West Bay Coastal Defence and Harbour Improvement Scheme

Physical Model Study

Report EX4064
January 2000

HR Wallingford

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Contract

This report describes work commissioned by West Dorset District Council whose representative was Mr Keith Cole, Engineering Manager. The HR Wallingford job number was CER2659. The HR Wallingford Project Manager for the physical model tests was Dr Phil Besley who also wrote the report. The model tests were carried out by Mr Ian Mockett, Mr A Steele and Mr Ian Wilcock. The overall HR Wallingford Project Manager for the ongoing studies at West Bay is Eur Ing Paul Sayers.

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Summary

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In 1997 West Dorset District Council (WDDC) commissioned HR Wallingford to undertake a series of preliminary studies into the design of a new coastal defence and harbour improvement scheme for West Bay. The first stage of these studies involved a series of desk studies firstly to understand the physical processes acting at the site and secondly to propose alternative methods of achieving an optimal improvement to the existing coast defences and harbour layout.

In April 1999 HR Wallingford were commissioned by WDDC to verify and optimise the preferred option identified by the desk studies through the use of physical model tests.

The objectives of these studies were:

a) To investigate the performance of East Beach during both moderate and severe storms.

b) To quantify shingle intrusion into the new entrance to Bridport Harbour (and design mitigation measures as necessary).

c) To assess the movement of river borne fine sediments in the existing and proposed harbour.

d) To assess the stability of the new West Pier armour.

e) To measure wave disturbance in the Navigation Channel and Bridport Harbour and infer impact on navigation and mooring tenability.

f) To quantify overtopping of the west seawall and harbour walls.

g) To investigate armour stability of the preferred West Breakwater design at various stages of construction (i.e. temporary works conditions).

h) Provide wave conditions to support preliminary investigations into the concept of a half tide sill.

The preferred layout identified by the preliminary desk studies includes a replacement West Breakwater for the existing West Pier, a rock extension to the East Pier and a recharged West Beach held by a rock groyne. Navigation into the harbour was improved by widening the harbour entrance.

The physical model tests show that the wave heights within the harbour following construction of the preferred layout are approximately 50% lower than for the existing harbour for all the wave conditions tested. Consequently the overtopping of the inner harbour frontage was reduced by a factor of between 10 and 100.

Summary continued

It was also shown that the new West Breakwater provides additional shelter to the East Beach. The wider shingle beach created as a result of this increased shelter in turn provides greater flood protection to the town of West Bay. Recharging the West Beach also provides an additional defence.

The armour protecting both the West Breakwater and the East Pier Extension was modified during the course of the physical model tests to achieve a design that was considered to be stable under all conditions tested.

Further tests were then undertaken to assess the performance of two construction phase layouts under the 1:10 year condition. These tests show that the scar end forming a temporary roundhead during construction was stable during the 1:10 year storm condition (a compensation weather event under the New Engineering Contract). Wave conditions at the entrance to the harbour were slightly higher for the part constructed shorter breakwater length than for the existing layout, probably due to wave reflections from the unprotected end caisson of the new Western Breakwater.

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1. INTRODUCTION

West Bay (Plate 1) is situated in Lyme Bay between the neighbouring ports of Lyme Regis and Weymouth on the south coast of England (Figure 1). Throughout the history of West Bay the two piers, which provide an entrance to Bridport Harbour through the shingle bank of Chesil Beach, have suffered damage. The Piers have been repaired a number of times. The harbour is the home of 15 vessels engaged in full time fishing activities and also has moorings for another 95 boats used for leisure and recreational pursuits.

In April 1999 West Dorset District Council (WDDC) approached HR Wallingford (HR) to advise on modifications to the layout of West Bay Harbour. The requirements were to reduce wave agitation in and around the harbour and to reduce the frequency of overtopping of the seawalls. Any proposals should also minimise the probability of breaching adjacent beaches and there should be no adverse impact on navigation.

At the start of the project, representatives of WDDC and HR established design wave conditions and discussed various options to improve the coast protection. Desk study techniques were used to assess the potential for each of the options and preliminary designs were suggested. The initial studies showed that three-dimensional physical model effects such as refraction and diffraction dominated the response of the harbour, beach and seawall structures. Consequently, a three-dimensional physical model was recommended to compare the performance of the existing and proposed structures.

This document outlines the work undertaken to assess the wave penetration into the harbour, the hydraulic performance of the existing and modified structures and the response of the East Beach. The design, construction and testing of the models, together with the analysis of the results and the conclusions drawn are discussed in this report.

This report should be read together with the Coastal Strategy Report (Reference 1).

1.1 Terms of reference

The Terms of Reference for the project are set out in a proposal submitted by HR Wallingford to WDDC in March 1999 and may be summarised as:

a) To investigate the performance of East Beach during both moderate and severe storms.

b) To quantify shingle intrusion into the new entrance to Bridport Harbour (and design mitigation measures as necessary).

c) To assess the movement of river borne fine sediments in the existing and proposed harbour.

d) To test stability of the new West Breakwater including the crest, toe, front face, back face and roundhead.

e) To measure wave disturbance in the Navigation Channel and Bridport Harbour and infer impact on navigation and mooring tenability.

f) To quantify overtopping of the west seawall and harbour walls.

g) To investigate stability of the preferred West Breakwater design at various stages of construction (i.e. temporary works conditions).

h) Provide wave conditions to support preliminary investigations into the concept of a half tide sill.

1.2 Outline of report

Following this brief introductory chapter, the remainder of this report is organised as follows. In Chapter 2 there is a short description of the environmental input conditions to the physical model. Chapter 3 discusses the details of the physical model, while Chapter 4 details its calibration. The test procedures and the performance criteria to which the responses of the harbour are compared are detailed in Chapter 5. Results of the tests are presented in Chapter 6 and the performance of the existing and preferred layouts compared in Chapter 7. The conclusions from the study are presented in Chapter 8.

An unedited video of the model tests accompanies this report.

2. WAVE CONDITIONS AND WATER LEVELS

2.1 Selection of wave conditions

Two wave directions were considered for both the existing and the proposed scheme as follows:

- 220°N - The most severe waves (in terms of wave height) approach from 220°N, as the waves move inshore, the waves refract, thus the wave crests tend to align with the coastline. This provides the worst case for stability of the trunk and roundhead of the proposed scheme. Waves from this direction sector also provide the worst case conditions in terms of the performance of East and West Beach; particularly the overtopping of the West Seawall and the potential breaching of East Beach.

- 160°N - The proposed new West Breakwater provides little shelter to the navigation channel and the harbour for waves approaching from 160°N. Tests were carried out from 160°N to assess wave disturbance within the harbour and to investigate the stability of the rear face of the proposed breakwater. These tests focus on severe storm events. In addition, there is concern regarding migration of shingle into the harbour entrance from East Beach. Wave conditions from 160°N represent a severe situation for the east to west movement of sediment. These tests focus on frequently occurring conditions.

2.2 Selection of extreme wave and water level combinations

To ensure all the important breakwater and beach responses are modelled, both low-tide, high water levels, and swell / storm wave conditions have been selected for the following reasons:

1. When investigating the stability of armour on the front face of the breakwater, the incident wave height and the nature of the wave impacts are the most important criteria. As the waves are limited by the depth of water at the site, the water level will influence the wave conditions at the structure. Most of the stability tests have been carried out at relatively high water levels to ensure the largest waves reach the structures, however one low water level case has been used to test the stability of the toe of the armoured structure.
2. To assess the degree of protection afforded to the West Beach frontage and the harbour area, wave overtopping is important, this is also strongly dependent on the water level.
3. When considering the movement of the shingle in the longshore direction frequently occurring events are important, whereas when considering the likelihood of a breach in East Beach it is important to consider extreme wave and water level combinations.
4. For tests on the construction phases shorter return periods are considered. The 1:10 year return period event is used as the most severe test of the part-constructed sections. The significance of this event is that it corresponds to the compensation event threshold, as defined in the new 'Engineering and Construction Contract' (Reference 2).

The combinations of waves and water levels for the two wave directions used are described below.

220°N

For the 220°N direction, eight wave and water level combinations were considered, including two swell conditions and one low water condition to test the stability of the toe of the new breakwater. The details of the wave and water level combinations for direction are given in Table 1.

Prior to the physical model study, the waves and water level combinations (not including swell conditions) for the 220°N wave direction were determined by searching for the maximum wave height reaching the -5mODN contour (where the toe of the proposed new Western breakwater is located). The maximum depth limited wave height was estimated by limiting the wave height to 0.55 times the depth of water. Information on the wave and water level combinations for each return period was provided by the joint probability analysis undertaken during the Coastal Strategy Report (Reference 1).

Information on swell conditions from 220°N was provided by the Swell and Bi-modal Wave Climate Atlas for England and Wales (Reference 3). The extreme conditions were selected for an offshore point in Lyme Bay assuming an average wave period of 16s. As swell waves propagate from the offshore to the boundary of the physical model (-10mODN), the wave height reduces slightly due to wave transformation processes such as refraction. In this case, using the wave transformation model (HINDWAVE), a reduction of 10 percent was observed for mean periods between 14 and 18 seconds. It should be noted that for swell conditions it is not possible to identify a corresponding water level using the joint probability analysis. Therefore, it has been assumed that the MHWS water level is appropriate based on the observation that both conditions are likely to happen when there is a reasonably high water level due to either a high astronomical high water with low surge or a low astronomical high water with large surge.

