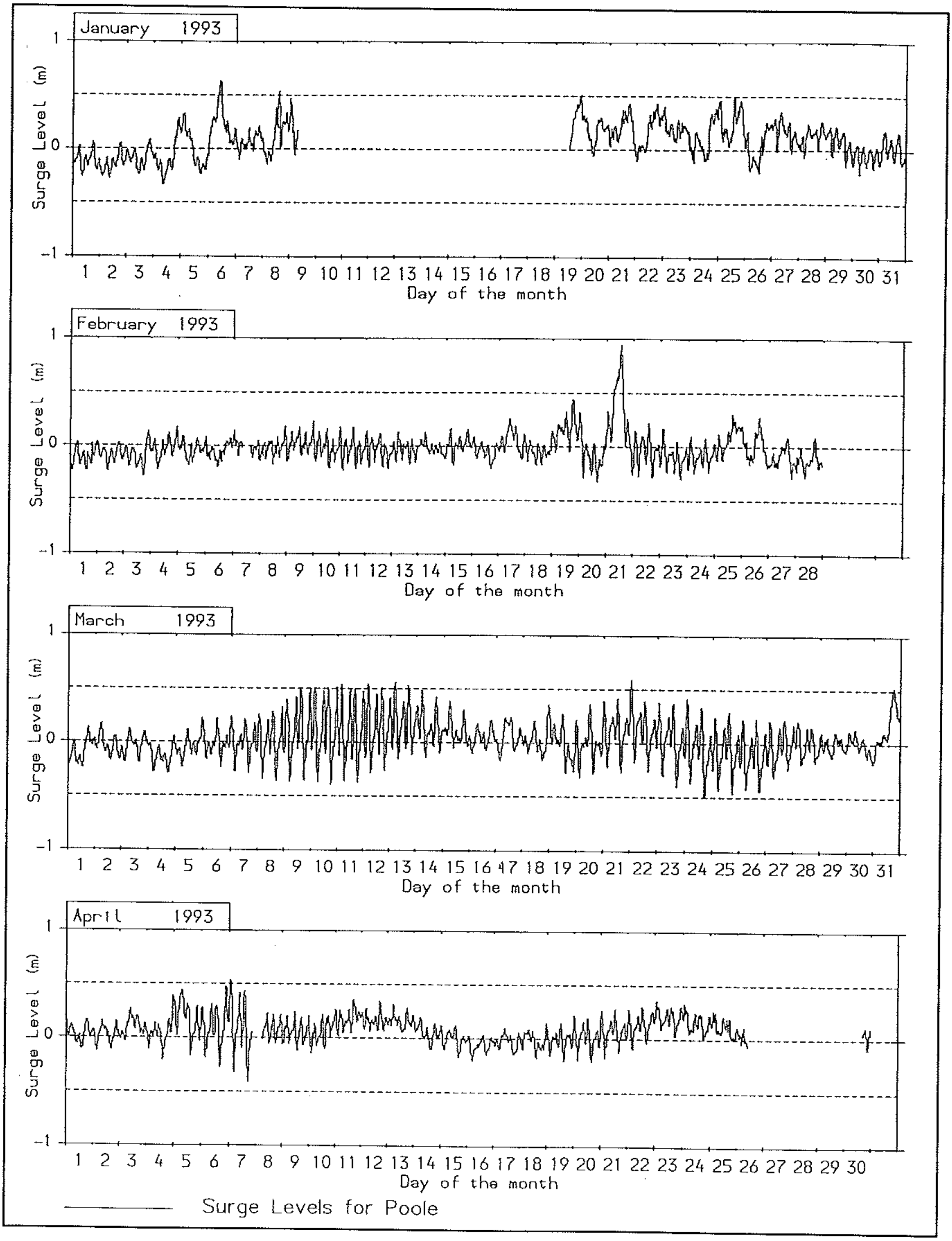


Figure 2.73 Surge levels in Poole Harbour. January - April 1993

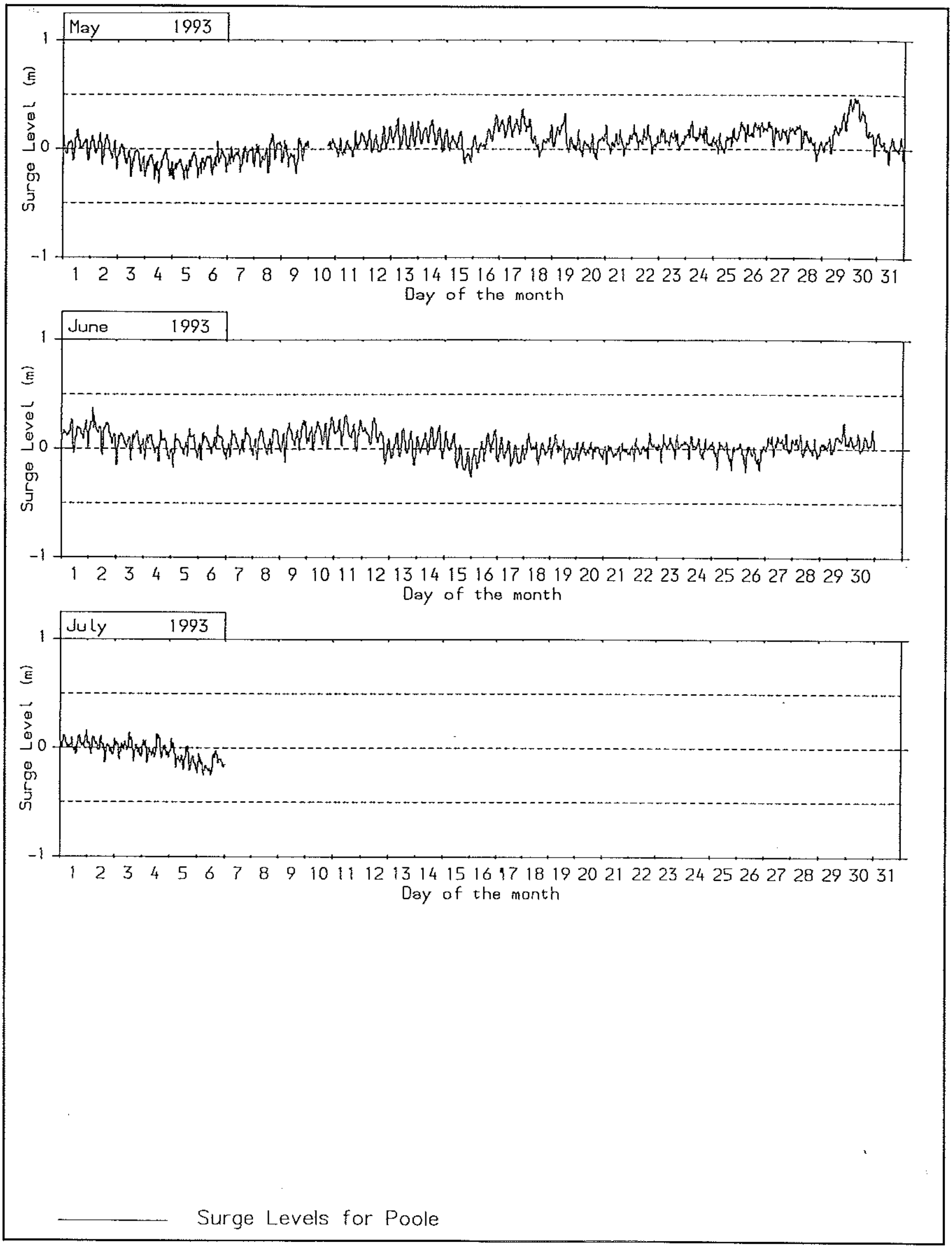


Figure 2.74 Surge levels in Poole Harbour. May - July 1993

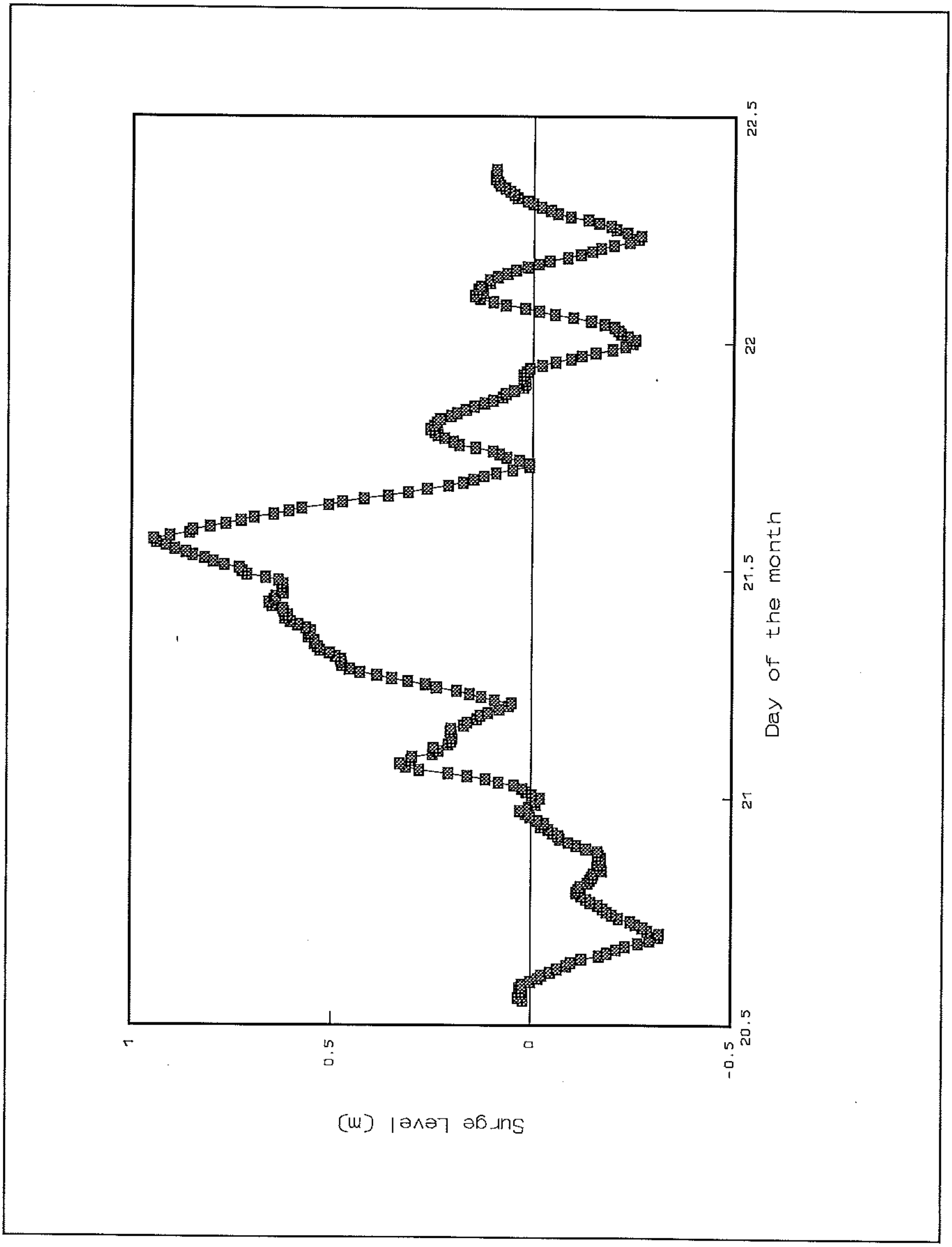


Figure 2.75 Surge in Poole Harbour 21 February 1993

Table Distribution of Hs and water level at Point 2 -30 to 210N

Data shown in number of occurrences

High water level (m OD)

Hs (metres)	1.30 - 1.50	1.50 - 1.70	1.70 - 1.90	1.90 - 2.10	2.10 - 2.30	2.30 - 2.50	2.50 - 2.70	2.70 - 2.90	Calms/ no dat	No.in row
4.50- 5.00	0	0	0	0	0	0	0	0	0	0
4.00- 4.50	0	0	0	0	0	0	0	0	0	0
3.50- 4.00	0	0	0	0	0	0	0	0	0	0
3.00- 3.50	0	0	0	0	1	0	0	0	0	1
2.50- 3.00	0	0	0	3	2	0	0	1	0	6
2.00- 2.50	0	0	0	5	3	1	0	1	0	10
1.50- 2.00	0	1	6	5	2	1	0	0	0	15
1.00- 1.50	0	14	9	24	6	2	2	0	0	57
0.50- 1.00	4	34	40	27	30	8	1	0	0	144
0.00- 0.50	13	36	68	59	25	4	4	1	0	210
No data/calms	0	0	0	0	0	0	0	0	0	0
No. in column	17	85	123	123	69	16	7	3	0	443

Direction sector analysed is from -30.0 to 210.0 degrees north

Records analysed from 1/ 1/1991 to 16/ 3/1992

Number of records analysed 443

Figure 2.76 Scatter diagram of H_s and water level at offshore point 2 for direction sector -30 to 210°N

Table Distribution of Hs and water level at Point 2 210 to 330N

Data shown in number of occurrences

Hs (metres)	High water level (m OD) 1.30 - 1.50	1.50 - 1.70	1.70 - 1.90	1.90 - 2.10	2.10 - 2.30	2.30 - 2.50	2.50 - 2.70	2.70 - 2.90	Calms/ no dat	No.in row
4.50- 5.00	0	0	0	0	0	0	0	0	0	0
4.00- 4.50	0	0	0	0	0	1	0	0	0	1
3.50- 4.00	0	1	1	2	2	0	0	0	0	6
3.00- 3.50	0	2	3	5	3	2	0	0	0	15
2.50- 3.00	1	3	4	0	9	4	0	0	0	21
2.00- 2.50	1	1	7	15	6	1	0	0	0	31
1.50- 2.00	0	1	10	6	7	3	1	0	0	28
1.00- 1.50	0	8	17	17	8	3	1	0	0	54
0.50- 1.00	1	9	23	25	15	4	3	0	0	80
0.00- 0.50	1	13	35	24	7	0	2	0	0	82
No data/calms	0	0	0	0	0	0	0	0	0	0
No. in column	4	38	100	94	57	18	7	0	0	318

Direction sector analysed is from 210.0 to 330.0 degrees north

Records analysed from 1/ 1/1991 to 16/ 3/1992

Number of records analysed 318

Figure 2.77 Scatter diagram of H_s and water level at offshore point 2 for direction sector 210 to 330°N

Table Distribution of wave height and water level at Point 2

Data shown in number of occurrences

Hs (metres)	High water level (m OD) 1.30 - 1.50	1.50 - 1.70	1.70 - 1.90	1.90 - 2.10	2.10 - 2.30	2.30 - 2.50	2.50 - 2.70	2.70 - 2.90	Calms/ no dat	No. in row
4.50- 5.00	0	0	0	0	0	0	0	0	0	0
4.00- 4.50	0	0	0	0	0	1	0	0	0	1
3.50- 4.00	0	1	1	2	2	0	0	0	0	6
3.00- 3.50	0	2	3	5	4	2	0	0	0	16
2.50- 3.00	1	3	4	3	11	4	0	1	0	27
2.00- 2.50	1	1	7	20	9	2	0	1	0	41
1.50- 2.00	0	2	16	11	9	4	1	0	0	43
1.00- 1.50	0	22	26	41	14	5	3	0	0	111
0.50- 1.00	5	43	63	52	45	12	4	0	0	224
0.00- 0.50	14	50	103	86	33	4	6	1	0	297
No data/calms	0	0	0	0	0	0	0	0	0	0
No. in column	21	124	223	220	127	34	14	3	0	766

Records analysed from 1/ 1/1991 to 16/ 3/1992

Number of records analysed 766

Figure 2.78 Scatter diagram of H_s and water level at offshore point 2 for all sectors

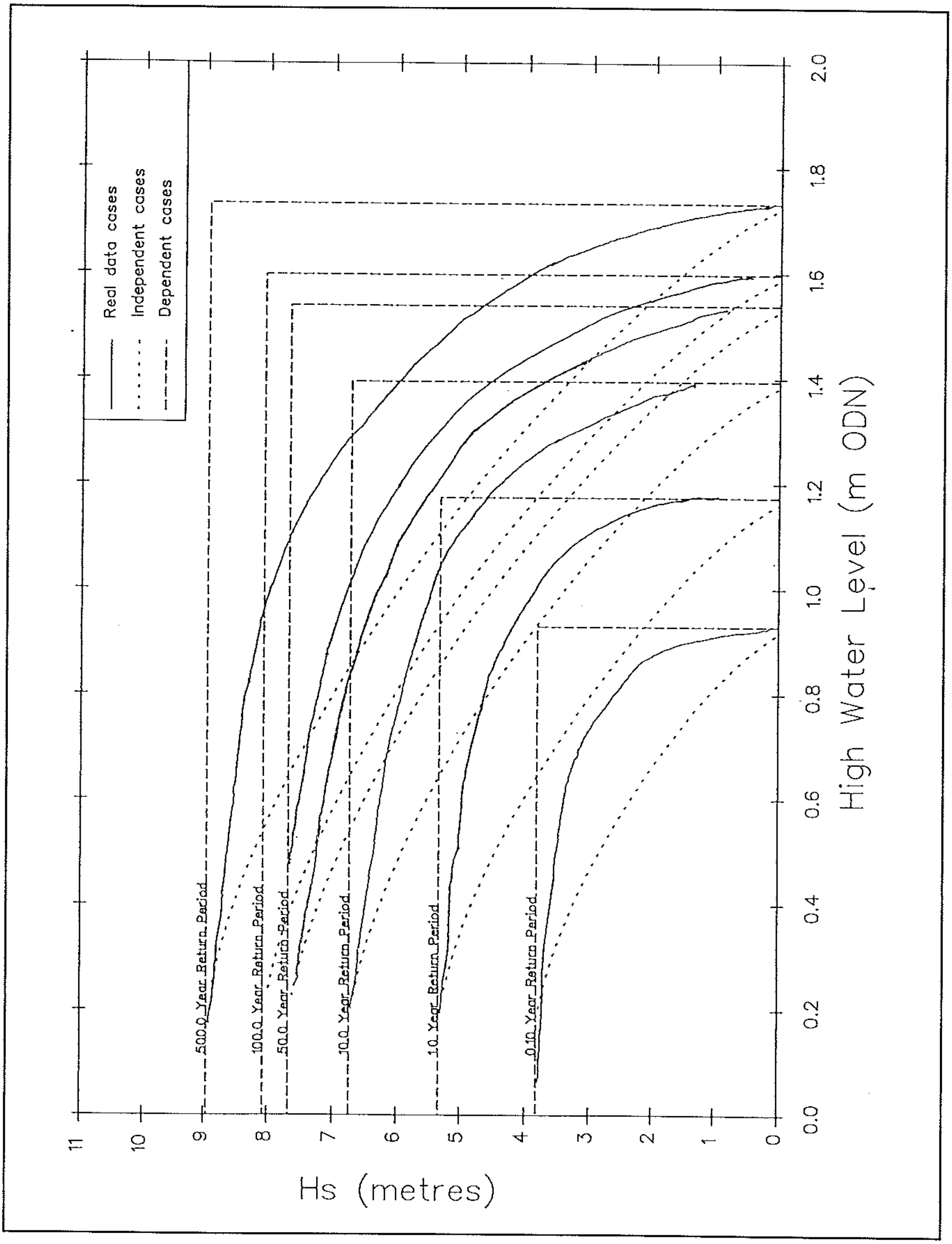


Figure 2.79 Extreme combinations of H_s and water level off Christchurch Harbour

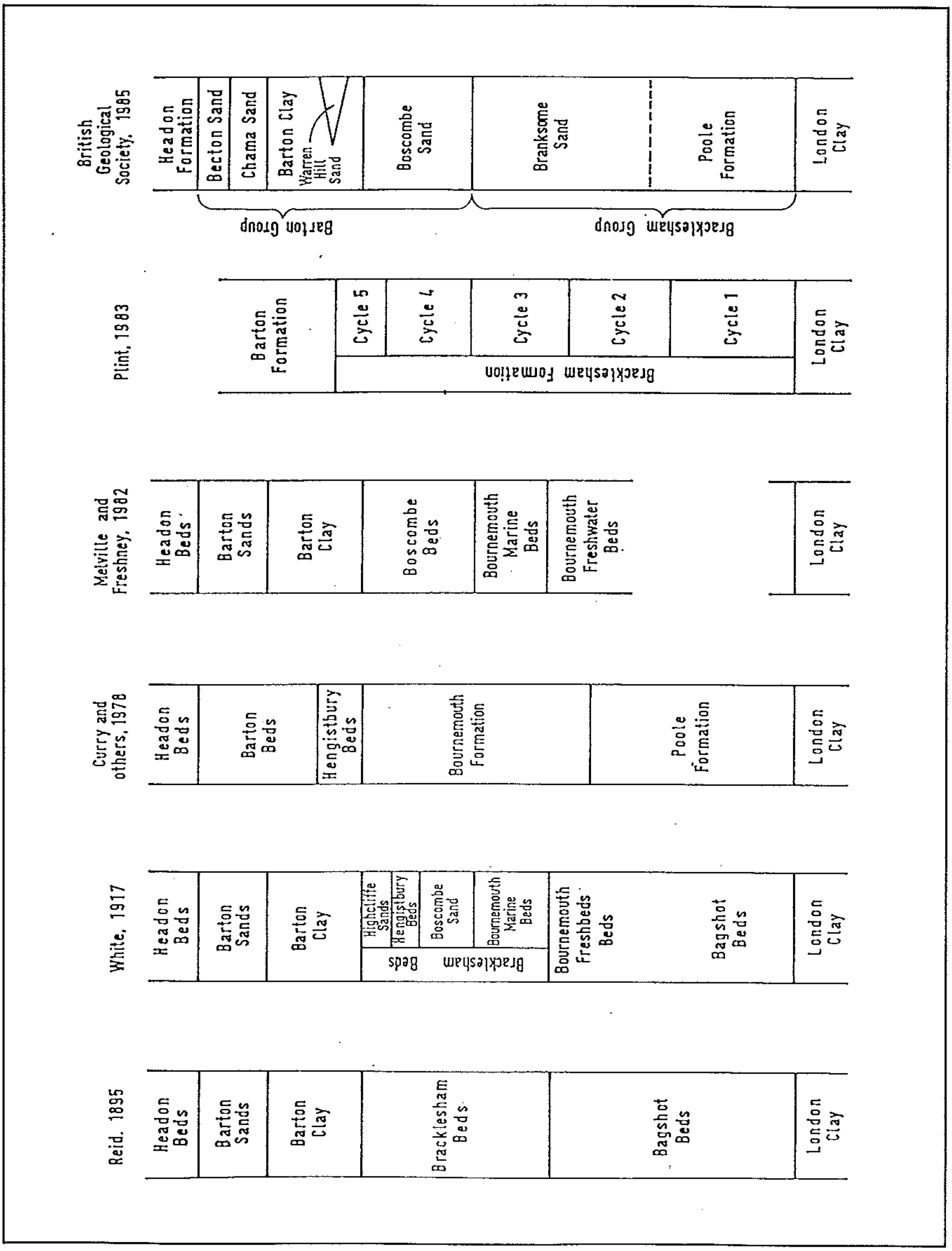


Figure 3.1 Evolution of the nomenclature of the eocene rocks of the Bournemouth district (from Golder Associates, 1990)

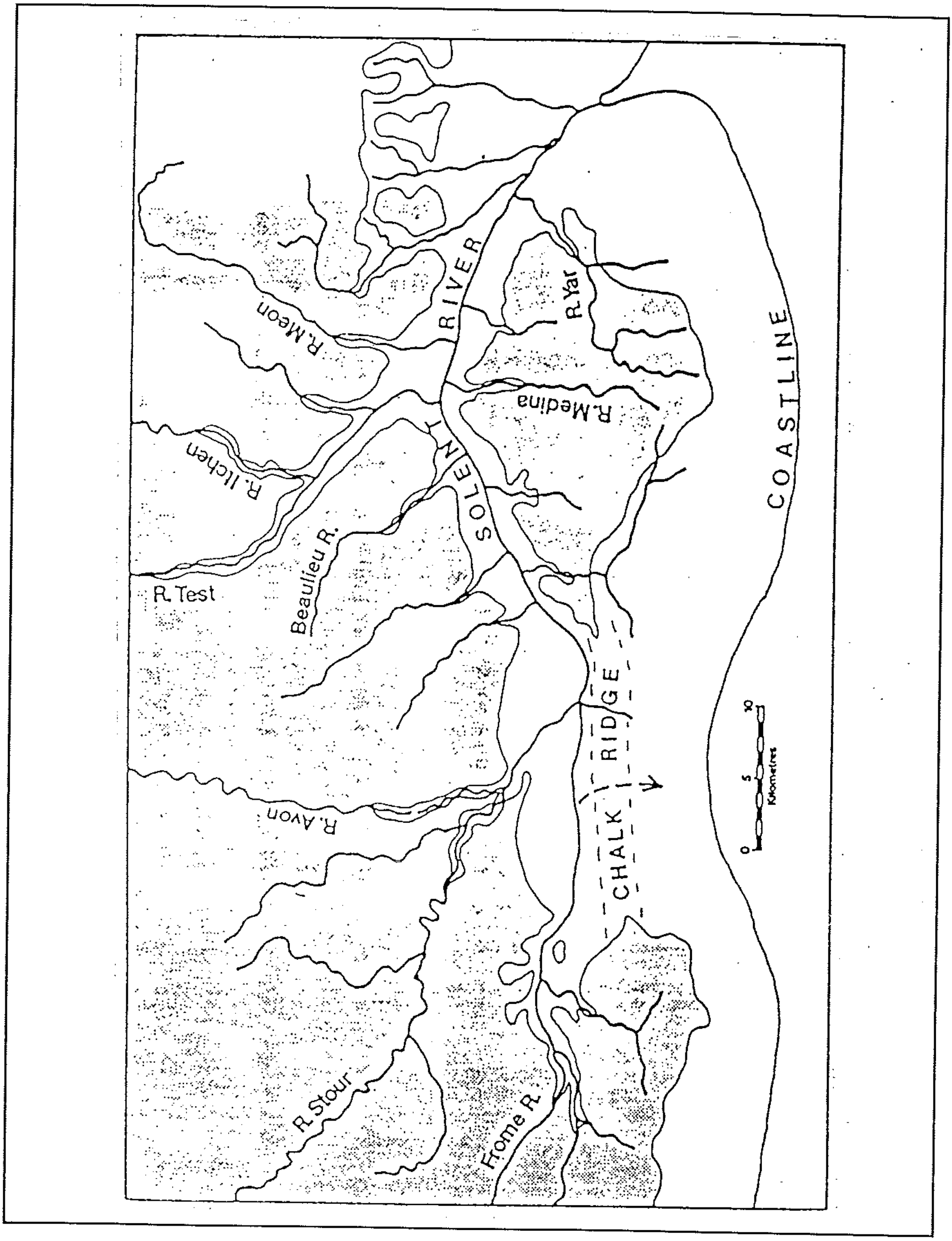


Figure 3.2 The Solent River and coastline of a million years ago

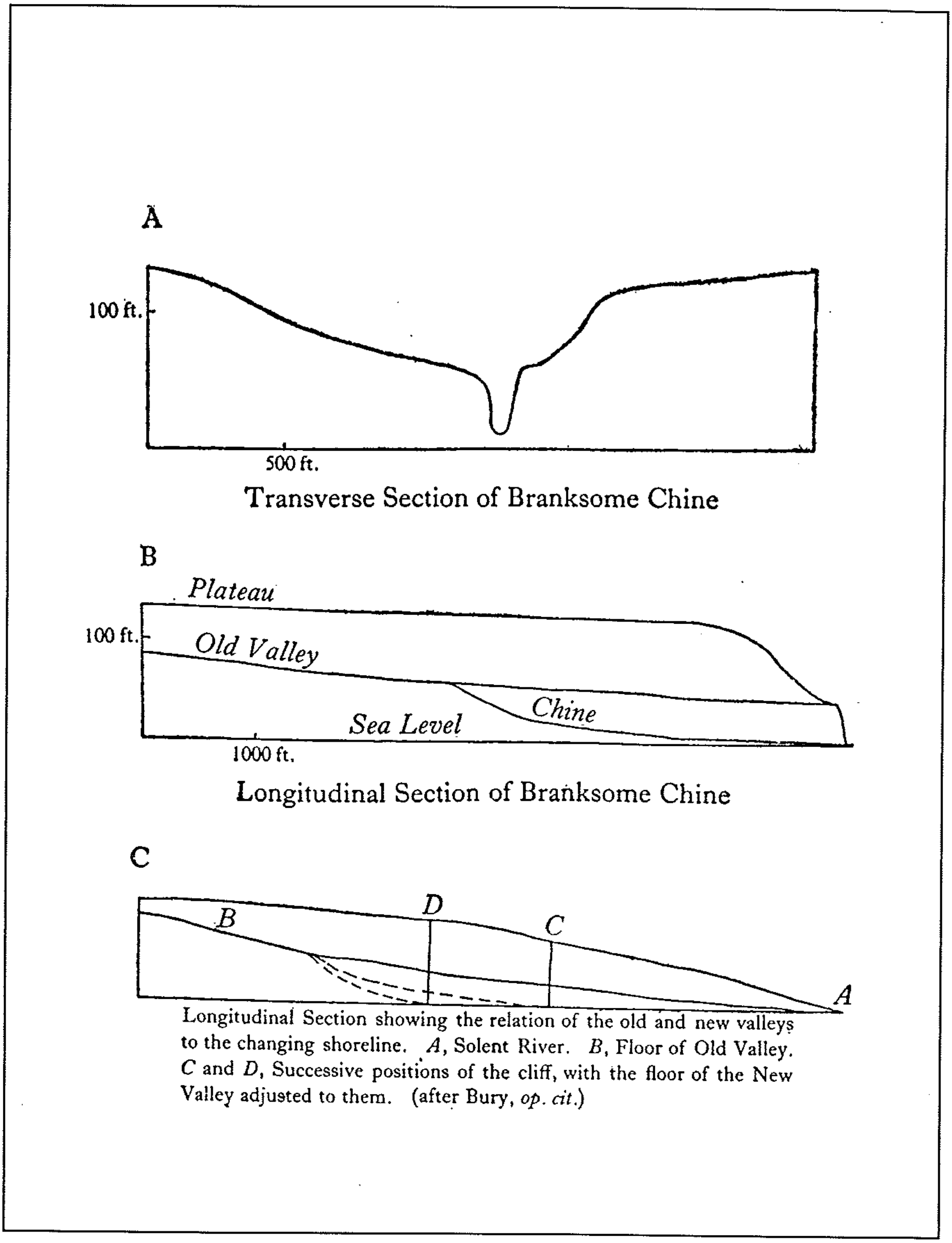


Figure 3.3 Transverse and longitudinal sections of Branksome Chine (from Steers, 1969)