160°N

Five wave and water level conditions were considered from 160°N (Table 1). Wave conditions approaching from 160°N are generally small and not depth limited, therefore the design conditions have been based on their selected marginal extreme return periods. Water levels were then selected using the joint probability analysis to link a wave height with a certain return period to a water level. The reason for this approach was to ensure a 'worst case' wave disturbance in the harbour entrance and inner harbour.

2.3 The influence of sea level rise

The aim of the model studies was to compare the performance of the existing and preferred layout under present day conditions. The influence of global warming on sea level rise has therefore not been included in the extreme analysis of water levels, i.e. all water levels described within this report are those considered to be applicable to the present day. Assuming the modifications to the harbour have a design life of 50years, a sea level rise of 0.25m (5mm/yr) should be considered, based on the design sensitivity guidance of MAFF (Reference 4). With the inclusion of this increase in water level, the 1:100 and 1:2000 joint probability events approximate to the 1:50 and 1:200 year events respectively. Therefore, the results of these tests have been used to indicate the performance of the design under future sea level conditions.

3. THE PHYSICAL MODEL

3.1 The model layout

The model was constructed in a wave basin measuring approximately 35m by 25m. The general layout is shown in Plate 2. A model scale of 1:45 was used which is sufficiently large to minimise scale effects, whilst at the same time providing sufficient area over which the waves can undergo transformation as they propagate inshore. The model covered an area of approximately 380,000m^2 from the sluice gate on the river Brit in the north to the -10mODN contour in the south, and from the second set of steps on the esplanade in the west, to the cliffs in the east. Seaward of the -8mODN contour a 1:10 approach slope was constructed to the floor of the wave basin, representing the -18mODN contour level. The wave generator was positioned on the flat basin floor. Wave guides were placed to maintain the wave energy up to the area of interest. These guides were angled to allow for refraction over the bed contours close to the harbour.

The moulded bathymetry extended far enough seaward (-10mODN) to ensure that the effects of wave breaking, shoaling and refraction are correctly reproduced as the waves propagate inshore. This ensured that the correct wave heights and directions were achieved at the shoreline. The harbour bathymetry was also modelled in full.

The seabed bathymetry, the harbour quays and seawalls were constructed in cement mortar on a compacted sand fill. Outside the study area the boundaries of the model were covered with wave absorbing material, either shingle embankments or flexible mattressing, to reduce wave reflections within the model.

East Pier and West Pier were moulded in cement mortar. West Pier was constructed in sections so that it could be removed easily during the construction of the proposed scheme and then replaced for the temporary works tests. The levels of the structures were checked using a surveyor's level and staff. The topography, structures and walls were built to an accuracy of +/- 1mm model (0.045m prototype.) The roughness of the navigation channel clad with sheet piling was modelled using timber trusses glued to the inside of each pier.

The mobile beach material of the East Beach was reproduced using anthracite. The armour rock of the existing West Beach and of the proposed new West Breakwater was reproduced from scaled limestone rocks. Details of the beach and rock scaling methods are given in Sections 3.3, 3.4 and 3.5 and Appendices 1,2 and 3.

The sources of the data required to construct and run the physical model are described below

3.1.1 Bathymetry

Two sets of bathymetric data were supplied by WDDC:

- A bathymetric survey produced in 1996 (WWDC drwg wbcds/sumre/sh2/3/2) .
- An indication of the bathymetry inside the harbour from sketches provided by WDDC after discussions with the Harbour Master (Letter to HR Wallingford, 26/4/99).

3.1.2 Topography

The hinterland behind the West Seawall was reproduced as far as the secondary defence wall north of the promenade. The topography behind the East Beach was reproduced accurately as far as the road. The position of buildings and walls were replicated in the model to ensure that the main flood routes passing into the harbour and West Bay were accurately reproduced.

Three sets of topography data were provided by WDDC:

- A site survey undertaken by Cartographical Services Ltd for the Strategy Study in 1997.
- Local surveys carried out by WDDC.
- Ordnance Survey Map (SY4690 & SY4590)

Numerous photographs of the harbour area were also used to provide detail of the site.

3.1.3 West Sea Wall

The existing beach in front of the seawall was modelled as a fixed bed. Both access points were constructed in the model. Information on the West Sea Wall was provided by WDDC in the following forms:

- Photographs of existing walls.
- Cross Sections of Defence Length 3b and 4.
- Aerial Photographs.
- Beach survey.

3.1.4 River Brit and sluice gates

The River Brit flows into the sea at West Bay Harbour through 5 sluice gates. The plan shape of the pond directly behind the sluice gates was placed in the model. The sluice gates were constructed in timber and fixed directly to the moulding. The cross section drawings and plan location of the gates were provided by WDDC.

Flow through the sluices was measured during a site visit to West Bay on 9 April 1990. The discharge rates through the sluice gates remained at an average of 16.27 m^3/s for approximately 60 minutes (Reference 5). The Flood Basin Model used in the strategy study predicted flows between 13 and 20 m^3/s. Therefore, the measured flow was used as the design flow in the physical model.

3.2 Development of the design

The existing layout shown in Plate 2 was modelled to assess the performance of the existing structures and to provide a baseline data set to which the performance of the proposed layout could be compared. The test programme is shown in Table 2. Results gained from the early tests were used to develop the general planshape layout of the proposed harbour.

The preferred layout identified by the preliminary desk studies includes a replacement West Breakwater for the existing West Pier, a rock extension to the East Pier and a recharged West Beach held by a rock groyne. The area between the root of the curtailed West Pier and the proposed West Breakwater was reclaimed and protected by a 1:3 sloping revetment armoured with 1-3t rock armour. This area would provide a storage area and slipway facility.

A typical layout is shown in Plate 3. Small modifications were made to the tip of the proposed West Breakwater and East Pier, however the main details of the layout remained the same throughout Test Series 3, 4, 5 and 6.

Typical cross sections through the proposed West Breakwater during Test Series 3, 4 and 5 are shown in Figures 2 and 3. The cross-sections show a structure designed with a core, underlayer and an armour layer. The Eastern Pier extension is described in Figure 4.

Following Test Series 5 the design of the West Breakwater rubble mound was changed significantly. The West Breakwater was constructed without core and underlayers. The change was considered to ease construction. In addition, the increase in the permeability of the rock armoured structures will reduce overtopping of the West Breakwater during severe storm conditions and increase the stability of the rock armour. The preferred harbour layout is shown in Figure 5. The armour size reduces towards the root of the breakwater where the incident wave conditions are less severe. Typical sections through the West Breakwater are shown in Figure 6. The design of the East Pier extension was also modified and is shown in Figure 7. Smaller 3-6t rock armour was placed at the crest of the East Pier extension, to reduce the possibility of beach users being caught in the large voids of the 10-15t rock armoured structure assessed during Tests Series 3,4 and 5.

3.3 Armour scaling

The primary rock armour used in the model was scaled to ensure the correct reproduction of armour stability on both the existing West Beach and the proposed West Breakwater and East Pier extension. Account has to be taken of the differences in densities between the fluids and armour unit material density used in the model and the prototype. In the case of the rock armour this meant using smaller rock in the model than was suggested by the ordinary geometric scale. The method of calculating the correct scaling for the armour rock is outlined in detail in Appendix 1.

All the rock armour was hand weighed to ensure that the correct grading was obtained. Rock that was excessively tabular in shape (i.e. the maximum axial length was greater than three times the minimum axial breadth) was rejected during this process as the elongated material is likely to exhibit poorer stability characteristics than more angular material. The armour was hand laid and carefully placed ensuring that no excess pressure was applied to the mound thereby resulting in the incorrect reproduction of its stability. The rocks were laid so that no individual unit was proud of the mean surface by more than $0.2D_{n50}$.

The present harbour utilises Dolos units to dissipate wave energy before the waves enters the inner harbour. This feature was replicated with model Dolos units. These units provided the correct reflection and dissipation properties, but were not scaled for stability.

3.4 Rock core scaling

Ordinary geometric scaling of the revetment core material would not have correctly reproduced prototype pressures and velocities in the model. The model material must therefore be made slightly larger in order to replicate the behaviour observed in prototype structures. The procedure used for calculating the size of the model core is outlined in Appendix 2. The material was prepared by sieving. In order to ensure that the correct grading was obtained the material was prepared in size sub-divisions and then mixed in the correct proportions. The core was laid and levelled off using a template and screeding beam. The slope angle of the revetment was then re-checked using templates

3.5 Scaling of beach material

When modelling any beach sediment the three main requirements are to reproduce the beach permeability, the sediment mobility, and the relative onshore/offshore movement. It is very unlikely that all three modelling requirements can be achieved simultaneously. Indeed, some compromise is always necessary in the selection of the theoretical characteristics of the model material.

The material used to form the mobile bed of the East Beach was anthracite coal which has been used successfully in a number of other physical model studies. Details of material selection are given in Appendix 3. The model sediment was scaled for onshore/offshore movement, and beach permeability. This was preferred because of the importance of onshore/offshore movement of material around the existing and extended East Pier. The threshold of the beach movement was increased (i.e. the longshore movement of beach material reduced) but this was compensated for by calibrating the physical model against the BEACH PLAN numerical model.

The typical sediment grading was defined by a site investigation on East Beach carried out in November 1996 for WDDC by Exploration Associates (Reference 6). The D_{50} of the sample was 7mm and the D_{10} was 4mm (Borehole 2, Depth 0.5m). In order to reproduce the same grading curve, different grades of anthracite coal were blended together in the correct proportions.

A typical design profile of East Beach was taken from the Bridport Flood Alleviation Scheme carried out by Wessex Water in 1986. The design profile had a 10m crest berm at +7.5mODN with a 1 in 6 slope leeward. Seaward of the berm the beach sloped at 1:3, merging into a 1 in 8 slope further offshore.

To form the model beach, material was placed and lightly compacted before being drawn into place using a template placed in section across the beach. For this study, a single template was used to represent the whole of the East Beach.

4. MODEL CALIBRATION

To ensure that the correct conditions were attained in the physical model, it was necessary to calibrate the following criteria:

- The wave and water level conditions.
- The longshore transport rate.
- The river flows passing through the sluice gates.

The calibration of these fundamental parameters is discussed in this chapter.

4.1 Wave calibration

The random sea conditions were produced by a long crested mobile wave generator 12m in length driven by an electro-hydraulic system using wave synthesizer software developed at HR. The random sea reproduces a real sea state, both in the way wave energy is distributed over various frequencies, and statistically in the way consecutive waves vary in height.

In deep water the pattern of random waves is continually changing, because waves of different periods have different velocities and move through each other. However, in shallower water all wave velocities tend towards the same limit and group patterns become more stable. This allows a real long period wave effect known as set-down to increase in magnitude whereby it can have noticeable and important effects both in harbours and on moored vessels. The wave periods in question are typically in the range 30s to 180s and can give rise to harbour resonance problems and flooding of low lying ground and quays. It is this latter phenomenon which may be of importance for Bridport Harbour with resonance typified by extreme changes in water level.

During calibration the breakwaters and other structures were covered with energy absorbing material to avoid reflections propagating back towards the offshore probes.

Wave heights and periods in the model were measured using twin wire resistance probes mounted on tripods. The accuracy of measurement of a wave probe is of the order ±0.003m in the prototype. The wave probes were regularly calibrated and checked for a linear response during the calibration sequence. The wave conditions were measured in deep water just in front of the generator, at the -8mODN contour at the -5mODN contour and at two locations inshore. These were on the -3mODN contour to the west of the harbour and the -4mODN contour to the east.

Initially a short repeating sequence of waves, defined by a JONSWAP spectrum, was programmed on the computer controlling the wave paddle. This repeating sequence created a calibration test length of approximately 10 minutes at model scale, which varied depending upon the wave period. Output from the wave probes was monitored and a spectral analysis was carried out using a fast Fourier transform technique. This method allowed the entire energy content of the spectrum to be measured, giving values of wave height, H_{mo} (which, in deep water, is equivalent to the significant wave height, H_s) and mean spectral wave period, T_m. Wave conditions measured at the calibration points over the -8mODN contour were then compared with the required conditions. Using an iterative process of altering the input conditions to the wave generator and measuring the response, the required wave conditions were achieved to within +/- 5% of the required values.

Once an acceptable agreement of the specified and measured spectrum was gained over the -8mODN contour, a longer non-repeating wave sequence using the same spectral parameters was programmed on the generator. Here a sample of the wave sequence was analysed using a technique based on wave counting where the statistics of the waves can be calculated and categorised. Measurements of surface elevation were made relative to the mean value of the water level. Wave heights, calculated from the sum of the maximum departure above and below the mean water level, were sorted in descending order from which statistical values of H_s and $H_{0.1\%}$ etc were found. The total length of the calibration period was divided by the number of waves to give the mean statistical wave period, T_m. In regions of non-breaking wave conditions the two techniques should give similar results. In shallow water however, the statistical approach is more reliable. The required and measured wave conditions achieved at the calibration point over the –8mODN contour are compared in Tables 1, 3 and 4. The wave conditions measured in deep water are also shown.

A comparison of the measured and theoretical JONSWAP spectrum for several conditions for waves from 160°N and 220°N are shown in Figures 8-12. The theoretical and measured data compare well for all of the conditions.

4.2 Fluvial flows

Sluicing of the harbour is an important function of the harbour maintenance and currently sluicing removes most of the fine sediment from the harbour. To monitor the performance of the sluice mechanism with the proposed layout, fluvial flows passing through the sluice gates were reproduced using a pump and manually operated gate. Flow rates were estimated from the results of HR Wallingford's Flood Basin Model (Reference 1), which compared well with data measured at the site during a survey on the 9 April 1990. The flow through the pump was calibrated by measuring the volume water flowing through the pump over a known period of time. A prototype discharge of $16m^3/s$ was used for all the sluicing tests.

4.3 Longshore transport rate

Using Froude scaling, the movement of shingle material (represented by coal Dn_{50} 2.0mm) in the model will be significantly faster than in the prototype. There was very little prototype data to compare with the physical model results, so a method was devised to calibrate the physical model longshore transport with rates derived from the numerical model BEACHPLAN. BEACHPLAN, in turn, had previously been calibrated during the course of the preliminary studies against 20 years of historical data. The methodology used in this study is described below.

<u>BEACHPLAN</u>
BEACHPLAN was run for 3 wave conditions from 160°N, and the longshore drift rate calculated.

<u>Physical model</u>
The longshore drift rate in the model was estimated by relating the longshore drift rate to the build up of beach material against an infinitely long groyne. (An infinitely long groyne was used to ensure that no beach material was lost from the system.)

The length of the East Beach was split into 6 cells. At the centre of cell 5 the beach profile was measured before and after each test. These profiles provided the change in the cell's cross-sectional area over the duration of the test. The position of the beach at the still water level and toe at the end of each cell were also measured. Combining the single profile line data with the position of the beach at the still water level and toe, the change in beach volume within each of the six cells was estimated.

The drift rate was calculated by dividing the total volume of material collected along the beach by the time over which it accreted (m^3/hour).

The drift rates measured in the physical model and those predicted by BEACHPLAN are compared below.

Test no	Longshore drift rate, BEACHPLAN (m^3/hour)	Longshore drift rate, Physical model (m^3/hour)	Scale factor
2c	73	658	9.0
2d	149	1141	7.6
2e	201	1533	7.6

The results above show that for all the test conditions, the longshore drift rate in the physical model is approximately 8 times faster than in the prototype (predicted by BEACHPLAN). This factor is used later in this report to indicate prototype time scales relating to beach movement.

5. TEST PROCEDURES AND PERFORMANCE CRITERIA

5.1 General

Measurements made during the study include the wave disturbance in and around the harbour, the amount of overtopping at various locations along the frontage, and the displacement of sediment and dye at various points. Tracers were used to assess wave induced currents and to ensure that the sluicing capability of the present sluice gates was not compromised.

Armour stability was assessed visually during the tests by using fixed camera positions to record movement before and after each test part. The overtopping performance of the frontage was measured in 9 calibrated tanks moulded into the bathymetry behind the seawalls. Wave heights and periods were measured at 16 positions, inside the harbour, and at the seaward boundary. Currents were measured using dye tracing techniques. The beach profile changes were monitored using an HR bed profiler recording a single cross shore profile and the planshape was monitored at 7 locations along the length of the east beach.

The techniques to measure the performance of each layout and the limiting criteria to which the model results were compared are described in more detail in the following sections.

5.2 Armour stability

Visual observations of armour stability were made throughout the test series. Notes were taken of armour displacements and fully extracted units. For quantitative information on the movement of the structures a series of photographs were taken before and after each test part from fixed camera positions around the structures. Subsequently, transparent prints were made of each picture allowing an overlay technique to be used to assess the movement of individual rocks. For this study the following categories of movement were used.

- Category 1 $0.5 - 1.0D_{n50}$
- Category 2 $> 1.0D_{n50}$

where D_{n50} is the nominal diameter of the median sized rock in the structure.

Armour stability performance criteria are dependent on the type of armour and the frequency of maintenance. For this study the following indicative criteria were used.

- **Initial damage criterion**
  Any solution should require no more than limited maintenance in the aftermath of a 1:100 return period storm.

A 1:100 year return period storm has approximately 45% chance of being exceeded during the 50 year design life of the structure.

- **Failure criterion**
  Any solution should be able to resist 'failure' (defined as total breakdown of the form of the design during a 1:2000 year return period storm.

  A 1:2000 year return period storm has a 5% chance of being exceeded during the 50 year design life of the structure.

The stability of the armour has been assessed according to the following classification reproduced from the British Standard Institution publication (Reference 7).

Damage	Description
Destroyed	Core of the breakwater affected
Serious	Core of breakwater visible
Much	Large gaps in primary layer; 5% of units displaced
Moderate	3% of units displaced
Little	2% of units displaced
Slight	1% of units displaced
Hardly	No damage

5.3 Overtopping discharges

During testing, extreme waves overtopped the defences at a number of locations. The water overtopping the structure was collected in 9 calibrated tanks behind the structure thus allowing the mean discharge to be calculated. The location of the tanks is shown in Figure 13. Tanks 1-5 in the east allowed measurements of flows over the East Beach area, tank 6 collected discharge at the north west corner of the harbour whilst tanks 7-9 measured the water discharged over the western promenade.

Overtopping performance criteria are discussed by Goda (Reference 8) and Fukuda et al (Reference 9). These papers are the basis for a number of international design manuals (Reference 10). Fukuda et al recommends that for a person to walk immediately behind the seawall with little danger, the discharge should be less than 0.03l/s/m. Goda recommends that for a seawall without a back slope and unpaved apron, discharges should be less than 50l/s/m for no damage to occur. For a paved apron the recommended threshold is increased to 200l/s/m. It should be remembered that the values recorded during this study have been quoted in l/s not l/s/m. This is because the water entering the tanks was from a number of directions and the length of entry was difficult to assess. An assessment of the discharge is given in Section 7.3.

The overtopping of the structures has been assessed according to the following criteria.