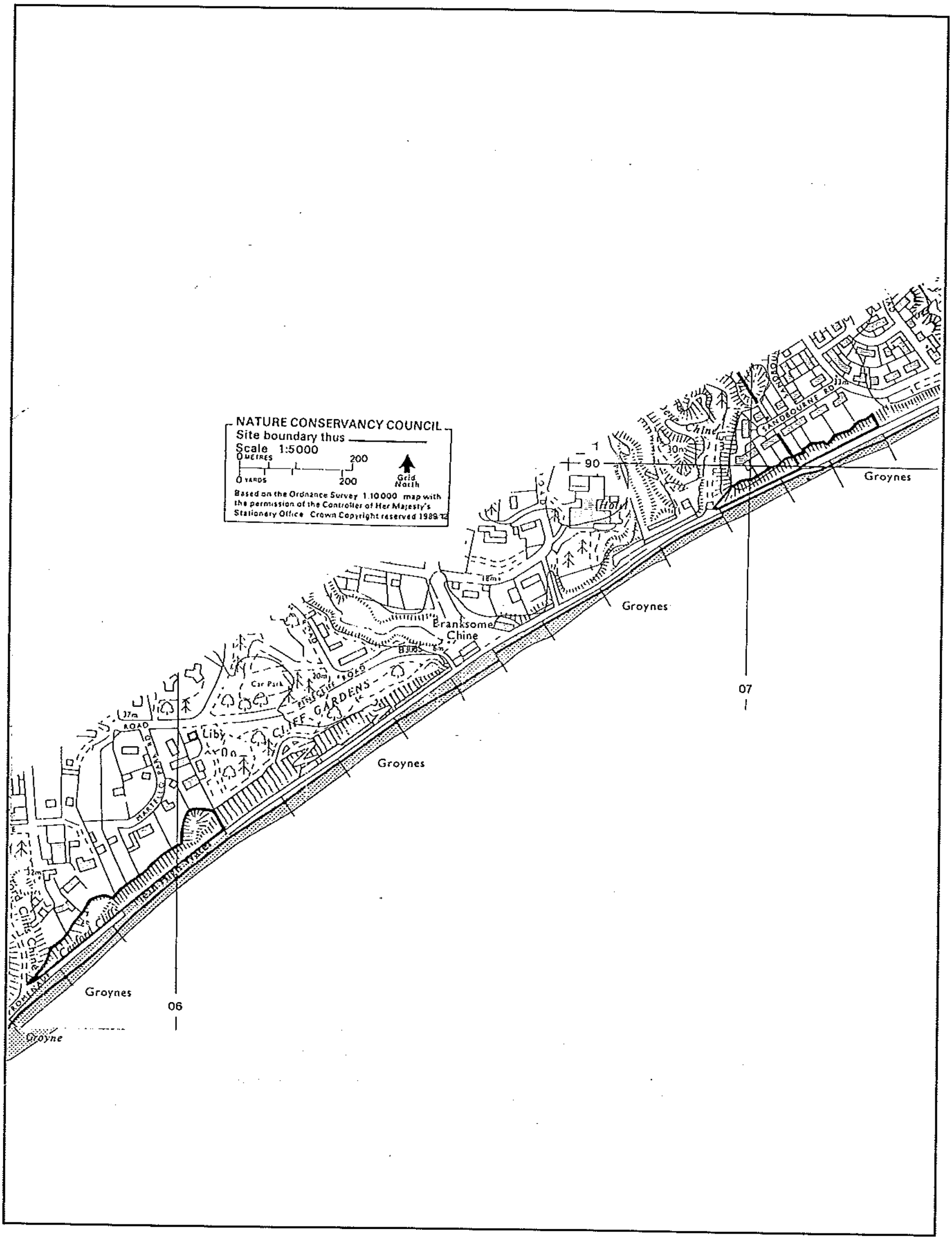


Figure 3.4 Geological SSSI's on PBC's Poole Bay frontage

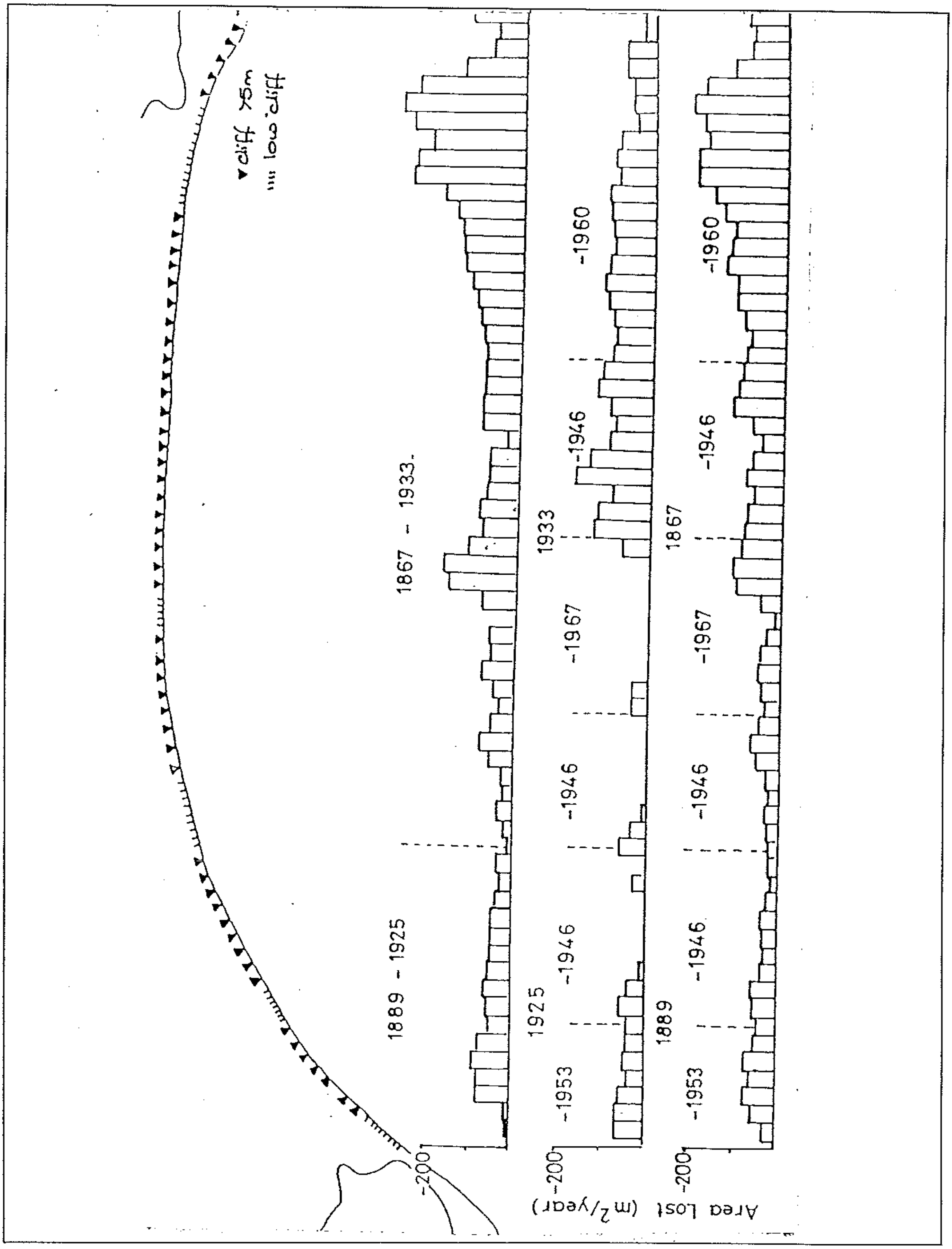


Figure 3.5 **Cliff top recession over the past century in Poole Bay (from Lacey, 1985)**

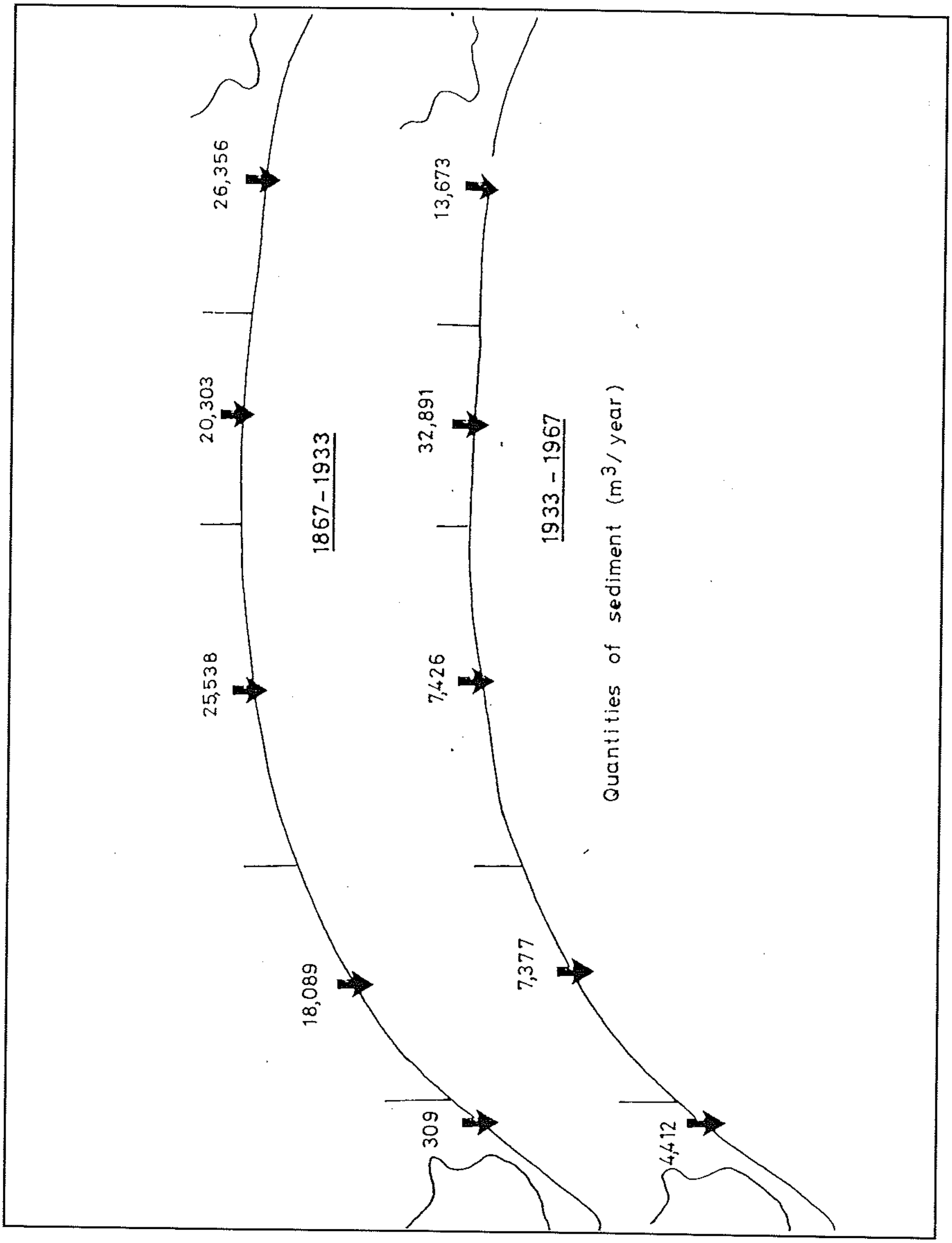


Figure 3.6 Contribution of sand and gravel to Poole Bay beaches (from Lacey, 1985)

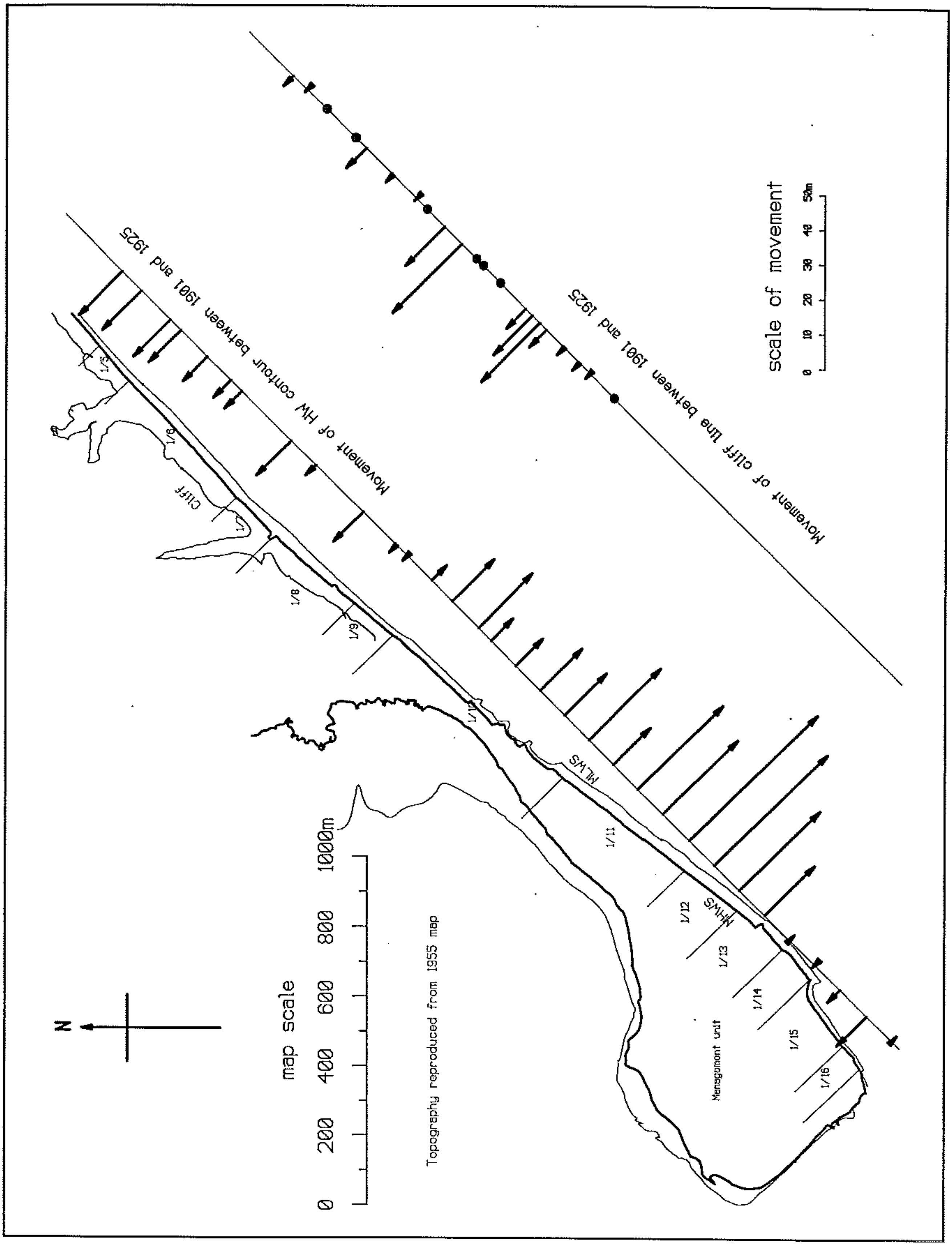


Figure 3.7 Movement of High Water Mark and cliff top between 1901 and 1925

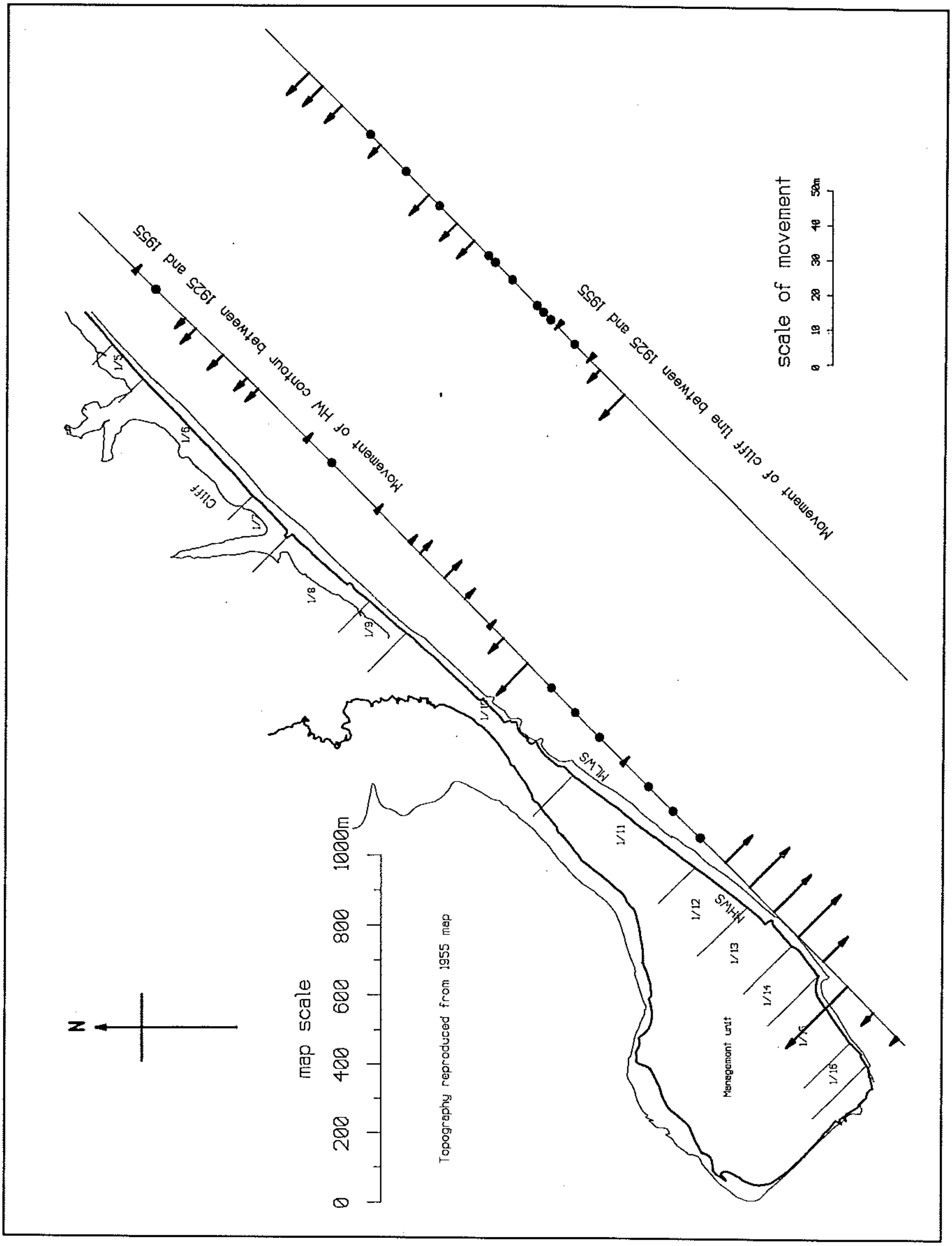


Figure 3.8 Movement of High Water Mark and cliff top between 1925 and 1955

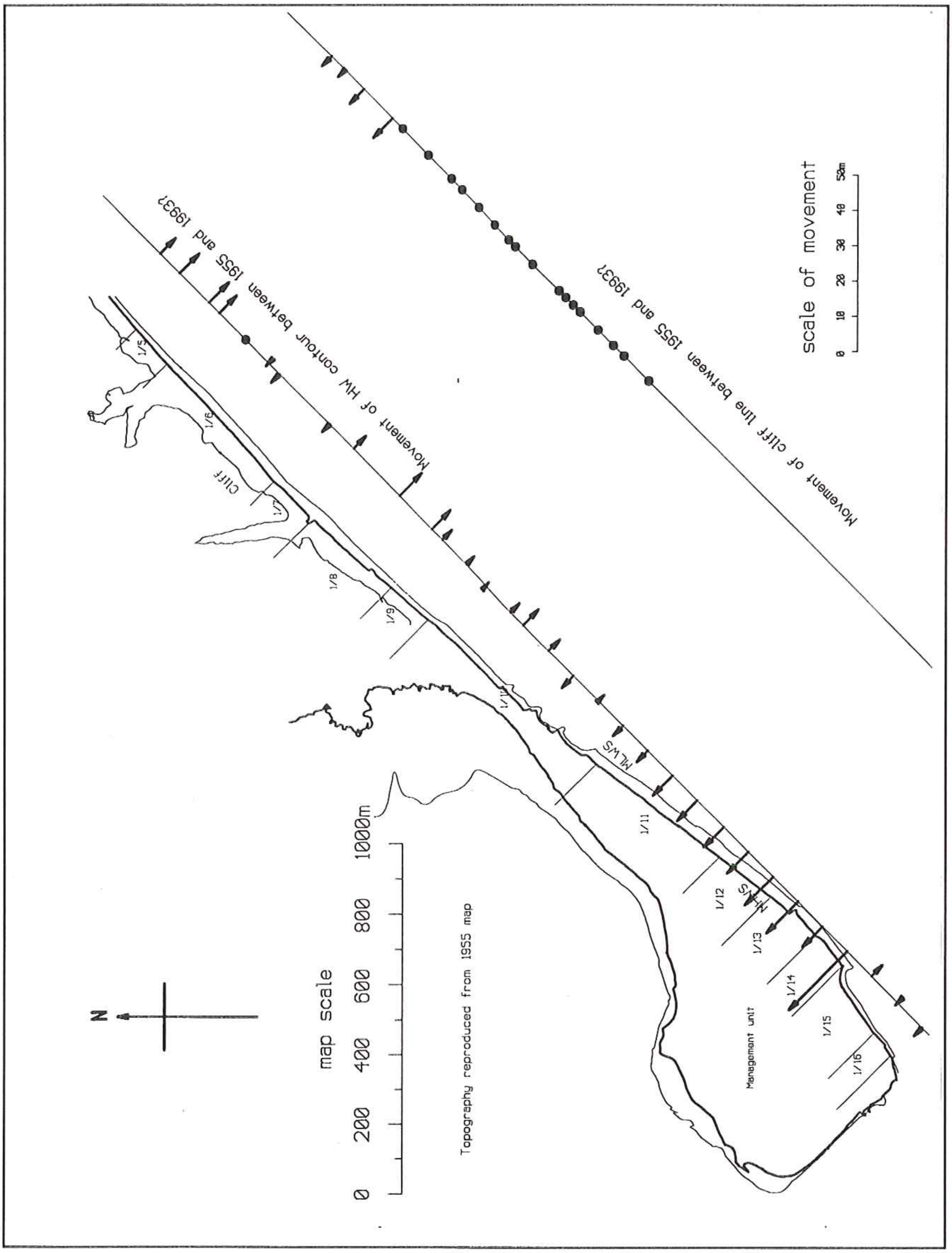


Figure 3.9 Movement of High Water Mark and cliff top between 1955 and 1993

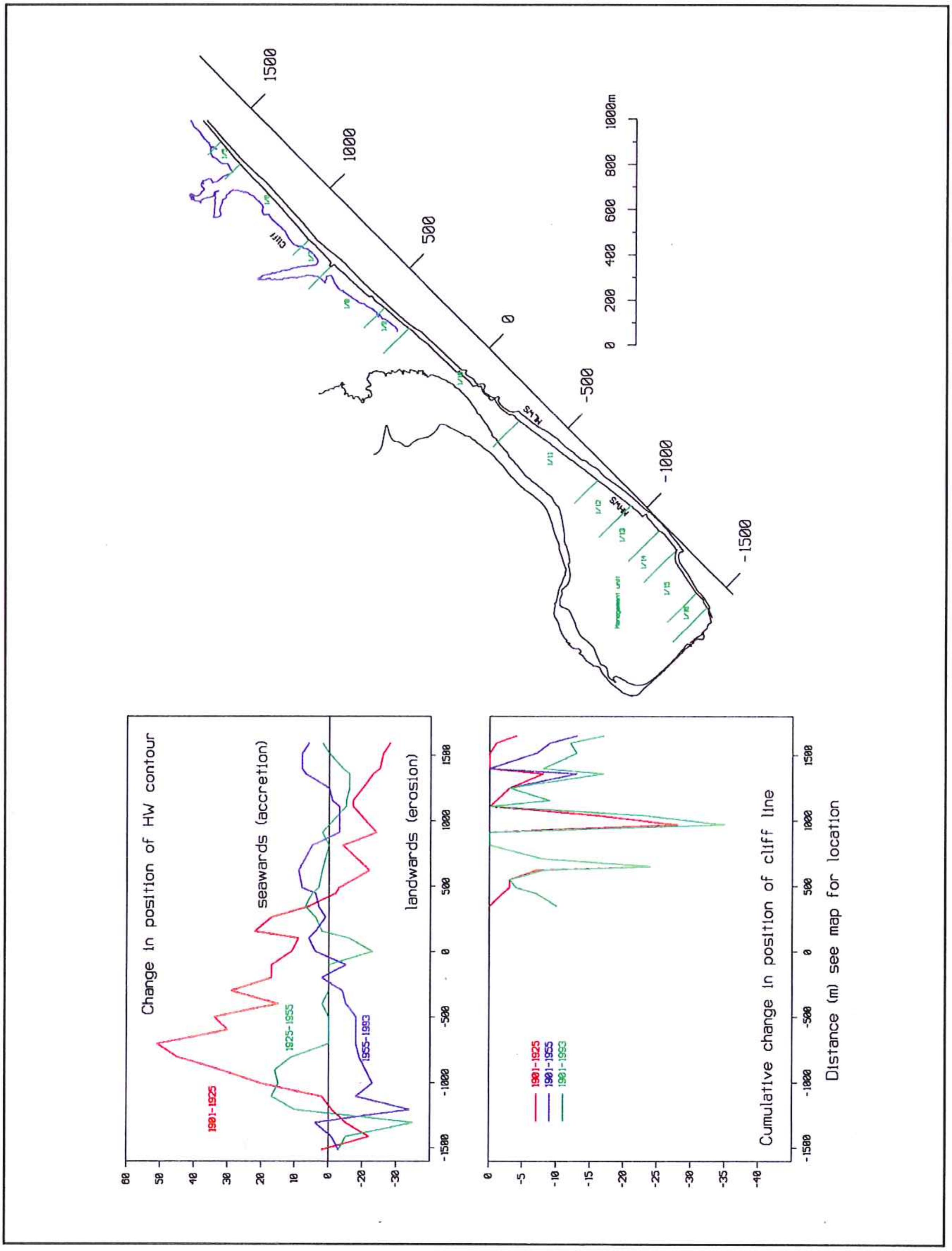


Figure 3.10 **Summary of cumulative movement of High Water Mark and cliff top between 1901 and 1993**