- **Pedestrian safety**
  Any solution should ensure the safety of pedestrians along the West Beach frontage and the harbour area during a 1:1 year storm (Tolerable overtopping discharge 0.03l/s/m).

- **Damage to paved surfaces**

Any solution should eliminate damage to paved areas under the 1:100 year condition (Tolerable overtopping discharge 200l/s/m).

5.4 Shingle and sediment movement

5.4.1 Profile

The area of beach to the east of the harbour was monitored quantitatively using a semi automatic bed profiler. This device produces cross sections of a beach by measuring the elevation of the beach surface relative to a datum point. The data is then reduced to prototype elevations and distances before plotting the profile.

5.4.2 Planshape

The planshape of the beach was monitored by measuring the position of the still water level and toe of the beach relative to a baseline at six sections. The location of these sections are shown in Figure 13. Overhead photographs to show the planshape of the beach were also taken before and after many tests.

5.4.3 Beach material depositing at the harbour entrance

Measurements were made of the amount of material bypassing the outer end of the East Pier and landward over the rear walls. The material was collected and weighed for conversion to prototype values. The volume of material passing the East Pier was used to calibrate the values obtained from the beach mathematical model (BEACHPLAN).

5.5 Dye tracing and currents

Both bed load sediment and dye tracer was used to assess the movement of material in and around the vicinity of the harbour. Overhead photographs were taken at known time intervals to record the location of both dye and tracer at various times. The movement of the dye in a given time period was used to calculate current velocities.

5.6 Acceptable wave heights in the harbour

Wave conditions were measured using 8 probes within the harbour, and a further probe was positioned at the harbour entrance. Wave probe locations during the 'existing' tests are shown in Figure 14. Probe locations during Test Series 3, 4 and 5 are shown in Figure 15, and 16. The probe locations during the tests on the preferred structure, Test Series 6, are shown in Figure 17.

Wave heights were split into short and long wave components, i.e. wave periods between 2.5sec to 20sec (H_{ss}) and wave periods greater than 20 sec (H_{sl}). It is usually that the short wave component influences boat mooring, as small craft gently rise and fall over the longer waves. The presence of longer waves within a harbour suggests that some form of resonance is occurring, longer period waves could cause localised flooding due to higher water levels in the area.

The total wave height, representing all the wave energy in the system at any measurement point is given by the relationship:

$$H_{st} = (H_{sl}^2 + H_{ss}^2)^{½}$$

It is useful here to have some idea of the general level of wave disturbance regarded as being acceptable for both fishing and pleasure craft in order to make an assessment of the performance of the modified layouts. The frequency of occurrence of these limiting conditions is expected to be one or two times a year.

Vessel length (m)	Beam/Quartering Seas		Head Seas	
	Period (s)	Height, H_s (m)	Period (s)	Height, H_s (m)
4 – 10	< 2.0 2.0 - 4.0 > 4.0	0.20 0.10 0.15	< 2.5 2.5 - 4.0 > 4.0	0.20 0.15 0.20
10 – 16	< 3.0 3.0 - 5.0 > 5.0	0.25 0.15 0.20	< 3.5 3.5 - 5.5 > 5.5	0.30 0.20 0.30
20	< 4.0 4.0 - 6.0 > 6.0	0.30 0.15 0.25	< 4.5 4.5 - 7.0 > 7.0	0.30 0.25 0.30

Notwithstanding the results given in the table, a generally accepted limiting wave height for harbours and marinas adopted by most authorities is about 0.3m significant with a return period of about once in 50 years. In fact, research (Reference 11) suggests that this limit is conservative, but does depend on the mooring arrangement.

For single point moorings where boats are spaced at wide intervals, experience at HR Wallingford suggests that storm waves up to 0.6m high may be acceptable in a harbour. This figure is also quoted in the grant aid application report as being the wave height that would cause difficulty if exceeded within the harbour entrance.

6. RESULTS OF THE 3-D MODEL STUDY

6.1 Tests to assess the existing layout for waves from 160°N and 220°N

6.1.1 Existing layout for waves from 220°N

The measured wave conditions and overtopping discharges are shown in Tables 5 and 6. The processes observed during the test are described below.

- **1:10 year condition (low water) (Test 1a)**

Some splashing water, indicative of spray, passed over the outer 35m length of both piers. Most of the West Pier up to the Dolosse was eventually wetted by splashing water overtopping the rocks on the West Beach adjacent to the root of the West Pier. The harbour entrance up to the Dolosse structure was quite choppy, possibly due to reflection of waves off the nibs. As a result splashing water often overtopped the first nib from the harbour entrance. There was no measurable overtopping within the harbour or along the west beach frontage.

Small eddies caused by long wave activity within the harbour were seen to shed off the concrete structure at the end of the Dolosse units. These eddies caused stirring of the material on the bed.

- **1:0.1 year condition (Test 1b)**

Flooding occurred at several locations around the inner basin during this test. Overtopping discharges were measured in Tanks 5 and 6 located at each of the rear corners of the harbour. Water regularly overtopped a 35m length of wall east of the Sluice. Waves also ran over the ramp at the east end of the inner harbour, the two walls surrounding the ramp contained the flooding. Water splashing over the inner section of the East Pier and by the two nibs, ran northwards to flow over the southern wall of the inner basin. On the west side of the inner basin, north of the Dolosse, waves overtopped the south western most wall, flooding the walled off area. This water also ran northwards along the harbour wall to the north west corner of the inner basin. Overtopping water also passed over the root of the West Pier causing extensive flooding.

Water ran around the end of the root of the East Pier, between the pier wall and Ship Cottage and ponded on the upper beach area.

- **1:1 year condition (Test 1c)**

Water discharging over the rocks on the west beach caused large scale flooding at the root of the West Pier. There was light discharge over the West Beach frontage for the first time. Waves overtopped between the rock armour and the first set of steps. There was not enough discharge for the water to reach tank 7, but the water came close to it.

Within the inner harbour area, water flowed into tank 6 through the nearest flood gate on the north harbour wall, and a small amount flowed north having overtopped the south west corner of the inner harbour. There was also 'minor spill' over the inner wall on the west side of the inner harbour.

Heavy overtopping passed over both piers, water spilled sideways onto the piers as the waves ran in through the entrance. There was ponding at the root of the East Pier as before but it was more pronounced on this test. Water flooding over the inner part of the East Pier did not quite reach the base of the Pier Terrace building. There was flooding and ponding around most of the low lying parts of the inner harbour.

No material form the East Beach passed around the outer end of the East Pier. The beach cut back with all material moving onshore or along the shore in an easterly direction. The beach was quickly eroded between sections 3 and 7. Further east, material built up on the lower foreshore beyond the original placement line. The toe of the beach was highly mobile and each wave caused re-suspension of the shingle material.

- **1:10 year condition (Test 1d)**

The response of the model during this test was very similar to Test 1c, however the beach was eroded at a faster rate and the number of overtopping events were increased. For example, the flow into tank 6 at the north of the inner harbour was increased by water overtopping the inner wave wall as well as flowing through the flood gate. Increased discharge onto the deck of East Pier caused flood water to reach the toe of the Pier Terrace building, with increased ponding around this area. There was extra flow up the ramp of the inner harbour and tank 5 received a measurable discharge.

There was again severe flow over the root of the West Pier but only minor overtopping of the rocks along the West Beach frontage.

- **1:100 year condition (Test 1e)**

Discharges increased over the whole area. The flood gates in the rear wall adjacent to tank 6 were closed at the request of the clients representative. The volume of water collected in this tank was recorded again during the wave collection sequence. The closure of the gate significantly reduced the measured discharges in tank 6. Overtopping of the West Beach frontage also increased with overtopping of the secondary wall. Water flowed east down to the inner harbour and eventually into tank 6. Both tanks 6 and 9 filled very quickly.

Erosion of the East Beach was rapid. Waves passed the initial crest line after 5:00 minutes (model) and onto the flat shingle 'breach' area after 8:00 minutes (model). After 37 minutes (model) a small volume of water passed through the 'breach'. The water flowed from the breach towards the west end of tank 2. There was some ponding on the topography east of the buildings behind East Beach.

Shingle material was washed on to the root of the East Pier along past the Pier terrace and was finally deposited in the inner harbour. A substantial amount of material was deposited on the deck of the East Pier.

- **1:2000 year condition (Test 1f)**

Waves flooded through the breach in the East Beach after 25:30 (model), however no water reached tanks 1 or 2. Flooding was extensive in all areas, with particularly large volumes of water overtopping the root of the West Pier. Some of the rocks armouring the revetment had been displaced at this location very early in the test. Overtopping of the West Beach frontage increased with water overtopping the secondary wall with relatively small volumes of water flowing into tank 8. Water flowing east along the promenade dropped down into the inner harbour basin and flowed into tank 6, in general flows were constrained to the harbour side of the secondary wall.

- **Swell (1 year) (Test 1g)**

Many waves overtopped the outer ends of the Piers, the volume of water ponding on the piers was increases by sideways spillage over the piers as the waves ran in through the harbour entrance. Overtopping water flowed in both directions along the East Pier. Water passed onto the East Beach and also flowed past Pier Terrace to run off into the harbour at the south west corner of the inner basin. This flooding was not as severe as the flooding in the previous tests.

There was overtopping of the West Beach frontage between the western end of the rock armour and the first set of steps. Some of this water reached tank 9 by running westward. There was no direct discharge over the wall in front of tank 9.

Water trickled into tank 6 continuously from the east through the nearest flood gate. There was an occasional surge of water from the west as waves overtopped the harbour inner wall and flowed northwards, then eastwards to the gate. Generally flooding was constrained by the secondary walls. Water overtopping at the ramp ran towards tank 5.