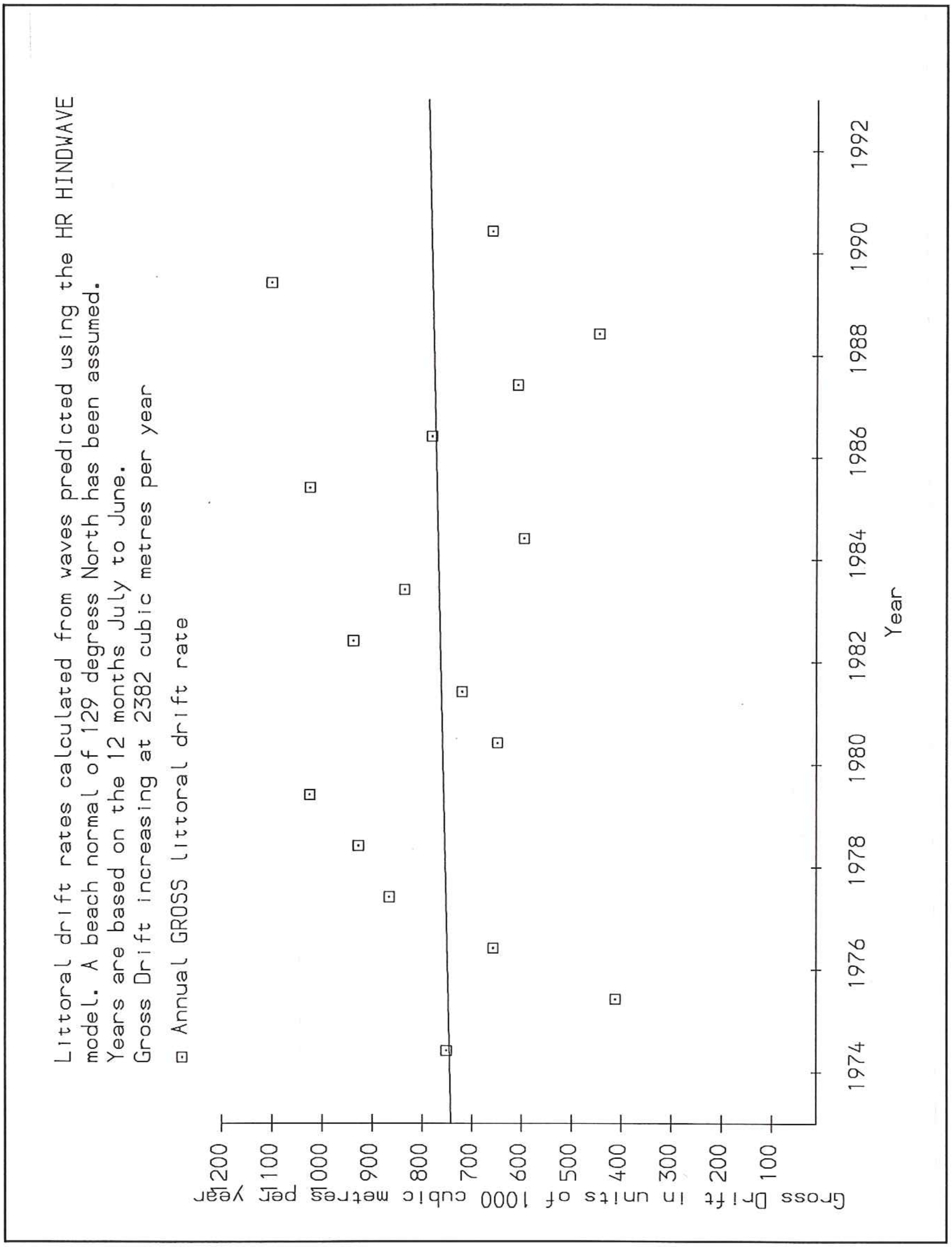


Figure 3.11 Annual gross potential drift rates 1974 to 1990 Point B

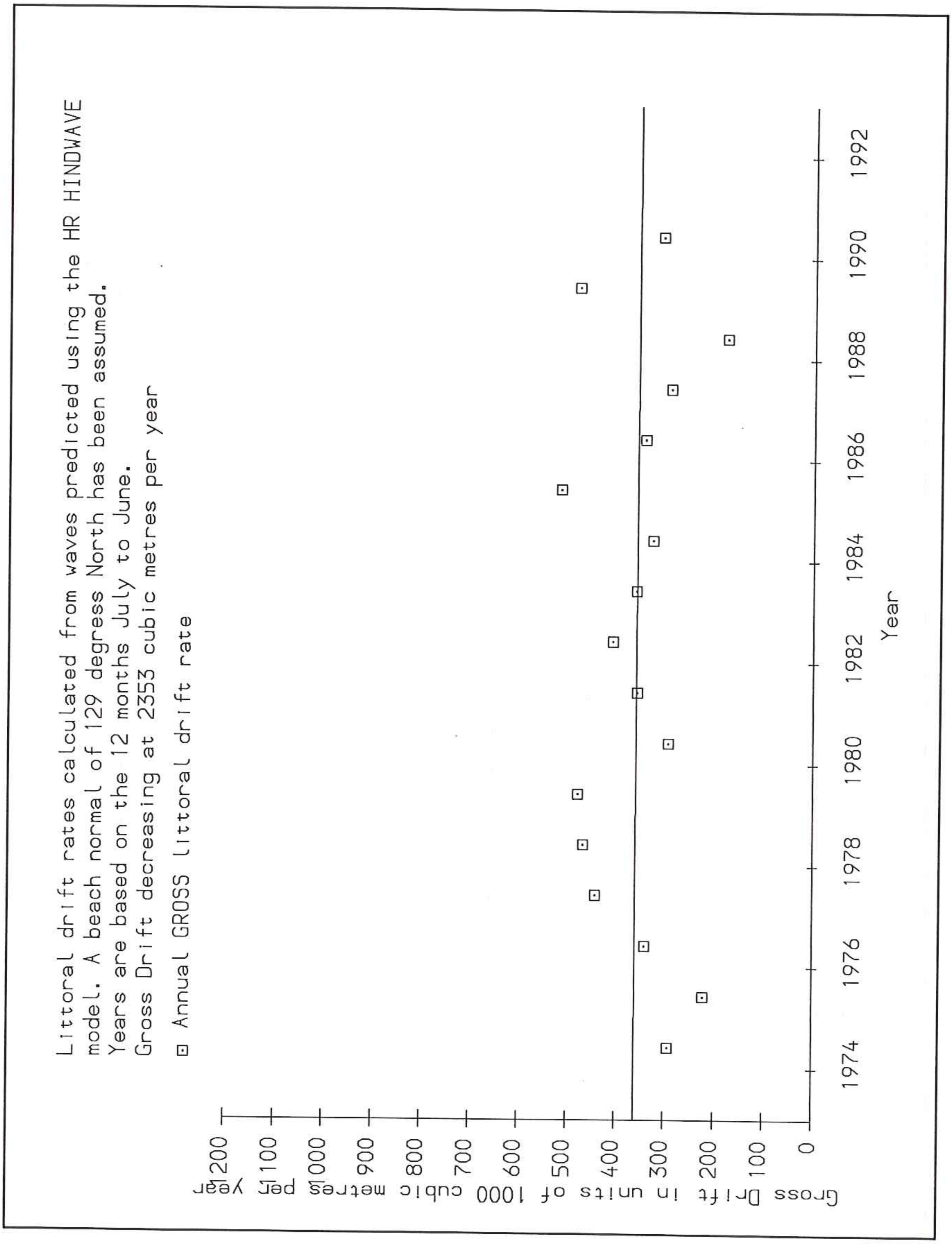


Figure 3.12 Annual gross potential drift rates 1974 to 1990 Point C

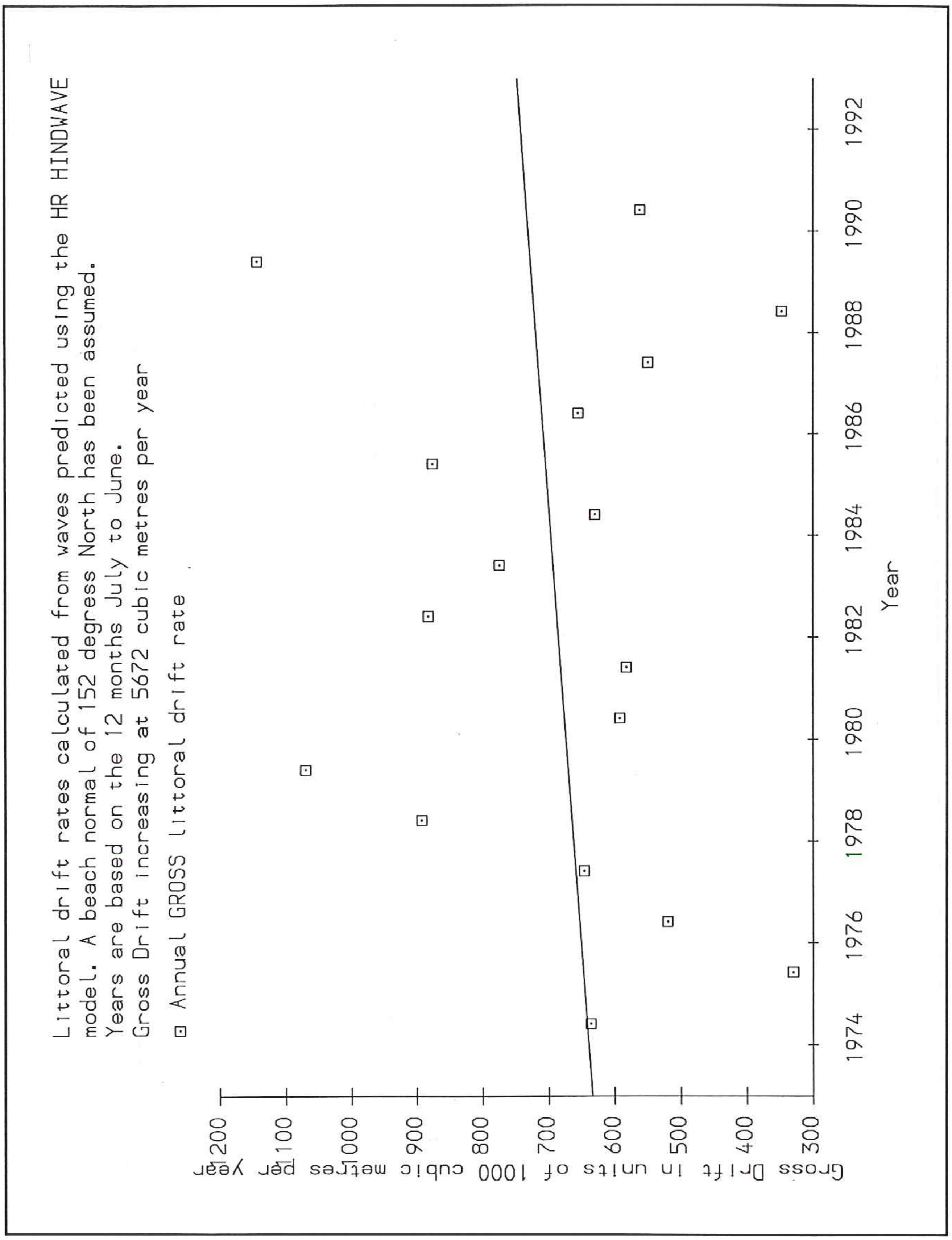


Figure 3.13 **Annual gross potential drift rates 1974 to 1990 Point D**

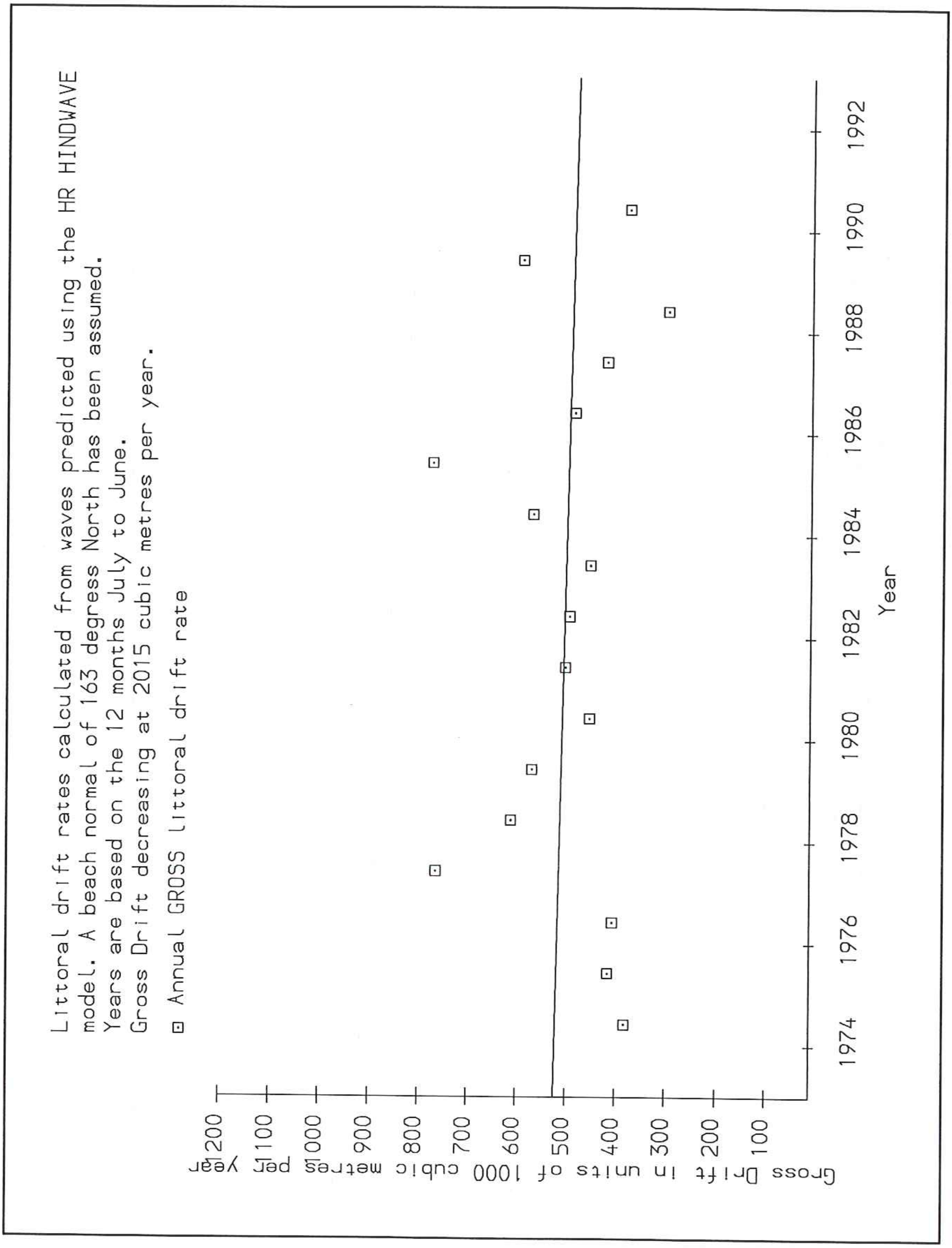


Figure 3.14 **Annual gross potential drift rates 1974 to 1990 Point E**

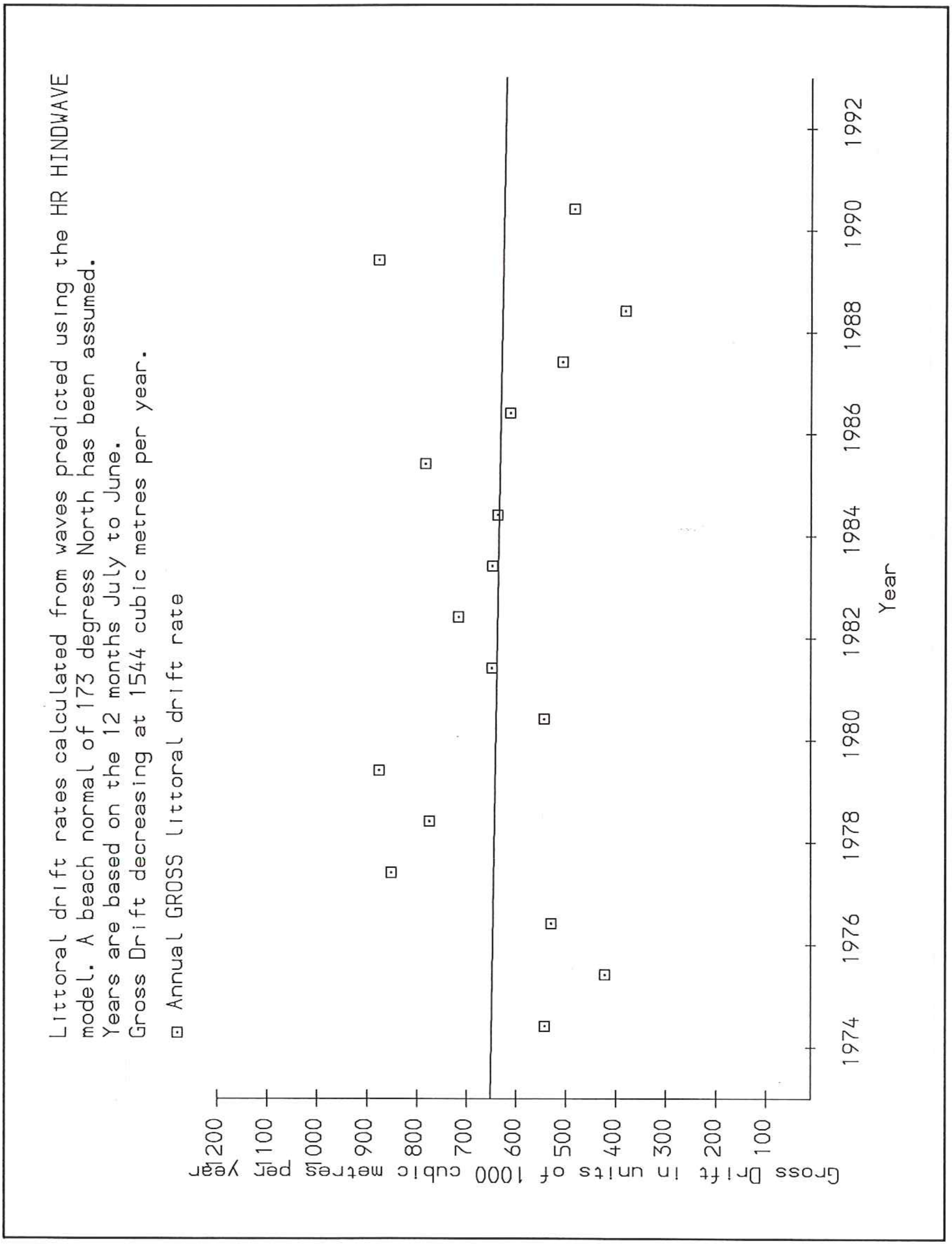


Figure 3.15 **Annual gross potential drift rates 1974 to 1990 Point F**

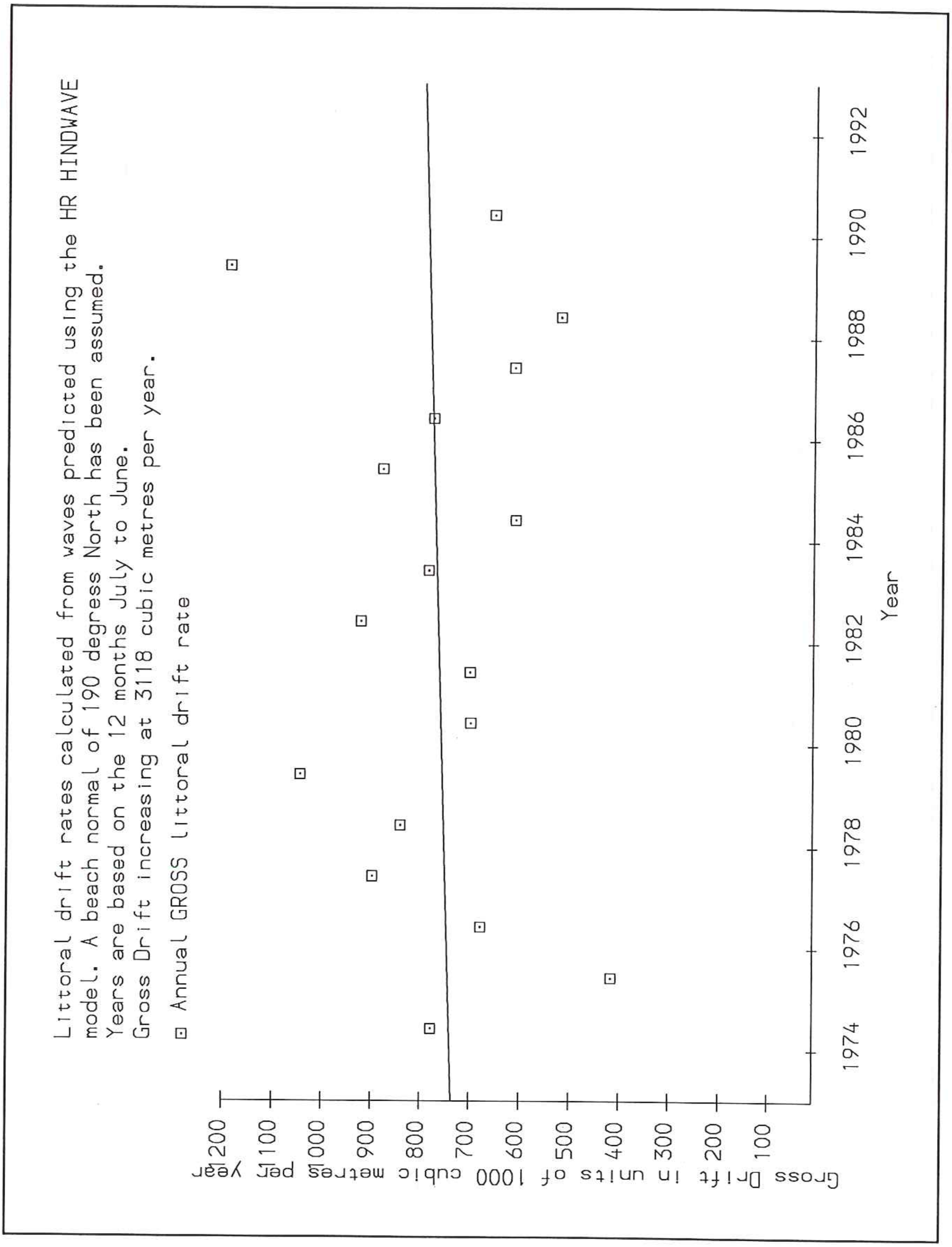


Figure 3.16 **Annual gross potential drift rates 1974 to 1990 Point G**

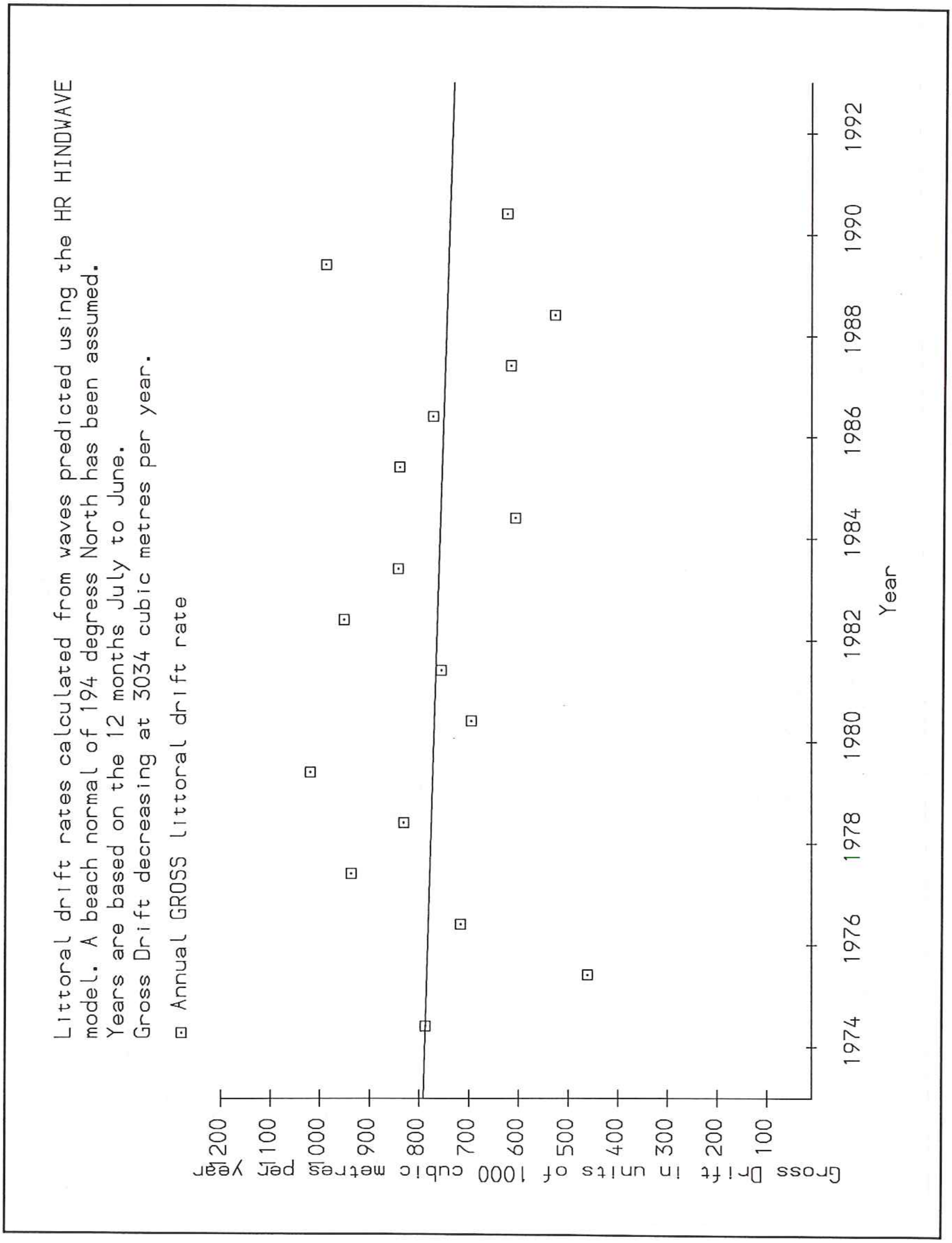


Figure 3.17 **Annual gross potential drift rates 1974 to 1990 Point H**

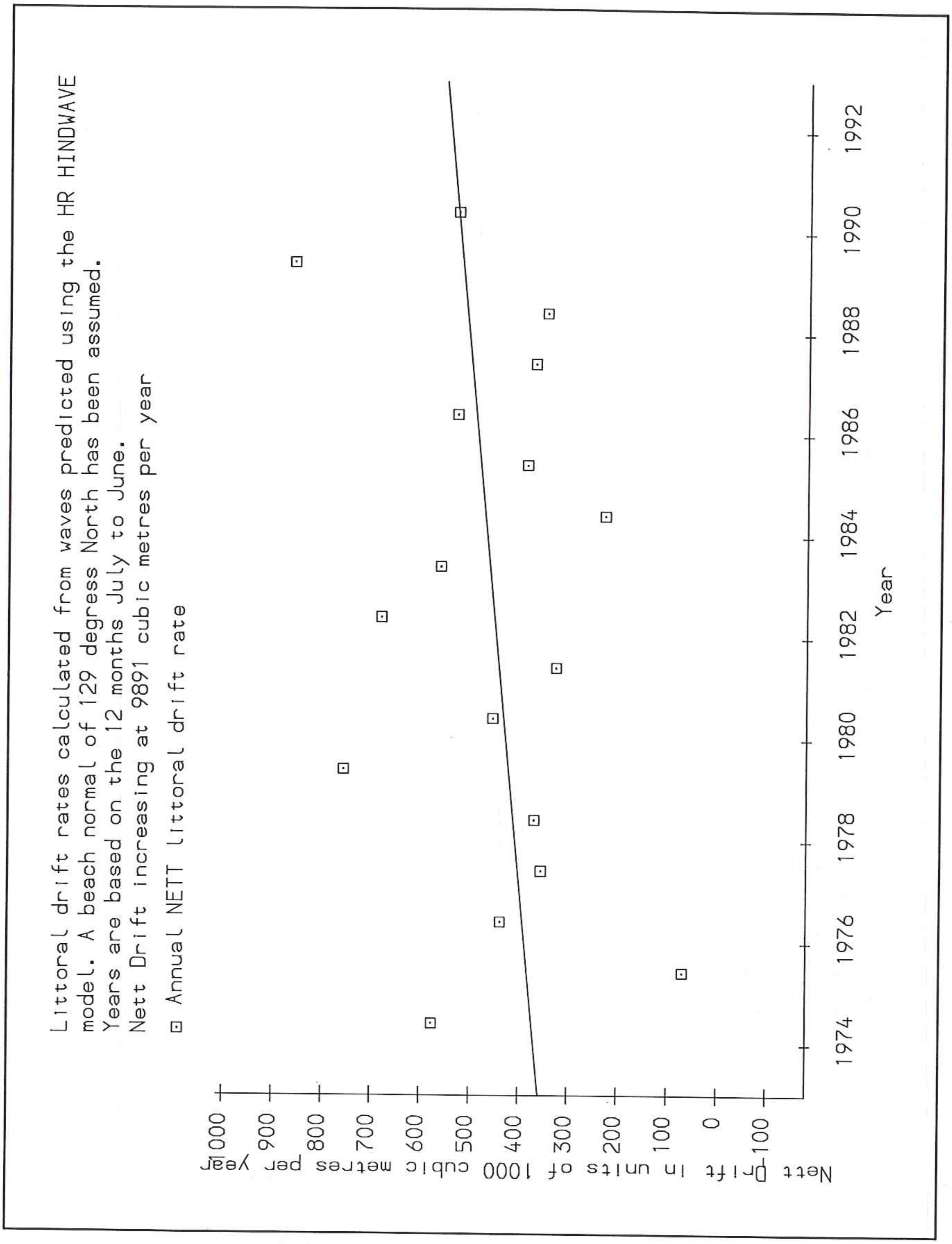


Figure 3.18 **Annual nett potential drift rates 1974 to 1990 Point B**

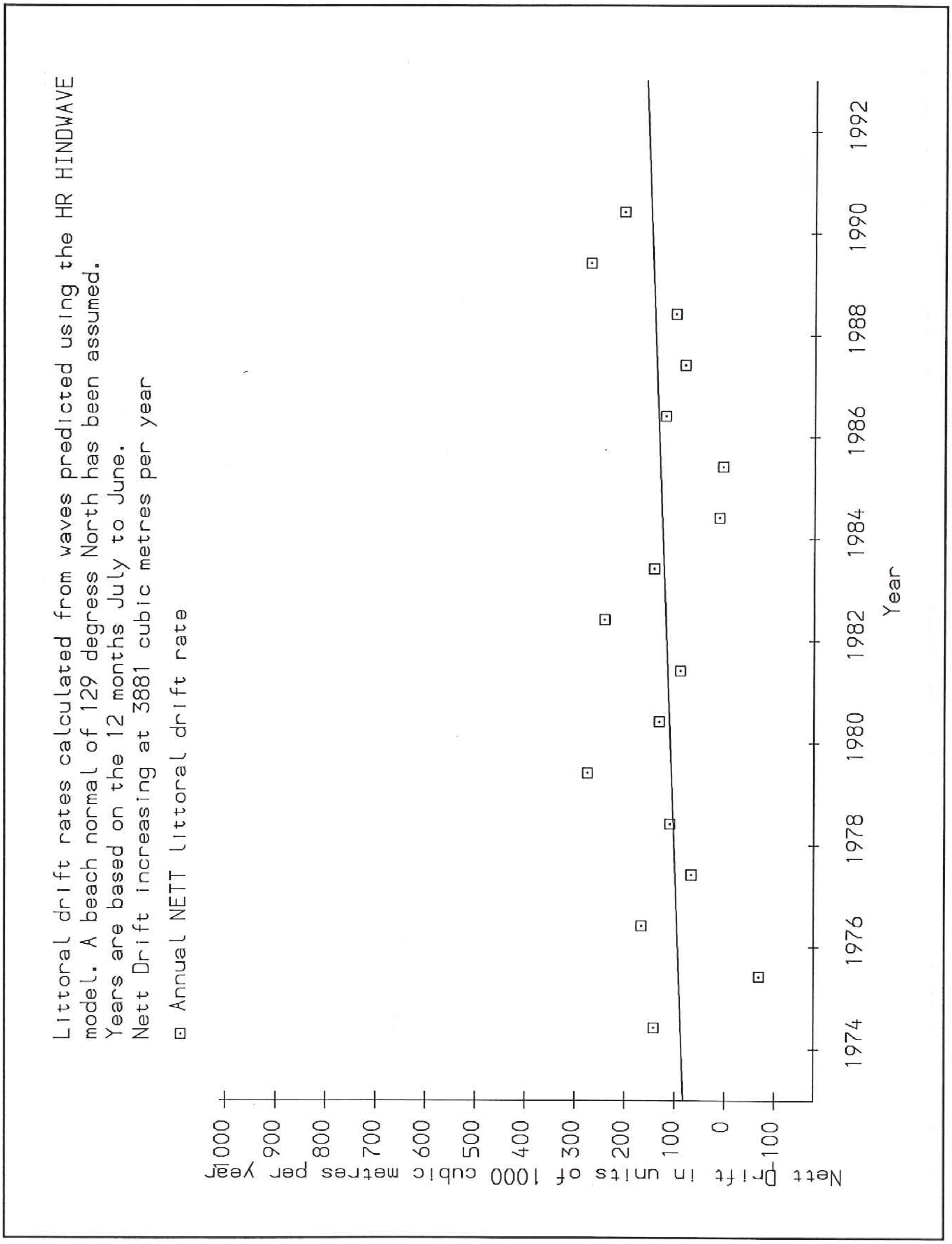


Figure 3.19 Annual nett potential drift rates 1974 to 1990 Point C

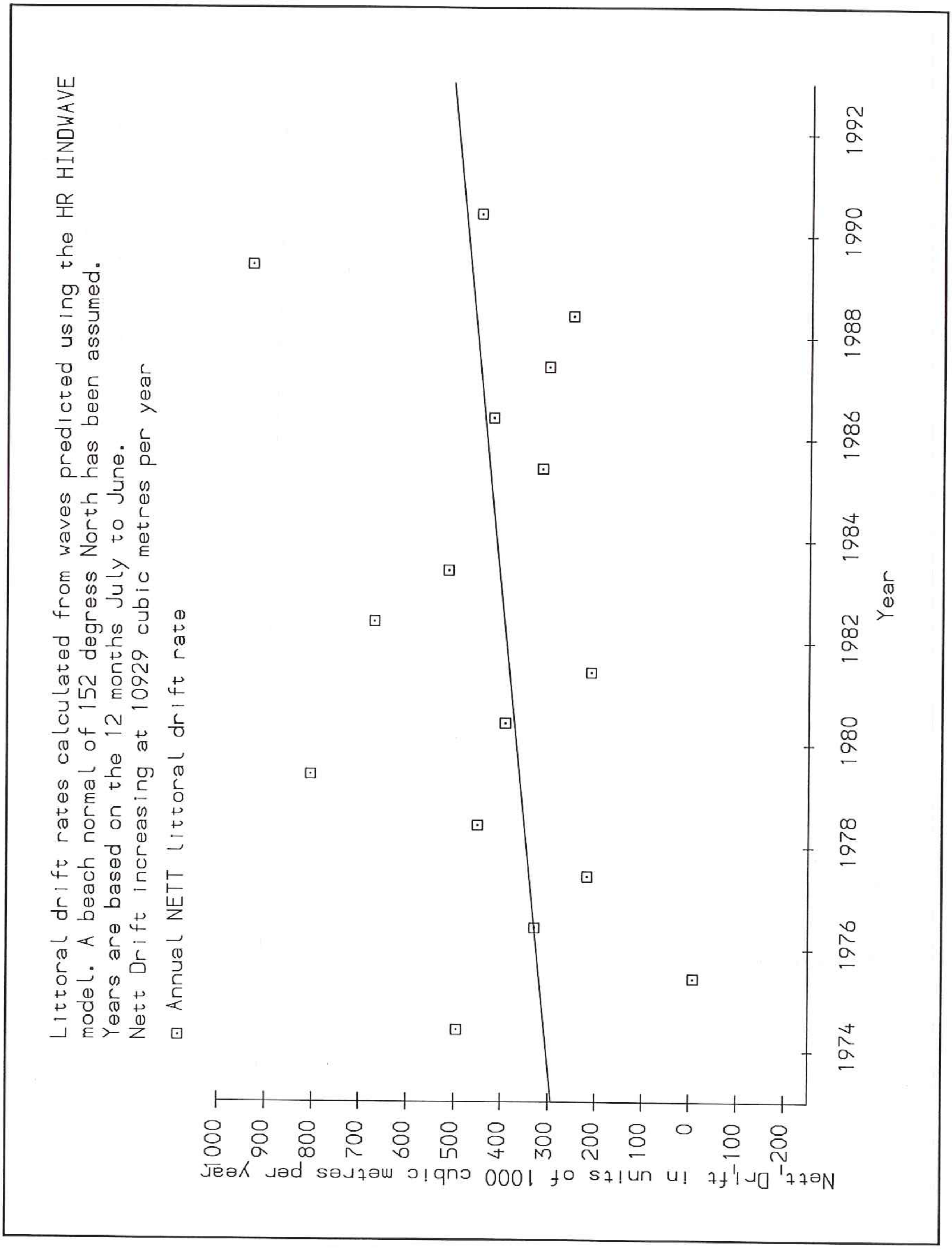


Figure 3.20 **Annual nett potential drift rates 1974 to 1990 Point D**

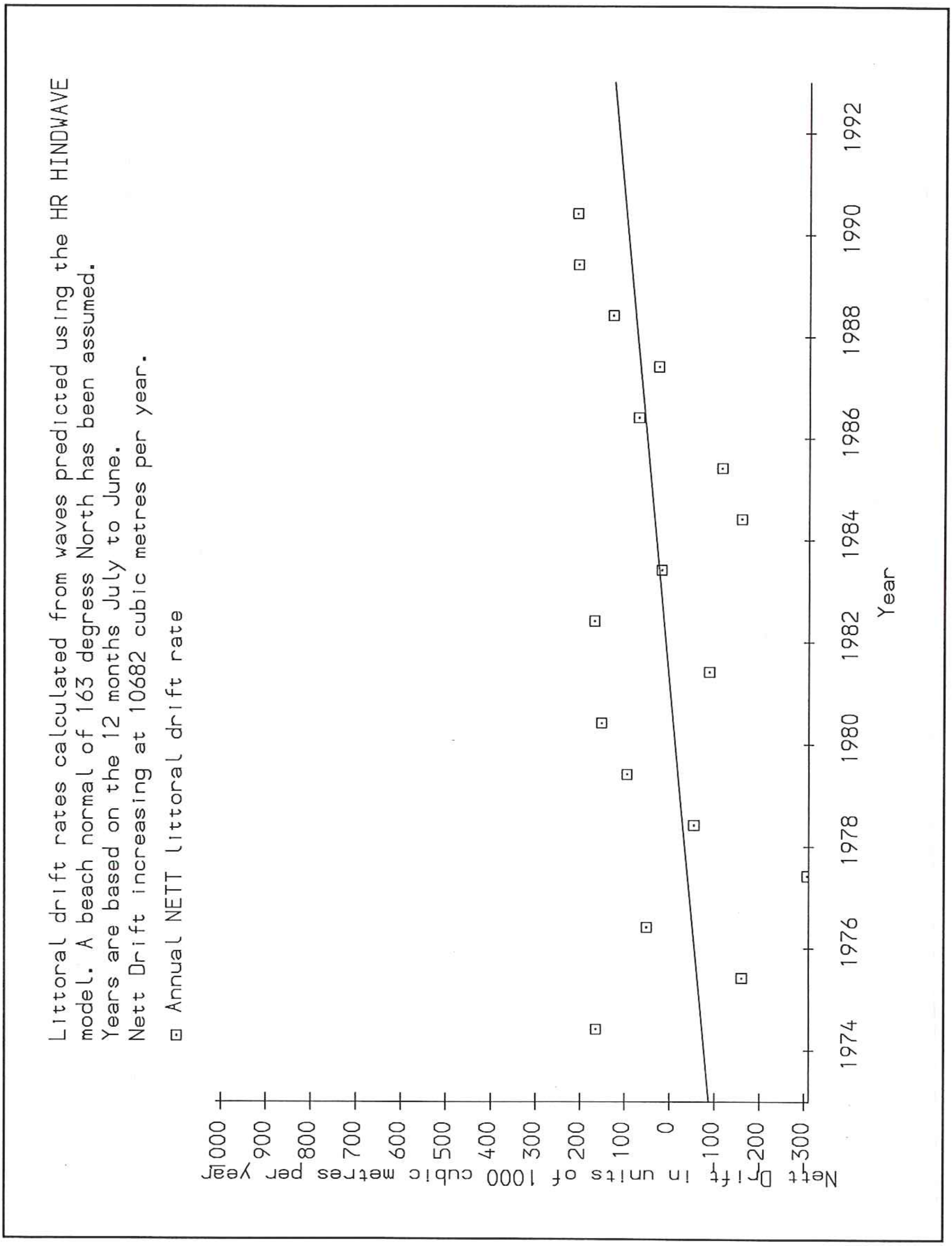


Figure 3.21 **Annual nett potential drift rates 1974 to 1990 Point E**

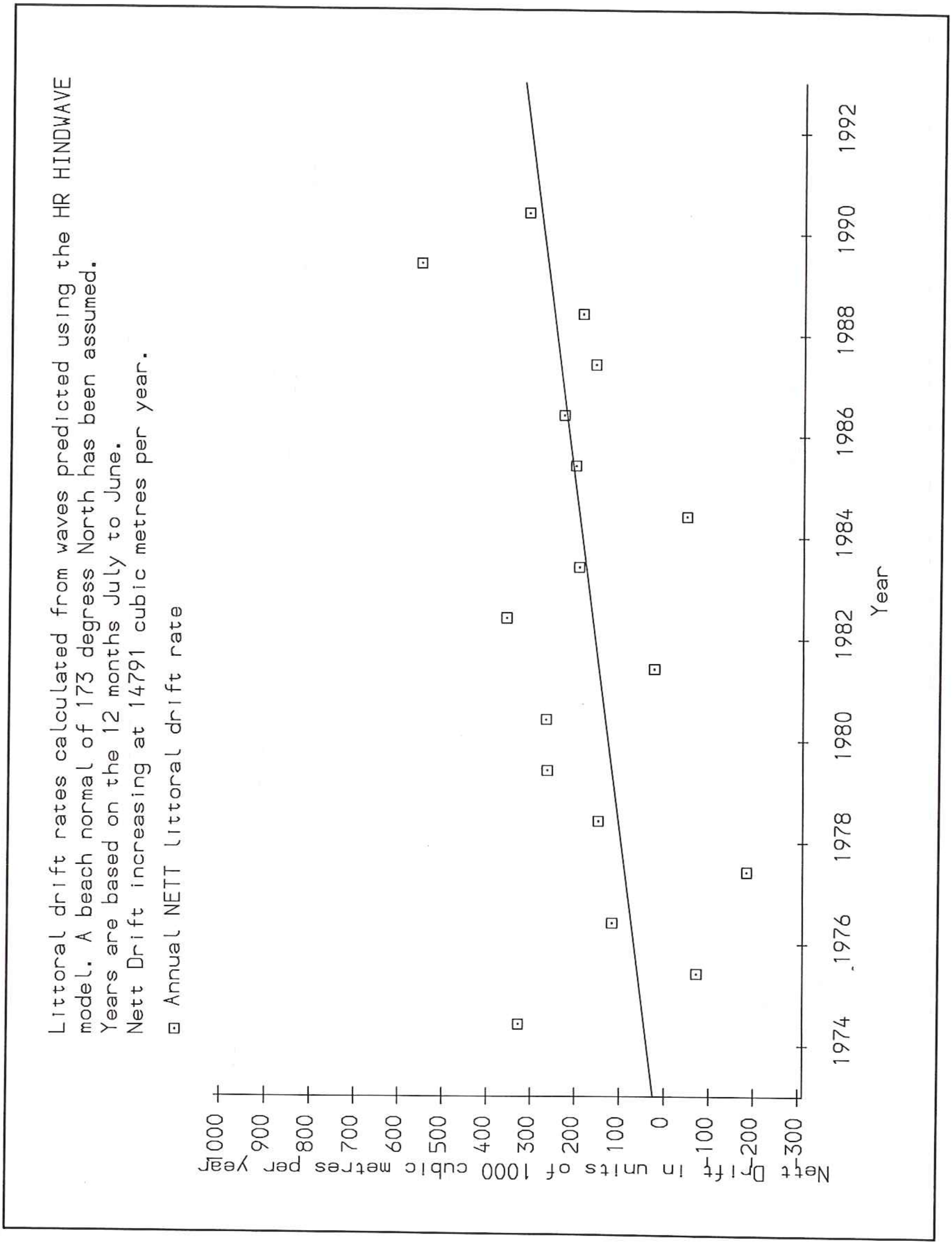


Figure 3.22 **Annual nett potential drift rates 1974 to 1990 Point F**

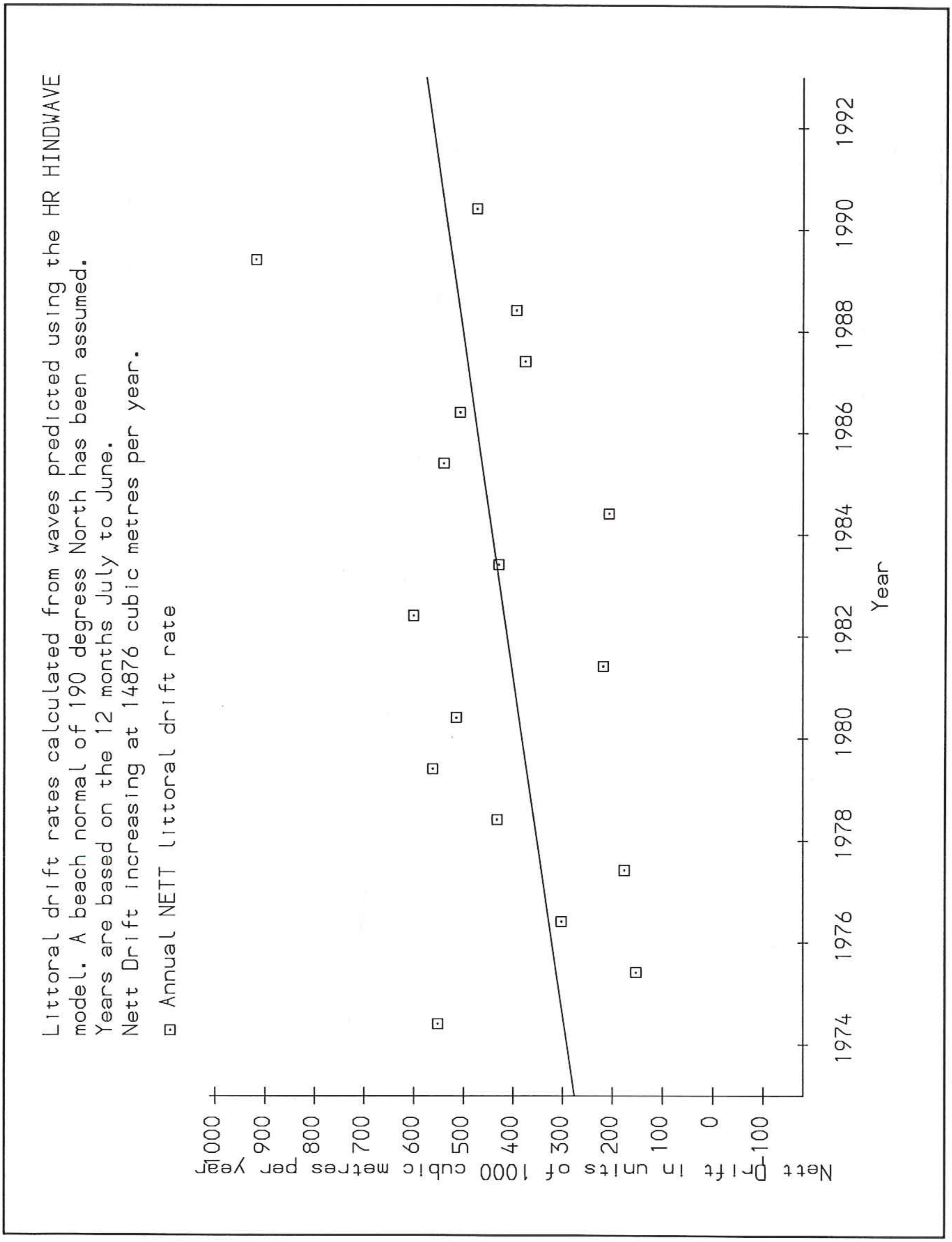


Figure 3.23 **Annual nett potential drift rates 1974 to 1990 Point G**

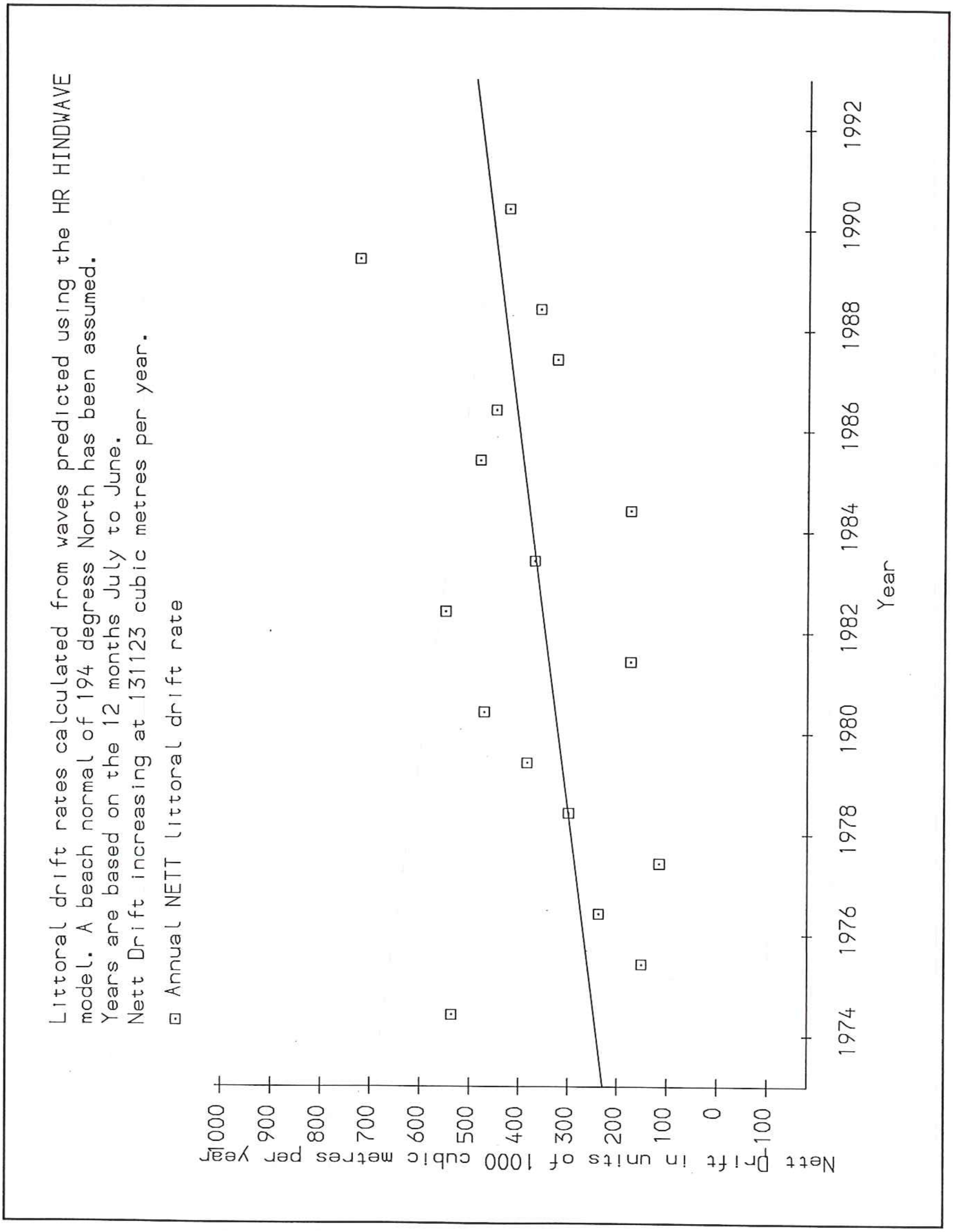


Figure 3.24 Annual nett potential drift rates 1974 to 1990 Point H

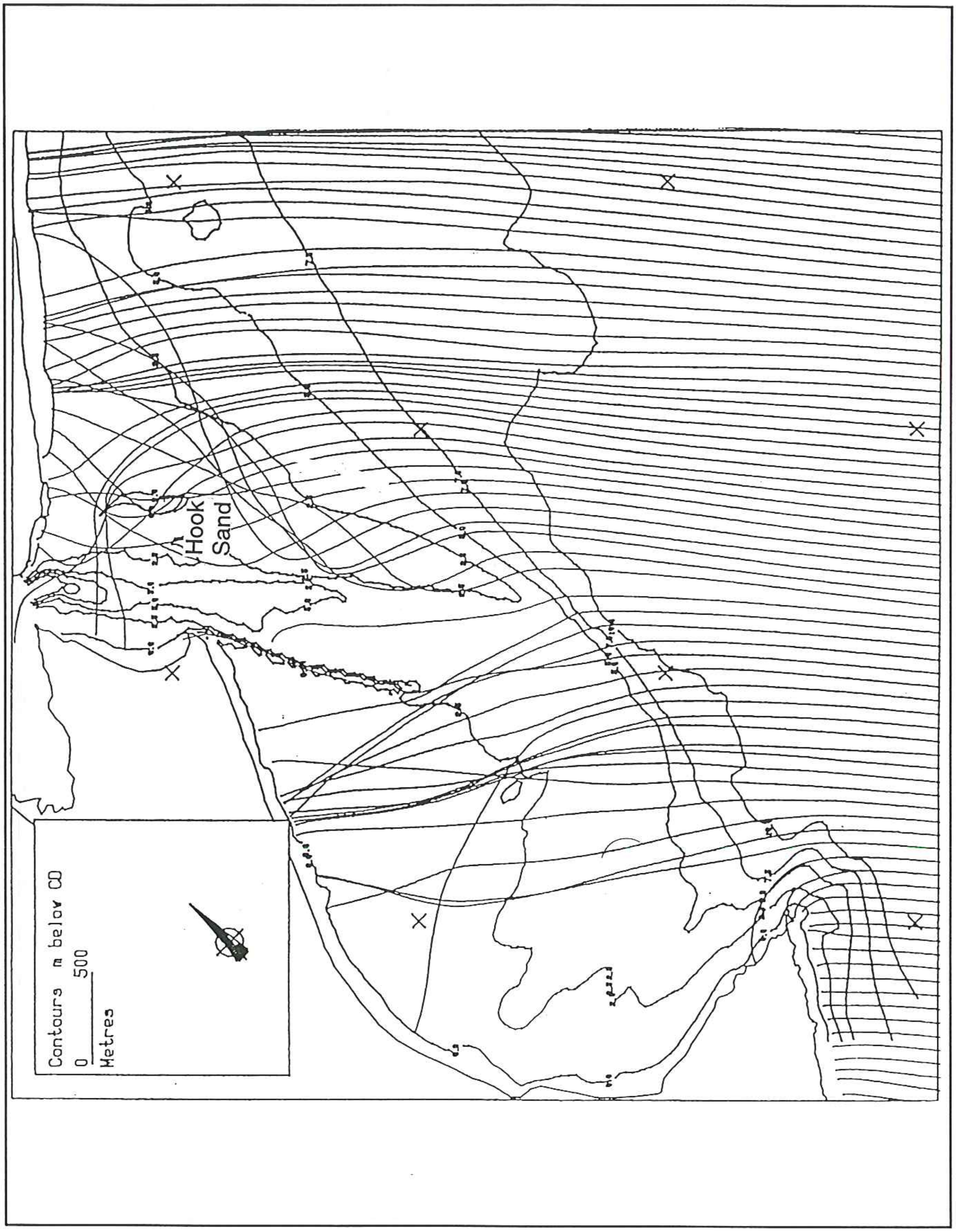


Figure 3.25 **Ray patterns for waves from 140° showing reflection off Hook Sand (from EX 2228, 1991)**

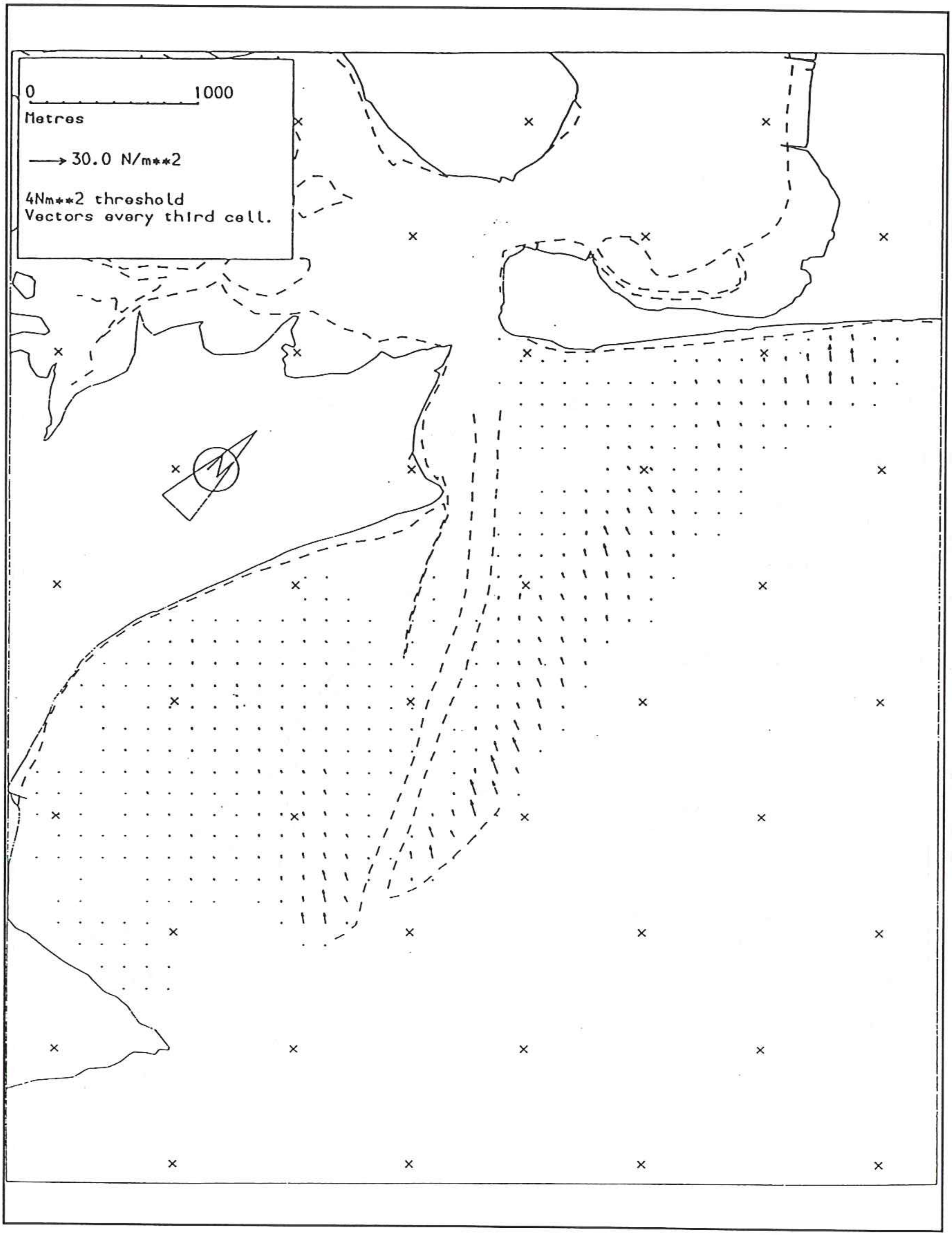


Figure 3.26 Storm radiation stress forces (from EX 2228, 1991)