At the root of the East Pier the solid model bathymetry was exposed within the first few moments of testing. The shingle was moved along the beach to the east. The wave conditions during this test appeared to support deposition of shingle at the crest of the beach . The beach continued to build up between the 'breach' and the eastern edge of the model throughout the run.

Despite the crest build up, there was no build up of the beach around the still water mark. The shingle below the water level was again highly mobile.

- **Swell (100 year) (Test 1h)**

Discharges over the West Beach frontage increased substantially compared with Test 1g. All three tanks (7-9) filled before the end of the test run. Overtopping of the East Beach caused a 'fan' of material to deposit over the breached area during the last 2 minutes of the overtopping sequence.

The crest of the eastern section of the East beach was again built up by the long period waves. During the wave recording period the beach toe moved inshore considerably more than during the previous tests.

No beach material passed into the harbour entrance. A small volume of beach material (74g model) was retrieved from the area between the root of the East Pier, and the harbour inner basin

6.1.2 Existing layout for waves from 160°N

The measured wave conditions and overtopping discharges are shown in Tables 7 and 8, and the observations are described below. A full pre test beach profile was recorded prior to Test 2c, d and e. Profiles were recorded at this location after each test part.

- **1:0.1 year condition (Test 2c)**

Overtopping only occurred at the nibs opposite the Dolosse units. Discharge events were very intermittent. There was no measurable discharge in any of the tanks.

During Test Parts 2c (a-d) the beach material moved in a westerly direction to build up at the east face of the East Pier. By the end of Test Part 2c (d) the toe of the beach extended past the outer end of the East Pier. During Test Parts e and f the shingle continued to accrete at the toe which now extended half way between the pier heads and filled an area 10m seaward of the ends of the piers. Further seaward movement of the toe was restricted, because the wave orbital velocities at the bed were not large enough to erode the beach material. By Test Part 2c (i), material moved into the harbour entrance and the build up continued on the last run, Test Part 2c (j).

- **1:0.5 year condition (Test 2d)**

There was occasional discharge onto the outer end of the East Pier. There was also continual minor overtopping onto the land between the two nibs on the inside face of the East Pier. Water then ran to the edge of Ship Cottage towards the inner harbour and also in the opposite direction along the East Pier. At the 'nib' opposite the Dolosse there was again continual minor overtopping. There was continual minor overtopping of the west and north inner harbour walls, especially around the sluices. An occasional wave overtopped the ramp. There was no measurable discharge during Test 2d.

Unlike Test 2c, the wave orbital velocities at the bed were large enough to move the shingle into deeper water at the Piers. By the end of the waves sequence, some grains had migrated around the end of the East Pier. Intermittent currents passing into and out of the entrance caused the grains to oscillate back and for. During Test Parts 2d (b - d) material moved along the entrance channel and built up against the west side of the East Pier. Material continued to feed around the end of the East Pier. Beach material did not reach the West Pier. The bypass rate increased substantially during Test Part 2d (e) and the channel was completely blocked part way through the test. This caused extra material to be forced past the entrance towards the West Pier. A lot of wave energy was prevented from entering the harbour by this build up. The build up was high enough to create an emergent mound of material at the entrance.

- **1:1 year condition (Test 2e)**

There was extensive flooding over the outer ends of both piers for the first time. Water ponded on the beach in front of Ship Cottage as it had on the earlier tests from 220°N. Water also flowed in front of Pier Terrace to drain into the south east corner of the inner harbour. There was considerable wave action up to the area of the two nibs on the East Pier. Further into the inner harbour the Dolosse had a considerable calming effect on the wave conditions reducing the 'choppyness' of the waves. There was flooding along most of the northern wall of the inner harbour with water from the sluice gate running along to tank 6. Both gates in the north wall were open for this run. None of the secondary walls in this area were overtopped. Water running up the ramp reached the base of the first wall but there was no direct flow into tank 5. The inner section of the West Pier remained fairly dry and was wet only on the inside edge due to waves spilling over from the channel. There was no overtopping of the West Beach frontage.

Some material bypassed the East and West Piers by the end of the first Test Part 2e (a). This material was retrieved and weighed. Some of the material in the channel reached a point 36m up the inside face of the West Pier. By Test Part 2e (d), material had built up in the channel enough to substantially reduce the wave action there.

- **1:10 year condition (Test 2f)**

No profiling of the beach was undertaken before or after this test, which was carried out to assess overtopping and wave disturbance only.

All the walls of the inner harbour were subject to increased overtopping and flooding, as were both piers.

To the east of the inner harbour overtopping water flowed past the walls directly into tank 5. To the west of the inner harbour the secondary wall was overtopped and the water flowed directly into tank 6. Water overtopping the wall to the west of the sluice also flowed into tank 6. There was overtopping onto the West Beach promenade for the first time during this test series. Water overtopped the set of steps at the far west end of the model and flowed along the frontage to tank 9.

- **1:100 year condition (Test 2g)**

No profiling of the beach was undertaken before or after this test, which was carried out to assess overtopping and wave disturbance only.

As on Test 2f there was an increase in the amount of overtopping over the whole area of the inner harbour basin. Water ran directly into tank 5 by both outflanking and overtopping the walls. Water ran into tank 4 after 40 minutes (model). Flood water over the inner part of East Pier reached the base of Pier Terrace. There was increased overtopping of both piers and water overtopped the rock armour onto the root of the West Pier. There was some splash over the West Beach frontage in front of tank 7, which reached the secondary wall after 60 minutes (model).

Shingle bypassed the East Pier 34 minutes into the test part, by the end of the wave recording period the amount of material in the channel had adversely affected the wave recording within the harbour area and therefore also the overtopping results

6.2 Tests to assess the performance of the East Pier extension and additional structures within the outer harbour (wave direction 160°N)

Following the completion of Test Series 2 the proposed West Breakwater structure was built, and the West Beach was constructed with a recharged profile. Prior to the main series of tests to assess the performance of the preferred harbour layout, preliminary tests were first carried out to identify the most appropriate design of two key structures.

- East Pier extension.
- An additional structure to reduce wave conditions within the outer harbour.

The results of the model tests and their impact on the designs are described below. The reasons for the changes to any of the designs are also discussed.

6.2.1 Design of the East Pier extension

Short tests were undertaken to determine a preferred alignment and cross-sectional arrangement for the East Pier extension. Three longitudinal sections were tested in the model, the height and the length of the structures were varied to assess their impact on the movement of beach material into the harbour entrance.

The adopted extension consisted of a 10-15t rock structure with a 10m flat topped berm at +4mODN, further seaward the structure sloped at a gradient of 1:6 to the bathymetry. The berm was $3D_{n50}$ long and $2D_{n50}$ wide where D_{n50} is the mean armour diameter. The front and rear faces of the breakwater sloped at 1:1.5. Cross sections of the extension are shown in Figure 4. The alignment of the extension centre line was parallel to the new West Breakwater.

A rock structure was used to minimise wave reflections in the harbour entrance. In addition the structure allowed wave energy, particularly from 220° N, to pass onto the East Beach and hence assist in the return of material in an easterly direction. The relatively low crest will reduce the need for beach maintenance.

6.2.2 Alternative structures to reduce wave conditions within the outer harbour

Additional benefit could be gained if a slipway was built to launch boats into the outer harbour. A slipway could be located somewhere along the new revetment to the north west of the outer harbour. The area in front of the proposed revetment could also be considered for seasonal mooring. With this in mind, short tests were carried out to try and reduce wave disturbance within the outer harbour. Construction of a 'stub' wall adjacent to the West Breakwater could provide additional shelter to the outer harbour area.

Although provision of a slipway and seasonal mooring may be beneficial to the scheme, there are a number of issues to be considered.

1. Wave disturbance in the harbour

a) Without the construction of a stub breakwater on the inside face of the West Breakwater, waves generally run along the West Breakwater quay wall and are absorbed by the rock revetment protecting the reclamation. This reduces the wave conditions adjacent to the East Pier and the entrance to the inner harbour and therefore wave heights in the inner harbour. Constructing a stub breakwater will cause waves to reflect off the structure increasing the wave heights on the eastern side of the outer harbour and increase the severity of wave conditions within the inner harbour.

2. Construction

a) A vertical wall would be relatively easy to construct given that block work will probably be used for the West Breakwater. A vertical wall will reduce transmission through and around the structure, but reflections off the structure will increase, creating difficult conditions in which to navigate.

b) A rock structure will absorb wave energy and reduce transmission. However, to maintain the stability of the structure a large footprint is required. Placing a vertical wall to support the structure would reduce the footprint to an acceptable limit.

3. Safety

a) A minimum turning radius of 40m was recommended by the harbour entrance desk study. Other safety issues, such as the possibility of vessels being carried onto rock structures should also be considered. Any rock structure will have to be well marked and sufficiently out of the way so that the likelihood of impact is reduced.

b) Public access to the stub breakwater may have to be restricted or controlled.

4. Harbour entrance width

PIANC guidelines on harbour entrances sets the design criteria for harbour widths to be 3 to 4 times the beam width. The design vessel assumed at West Bay is:

- 13m long
- 4.5m beam
- 1.6m to 2.0m draft
- 8-10 knots full speed (single engine)

Using PIANC guidelines, the design width should be in the region of 13.5m to 18m wide. This assumption is confirmed by Bertlin who states that a harbour entrance of 12 to 18m is usually adequate for Class V yachts. The channel leading to the entrance should not be less than 15m and preferable 20 to 30m wide. A navigation channel width of 20m was accepted.