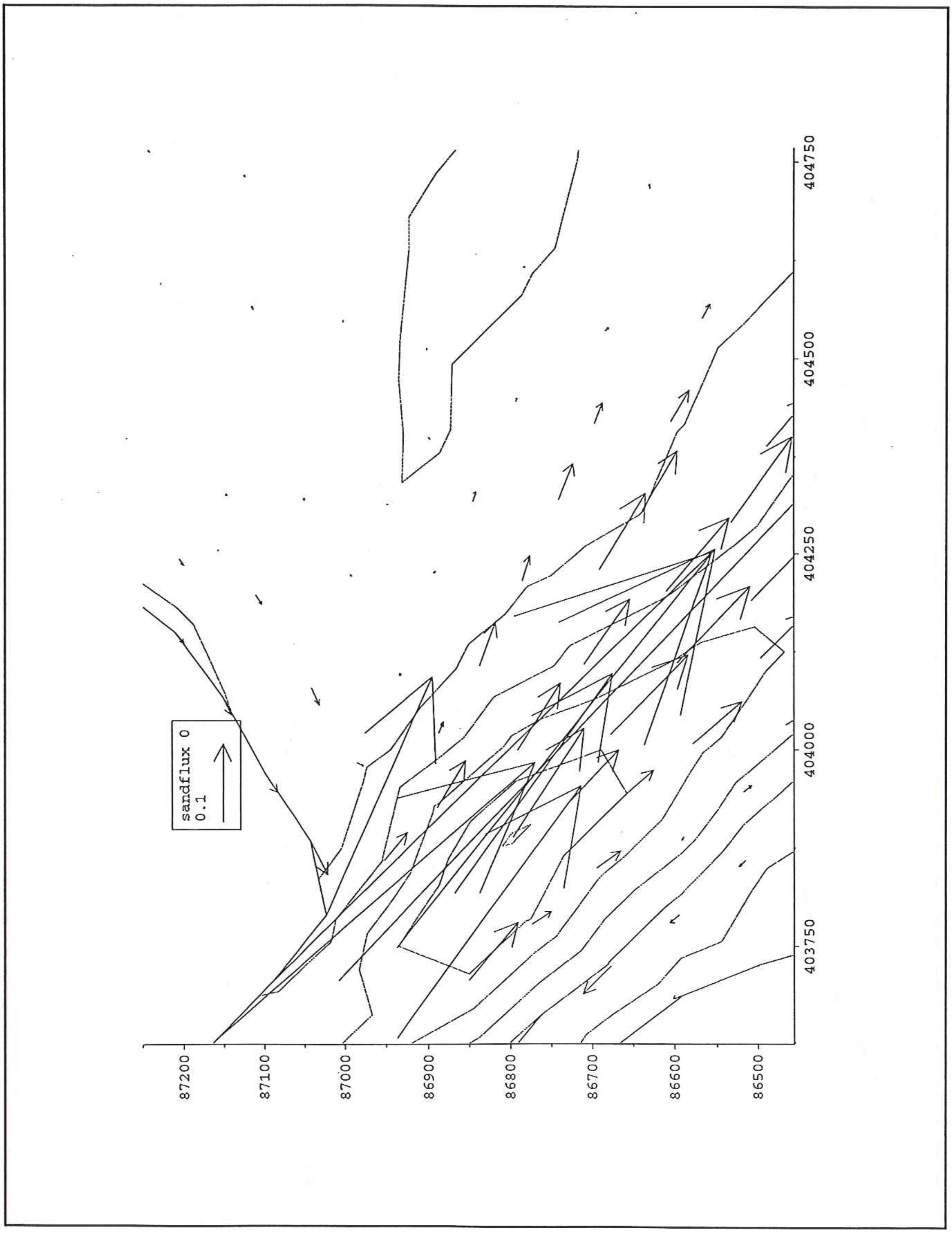


Figure 3.27 Nett potential sand flux (spring tide) at Poole Harbour entrance

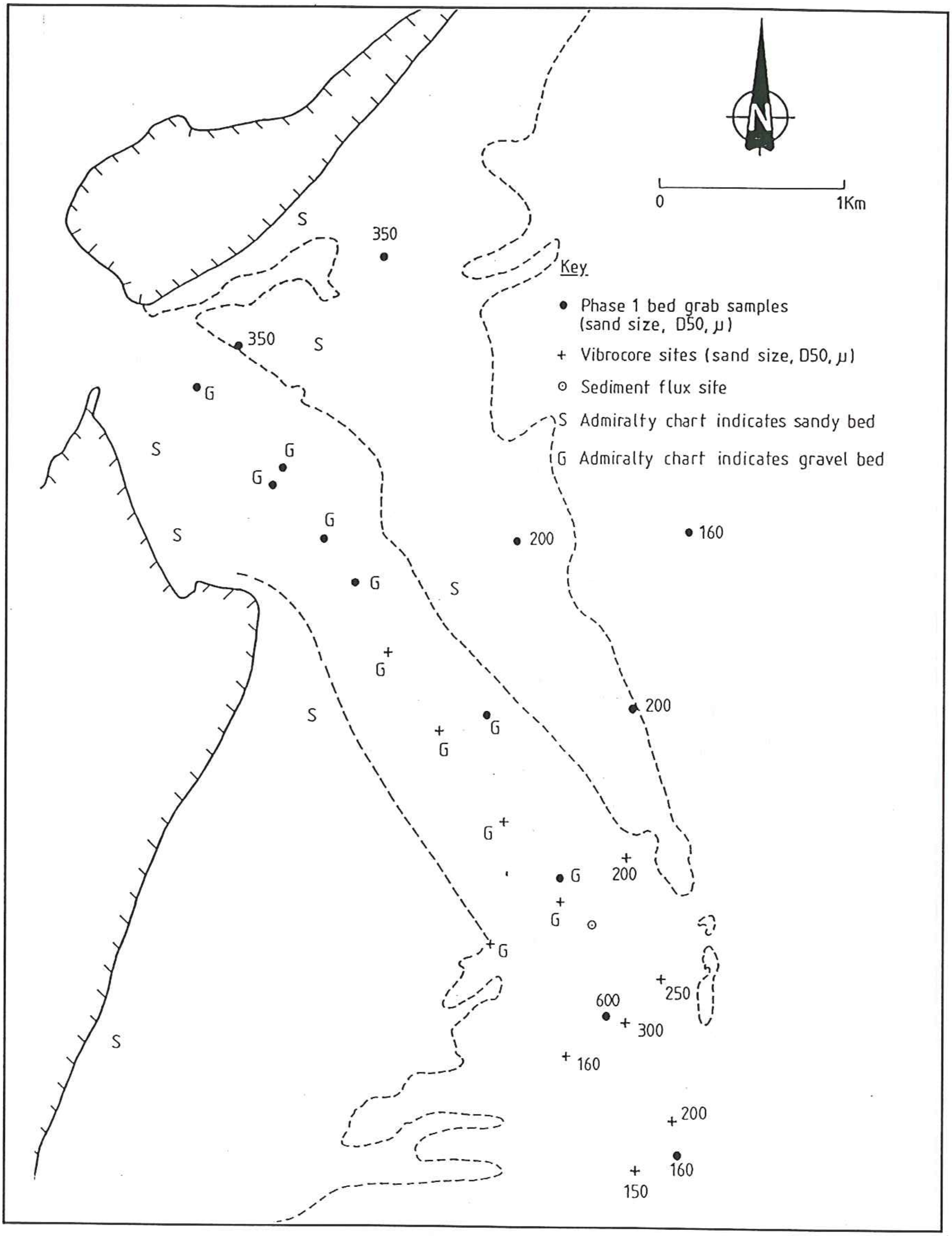


Figure 3.28 **Bed surface sediments (from EX2228,1991)**

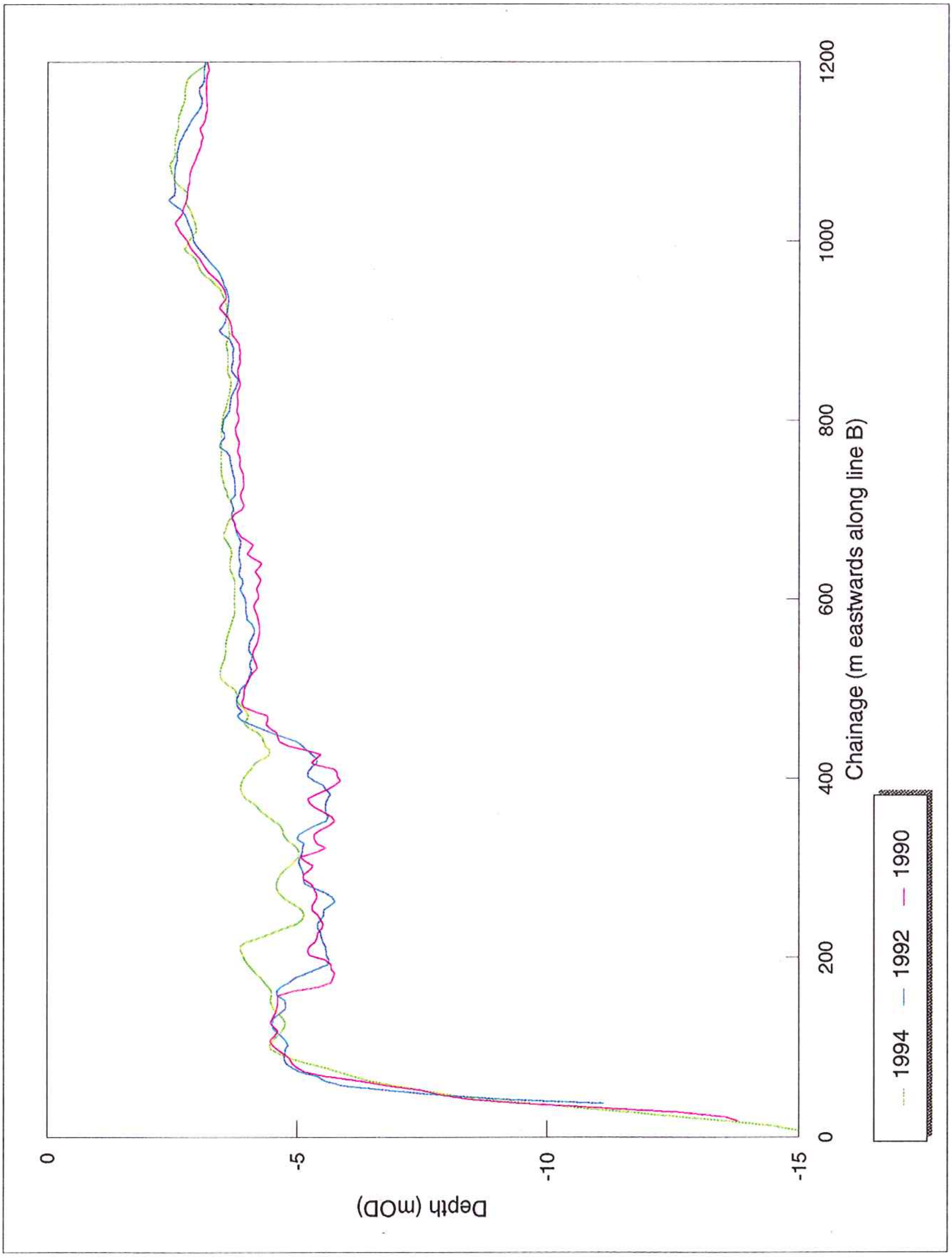


Figure 3.29 Depth changes in the East Looe Channel 1990 - 1994

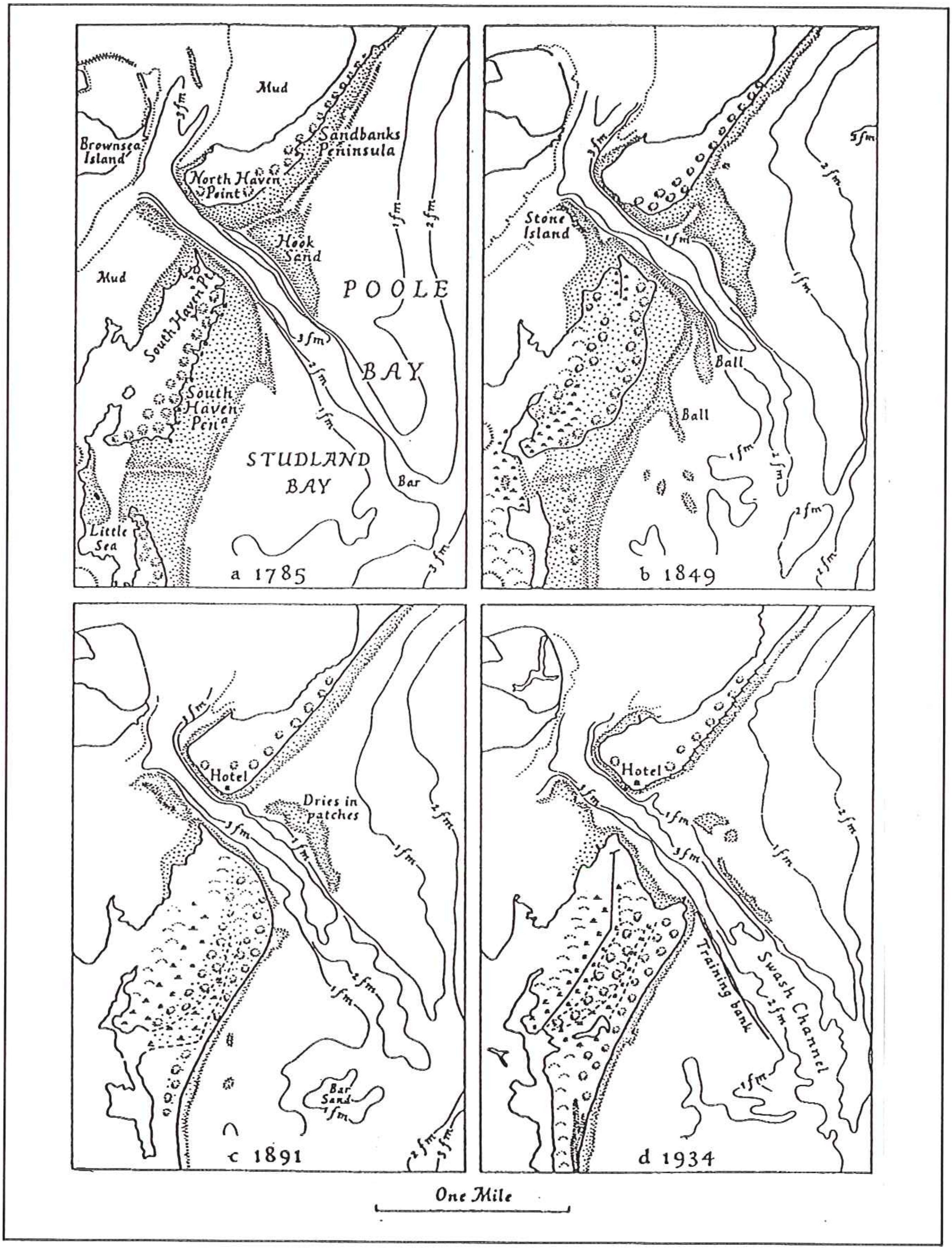


Figure 3.30 Stages in the evolution of the shore line at the entrance to Poole Harbour (from Robinson, 1955)

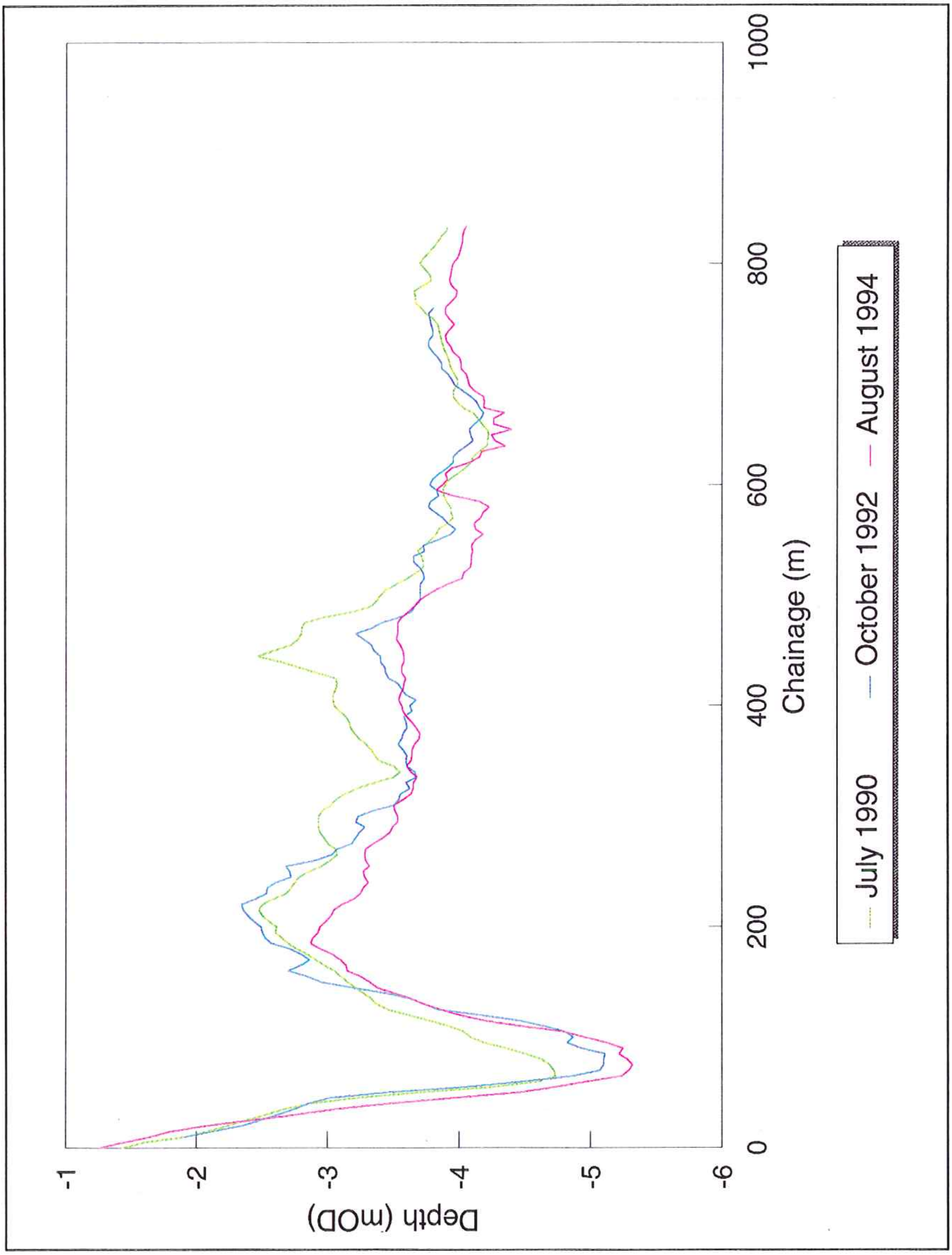


Figure 3.31 **Cross section through East Looe Channel and western flank of Hook Sand**