5. Hydraulic aspects

A number of different arrangements were considered. The types of structure and their advantages/disadvantages are discussed below. A number of options were put forward but were not considered to be acceptable.

a) A rubble mound armoured with rock will reduce reflections within the harbour. Rock armour must be placed to the crest of the breakwater at +4mODN to limit run up and therefore reflections. A large footprint will be generated.
b) A vertical face will provide a quay to moor against, however reflections will be high.
c) A vertical face with a protective rubble mound on the entrance side will lower reflections, provide a reduced footprint and an increased mooring capacity.
d) Wave screen structure (not assessed). A wave screen will be less effective than a rubble mound.
e) Semi-submerged breakwater, crest level +2mODN (not tested). A submerged structure will provide little protection and will be hazardous to incoming and out going craft. Waves will be transmitted through and over the crest of the structure.

Following the selection of the most appropriate 'stub' structures, each was tested in the physical model. The wave conditions within the inner harbour during the tests on each of the modified designs were compared with the existing layout , Test 2e, and with the new scheme without structures (Test 3a). For these tests the proposed West Breakwater was terminated with a rock armoured roundhead to reduce wave reflections from the exposed vertical surfaces (Plate 4). All the layouts shown in Plates 5-11 were assessed under the 1:1 year condition.

The wave conditions for Tests 3a- h are compared with the existing layout Test 2E in Figures 18-20 and Table 9.

- **Existing layout, Test 2e**

During Test 2e, the maximum wave height at the entrance to the channel was 1.9m, wave heights reduced to a maximum of 0.8m within the harbour. A long wave component, H_{sl} up to 0.4m, was measured within the channel and harbour. The longer wave activity was confirmed by dye tracing. The craft will gently rise and fall with these longer waves, so navigation and mooring within the harbour will not be affected. The interaction of the long waves with the shorter wind derived waves could cause localised overtopping of the piers.

- **Rock armoured roundhead, Test 3a**

Wave conditions at the entrance showed a small reduction in wave height probably due to wave dissipation over the new East Pier extension and the Western Breakwater roundhead. The maximum wave height at the entrance to the channel was 1.5m, 0.4m lower than for the existing situation.

Wave conditions within the harbour were significantly lower with wave heights up to 0.4m recorded. This is almost a 50% reduction when compared with the existing situation.

The distribution of wave conditions along the length of the outer harbour reclamation is shown in Figure 20. This shows that the largest wave heights occur close to the proposed West Breakwater, an H_{ss} of 1.0m was measured. Wave height gradually dropped towards the eastern end of the revetment. Wave conditions (H_{ss}) for the 1:1 year storm (160°N) reached 0.7m at this location. A slipway close to the root of the curtailed West Pier would therefore be the best option for this layout.

A small long wave component was again measured within the channel and harbour, however the removal of the West Pier reduced it's magnitude. The largest H_{sl} of 0.4m was measured close to the new revetment.

- **Outer harbour modification, Test 3b**

The 10m wide structure with a revetment on both sides (Plate 5) had very little influence on the wave conditions to the east of the revetment, however close to the new breakwater wave heights were reduced by 50% to approximately 0.5m.

The presence of the stub breakwater increased wave conditions within the inner harbour by approximately 0.05m

- **Outer harbour modification, Test 3c**

The 20m wide structure with rock armouring on the entrance side (Plate 6) influenced the wave conditions along the whole length of the outer harbour revetment with wave heights of just less than 0.5m recorded on both the east and west side of the revetment. Wave heights within the inner harbour were generally 0.1m higher when compared with the, Test 3a.

- **Outer harbour modification, Test 3d**

The 10m wide structure with rock armour on the entrance side (Plate 7) had no obvious sheltering effect.

- **Outer harbour modification, Test 3e**

The vertical walled structure positioned 90m from the revetment (Plate 8) provided significant shelter to the revetment. The slipway could be positioned anywhere along the revetment. However, wave conditions at the harbour entrance and within the channel were more severe making it difficult for boats to navigate into the harbour. Wave conditions within the inner harbour were also increased.

- **Proposed modifications**

None of the outer harbour modifications provided enough shelter for the provision of a slipway close to the root of the proposed western breakwater. In fact wave conditions were generally less severe close to the root of the old West Pier. The outer harbour modifications were therefore discarded.

During the tests on the outer harbour modifications it was noted that the promenent plan shape bulge on the East Pier did not provide the best alignement for boats approaching or leaving the harbour. The curvature was therefore reduced and the layout tested as Test 3f. Wave conditions within the harbour during Tests 3f are compared with the existing layout Test 2a in Figures 21-23.

- **Rock armoured roundhead , with re-aligned curvature of Eastern Pier, Test 3f**

Changing the alignment of the East Pier (Plate 9) increased wave conditions within the inner harbour by approximately 0.05m. Close to the proposed revetment in the outer harbour, wave conditions were very similar for the two alignments. Tests therefore showed that wave conditions within the harbour were marginally more severe following the realignment of the East Breakwater. As this realignment significantly improved navigation at the harbour entrance, the increased wave activity within the harbour was accepted.

The Bridport harbour master, Tony Preston, visited HR on the 6 July. Following lengthy discussions it was decided that the passage of boats over the submerged rock armoured roundhead would not be acceptable. The possibility of damage to the roundhead structure or the boats entering the harbour during severe conditions would be high. Warning masts and fendering were considered but these structures would be relatively costly to provide and maintain. The harbour master felt that the submerged rocks would hinder the passage of craft through the entrance and he suggested that a vertical wall would be more appropriate. It was therefore decided to remove the submerged rock armour by retaining the roundhead material with a vertical wall.

Two layouts were considered:

a) Removal of the submerged rock armour increasing the navigable channel width to 30m.
b) The vertical wall was moved to the location of the previous armour toe, reducing the channel width to 20m. These modifications were implemented in tests 3g and 3h.

- **Vertical wall entrance channel width 30m, Test 3g**

This arrangement is shown in Plate 10. Results show that the wave heights over the whole of the harbour were increased by up to 0.1m.

- **Vertical wall entrance channel width 20m, Test 3h**

This arrangement is shown in Plate 11. Measured wave heights reduced within all parts of the harbour. Wave heights ranging between 0.25 and 0.35m were recorded in the inner harbour, compared with wave heights of over 0.5m for the existing layout.

- **Interim conclusions**

The tests to assess the performance of the additional structures and modifications to the harbour entrance have been carried out for the 1:1 year condition from 160°N. The wave height and period associated with this storm are 2m and 5.3s respectively. Note that from 220°N the wave conditions for a 1:1 year storm are $H_s = 4.0$m and $T_m = 7.5$sec, significantly higher than the wave conditions 160°N. We would therefore expect significantly higher wave conditions to reach the inner and outer harbour areas for an incident wave direction of 220°N. It is also likely that for the yearly condition overtopping of the breakwater may prevent safe mooring or use of any slipway in the outer harbour area.

All structures placed along the inner face of the proposed breakwater increased reflections and therefore wave conditions within the inner harbour. It is unlikely that these structures will provide any great benefit, and no further tests on these structures were carried out.

The additional nose on the East Pier was re-aligned to improve navigation into the harbour (see Test 3f, Plate 9).

The head of the Western Breakwater was revised to improve navigation and reduce wave conditions within the harbour. The entrance channel was reduced to 20m and a vertical wall running parallel to the entrance channel supported the rock armour on the seaward side of the West Breakwater.

The stub of the demolished West Pier was rounded off to reduce disturbance due to eddy shedding caused by long wave activity within the harbour.

If a slipway is required wave conditions are significantly lower towards the root of the old West Pier. Unfortunately, the access to this end of the new revetment is limited. We would therefore recommend that any slipway should be constructed at least 10m away from the West Breakwater, outside the area of greatest wave activity.

6.3 Tests to assess the performance of the scheme for wave conditions from 160°N and 220°N

6.3.1 Waves from the 160°N (Test series 4)

The West Breakwater outer end was constructed as a rock armour roundhead supported by a vertical face adjacent to the entrance channel (Plate 11). The East Pier extension was built to the profile used during Test Series 3. The measured wave conditions and overtopping discharges are shown in Tables 10 and 11.

- **1:0.1 year condition (Test 4a)**

There was no overtopping of any of the harbour structures during this test. Some splash was created on the East Pier extension rocks as waves ran up the 1:6 slope. No splash reached the top of the East Pier and no splash was seen on the West Breakwater. The dye tests showed no indication of any movement within the harbour except two areas of possible long period oscillation.

- **1:0.5 year condition (Test 4b)**

Observations were similar to Test 4a. There was minor splashing over the end of the East Pier, no other overtopping was noted.

- **1:1 year condition (Test 4c)**

Waves reached the top of the inner harbour ramp, small volumes overtopped the ramp and the eastern inner harbour wall. The overtopping was not severe enough for the water to flow to tanks 5 or 6.

To the west, there was no overtopping of the West Beach frontage. Splash from waves breaking onto the rocks at the West Breakwater roundhead reached the concrete crest. Waves ran along the inside face of the West Breakwater below the level of the roadway.

Overtopping was observed at the outer end of the East Pier and at the inside face of the 'nib'. No measurable discharge was collected in any of the tanks.