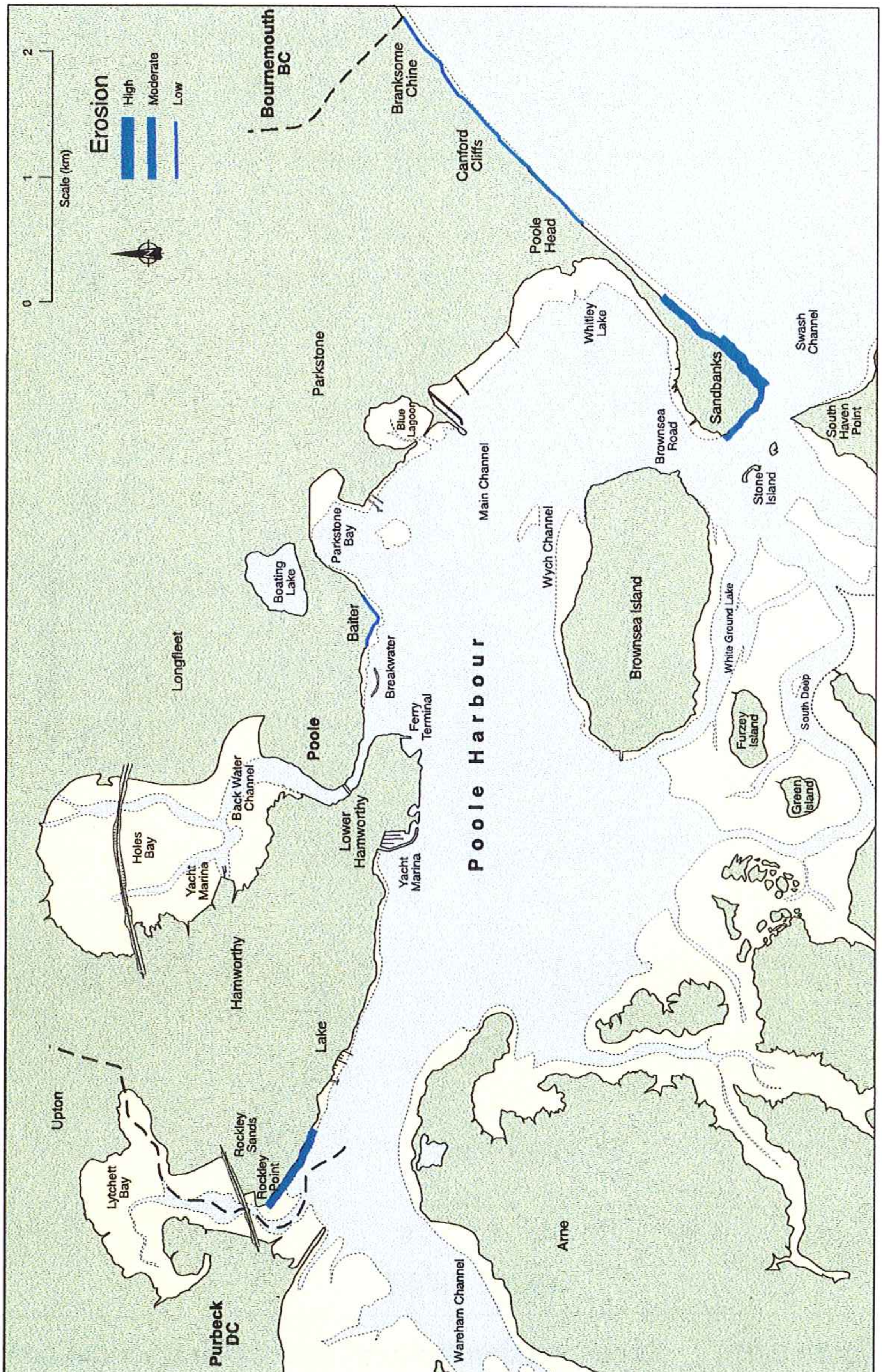


Figure 3.32 Summary of areas of erosion

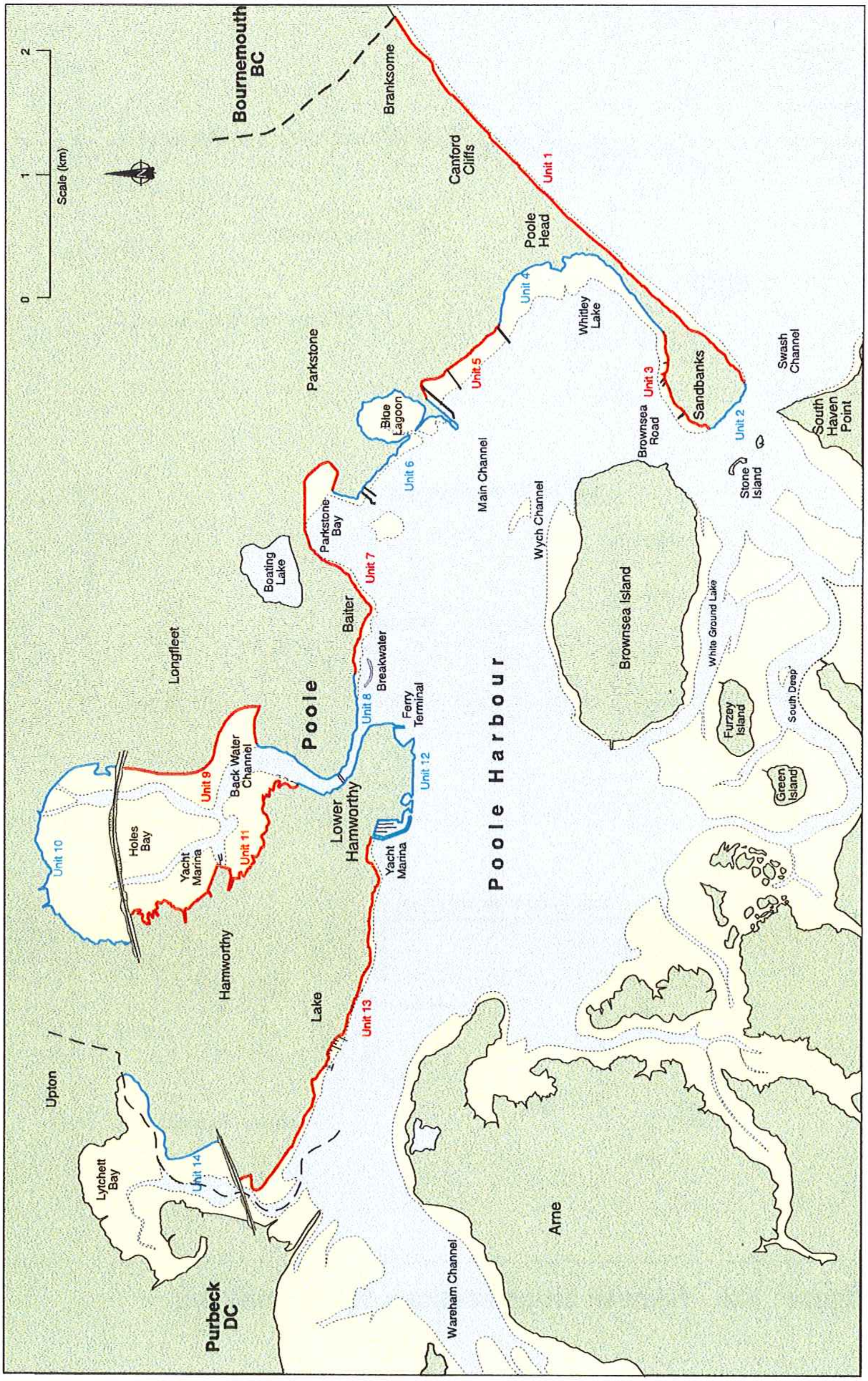


Figure 4.1 Management Units

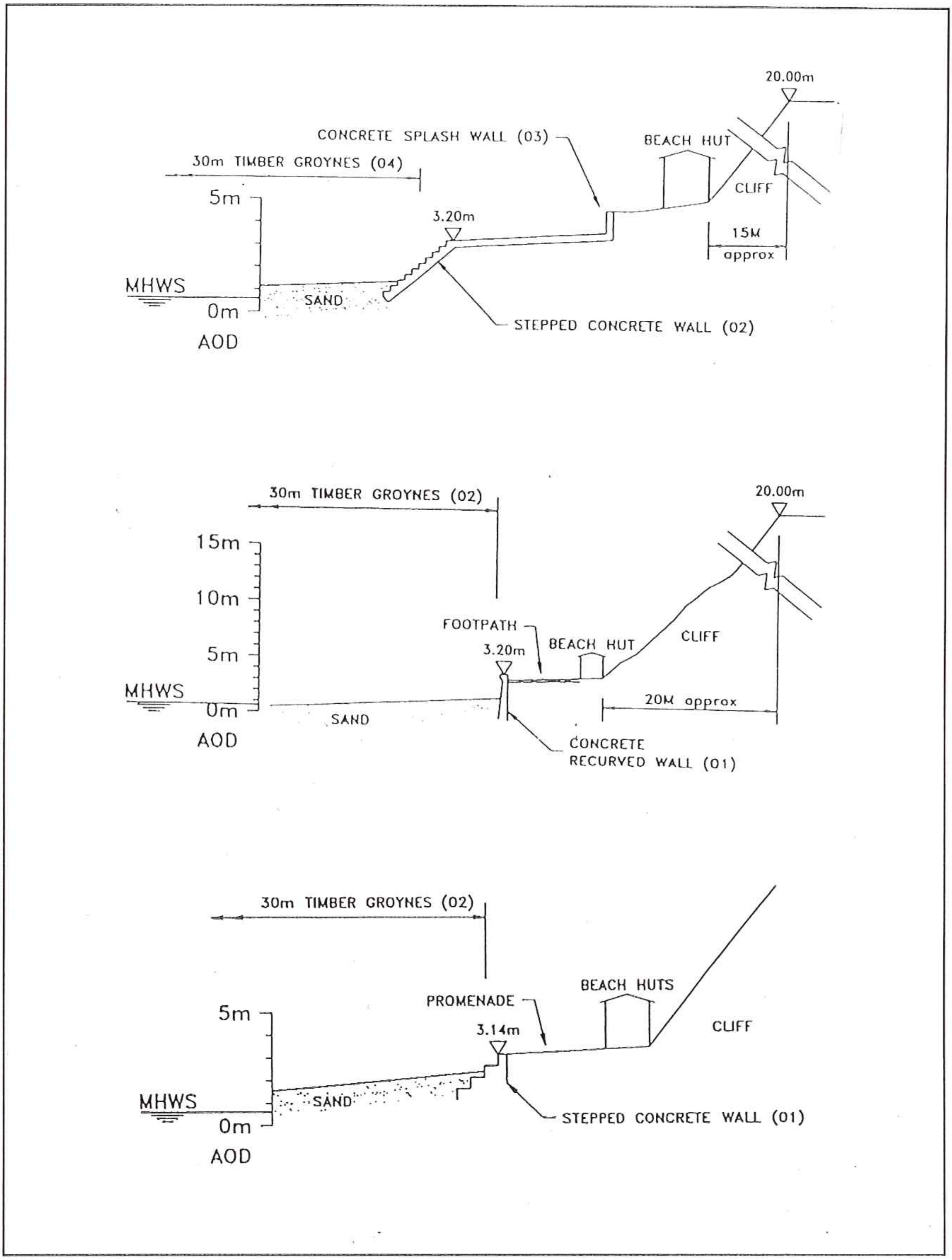


Figure 4.2 Seawall cross-sections 1/4, 1/6 and 1/8

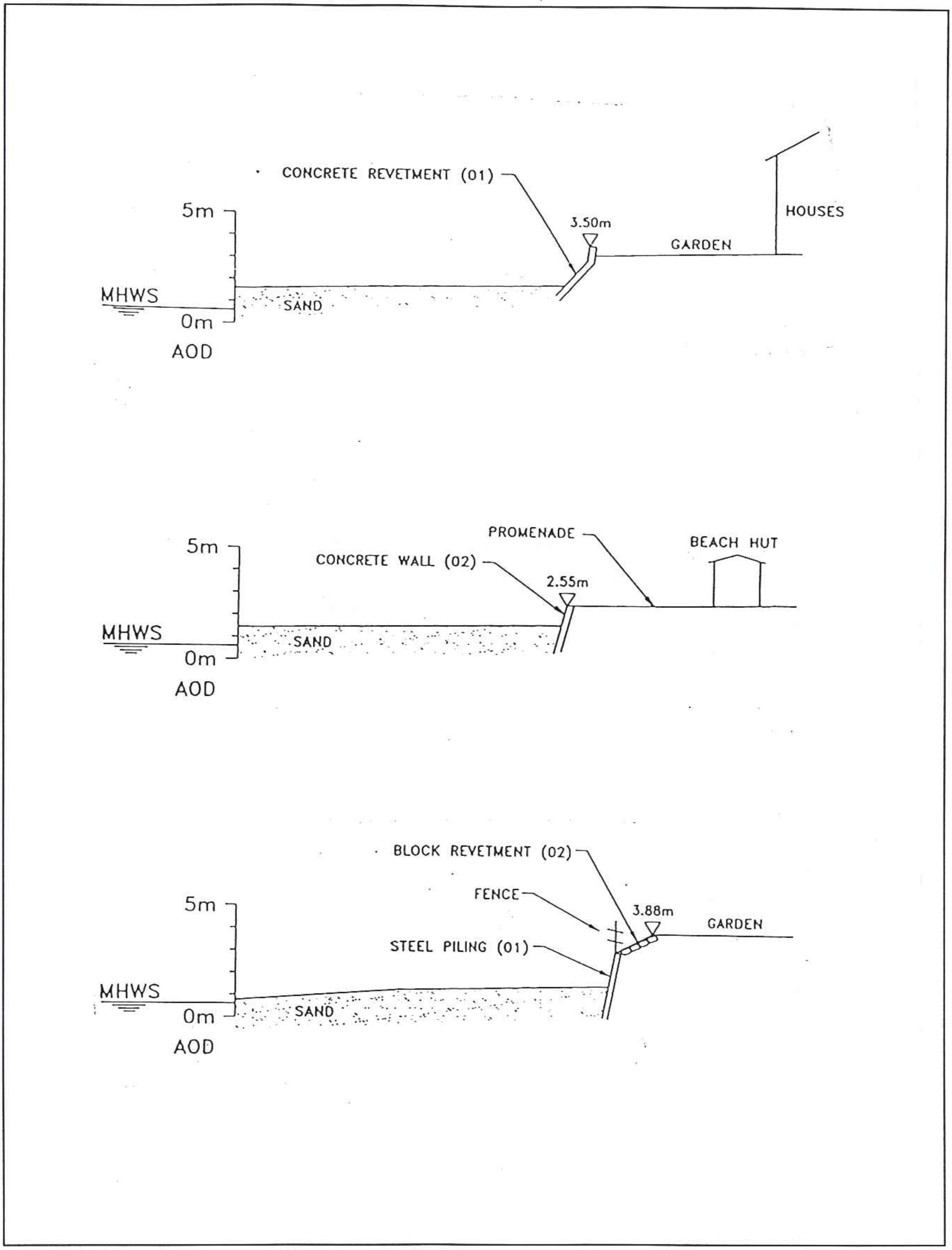


Figure 4.3 Seawall cross-sections 1/10, 1/11 and 1/14

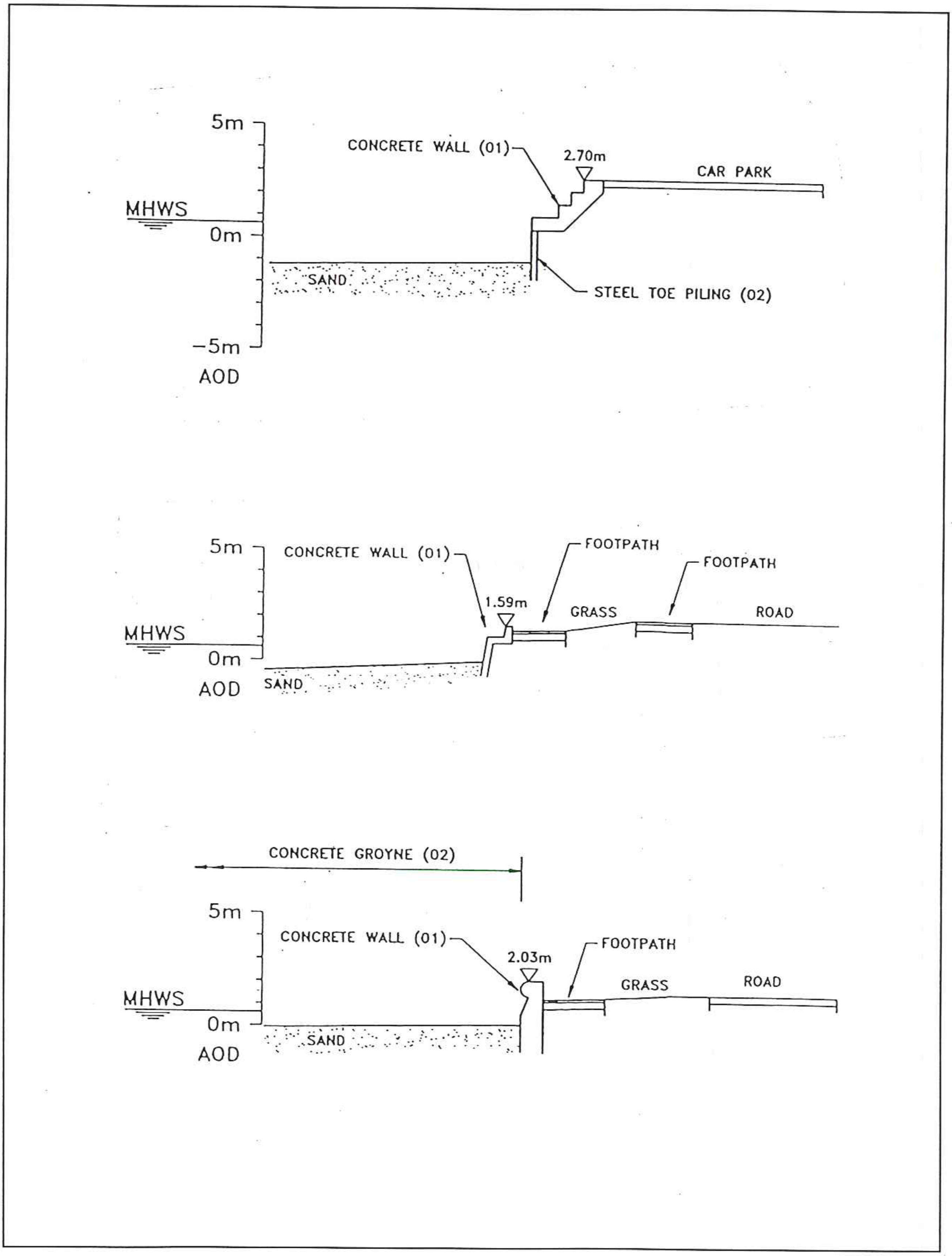


Figure 4.4 Seawall cross-sections 3/3, 4/2 and 4/5

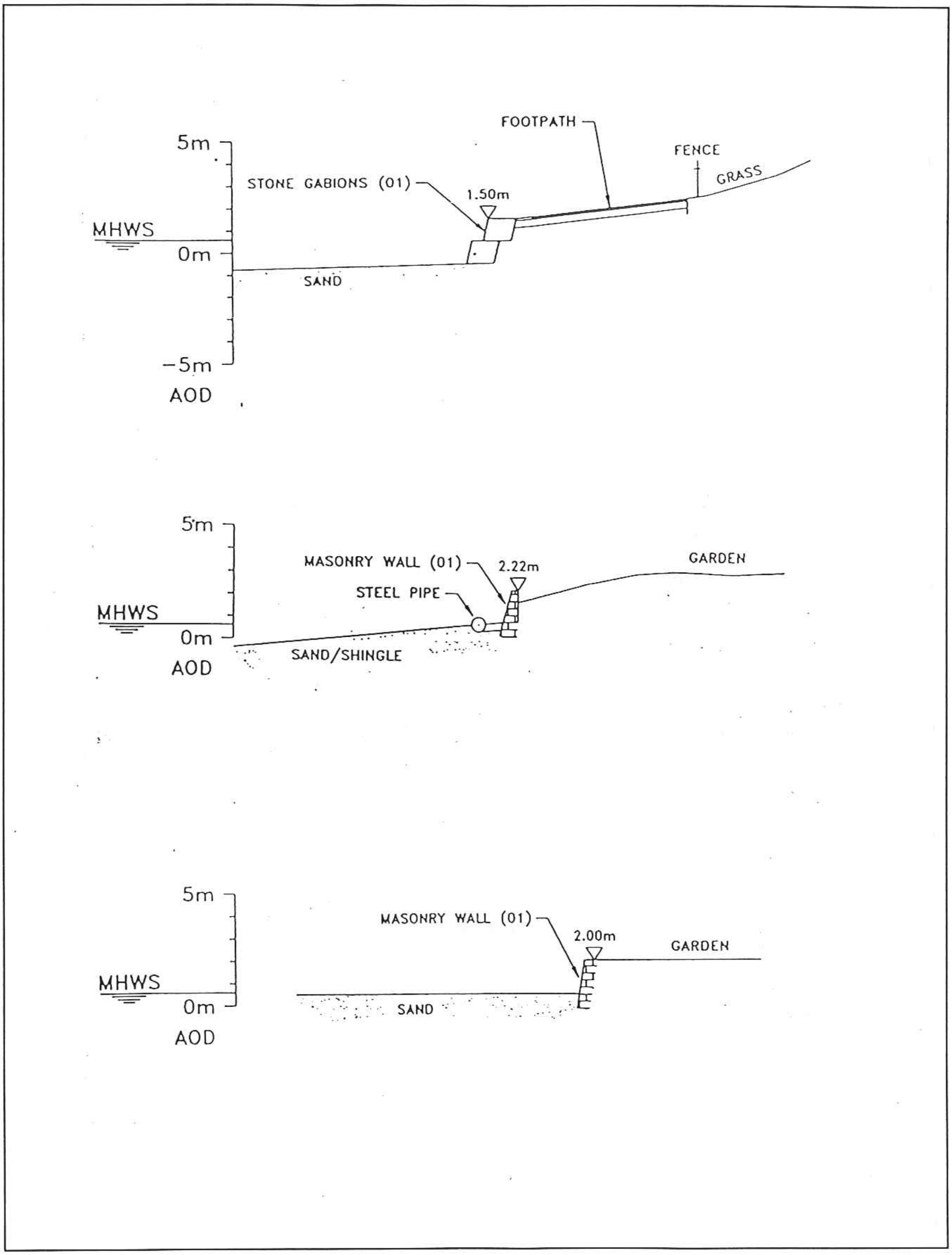


Figure 4.5 Seawall cross-sections 5/2, 5/4 and 6/3

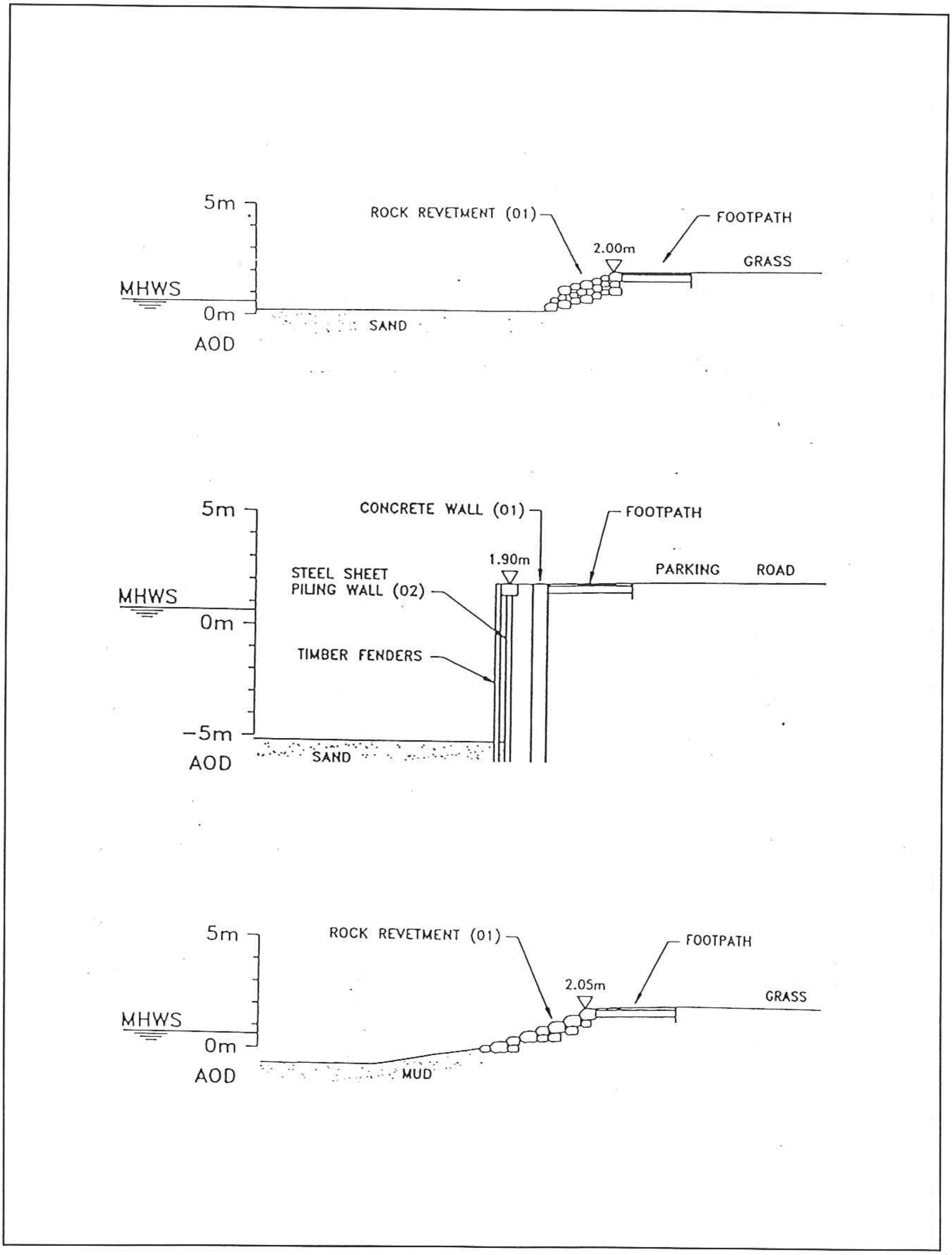


Figure 4.6 Seawall cross-sections 7/4, 8/2 and 9/2

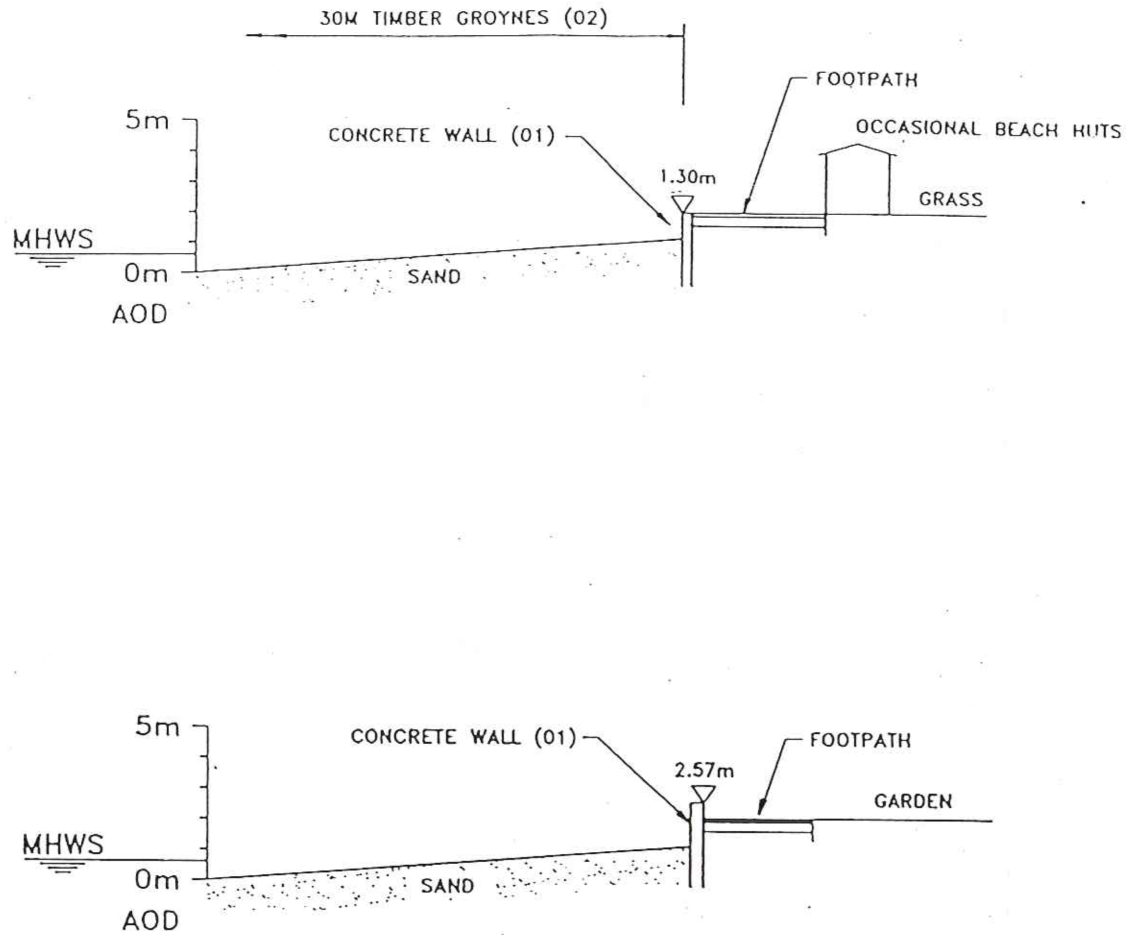


Figure 4.7 Seawall cross-sections 13/2 and 13/4

Limiting values of $\bar{Q}$ for different design cases have been suggested, and are summarised in Figure 152. This incorporates recommended limiting values of the mean discharge for the stability of crest and rear armour to types of seawalls and or the safety of vehicles and people.