- **1:10 year condition (Test 4d)**

Occasional green water events overtopped the roundhead of the West Breakwater, however, there was no overtopping over the trunk of the West Breakwater. A small number of waves within the outer harbour spilled over the inside face of the West Breakwater. The reclamation was occasionally overtopped at the western corner close to the proposed slipway. The East Pier was often awash as waves overtopped its outer end. Increased waves conditions in the inner harbour area caused minor spillage of the walls, mainly the west wall, and tank 6 began to fill during this test. The closure of the gate in the rear wall significantly reduced the collection of water in tank 6. No other tanks recorded any discharge under this condition. Waves barely reached the promenade wall along the West Beach frontage.

- **1:100 year condition (Test 4e)**

The overtopping of the outer end of the West Breakwater was increased. In fact, green water ran along the deck towards the reclamation. There was still no overtopping over the trunk of the West Breakwater. The East Pier was also subject to increased overtopping. There was increased flooding of the inner harbour walls, especially at the ramp where the wall was often awash. Even though the secondary walls retained the vast majority of the spillage a small volume of water flowed into tank 5. Waves running up to the promenade wall protecting the West Beach frontage were reflected seaward. No overtopping was observed at this location.

6.3.2 Waves from the 220°N (Test Series 5)

All the previous tests were carried out with a typical East Beach profile. It was assumed that following the construction of the West Breakwater and the East Pier extension the beach would be protected and would therefore build up at the East Pier during storms from the south east. Before the start of Tests 5c to 5f, additional shingle was placed in the sheltered area behind the East Pier extension to simulate the protected beach planshape.

Tests 5g and 5h for the 1:100 and 1:2000 year events commenced with the typical East Beach profile so that the sheltering provided by the proposed layout could be assessed by comparing the results from the existing and proposed layouts.

The measured wave conditions and overtopping discharges are shown in Tables 12 and 13, and the observations are described below.

- **1:10 year condition (Low water) (Test 5c)**

Spray reached the rock armour crest along the outer half of the West Breakwater, although no water reached the deck. No water was collected in any of the overtopping tanks.

The beach in the lee of the East Pier extension cut back fairly quickly leaving a steep ridge along the whole length of the beach.

- **1:0.1 year condition (Test 5d)**

Waves ran half way up the ramp in the inner harbour. Waves breaking onto the West Beach occasionally reached the toe of the wall. Spray reached the crest of the rock armour along the outer section of the West Pier. There was no overtopping observed within the harbour or along the frontage.

After 11 minutes (model) the crest of the East Beach had cut back to the intersection of the East Pier extension with the East Pier. Toe rock movement on the outer end of the East Pier extension was noted.

- **1:1 year condition (Test 5e)**

Heavy overtopping was observed at the outer end of the West Breakater. Overtopping water ran along the deck of the West Breakwater to spill into the harbour entrance area. Water also splashed over the West Breakwater all the way to the root and along the West Beach frontage to the first set of steps. There was no overtopping of the rock reclamation at the root of the West Breakwater.

Side spilling waves overtopped the west inner harbour wall. Waves also reached the top of the ramp in the inner harbour but were retained within the walled area. No water was collected in the tanks.

- **1:10 year condition (Test 5f)**

Waves overtopped the whole length of the West Breakwater with ponding on the reclamation area at the root. Water also reached this area from inside the harbour by discharging over the reclamation revetment. Water discharging onto the outer end of the West Breakwater flowed into the outer harbour. Water overtopping the East Pier ran along the pier towards the buildings of Pier Terrace. There was increased flooding onto the western wall of the inner harbour, but the water was retained by the secondary walls. At the ramp water passed beyond the walls but did not pass into tank 4 and 5.

Water passed over the West Beach frontage for the first time. Overtopping water flowed along the roadway into tank 9 but nothing overtopped the rear wall on the promenade.

Currents through the harbour entrance produced a strong intermittent eddy between the East Pier and the Western Breakwater. The strong localised circulation was visible adjacent to the West Pier roundhead.

- **1:100 year condition (Test 5g)**

The area between the root of the West Breakwater and the first set of steps on the West Beach frontage was regularly overtopped. In time, water flowed west along the promenade. At the end of the test, tank 7, behind the second flood wall, had measurable volumes for the first time. Tank 9 was full half way through the run. There was apparently less overtopping at the outer end of the East Pier and the area leading to the Pier Terrace compared with Test 5f.

Throughout Test 5g material was pushed from the East Beach onto the East Pier forming a heap on the deck just landward of the bus shelter. By the end of the test this build up passed over the pier to deposit in the entrance to the inner harbour adjacent to the inside face of the East Pier.

- **1:2000 year condition (Test 5h)**

Waves overtopped the whole length of the West Beach frontage, the largest overtopping events occurred at the root of the West Breakwater. There was no flow down the roadway into the inner harbour. At least 70% of the waves broke onto and over the West breakwater. A small number of rock armour units were seen to move, particularly at the roundhead. By the end of the test he armour layers remained intact.

The East Pier extension was sheltered by the head of the West Breakwater so few waves broke onto the rubble mound extension. The water within the outer harbour was choppy due to reflections from the vertical walls, but the inner harbour landward of the Dolosse units was much calmer. Water flooding up the inner harbour ramp caused measurable discharges in tanks 4 and 5. Tanks 7, 8 and 9 were filled by the end of the test. Overtopping of the second promenade wall was caused by waves skipping over the flood water ponding in the roadway.

Shingle was moved onto the East Pier by wave action, the shingle moved further landward than on test 5G. Beach material fell into the outer harbour area adjacent to the East Pier.

- **Swell (1 year) (Test 5i)**

The larger waves overtopped the outer half of the West Pier and East Pier. Smaller waves hitting the end of the West Pier passed into the entrance, causing disturbance in the outer harbour. These waves passed through to the inner harbour causing choppy conditions. The West Beach frontage overtopped along the central section, water then ran along the roadway into tank 9. Tank 9 was the only tank to register any discharge. The turbulent eddies previously noted in the entrance, were enlarged during this run probably because the longer period waves caused larger volumes of water to pass in and out of the inner harbour.

- **Swell (100 year) (Test 5j)**

There was continual overtopping of the West Beach frontage. A significant proportion of the water recorded in tank 6 flowed from the upper promenade area, past tank 7 and down the western inner harbour wall. Flows past the western inner harbour wall first occurred after 35 minutes (model). As on previous tests the discharge in tank 8 was collected following overtopping of the rear promenade wall.

The larger waves in the sequence overtopped the whole length of the West Breakwater. Waves overtopping the West Breakwater increased the wave activity in the outer harbour, this caused significant overtopping of the East Pier between the roundhead and the buildings on Pier Terrace.

6.4 Tests to assess the performance of the preferred layout for wave conditions from 220°N, Test Series 6

Following the completion of Test Series 5 a number of modifications were made to the design. These included modification of the West Breakwater and East Pier extension.

1. A circular caisson was provided at the end of the West Breakwater.
2. A stepped wall was provided to support the rock armour on the West Breakwater roundhead.
3. The armour crest dropped at a slope of approximately 1:3 from 4.8mODN at the front face of the breakwater to 1.5mODN at the steps.
4. The rock revetment was constructed of a homogeneous armour layer placed on a bedding layer.
5. The armour size was reduced towards the root of the breakwater (Figures 5 and 6).
6. The crest wall at the root of the West Breakwater was raised to 6.2m.
7. The section through the Eastern Pier extension was modified (Figure 7).
8. The slipway in the outer harbour was constructed 10m away from the inner face of the West Breakwater.
9. The alignment of the East pier was straightened.
10. The West Beach groyne was constructed

The measured wave conditions within the harbour and the overtopping discharges along the frontage are shown in Tables 14 and 15 respectively. Photographs showing details of the layout are shown in Plates 12, 13a and 13b. Observations during the tests are discussed below:

- **10:1 year condition (Low water) (Test 6a)**

There was light splash over the armouring along the West Breakwater and East Pier extension. One or two rocks were displaced from the West Beach groyne but these were not supported by the bathymetry at the toe of the structure. There was no overtopping of any of the harbour structures.

- **1:0.1 year condition (Test 6b)**

The larger waves occasionally reached the base of the promenade wall at the West Beach. Waves ran part way up the rock armour protecting the West Breakwater but none reached the crest. There was light but continual splashing of the deck of the West Pier. By the end of the test the outer end of the West Pier was flooded.

Significant wave induced currents were noted at the steps on the outer end of the West Breakwater. Strong currents were also observed through the inner harbour entrance, which caused eddies to form around the wooden poles seaward of the Dolosse units in the inner harbour, and at the tip of the curtailed Western Pier.

- **1:1 year condition (Test 6c)**

There was increased overtopping of the West Breakwater, large droplets of water were thrown over the breakwater between the roundhead and reclamation revetment. There was also light splashing of water onto the stepped area above the reclamation. Waves ran along the inside face of the West Pier but did not reach the deck at +4.0mODN, these waves ran to the top of the revetment ramp. The increased wave action forced water to the top of the inner harbour ramp, however, no water reached the overtopping tanks. Occasionally waves reached the crest of the outer end of the East Pier extension.

Waves reached the promenade wall along the whole length of the West Beach frontage and there was occasional overtopping of the wall at the root of the West Pier. Reflections off the promenade wall were also evident. There was no overtopping of the inner harbour walls.

The strong currents at the entrance to the inner and outer harbours continued to shed eddies off the structures.

- **1:10 year condition (Test 6d)**

Overtopping of the West Breakwater increased to heavy splash along the whole length of the structure. An occasional greenwater event overtopped the outer end of the West Breakwater. Waves overtopping the West Breakwater caused flooding of the proposed reclamation. A small number of events overtopped most of the inner harbour walls. Overtopping water was collected in tank 6. No water spilled sideways onto the East Pier as the waves ran into the inner harbour.