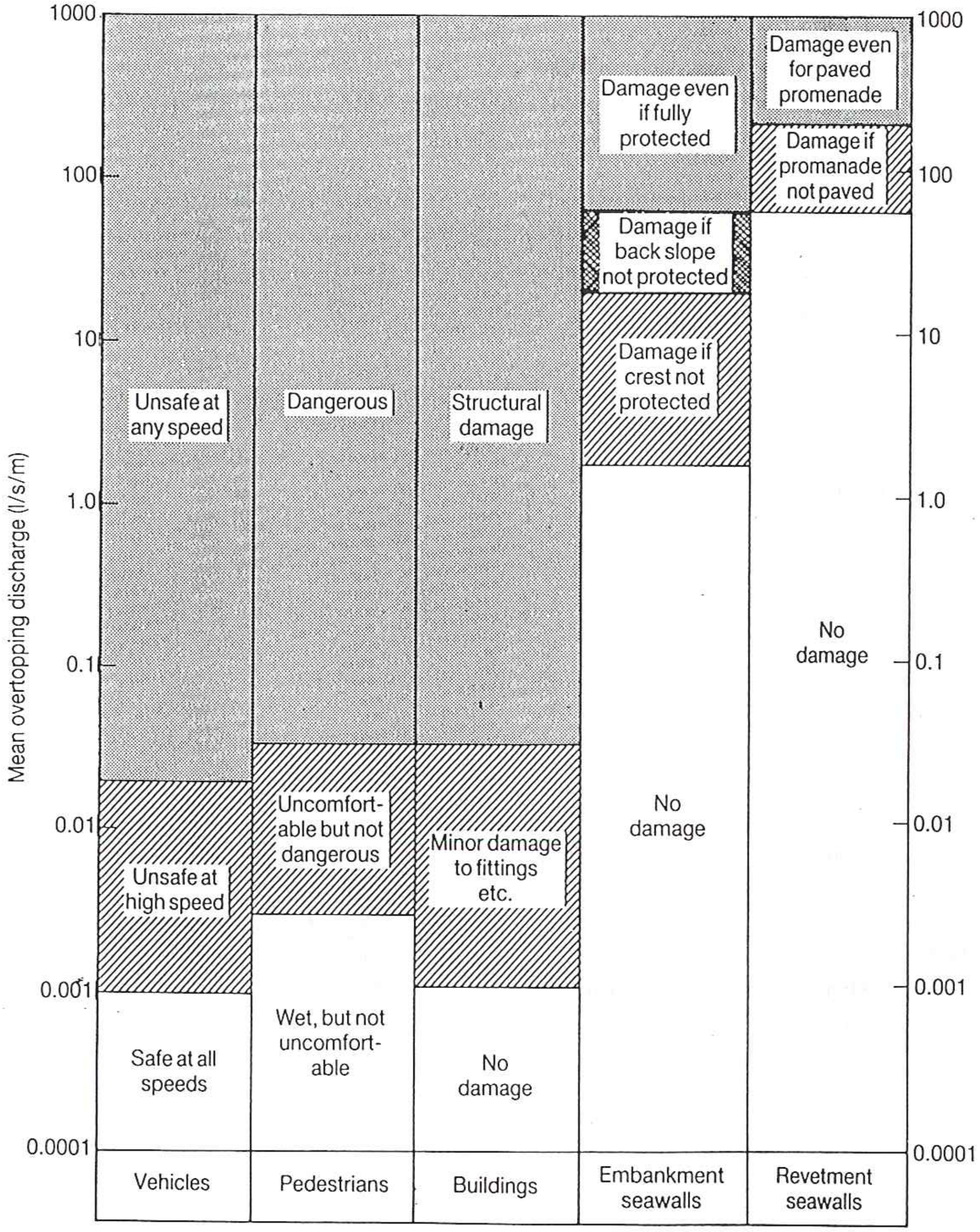


Figure 4.8 Critical overtopping discharges

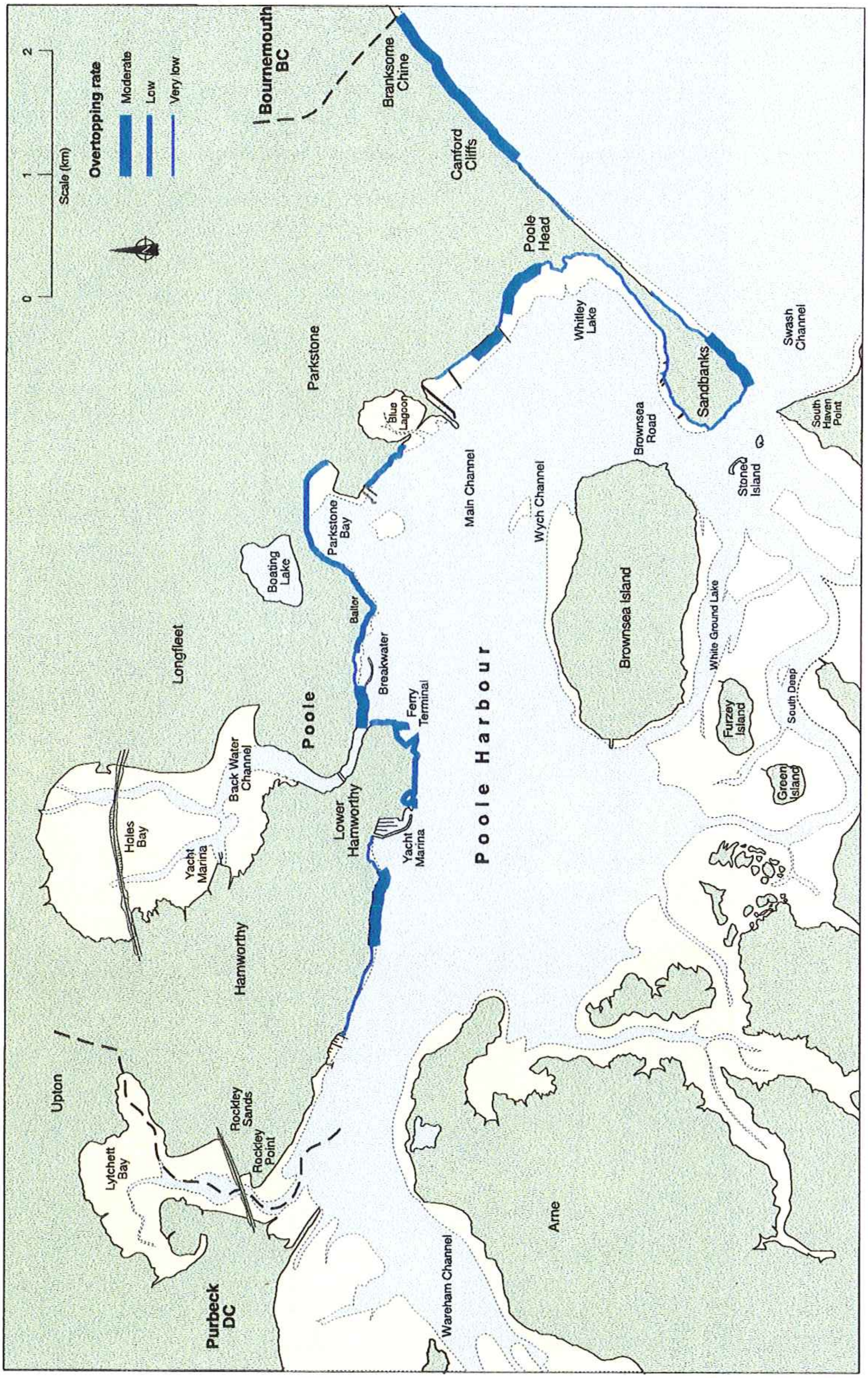


Figure 4.9 Summary of results of overtopping analysis

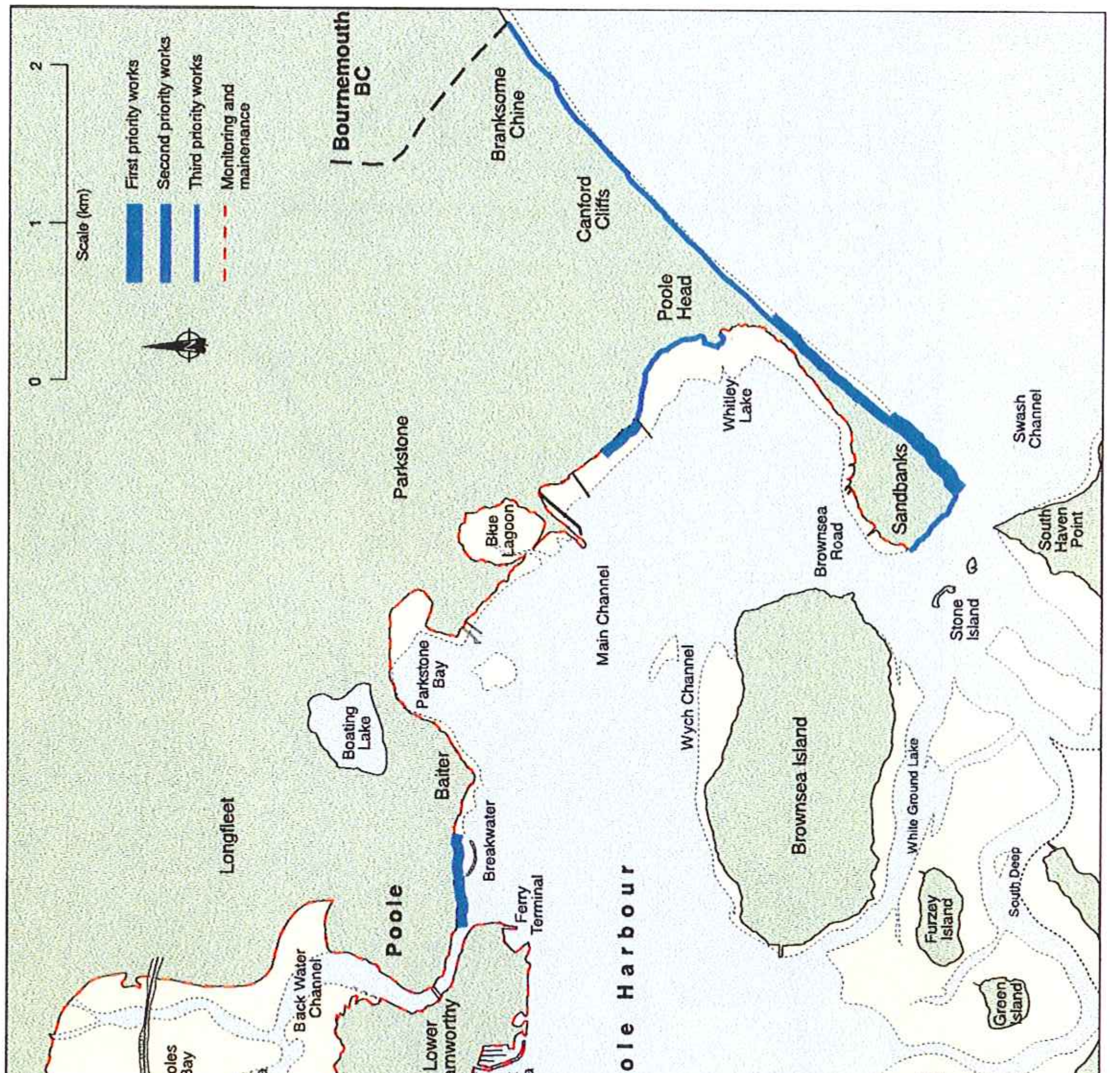


Figure 5.1 Priorities for future work

Plates

Plate 3.1 Sandbanks in 1957 showing sand buildup on the western side of groynes

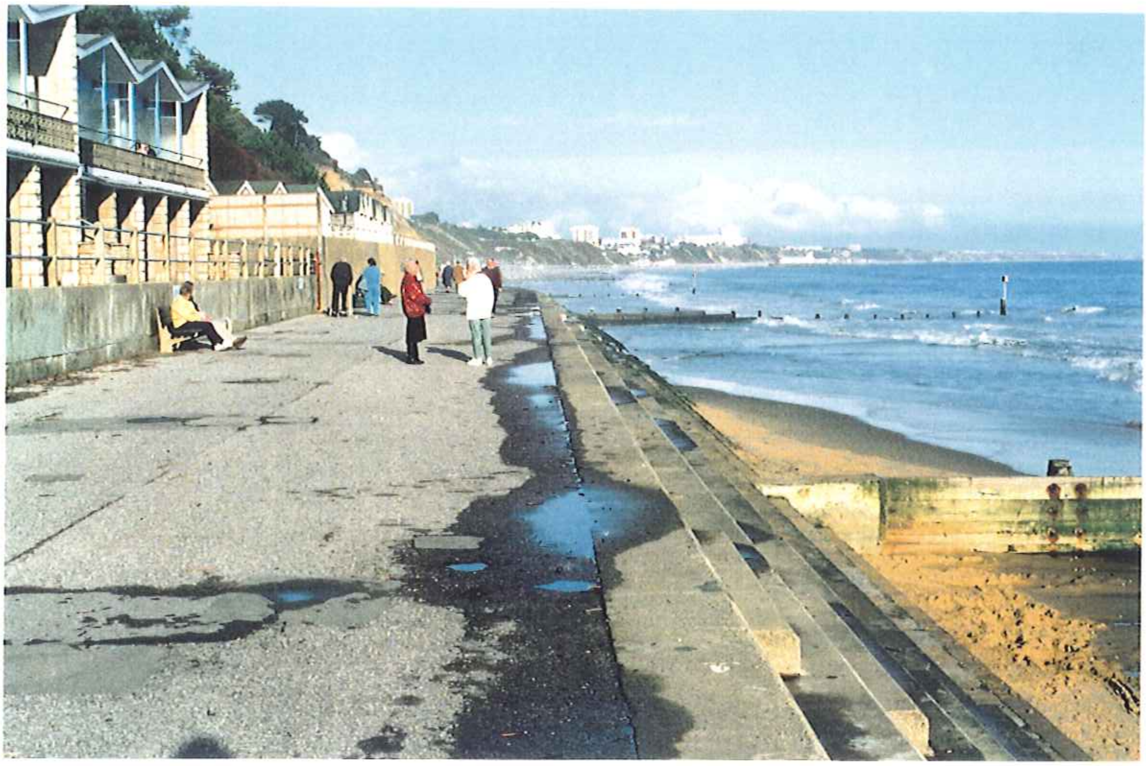


Plate 4.1 Management Unit 1/2

Plate 4.2 Management Unit 1/7

Plate 4.3 Management Unit 1/11

Plate 4.4 Management Unit 1/14

Plate 4.5 Management Unit 2/14

Plate 4.6 Management Unit 3/3

Plate 4.7 Management Unit 4/2

Plate 4.8 Management Unit 5/2

Plate 4.9 Management Unit 6/3

Plate 4.10 Management Unit 7/3

Plate 4.11 Management Unit 7/8

Plate 4.12 Management Unit 8/2

Plate 4.13 Management Unit 9/2

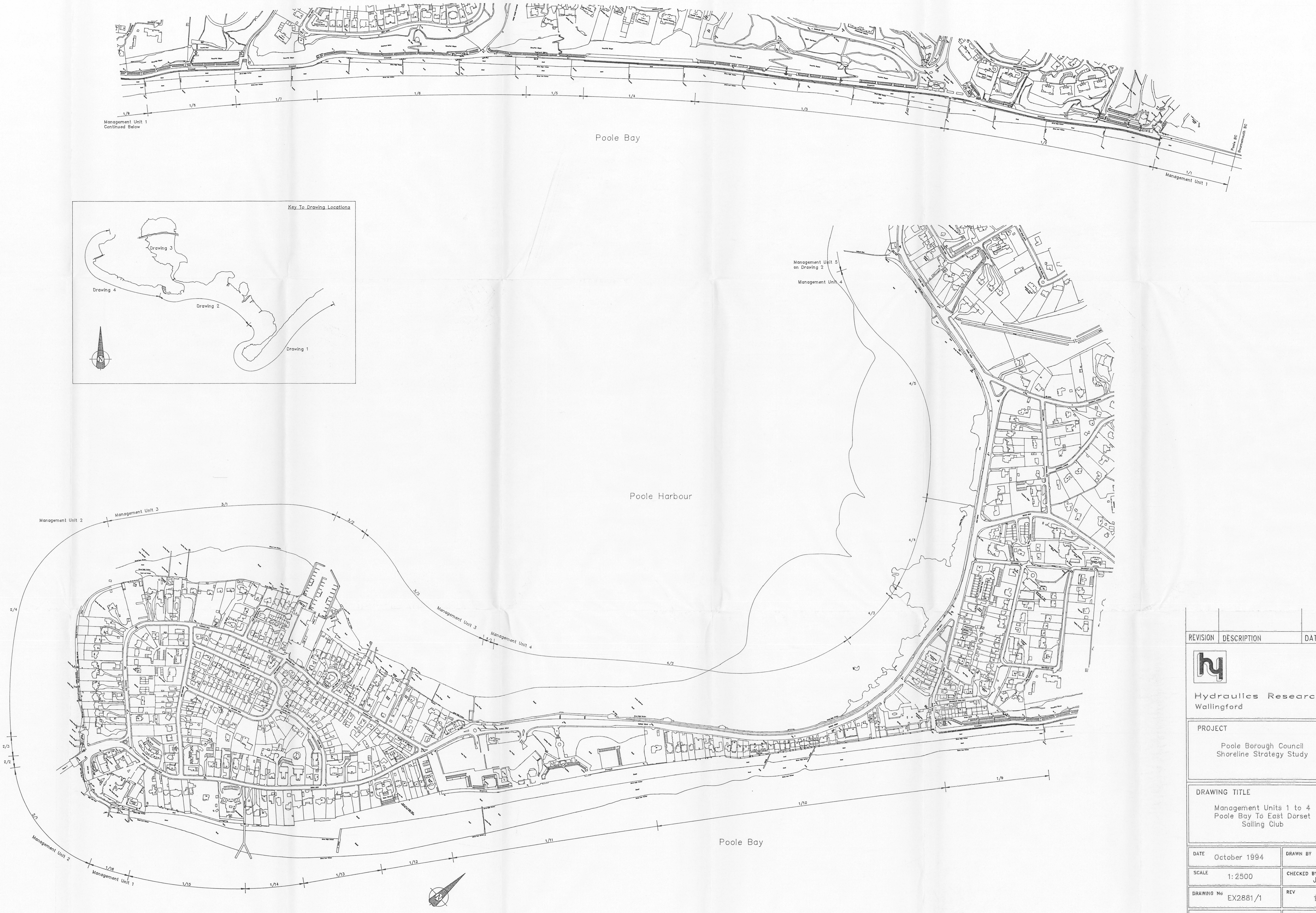
Plate 4.14 Management Unit 10/1

Plate 4.15 Management Unit 11/3

Plate 4.16 Management Unit 13/3

Plate 4.17 Management Unit 13/10

Plate 4.18 Management Unit 14/2



REVISION	DESCRIPTION	DATE

Hydraulics Research
Wallingford

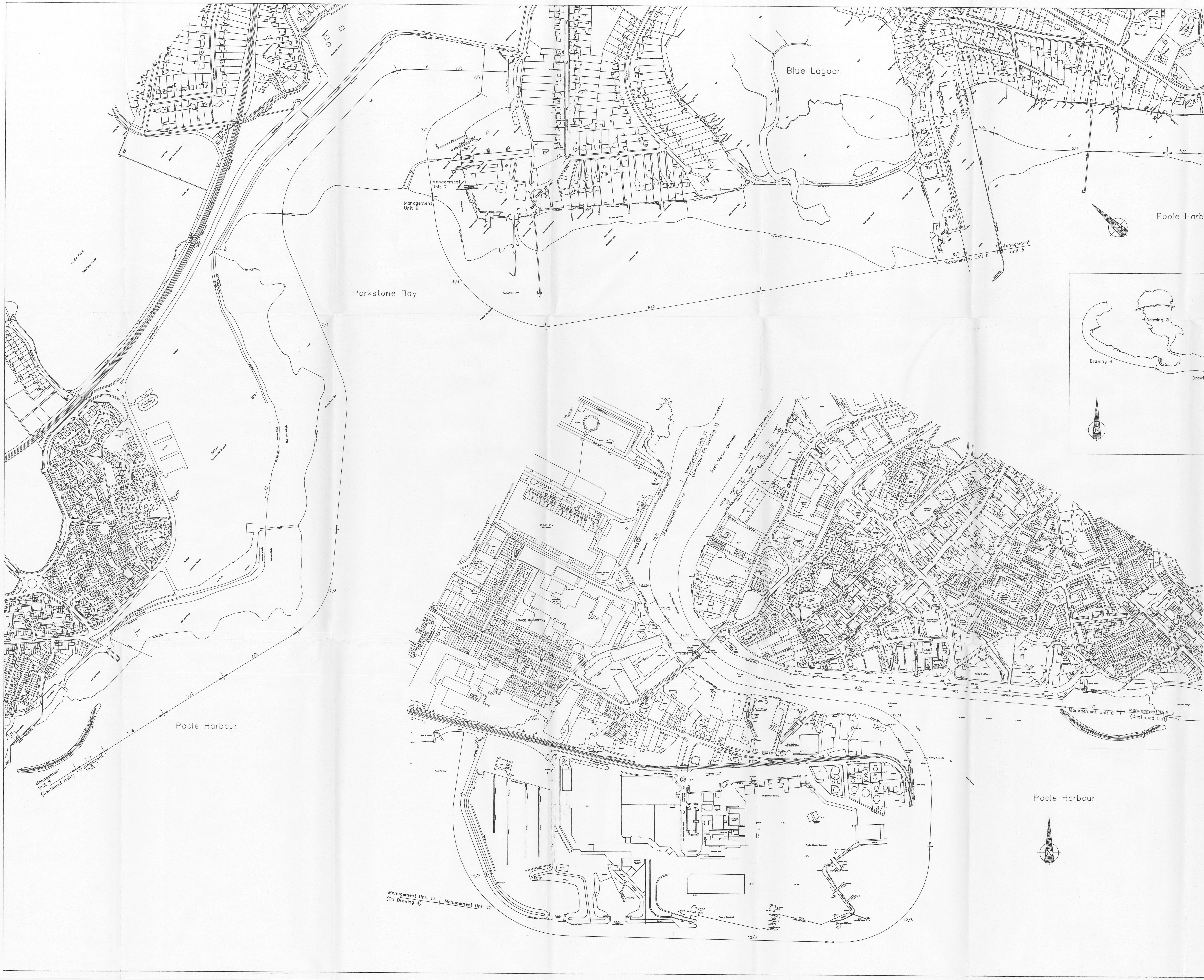
PROJECT

Poole Borough Council
Shoreline Strategy Study

DRAWING TITLE

Management Units 1 to 4
Poole Bay To East Dorset
Sailing Club

DATE	October 1994	DRAWN BY	N.R.
SCALE	1:2500	CHECKED BY	J.W.H.
DRAWING No	EX2881/1	REV	1
CLIENT	P.B.C.	JOB No	CBR1609



REVISION	DESCRIPTION	DATE

Hydraulics Research
Wallingford

PROJECT

Poole Borough Council
Shoreline Strategy Study

DRAWING TITLE

Management Units 5 To 8
And 12

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CLIENT	P.B.C.	JOB No	CBR1609

REVISION	DESCRIPTION	DATE

Hydraulics Research
Wallingford

PROJECT	Poole Borough Council Shoreline Strategy Study
DRAWING TITLE	Management Units 8 to 11 Holes Bay

DATE	October 1994	DRAWN BY	N.R.
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DRAWING No	EX2881/3	REV	1
CLIENT	P.B.C.	JOB No	CBR1609

REVISION	DESCRIPTION	DATE

Hydraulics Research
Wallingford

PROJECT
Poole Borough Council
Shoreline Strategy Study

DRAWING TITLE
Management Units 13 And 14
Lower Hamworthy To
Lytchett Bay

DATE	October 1994	DRAWN BY	N.R.
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DRAWING No	EX2881/4	REV	1
CLIENT	P.B.C.	JOB No	CBR1609

Appendices

Appendix 1

Terms of reference

Appendix 1 Terms of reference

1. REGIONAL PROCESS MODELLING (PORTLAND BILL TO ISLE OF WIGHT)

1(A) Tidal flow modelling

Background
The marked variations in tidal level between Weymouth and Hurst Castle, and the often complex tidal current patterns, are a major feature of the general study area. For shoreline management, it is important to have good information on these aspects, especially on the currents, which help to transport sediment and shape the beaches. HR have carried out a variety of studies of tidal flows along this section of the coast over the years. It is a very complicated area, from a tidal viewpoint, with double high-waters in the east, double low-waters in the west, and a very large variation in tidal ranges.

Previous models have had rather different aims to those of the present study and do not represent the tidal currents very accurately close to the beaches, because of inaccuracies caused by using a rectangular grid. However, these models provide a wealth of information on the seabed contours (the bathymetry), and on the deep-water tidal conditions which need to be specified to "drive" a numerical tidal model (the boundary conditions). Also, these previous models have been carefully checked against measured currents, either from fixed instruments or from surface flow observations carried out using a radar system deployed on the cliffs (OSCR).

Methods
Information from previous models, as described above, will be used in the proposed tidal model for this study. To accurately resolve the complicated shape of the coast, however, we will use a new flow modelling method, TELEMAC-2D, which predicts depth-averaged tidal currents on a varying-size triangular mesh (see enclosed leaflet). This will allow us to represent the seabed and coast more accurately, where required, than by using existing models which have a minimum grid size of 400m square. Apart from the normal tidal constituents needed to represent tides in UK waters (S2, M2, M4), 6 others will also be used (01, KI, N2, MS2, MS4 and M6) in this model. Poole Harbour will be represented, but only at coarse resolution. Both a mean Spring tide and a mean Neap tide will be run through the model, with tidal currents and water levels.being output at a large number of points throughout the modelled area. This modelled area will stretch from St Catherines Point (IOW) to Portland Bill, and include Weymouth, Poole and Christchurch Bays.

Outputs
The tidal model will provide:

Through-tide predictions (at 12 minute intervals) of water level, current speed and direction at about 100 locations to be specified during the course of the study. Results from these locations will be available to Poole Borough council on floppy disks at the end of the study.

Results (at 1 hour intervals) from about 30 of these locations will be presented in the final report in the form of tables.

Vector diagrams showing depth-averaged currents at various stages during Spring and Neap tides will also be provided in the final report, thus providing local tidal stream atlases for this part of the coast

The model developed in this study will be retained by HR, and will be available for future studies for Poole Borough Council, or other members of SCOPAC.

1(B) Offshore wave climate evaluation

<u>Background</u>
Wave conditions along the coastline between St Catherines Point and Portland Bill are best evaluated by using a two-stage approach. In deep water, ie greater than about 25m in the present case, the effects of the seabed on the character of waves are small in comparison to the effects of winds. Closer to the shore, the effects of the seabed and headlands become more important, and dominate effects due to the wind. In this study, the proposed numerical simulation of the generation of waves in deep water, and of their transformation in shallow water, considers the two aspects separately. In a previous study in Poole Bay, wave measurements have verified that this approach provides reliable estimates of actual conditions.

<u>Methods</u>
We have already carried out several studies of wave generation in this area. A standard HR computer model, HINDWAVE, will be used with input wind data measured at Portland Bill from 1974 to 1992. Most of this wind data is already held at HR, but to improve the joint probability study (see Section 2C), it is recommended that about 2 years of recent measurements are obtained during this study to extend the wave "hindcasting" to cover the whole 19 year period. Five locations offshore from the coast will be selected, two west of Durlston Head, and the remaining 3 further east. For each location, hourly wave conditions will be predicted, and a synthetic wave climate will be evaluated.

Statistical methods will be used to predict extreme wave conditions at each of the offshore locations, and this information will be used in the evaluation of nearshore extreme wave conditions (Section 2A) and in the joint probability study (Section 2C).

<u>Outputs</u>
The proposed offshore wave modelling will provide:

Predicted hourly wave conditions at five offshore locations, which will be available on floppy disks to Poole Borough Council at the end of the study.

Tables and diagrams (eg wave "roses") in the final report summarising the overall wave climate for each location.

Predictions of extreme wave conditions, for a variety of direction sectors, and for a variety of return periods.

2. LOCAL PROCESS MODELLING

2(A) Derivation of nearshore wave conditions (Poole Bay)

Background
As waves travel into the interior of Poole Bay from the open waters of the English Channel, they are modified by the changing water depths. The partial protection provided by Durlston Head is also important, especially for Studland Bay and the eastern part of the Poole Borough frontage on Poole Bay. Wave conditions during a westerly storm are therefore very different at Hengistbury Head and at Poole Harbour entrance. Numerical modelling will be used to quantify these changes, as an essential step in producing "nearshore" wave climate information at a number of locations around Poole Bay.

Methods
The wave transformation modelling proposed in this study will be heavily based on previous HR studies of Poole Bay. An existing computer model will be revived, and combined with the offshore wave climate information (see Section 1 B) to calculate corresponding "nearshore" wave climates at a number of locations both along the Poole Borough frontage, and elsewhere.

This wave climate information will be used in later stages of the study to predict extreme wave conditions for use in design studies, and to calculate longshore drift rates for the sandy beaches of Poole Bay (see Sections 2D and 2E).

This modelling will assume a fixed tidal level (MHWS), thus providing a good estimate of wave conditions occurring during the important upper part of the tidal cycle.

The sequential wave conditions predicted for the nearshore locations will be used at a later stage in the study to provide information on the year-to-year variability of longshore drift rates along the sandy coastline of the Borough.

Outputs
The nearshore wave climate information will.b.e presented in the form of a number of tables and diagrams in the final report, in a similar manner to that used for the offshore wave climates.

In addition, standard statistical methods will be used to predict extreme wave conditions, ie significant height, period and direction, at each nearshore location.

The sequential nearshore wave data will be available to Poole Borough Council on floppy disks at the end of the study.

2(B) Derivation of nearshore wave conditions (Pool.e Harbour)

Background
Waves in Poole Harbour are almost entirely due to local wave generation within the Harbour itself. The height of the waves depends largely on the wind speed (over about the last hour), and the distance across the Harbour in the wind direction to the point of interest, ie the "fetch". It also depends on the fetch "width", calculated using fetch lengths at angles either side of the wind direction. Because of the substantial drying areas within the Harbour, fetch

lengths, and waves, are smallest at low water, and highest at high tidal levels. The wind strength will also tend to be greatest at high water because the water surface is "smoother" than the saltmarsh and mud-flats at low tide. Because of this, even the relatively small waves on this part of the Poole Borough coastline can combine with high tidal levels to cause damage and flooding.

The complex shape of the Harbour, with various islands which shorten some fetch lengths, and change the fetch widths, means that wave conditions will change along even a short stretch of coastline. However, because the waves generated within the Harbour have a rather short period, they will be affected only slightly by changes in water depth.

Method

Because of the rapid spatial changes in wave conditions around the coastline of Poole Harbour, it is not possible in this study to provide wave climate information at every point. This part of the study will therefore be carried out in three parts, as follows.

First we will assess the various sources of wind data available for use in calculating wave conditions within the Harbour. There are wind records from Hum, Portland, St Catherine's Point and other locations. A recommendation on which wind data to use, and how it needs to be modified for the area will be made, and a suitable wind-rose (and frequency occurrence table) produced. Extreme wind conditions, from various directions, will also be calculated.

In the second stage, a number of locations (6) along the Poole Borough side of the Harbour will be selected, and a computer model of wave generation developed by H R Wallingford will be used to produce wave climates and extremes at these chosen locations. This information will provide an input to the joint probability study (see below).

Finally we will pass over the computer model, together with user notes and worked examples, to Poole Borough Council staff, for them to apply at any other location(s) of interest. This hand-over will be made during the study, so that any problems can be solved by HR staff before the final report is issued.

Outputs

This section of the study will provide (high tide) wave climate information at 6 locations along the Harbour frontage of the Borough, together with extreme wave predictions for each site.

It will also provide computer software, and wind data, to allow Borough staff to calculate similar information for other locations along that frontage, or elsewhere in the Harbour.

2(C) Tidal data assimilation and analysis

Background

The Poole Bay area is poorly served with well-documented numerical information on tidal levels. The nearest A-Class tide gauges are at Portsmouth (where tidal conditions are very different) and at Weymouth. This latter gauge has only been installed recently, and cannot therefore provide long-term records on extreme water levels. The previous A-Class gauge in Portland

Harbour did operate for many years but the measurements have not been digitised and are not therefore available for this study.

Method

We will obtain tidal measurements from Poole Harbour Commissioners, recorded within the harbour. It has been assumed in pricing this study that data for the last 4-5 years in available in digital format, in a computer readable format, at no cost to HR. This data will be the basis for the analysis of both extreme water levels and the joint probability of such levels occurring with large waves.

Information on the spatial changes in tidal levels in . side Poole Harbour and in Poole Bay will be obtained both from the Harbour Master and (existing) numerical models of tidal flows.

Digital tide level data will be entered into our computer system at HR, and standard analysis software used to determine the various tidal constituents. These constituents will then be used to "hindcast" the expected (astronomical) tide, and the differences (residuals) between these predicted and the measured tide levels will be calculated. The times and levels of the (higher) high water values for each tide will also be identified and used in calculations of extreme high water levels.

Outputs

The analysis of tidal levels will provide:

A time-history of the (higher) high water level for each tide (at the tide gauge site), with times and calculated residuals (ie surges) at those times. This will be used in the joint probability study.

A list of the highest recorded water levels, with dates and times, for subsequent use in reviewing historical flood/damage events (see Section 5A).

Statistical estimates of extreme water levels (at the tide gauge site) for a variety of "return periods", eg the level only likely to occur once in 50 years.

An estimate of the differences between water levels at high tide at the tide gauge site, and at other locations within Poole Bay and Harbour. This information will be used in extending the joint probability study to other locations along the Borough's frontage (see Section 2C).

2(D) Joint probability and design wave & tide level conditions

Background

Flooding and damage to coastal properties rarely arises just because of a very high tide, or just because of severe wave conditions. In almost all cases it is a combination of these two factors that is involved. Because of the modest (astronomical) tidal range in Poole Bay, the highest recorded tidal levels always include a significant weather-induced residual or "surge". As such surges typically result from low atmospheric pressure and strong winds, eg blowing along the English Channel, it is likely that the waves will also be significant at the time. There is a likelihood of at least a partial correlation between high water levels and large waves in Poole Bay. In Poole Harbour, where waves depend on the local wind strength and its precise direction at the time of high water any correlation may be different.

By ignoring such correlations, there is a danger of under-estimating the chances of flooding or damage. On the other hand by taking a pessimistic view, and assuming the worst wave and tide conditions will always occur simultaneously, it is probable that coastal protection works will be over-designed and unduly expensive. An accurate assessment is therefore needed of the joint probability of these two factors, and a derivation of appropriate combinations of them for use in the design of new coast protection works (or the assessment of existing structures).

Methods

The only satisfactory way of establishing "joint probabilities" is by analysis of simultaneous recorded (or hindcast) wave conditions and tide levels.

The main danger of damage and flooding occurs at the top of each tide, ie at the peak water level. BY combining the information of these peak levels for each tide with the hindcast (offshore) wave conditions at the same time, we will be able to calculate the probability of the joint occurrence of large waves and high tides in Poole Bay.

In Poole Harbour, the calculations are slightly different in that the information on peak levels will be combined with the calculated nearshore wave conditions, which vary depending on the exact location within the harbour.

Having derived the individual (ie marginal) probabilities of both wave conditions and of tidal levels, together with the joint probability, it is then relatively straightforward to derive appropriate combinations of these two factors, with a specified probability of occurrence.

Outputs

The output from the joint probability study will be a series of tables in the final report which give the probabilities of high tidal levels and simultaneous large waves, both offshore in Poole Bay and at a number of locations in Poole Harbour.

In addition, tables listing recommended combinations of high tide level and wave heights for design purposes will be produced for each chosen location around the Poole Borough coastline.

Finally, we will adjust these 'present day' design conditions to take into account predicted increases in sea levels caused by global warming.

2(E) Longshore drift modelling

Background

Long term changes in sand or shingle shorelines are usually caused by variations, from point to point, in the alongshore transport of beach material. Poole Bay is a classic example of this general rule. East of Bournemouth, the long term "drift" of beach material is eastward, ie toward Hengistbury Head. From Canford Cliffs to Poole Harbour entrance, however, the nett drift is westward, and has produced the characteristic "spit" at Sandbanks. There is therefore a "drift divide" in Poole Bay, ie a point at which sand is carried away to both sides by wave action. Until recently this diverging drift pattern was supplied by erosion of the sandy cliffs, for example at Bournemouth. The longshore transport of sand and the erosion of the cliffs, and the beaches in front of them, are clearly linked.

Areas of erosion (or of accretion) around the Poole Borough coastline are therefore likely to be linked to the longshore drift rates. Quantifying the volumes of material involved is a vital first step in deciding on an appropriate management strategy for a sandy, or shingle, beach.

Methods

As with many other parts of the modelling of the coastline of Poole Borough, we will be able to make use of previous investigations. Where the movement of beach material is dominated by wave action, it is relatively easy to predict the annual sediment transport rates using a standard drift formula and the nearshore wave climate information obtained at an earlier stage in the study. This approach can be used both along most-of the Poole Bay and Harbour frontages of the Borough, where the beach is of sand or shingle.

The situation is much more complicated near the entrance to Poole Harbour because tidal currents also become important in the transport of beach material. It would be disproportionately expensive to fully analyse and model sediment transport in this area during the strategy study. Here we will carry out some sample calculations to demonstrate the importance of the tidal currents, as well as of wave action. We will then provide recommendations for any further modelling required of this very complex area.

Outputs

Mean annual drift rates will be presented for approximately 8-10 sites in both diagrammatic and tabular form in the final report. The likely accuracy, and the inter-annual variability, of those mean drift rates will also be discussed, by reference to observed beach behaviour and previous similar studies.

3. HISTORICAL REVIEW OF LOCAL COASTAL EVOLUTION

3(A) Geology and geomorphology

Background
It has been found in previous studies of the development of the coastline that a general understanding of its solid geology, and its geomorphological development, is extremely useful in understanding present-day processes and changes, and hence in proposing a sensible management strategy. In Poole Borough, the geology of the cliffs is of considerable interest in its own right, and Sites of Special Scientific Interest (SSSI's) have been established because of this. The future management of the coastline will need to take full account of these sites, as in previous years, and be sensitive to their scientific value.

Methods
Existing reports, papers and books describing the coastlines of Poole Bay and Poole Harbour have already been obtained by HR in connection with other studies. A review of this information on the geological character of the coast, particularly its development since the last Ice Age, will be undertaken. Any differences of opinion as well as areas of agreement, will be highlighted. Sources of more detailed information will be identified and appropriate references given.

Outputs
A simply-written, non-technical chapter of the final report will describe the (geologically) recent evolution of the Poole Bay coastline from Hengistbury Head to Durlston Head, including Poole Harbour.

An Appendix to the final report will deal with the geological SSSI's within Poole Borough, with comments on their preservation and management.

3(B) Review and interpretation of shoreline changes in Poole Borough

Background
It is important to review the shoreline changes that have taken place along the shoreline of Poole Borough over the last 100 years or so, since this information often gives useful guidance to potential continuing or future erosion problems.

On the Poole Bay frontage, the erosion rates before installation of coastal promenades and seawalls will allow an estimate to be made of the possible consequences of not maintaining such structures, an important "baseline" calculation required in the justification of any future major improvements or changes to those works.

Within Poole Harbour, the direct influence of coast protection works on changes in the shape of the estuary is probably smaller, but natural variations are more marked, and more difficult to predict. An examination of the effects of the various reclamations along the Poole Borough side of the Harbour is also important in the context of possible future works to protect property or prevent flooding.

Methods
The review of coastal changes over approximately the last 100 years will largely be based on an analysis of Ordnance Survey maps. Changes in the

position of the cliff top, the high water mark and the low water marks will be examined around the coastline. These will need to be compared with the dates of construction of coast protection works in order to understand the patterns of erosion (or accretion).

In addition to the comparison of maps, we will also undertake an analysis of beach profile information for the Poole Bay frontage. Some of this data is available from Poole Borough Council surveys (as noted in the Terms of Reference). Other information has been collected by British Petroleum, and will be available for this study. Finally it is hoped that beach profile data collected for almost twenty years along the adjacent frontage within Bournemouth can also be used in the present study.

The interpretation of the observed changes in the coastline will involve the results of the longshore drift calculations (see Section 2E), and a description of the impacts of the various protection works that have been installed over the years.

Outputs

Figures summarising the changes in the shoreline around the Borough will be presented in the final report along with the results of analysis of those changes (eg historical cliff erosion rates).

Areas of continuing erosion, accretion, or marked variability will be identified and commented upon.

Explanations for the observed changes, and what they mean for future management of the coastline, will also be included in that report.

4. DEFINING COASTAL "UNITS" ALONG THE POOLE BOROUGH SHORELINE

4(A) Review of land use, beach types and hydraulic regimes

Background

The future management of the coastline of Poole Borough will not be considered on a metre-by-metre basis, but rather by defining sections or "units" of perhaps 1000m to 3000m long within which a uniform method of management can be applied. This type of approach has been adopted in previous management studies, on the east coast of England. Typically each unit will have a consistent type of beach and hydraulic regime, and usually the land use (eg industrial, residential) behind the shoreline will also be homogenous. By defining these main parameters along the whole coast, a useful guide to the boundaries of these management units can be obtained.

Methods

Information on land use, and on the level of the hinterland (eg cliff top, below highest tides) will be provided by the Council. Information on the hydraulic regime for each stretch of coast will be available from the modelling described in sections 1 and 2 above. Information on the beach types around the Borough is already available from photographs, maps and previous site visits. All this information will be drawn together and summarised at locations about 50-100m apart around the whole coastline.

Outputs

All the above information will be collated using a spread-sheet, which will form an input to the definition of coastal "units" (see Section 4C).

4(B) Defining management "units" for the Borough's coastline

Background

When defining the coastal "management units" precisely, it is important to understand any interactions between various parts of the coast. It is generally important to consider the impacts of coast protection works within a natural coastal "cell" or "sub-cell", as recently proposed in an HR report to MAFF. For shoreline management in Poole Borough, impacts may be totally within the Borough, o @ they may affect beaches in adjacent authority areas. In the latter case any management strategy will need to involve those authorities. Even if the stretches of shoreline affected are entirely within the Borough, agreement may have to be reached with frontages (eg the Harbour Commissioners) before achieving a practicable management plan.

Methods

Natural or man-made "breakpoints" along the Poole Borough coastline will need to be identified. These may be large groynes, breakwaters, or other features which disrupt the coastal processes, eg interrupting the movement of sediment along the coast. This will be achieved by a combination of examining maps and inspecting the whole coastline.

Where the coastline changes character without such breakpoints, then there is a danger of the management of one part, or "unit", of the coast affecting another. These dangers also need to be identified, before the "unit" boundaries are defined.

Outputs

The various natural breakpoints along the coast will be identified on maps, and this information used in the following section of the study.

5. REVIEW OF EROSION AND FLOODING RISKS

5(A) Review of historical damage and flooding events

Background
It is always useful, during a shoreline management strategy, to gather information on past events which have caused flooding and damage to the coastline. By providing concrete examples of such events, which are often soon forgotten, it is much easier to explain the ideas behind the joint probability study, and the subsequent derivation of design conditions for assessing coastal defences. Any information on the damage caused is also valuable in the future assessment of the extent of the benefits of a proposed coast protection scheme.

Methods
Some guidance on the likeliest dates for such past events, at least in recent years, will be provided by the analysis of tidal levels. Further information will be sought from the Council itself, the NRA (Wessex Region), and perhaps local residents and businesses. The exact day and time of such events will be useful in the testing of the present defences (see below), since it should be possible to estimate the tidal levels and wave conditions which caused the problems.

Wherever possible, the extent of any flooding or damage will also be determined for use in future benefit appraisals. In previous studies, however, this has proved to be a particularly difficult task.

Outputs
It is hoped to provide a useful body of information regarding the major coastal protection and flooding problems experienced in Poole Borough by this examination of past events. This will also help to 'calibrate' the numerical models to be used in the following part of the study, which will assess the standard of protection presently provided by the coastal defences around the Borough.

Finally, by presenting this information it is hoped that the extent of any problems can be made clear to the public, and will hence help justify the need for a shoreline management strategy for Poole Borough.

5(B) Review the of existing defences

Background
An important part of the proposed strategy study is to take stock of the present coastal defences in Poole Borough. Not only does the present standard of protection they provide need to be assessed, but the possible changes in this standard in the medium term future also have to be considered.

This information is of primary importance when the priorities for instigating shoreline management plans are drawn up at.a later stage in the study.

Methods
The assessment of the present standard of coastal protection around the shoreline of the Borough will fall into two main parts. First it will be necessary to gather information on the existing defences. ie their type, cross-section etc.

In this we will be helped by the records held by the Borough Council itself, and the recent Coast Protection Survey commissioned by MAFF. It is anticipated, however, that site visits will be undertaken to inspect the defences at first hand during the study. This will assist in the estimation of the likely changes in the condition of those defences in coming years. and more particularly in the beach/foreshore levels immediately in front of them, which have a considerable impact on their performance in storm conditions.

The next step will be to calculate the efficiency of these various defences using numerical models of their capacity to prevent over-topping by waves. In this part of the study, we will use the information on design wave and tidal conditions derived at earlier in the study. Where sensible to do so, we will also investigate what changes in the existing structures would be required to increase their performance to achieve an acceptable standard of defence. This may involve, for example, adjusting the crest levels of seawalls or the level of the beach at the seawall toe, in the numerical modelling exercise.

We will also make allowances for the potential impacts of climate changes, eg sea level rise. This will be carried out by adjusting the extreme tidal levels used in the testing of the defences, or by assuming severe storms will occur more frequently in the future.

<u>Outputs</u>
The review of the existing coastal protection around the Borough's shoreline will provide:

> An appraisal of the present standards of the defences against flooding an damage to property, indicating areas where that standard is lowest.
>
> Information on the increases in dimensions of existing defences, or in the foreshore level directly in front of them, to provide an acceptable standard of defence.
>
> An assessment of the sensitivity of the erosion/flooding risks to climate change, particularly to sea level changes.

5(C) Consequences of a "do nothing" approach

<u>Background</u>
One of the most important issues to be considered in a coastal strategy study is the consequences of <u>not</u> intervening. It may well be that in some areas the present defences will continue to be entirely adequate for the foreseeable future, and these stretches of coast require no more than periodic monitoring. In this sense, "adequacy" is taken to mean providing the appropriate level of protection, which on certain stretches of shoreline may be rather less than provided today.

Equally well, from the knowledge of present erosion rates, of the performance of existing defences, and the sensitivity of that performance to a modest rise in sea level, it may be clear that the "do nothing" option will rapidly result in unacceptable flooding and/or permanent loss of land. An appraisal of the consequences of not intervening in the medium term future (ie up to 10 years) is therefore a first step in assessing the need for, and type of management needed.

Methods
For each management unit, a number of factors need to be taken into account. These include

the present and historical rates of shoreline change
the calculated present performance of the defences
the residual life of the defences
the past history of flooding or damage
the sensitivity of the defence standards to climate change
the land use, and value, behind the defences.

A full economic evaluation of the "do-nothing" option would require each of these factors to be carefully looked into, and quantified, and this is well beyond the scope of the present study. It will be possible, however, to provide a reasoned argument about whether or not such an approach is safe in the medium term future (ie the next ten years). Depending on the particular frontage, the most crucial factor will differ. For agricultural land, for example, the likely costs and environmental impacts of installing defences may outweigh any concerns about an increased risk of flooding. Conversely, for a low-lying business area already experiencing regular flooding, there is a strong chance of the situation worsening in the coming years.

Outputs
For each defined "management unit" of the Poole Borough coastline, we will provide a statement of the likely acceptability of a "do nothing" approach to shoreline management over the next 0-1 0 years. For the frontages where such an approach is not appropriate, we will then proceed to an analysis of alternative management options.

6. ASSESSMENT OF MANAGEMENT OPTIONS

Background

Following the review of the Borough's shoreline, the areas where improvements to the coast protection are required will have been identified. There are an ever increasing variety of techniques available to the shoreline manager for this purpose, ranging from traditional "hard" structures such as seawalls to novel methods such as managed retreat and beach drainage systems. In this section of the study, therefore, we propose to provide guidance on the options most suitable for Poole Borough.

It is not appropriate to attempt to design or price alternative options during the present study, or to make firm recommendations for any particular coast protection scheme. The opportunity should be taken, however, to assess the suitability of the various alternative techniques for each "management unit".

A number of factors have to be taken into account in such an exercise, for example the costs of a scheme and its potential impact on other parts of the coast. Because the character of the Borough's shoreline is so varied, it is likely that a suitable method on one frontage may be entirely inappropriate on another. An overview of the methods available to protect the coast, and of the most promising options for each "management unit" is therefore required.

Methods

We will employ a multi-criteria analysis method to rank alternative shoreline management options, for those stretches of coastline where the "do nothing" option seems likely to be unacceptable. This type of analysis has been successfully applied in other coastal strategy studies, and provides clear, well-reasoned guidance on the most promising options for protecting each part of a coastline. The first step is to establish suitable assessment 'criteria'. These will include, but not be limited to:

Expected scheme life
Capital costs and maintenance commitments
Effects on adjacent lengths of the coast
Impacts of the scheme on the natural environment
Public acceptability (character, aesthetics, safety)
Effects on access to the beach/sea and on navigation
Whether the scheme is well-proven or 'novel'

Of all these criteria, the likely costs and the potential impacts on the environment are usually the most critical. This will require consideration of aspects such as the delivery of materials and the methods of construction, as well as the character of the finished scheme. In Poole Harbour, the possible damage to the fragile saltmarsh and mud flats are likely to be seen as a major issue in proposals for coastal protection works.

The next stage in the analysis will be to draw up a list of shoreline management options which may be useful for the Poole Borough coast. These will include

Managed retreat, ie allowing some areas to become tidally flooded

Maintaining, modifying or re-building existing defences, for example

Increasing seawall crest levels
Improving beach levels in front of seawalls
Adding energy absorbtion to seawalls or groynes

Construction of new defence structures, either
At or close to the existing high water line
Landward of existing defences or of the high water line
Seaward of the existing defences or of the high water line

Beach management methods such as
Nourishment, perhaps with periodic 'top-up" operations
Re-cycling of beach material along the coast (to counter drift)
Re-grading or re-profiling of beaches (periodically)
Installing beach drainage systems

Installing beach control structures, eg groynes, breakwaters, cills.

Finally, for each management unit, the various options will be ranked using the appropriate criteria for that unit. This will certainly eliminate many of the possible management techniques, and will usually suggest one or two methods which appear to be suitable for the particular frontage. We will keep in close contact with Borough Council staff during this analysis, to ensure that local sensitivities are taken into account.

Outputs

The multi-criteria analysis will Provide a list of the most promising shoreline management options for each "management unit" around the coastline of the Borough. This does not mean, of course, that intervention in the present development of the coastline will be required in all areas. Indeed it is likely that continued monitoring is all that will be required in many areas.

7. RECOMMENDATIONS FOR FUTURE ACTION

Towards the end of the study, we will begin to formulate recommendations for future action, arising from the study findings. These recommendations will cover, at least, the following topics:

1) After consultation with the Borough Council staff, we will draw up a priority ranking for required shoreline management along the Bay and Harbour coastlines. For each "management unit", the urgency of any improvements to the coastal protection works will be ranked, bearing in mind the present standard of protection, the likely consequences of "doing nothing" and the possible impacts of climate change in coming years.

2) We will provide an assessment of the information available to assist in the optimal management of the Poole Borough shorelines, and on the need for any changes to the present monitoring arrangements. Post-project appraisals will soon become essential for coast protection schemes, and establishing a suitable set of information prior to any works is likely to accelerate any central government grant aid provision.

3) If necessary, we will recommend any further studies or investigations following on from the present study to improve proposed management methods.

4) Where the proposed management strategies for any part of the coastline will profit from an agreed approach with other interested organisations, such as adjacent local authorities, we will indicate the main issues to be tackled.

Appendix 2

The HINDWAVE wave hindcasting model

Appendix 2 The HINDWAVE wave prediction model

Introduction

The HINDWAVE model was developed to meet the needs of coastal engineers for large quantities of wave data at specific sites, quickly, and at low cost. It strikes a good balance between sophistication and price. The input to the model consists of two sets of variables, one representing the shape of the wave generation area and the other the changing wind velocity in that area. The programs are written in an efficient way enabling long sequences of wind data to be processed rapidly.

The model may be used on its own in order to estimate a directionally dependent wave climate distribution, or it may be used in conjunction with measured wave data. Once calibrated against recordings, it can be used reliably at other nearby locations and with other or longer periods of wind data. For example this procedure can be used to put a single year of wave records into a longer term perspective, or to predict wave conditions at a point nearby which has a different exposure to wave action. It is certainly better to make design decisions based on 10-15 years of synthetic wave data, rather than just one year of measurements. In addition, the model adds the important wave direction parameter which is not usually available with measurements. This is particularly valuable for coastal or harbour engineering projects.

The model has been used successfully on about twenty recent projects at various sites around the British coast, and in several cases has been proved against measured wave data. It is suitable for offshore use at most coastal sites where a directional wave climate and extremes are required, whether or not wave recording is also undertaken. It does not include shallow water effects, but may be used to provide input to a refraction model to provide nearshore wave conditions. The model is dependant upon having a long sequence of high quality hourly wind velocities, which are assumed constant across the wave generation area. The calculations do not include an estimation of long period swell, which may render the model unsuitable for use in certain areas of applications.

The HINDWAVE model

The computations are split into two main parts. The first consists of production of a table of about one thousand possible wave conditions, derived from a similar number of specified wind conditions. Fetch or open water rays are measured at, say 10° intervals around the wave prediction point for use as input to the first element of HINDWAVE, which is the JONSEY wave generation sub-model described below. The second part consists of analysis of hourly wind speed and direction records. For each hour in the sequence, the wind (and hence wave) condition most closely corresponding to actual wind activity at the time is chosen from the table. The analysis requires measured wind data collected at hourly intervals over a period of several years.

The JONSEY program is used to assign a particular significant wave height (H_s), peak period (T_p) and mean direction (θ) to each member of a particular set of wind conditions. The set comprises all possible combinations of values of wind speed, direction and duration sufficient to cover the range expected at that location. The predicted heights, periods and directions are stored as a look-up table for use by the second stage of the HINDWAVE model.

Hourly wind speeds and directions are obtained from the Meteorological Office in the form of a computer data file. For each hour in turn, the model determines, for the chosen group of durations, the dominant set of wind conditions at the prediction location, with reference to the H_s table. This is achieved by vectorially averaging the wind velocities over the various chosen durations leading up to that time in order to obtain an average speed and direction for each. The largest value is then selected from the corresponding set of H_s values. This figure is retained together with the appropriate peak period and wave direction.

Suitable wind data is available from coastal stations only, which may not be representative of conditions over the sea. A speed-up function is necessary, which may be dependent upon both speed and direction. Calibration of the model usually takes the form of adjustment of this factor. Its magnitude may be determined by a general examination of the anemograph site, or by comparison with offshore wind frequencies, or by a comparison between predicted and measured wave heights, if available.

An additional option is to analyse storm persistence as the hindcasting proceeds. For this purpose a storm is defined as beginning when a give threshold H_s is exceeded and ending when H_s first subsides below that level. Variations in wave severity within a particular storm are not recorded. The results are tabulated as numbers of storms of various durations for several different threshold levels.

A further option is automatic extrapolation to extreme wave heights, for different direction sectors, based on the overall predicted distribution of H_s. The results are tabulated for various directions and return periods.

The JONSEY model

It is observed that wind generated waves show some directional spreading about their mean direction of propagation. To incorporate this effect in the model, components of the total wave direction spectrum are calculated for directions either side of the mean, and then a weighted average is taken using a standard spreading function.

The component directions (i = 1 to n) are spaced at regular intervals ($\Delta\theta$) in the range ±90° from the mean (θ_o), the mean JONSWAP equation (1973) representing a growing wind sea, is used to define the spectrum (E_i). The summation of the component spectra is then performed using the Seymour equation (1977), which includes the cosine-squared directional spreading function.

If the fetches are measured at say 10° intervals, then the effective wave spectrum (E) for a particular direction is calculated as the weighted average of seventeen component spectra (E_i (θ_i), θ_i = -80°, -70°, ..., 80° for i = 1, 17), as indicated in equation (1).

$$E = (2\Delta\theta/\pi) \sum_{i=1}^{17} E_i \cos^2 (\theta_i - \theta_o) \qquad (1)$$

Although it is not part of the original theory, experience at HR indicates that cosine-sixth, or exceptionally cosine-thirtieth, is sometimes a better spreading function to use. These alternatives are useful when the wave generation area is unusually narrow or the peak period is unusually long.

The three main wave parameters, H_s, T_z and θ are found by numerical integration of equation 1. T_p is the peak period of the one-dimensional wave spectrum. The directional spectrum, as a discrete function of both frequency and direction, may be retained for transfer to a wave refraction program, or elsewhere.

Typical results

(i) Hourly listing or graphs of H_s and T_p.
(ii) Monthly scatter diagrams of H_s and θ.
(iii) Seasonally and annually averaged scatter diagrams of H_s and θ.
(iv) Tables of storm persistence.
(v) Extremes analysis for various direction sectors.

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Appendix 3

Prediction of extreme wave conditions

Appendix 3 Prediction of extreme wave conditions

There are several different methods of estimating extreme events from limited data. They are based upon the idea of fitting a standard probability distribution to the range of data which is available. The extreme wave heights are then obtained by substituting the corresponding extreme probability levels into the fitted equation.

For this approach to work properly, the data should be a representative sample, for example one year of continuous record, and not be unfairly weighted in favour of one particular time of the year. In addition, the probability theory demands that the recorded events be independent. A suitable method is to use a large number of regularly measured H_s values and to assume that the lack of independence between neighbouring values will be overcome by virtue of the volume of data involved (Reference 1).

The three-parameter Weibull distribution (Equation 1) has previously been found to be the most reliable and consistent method of fitting distributions of wave data. The parameters of the distribution are calculated after plotting the various exceedence levels on Weibull scaled graph paper (Equation 2), and drawing the best fit straight line through the points. As a check, this procedure is reproduced by a computer program and the results compared.

Extreme Value Distribution

$$P(H_s) = 1 - \exp[-\{(H_s - a)/b\}^c] \qquad (1)$$

where H_s = significant wave height
P = probability less than H_s
a, b, c are parameters to be found

Weibull Scales

$$\log\{-\log(1-P(H_s))\} = c\,\{\log(H_s-a) - \log b\} \qquad (2)$$

$y = \log\{-\log(1-P(H_s))\}$ x and y are plotted
$x = \log(H_s-a)$ on linear scales

Waves of a given return period (N years) are determined graphically from the appropriate probability. In order to calculate the correct probability, it is necessary to set the duration or persistence of the return period event. For example, if three hours were chosen (as in this study), there would be a total of 2922 three hour periods per year, and the probability of the 10 year return period event would be:-

$$P(10\text{ year event}) = 1 - 1/(10 \times 2922)$$

$$= 0.9999658$$

Note that the expected highest individual wave (H_{max}) in a sequence is related to H_s by the approximate formula:-

$$\frac{H_{max}}{H_s} = (\tfrac{1}{2}\ln N)^{\frac{1}{2}}$$

where N = the number of waves in the sequence

Reference

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Appendix 4

The OUTRAY Wave Refraction Model

Appendix 4 The OUTRAY wave refraction model

Introduction

The OUTRAY wave refraction model was developed to enable wave activity at a coastal site to be predicted from offshore wave data. The model uses a reverse tracking ray method which is economical to run and can deal with complex bathymetries. It takes as input a directional wave energy spectrum and calculates a wave spectrum which gives the distribution of wave energy with frequency at an inshore point. This is an advantage over other wave refraction models which take only a single offshore component and calculate a representative inshore wave height and direction.

OUTRAY may be used to calculate wave conditions required for the design of coastal engineering projects such as harbours, sea walls or offshore breakwaters. In such studies the offshore conditions which give the worst inshore wave heights are not usually known in advance. It is therefore necessary to study a large number of different wave conditions. For this the computer model has to run as inexpensively as possible, with a minimum of manual calculation. Therefore the reverse tracking is ideal for most studies of specific sites.

The model has been used successfully at numerous sites both around the UK and in other parts of the world. It was specifically designed to represent the effects of depth refraction and shoaling and does not normally include the effects of currents and energy dissipation processes. In many cases this will not be a significant drawback and the model will give accurate predictions of inshore wave conditions, even in very shallow water. If necessary the effects of currents on wave refraction can be incorporated into the model, but it will require separate model runs for each configuration of current strengths and directions being studied. Where the seabed is extremely rough neglecting frictional effects will overestimate wave heights at the shoreline. The reverse tracking ray method will, however, still identify the critical offshore wave conditions. For these conditions a more expensive forward tracking or finite difference refraction model, which includes the effects of wave breaking and friction, can be used to give accurate results at the point of interest.

The back tracking wave refraction model

The computations for the OUTRAY model are split into two parts. The first stage involves generating a set of functions which describe the transfer of energy between an offshore area and an inshore point. The second part of the model uses these functions and offshore wave data to calculate inshore wave conditions. In this section the theoretical background of this process is described.

As with any mathematical model certain assumptions need to be made in order to simplify both the theory and the calculation method. In this case the first such assumption is that in the offshore area of interest a wave energy distribution $S(\theta,f,r)$ exists, where θ is the wave direction f the wave frequency and r a position vector. In a typical situation the wave energy in deep water will depend only weakly on r. On the outer boundary of the area being considered, it is thus assumed that a homogenous sea state exists and is described by $S_o(\theta,f)$, the wave energy being considered to depend solely on direction and frequency. (The subscript o is used to denote quantities offshore.) The purpose of the wave refraction method is to provide information on the wave conditions, or energy distribution at some point close to the shore $S_p(\theta,f)$, for a variety of offshore conditions, ie, different values of $S_o(\theta,f)$.

In order to link the energy distribution offshore to that inshore we need to consider ray paths. If a ray starts from the outer boundary of the area with direction θ_o and frequency f_n and reaches point P with direction θ_p and frequency f_n then the functions S_o and S_p are linked by:

$$S_p(\theta_p,f_n) = \mu(f_n)\ S_o(\theta_o,f_n)$$

Here $\mu(f_n)$ is a function of the phase speed and group velocity at the offshore boundary and the inshore point. Both quantities can be found if the depth at the outer boundary and the inshore point are known. Provided that enough rays can then be found linking the offshore boundary with the inshore point the above equation can be used repeatedly to build up a picture of $S_p(\theta,f)$ for any function $S_o(\theta,f)$. To find such rays would be difficult if it were necessary to start at an outer boundary. Fortunately, the paths of rays, like those in optics, are completely reversible and this makes the task relatively straightforward.

To start the process a variety of wave frequencies are chosen. In a typical study these would lie in the range 0.05Hz-0.30Hz, and there would be between ten and fifteen frequencies selected. for each frequency a fan of rays is sent out from the point of interest. Each ray is separated from its neighbour by a small angular increment. The rays are then tracked seawards from the inshore point until they run ashore or reach the outer boundary. The rays paths are determined using Snell's law at each cell of a triangular mesh, the seabed depths being specified at the grid intersections. The wave rays are followed through each cell from some given entry point and direction. As the ray leaves one cell, its position and direction become the entry conditions for its path across the next. The results from this stage of the operation take the form of a list of those rays which connect the point to the boundary, for each ray its frequency, f_n, its direction of leaving the point, θ_p, and its direction at the outer boundary, θ_o, are specified. Typically this list contains information on several thousand rays.

For convenience this list is converted to three matrices which are called 'transfer functions', because they contain all the information necessary to evaluate the transfer of energy from the outer boundary to the inshore point. Each matrix column represents a frequency band and each row a sector of offshore wave directions. The transfer functions therefore contain the information on how the seabed topography changes the waves of each frequency and direction. These transfer functions apply to a specific point for a single tidal level. The model can of course be rerun for other points and different tidal levels.

Having set up the transfer functions we then move onto the second stage of the model. The offshore wave conditions at the site are assembled in similar matrix form, with the wave spectra split into intervals of frequency and direction. It would then be possible to completely evaluate $S_p(\theta,f)$, the energy distribution at the inshore point. However, all that is normally required is the mean inshore direction and directional spread of the waves together with the distribution of energy over frequency, which will allow the derivation of the inshore significant wave height (H_s) and mean zero crossing period (T_z). All of these parameters can be found by a series of relatively straightforward operations involving the transfer functions and the matrix of offshore wave conditions. This method of calculating wave refraction makes it very economical to compute wave conditions at a site for many different incident wave spectra.

Typical results

(i) Tables of H_s, T_z and mean direction at an inshore point for a variety of offshore conditions. For example, the data for the wave refraction model may correspond to extreme offshore wave conditions for a particular area for given return periods.

(ii) Inshore frequency spectra for a set of specified offshore conditions. Information of this type would normally be required as input to a numerical harbour model or a mathematical model of beach processes. In addition this information would also be needed at the wave paddle positions in a physical model in order to generate the correct random wave sequence for design studies.

(iii) The model can also be used in conjunction with the HINDWAVE wave prediction model to generate an annual inshore wave climate at a particular site for use in sediment transport calculations.

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Appendix 5

The TIRA tidal analysis software

Appendix 5 The TIRA tidal analysis software

HR Wallingford operates the standard TIRA tidal analysis software under licence from the Proudman Oceanographic Laboratory (POL). The following description is based on lecture notes provided by POL.

The process of analysis is one of reducing a set of measurements, which in the case of tides amounts to approximately 9000 hourly values in a year, to a manageable set of parameters which completely specify the measurements. It is important that the parameters chosen have some physical meaning so that they can extend the knowledge of whatever is being studied.

The oceans generally respond to forces such as the gravitational attraction of the Moon and the Sun, or the radiational input from the Sun, or to forces generated by the meteorology, for example the equatorial current, the gulf stream and wind waves. Well over 90% of the energy contained in sea level records is caused by tidal movements. This is easy to separate from the other components because it is coherent in time, whereas, for example, wind waves occur randomly. All methods for tidal analysis make use of this property of coherence.

Although the tides existing on the surface of the Earth cannot be solved analytically, an examination of the forces causing the tides leads some way toward a solution, but not very far. The gravitational attraction of the Moon and Sun on the Earth produces diurnal and semi-diurnal tidal bulges both of which are waves of planetary dimensions. These waves have different frequencies and are usually denoted by the symbols M_1 and M_2 for the Moon and S_1 and S_2 for the Sun. If the Moon and the Sun remained permanently on the equator then only the constituents M_2 and S_2 would exist, and in practice they do dominate most tidal records. There are some areas where the tides are predominantly diurnal, and some where the regime is mixed, ie the diurnal is a respectable proportion of the semi-diurnal tide.

The Moon and the Sun are only above the equator twice a month and twice a year respectively, leading to a more complex form of the tidal potential in which the main constituents are modulated at periods of 1 month, 1 year, 8.85 years, 18.61 years and 21,000 years. So although tides are regular they are only repeatable over a very long time scale.

The tidal potential so far discussed explains only the diurnal and semi-diurnal species of the tide and can be extended to include other tides. However, the spectrum of a tidal record shows clearly that higher order species exist except where the measurements were made at an oceanic location. These compound tides are generated by the linear tides as they propagate into shallow water or as they encounter frictional forces. They are given names such as M_4 to denote a multiple harmonic of M_2, or MS_4 to denote interaction between the M_2 and S_2.

So far the proposed model for the tides on the surface of the Earth is based upon the forces that generate them. However, a comparison of the equilibrium tide at, say, the latitude of Liverpool with the measured tide at the same place, shows little similarity. The equilibrium tide is much more diurnal than is the observed tide at these latitudes. This is not unexpected if one looks at the progression of the equilibrium and observed tide in the north-east Atlantic.

Whereas the former should propagate from east to west the latter propagates in a northerly direction. The difference is because the dynamics of the ocean have not been taken into account in the derivation of the equilibrium tide.

The model that has been derived for the equilibrium tide is not completely without use because it does provide a knowledge of the frequencies of the tidal constituents and of their relative amplitudes, giving an idea which constituents will be important in the real tide. If these constituents are combined with certain amplitude and phase relations, the profile of the observed tide can be generated. For example from a combination of M_2 and M_4 the double high water seen at Southampton can be demonstrated. In the latter the phase relationship between M_2 and M_4 is important. If these two constituents are in phase with one another then a stand in the tide following high water is produced. If they are out of phase by quarter of a cycle then this creates the double high waters.

Analysis is the direct opposite, consisting of decomposition of a complex record into a simpler set of components which in the present model of the tides are harmonic constants. For each location there exists a unique set of harmonic constants which define the complex tidal regime at that place.

In the harmonic method of tidal analysis the objective is to fit the best possible sinusoidal wave of each frequency to the measured data. Harmonic constants are invariate with time in that they do not depend on the epoch when the tide was recorded. Also, although they are derived for a particular place, the propagation of each constituent through an area tells something of the tidal dynamics. The least square procedure of TIRA relies on the ability of the computer to solve large sets of simultaneous equations involving the required harmonic constants.

Harmonic analysis consists of fitting a finite number of harmonic constituents to the data and minimising the residuals by the principle of least squares. The constituents, being sinusoidal in time, then fit the coherent part of the signal and leave behind any non-coherent energy, say from meteorological forcing. The analysis relies on the correct assessment of which constituents to use since the energy belonging to a constituent which is not included will be spread across neighbouring constituents.

In short-term analyses involving less than 1 month of data, because surge energy may not be totally random over this period, it is possible that some of the surge energy will be absorbed by the harmonic constituents, which adversely affects the constituents and leaves an artificially small residual. So some care must be exercised in tidal analysis.

Running TIRA correctly is largely a matter of experience. For each different length of data a different number of constituents must be used and a variable number of related constituents must be included.

Estimating that an analysis is correct is largely concerned with knowing if the results make sense for the region from which the data was extracted. This can be helped by examining the ratios between various constituents eg M_2 and S_2 and calculating the age of the tide, both of which have some regional stability. The standard deviation of the observed sea level and of the non-tidal residuals are output by the program. If the latter is unusually large then it may indicate some errors in the data. An examination of the non-tidal residuals, especially by plotting, will help to confirm this.

In general a ratio between residual and observed standard deviations of 0.1 is indicative of a good analysis. However, the ratio may vary considerably from this value. In shallow areas the residual standard deviation will be large (25cm say) and the above ratio will vary correspondingly. Also, near an amphidrome it will increase because the tidal range is low.

The standard deviation represents a statistic of the oceanic noise associated with observed data and it should have a value representative of the site. Therefore it will remain reasonably constant from year to year but may vary monthly due to the seasonal distribution of weather patterns. Its value may also rise systematically as surges propagate into shallow water and become amplified.

The major decision in tidal analysis is how many constituents to derive from a given length of data, and how many constituents to relate and where to find the relationship. Although the lengths of data below give guidelines, one is seldom in the fortunate position of having an exact length of data that suits an exact set of constituents. There are certain synodic periods in the tide which make the lunar and solar tides commensurable and these periods, and multiples of them, are useful in tidal analysis eg 29.53 days (synodic month) and 355 days.

Length of Data	No of Constituents	No of Related
1 year	60	0
6 months	54	5
29 days	26	8
15 days	15	15
1 day	5	0

Whether adjacent constituents in the tidal spectrum can be separated is a function of the data length. A guide to the number of constituents separable from different data is given in the table above, but the actual number is dependent on the background noise continuum associated with the data. If this is low then constants may be separated from data which is theoretically too short, but it is well to be wary of trying to extract too many harmonics from the data. One exception to this is numerical model output where the noise is purely rounding errors. In this case harmonics can be extracted from about one-quarter of the theoretical length.

All constituents of a group which are not separable must be included in analyses, unless their amplitude is small, otherwise the results will be in error. In the harmonic method this is done by relating the amplitude and phase of the smaller constituents to the largest in the group.

Appendix 6

Recommendation on management of geological SSSIs

Appendix 6 Recommendation on management of geological SSSIs

The sedimentary cliffs on PBC's frontage are rare exposures of tertiary and quaternary deposits. In recognition of their geological significance the most important exposures have been designated as Sites of Special Scientific Interest. Potential conflict exists between permitting ongoing erosion which is necessary to maintain good exposures and ensuring public safety and protecting properties at the top of the cliffs.

Before construction of the promenades, which now extend along the entire length of the cliff toe within PBC boundary, wave undercutting was a principal mechanism of erosion. Now that wave erosion of the cliff toe has been prevented, management of the cliffs is essentially a geotechnical rather than a hydraulic problem. Recommendations on cliff management have therefore been included in this Appendix rather than in the main text which deals with the active coast. Moreover, the recommendations are general in nature since specific study was beyond the scope of this study.

Instability of the cliffs is now attributable to surface weathering due to saturation of the sediments at times of heavy rainfall, and, of larger scale significance, to perched water tables at clay layers leading to locally reduced effective stresses. The natural cliffs have an overall angle of about 50°, steepening to 60° to 80° in parts, yet previous research suggests that at this angle the factor of safety against slope failure can, under certain conditions, be significantly less than 1.0 (Golder Associates, 1990).

The consequences of continuing erosion are site-specific. Where properties or roads are close to the cliff edge the need to resist erosion is urgent. Elsewhere there are wide gardens between properties and the cliff edge. Nonetheless, the value of land on the edge of Poole Bay is high and whilst actual properties may not be lost, the economic consequences (in terms of reducing property value) of continuing erosion are significant along the entire cliff edge. It is worth noting that since construction of the promenades, cliff retreat rates have been very slow indeed (see Section 3.3.3). At most sites it should be possible to permit slow erosion to maintain the quality of exposures whilst not threatening cliff top properties.

Erosion of the cliff is also of consequence at its toe. Consequences range from a maintenance requirement to clean weathered sediments off the promenade, to the remote, but possible, safety risk associated with sudden, large-scale cliff failure.

In the past a number of techniques have been adopted to manage the coastal cliffs:

(i) Regrading to a less steep slope which has then been vegetated has been carried out at some sites.

(ii) Bournemouth Borough have installed a number of vertical drains to reduce pore pressure buildup and drain perched water tables. It is reported that these drains reduced seepage from the face by 50% (Golder Associates, 1990).

(iii) Walls have been constructed at the cliff toes to contain weathered sediment and minor slips.

In view of the success or otherwise of these approaches it is recommended that:

(i) Regrading should not be adopted except as a last resort at locations where irreplaceable property or infrastructure is under immediate threat.

(ii) Monitoring of cliff recession rate and slope should be carried out on all eroding cliffs, forming the basis of a planned reactive cliff management strategy.

(iii) Consideration should be given to the potential use of vertical drains at critical sites where cliff top properties are threatened with erosion (this should be the subject of specific geotechnical study).

(iv) At Canford Cliffs where there is no toe wall, one should be constructed, separated from the cliff by a collection zone between 2 and 5m wide. This will serve to prevent sand from washing onto the promenade. It should be high enough to prevent access to the cliff toe by the public.