There was increased overtopping on the West Beach frontage, but this was mainly at the root of the West Breakwater and no overtopping occurred west of the first set of steps. Water overtopping at the root of the West Breakwater ran along the promenade to collect in Tank 9.

- **1:100 year condition (Test 6e)**

A further increase in overtopping of the West Pier caused green water waves to run along the length of the structure. This flowed into the outer harbour via the reclamation revetment. Water frequently overtopped the ramp, flooding the new reclamation area. The overtopping of the inner harbour walls was also increased.

The number and volume of the waves overtopping the root of the West Breakwater increased, the overtopping water flowed west along the promenade. The first damage to the East Pier extension was noticed on this test with a flattening of the 3-6t rock at the crest of the structure. No major armour movements were observed along the West Pier. A small number of rock units armouring the outer harbour reclamation were displaced. A number were moved onto the slipway.

- **1:2000 year condition (Test 6f)**

Overtopping of the West Breakwater caused extensive flooding of the reclamation and the promenade. Water flooded down the ramp and steps at the root of the breakwater onto the reclamation area. There was also increased overtopping of the East Pier, with water passing over the rock extension onto the outer end of the concrete pier. More armour rocks moved on the reclamation by the outer harbour ramp.

6.5 Tests to assess the performance of the West Breakwater during construction

The layout of the breakwater and piers for Tests 7a and 7b is shown in Figure 17 and Plates 14-17. A wave direction of 220°N was used during Test Series 7 to ensure that the temporary West Breakwater roundhead was stable, and to ensure that wave conditions within the harbour were not significantly increased during the construction period. Any increase in wave height within the inner harbour may affect the harbour operations. The wave conditions measured during the tests are shown in Tables 16, and the observations are described below.

6.5.1 Short length of West Breakwater installed, Test 7a, 1:10 year condition

Large waves entered the existing channel unhindered by the temporary West Breakwater roundhead. The reduced protection allowed heavy overtopping of both the existing East Pier and (shortened) West Pier as the waves ran through the entrance, spilling sideways over the decks. The overtopping water reached the base of the Pier Terrace in the inner harbour. All the inner harbour walls were subject to flooding and some parts of the north and west secondary walls were regularly overtopped. Water also ran up the existing inner harbour ramp and caused a measurable discharge in Tank 5. This last occurred on Test 6e (1:100 year event of the preferred layout).

Water overtopped the root of the West Breakwater and subsequently flowed in a westerly direction towards tank 9. There was also overtopping of the existing wall at the location of the proposed reclamation between the old West Pier and the new West Breakwater. Most of this water flowed towards tank 6 at the north side of the inner harbour. Reflections from the rear face of the proposed reclamation caused a great deal of choppy wave activity in the new outer harbour area. The waves within the inner harbour washed onto the deck of the new West Breakwater. There was heavy overtopping of the outer face of the West Breakwater along most of its length, however only one 6-10t rock was observed to roll down the slope.

6.5.2 Long length of West Breakwater installed, Test 7a, 1:10 year condition

The harbour responses observed during Test 7a were repeated for Test 7b. For this layout, however the wave conditions and overtopping of the inner harbour was much reduced due to the increased protection across the inner harbour entrance. For instance, the flood water on the eastern inner harbour wall did not reach the Pier Terrace property and the secondary inner harbour walls were rarely overtopped. There was no overtopping of the proposed reclamation area and, consequently, a reduced amount of water washed over the decking of the West Breakwater. The incident wave conditions overtopped the West Breakwater.

A comparison of the wave conditions between the entrance and the inner harbour for the existing and the preferred layout compared with the Construction Phase Tests (7a and 7b), is show in Figure 24. Similarly, wave conditions within the outer harbour are shown in Figure 25. The 1:10 year storm event was considered during all the tests. Clearly the reduced protection afforded by the part constructed West Breakwater will mean that the wave conditions within the harbour will be more severe than for the completed structure. The exposed inner harbour will therefore continue to overtop as the West Breakwater is being constructed. However, even the uncompleted structure will provide more protection to the entrance than the existing layout.

7. A COMPARISON OF THE PERFORMANCE OF THE EXISTING AND PREFERRED LAYOUTS

7.1 Wave conditions

A comparison of the existing wave conditions between the entrance and the inner harbour with those for the preferred layout for the 1:1 and 1:10, conditions from 160°N and 220°N are shown in Figures 26 and

28. Similarly the waves conditions within the inner harbour are shown in Figures 27 and 29. For the preferred option, wave heights within the harbour were significantly reduced, and as a consequence overtopping discharges around the harbour walls were also reduced.

The reduction of the wave conditions, orbital velocities and bed shear stresses in the harbour could encourage accretion of fine material within the more sheltered areas of the inner and outer harbours.

7.2 Movement of the eastern shingle beach

The build up of the East Beach against the existing East Pier, for waves from the south east, is shown in Plates 18a-d. The build up is relatively fast allowing the shingle material to pass the tip of the East Pier and deposit in the harbour entrance. It is interesting to note that no material passed the existing West Pier during any of the tests.

The new West Breakwater and East Pier extension provided a sediment trap for waves from 160°N (Plates 19a-c). The increased length of the East Pier extension allows the beach to accumulate before bypassing the rubble mound structure. The wider shingle beach will act as an additional flood defence and will therefore provide a higher standard of service to the town of West Bay.

The existing East Pier provides some shelter to the East Beach for waves from the south west. A relatively rapid erosion of the East Beach in front of West Bay is shown in Plates 20a-d for the existing situation. The new West Breakwater and East Pier extension provides more shelter than the existing layout, however waves from the south west (Plates 21a-c) still pushed the beach in an easterly direction, this will therefore reduce the magnitude of beach re-cycling required as part of the beach management plan for East Beach.

Beach profiles taken during the 1:100 and 1:2000 year storm for the existing layout and the layouts assessed during Test Series 5 and 6 are shown in Figure 30. These profile lines again suggest that the preferred breakwater scheme provide additional shelter for the East Beach for wave from 220°N. The similarity of the profiles for both schemes showed that they both provided a comparable protection to the East Beach adjacent to the East Pier. These results suggest that the probability of breaching will reduce once the scheme is constructed.

7.3 Overtopping

The West Beach frontage is protected by the root of the proposed breakwater for waves from 220°N. Further west the recharged beach also provides protection. For all conditions tested the overtopping of the seawalls and inner harbour was therefore less for the preferred layout than for the existing layout.

The reduction in discharge can be clearly seen by comparing the data in Tables 6 and 8 for the existing layout and Tables, 11, 13 and 15 for the modified layouts. The overtopping discharge is reduced by at least 40%, with even greater reductions in the harbour area.

Another important point is the effect of the flood gates in the north wall of the inner harbour. On some tests the discharges were monitored with these gates open and closed. The closed layout are represented by bold values on the above tables. The effectiveness of these gates in reducing discharges is quite clear, both between the existing and preferred layouts, and within each layout.

The overtopping discharges quoted in these tables are measured in litres per second, l/s, prototype. Usually discharge is quantified in l/s/m length of wall but this is not possible in this case. The method of entry of the water into most of the tanks precluded a value for the length of entry being calculated. However it is important to be aware of the likely effect of different rates of discharge both on people and structures are discussed here.

- **1:1 year condition**

For both the existing and the preferred layout there was no overtopping of the frontage or inner harbour area. Pedestrians will not be in danger during the 1:1 year storm.

The proposed West Breakwater is relatively low so even during the 1:1 year condition spray caused by waves breaking onto the rock armour will pass over the structure. We would recommend that pedestrians should not be allowed onto the breakwater for storm greater than the 1:1 year storm.

- **1:100 year condition**

Overtopping volumes during tests on the existing layout, Test Series 1, are shown in Table 6. For the1:100 year event at 220°N, tanks 7 and 9 collected 133 and 508 l/s respectively. Overtopping volumes during tests on the preferred layout, Test Series 6, are shown in Table 15. For the1:100 year event at 220°N, tanks 7 and 9 collected 100 l/s and 264 l/s respectively. If we assume that these two tanks collected water directly along their whole length, and only along their length, then the entry length can be taken as 45m for tank 7 and 38m for tank 9. The overtopping discharges then become 2.97 l/s/m and 13.4 l/s/m for the existing layout and 2.22 and 6.94 l/s/m for the preferred. Although this represents a marked reduction 30% and 53% respectively the values are still very high compared with the tolerable overtopping guideline figures quoted for pedestrians in Section 5.3, however discharges of this order will not damage paved areas.

7.4 Accretion of material in the harbour entrance

The cumulative volumes of material deposited in the harbour entrance (during each test sequence) is shown in Table 17. The position of the measured beach profile at the still water level is also indicated. This data was used in the BEACHPLAN numerical model to assess the long term beach development on both sides of the harbour entrance

The accretion of material with time is shown in Figure 31. The efficiency of the new groyne can be assessed by comparing results for the same test conditions for the existing and proposed layouts (Test 2c with Test 4a, Test 2d with Test 4b and Test 2e with Test 4c). Results show that the initial accretion of material in the entrance is delayed by the groyne, this delay may be further increased if the groyne was constructed with an impermeable core. The beach subsequently builds up behind the groyne, however during this time material passes through the permeable structure. Eventually, as the material builds up the shingle starts to pass over the groyne

Very little material passed around the end of the groyne as the wave orbital velocities at this location were to small to erode the material at the toe of the beach.

The method of measurement was changed between the tests on the existing and proposed layout. During the tests on the existing layout the material was extracted from the entrance, weighed and then replaced. This method was implemented so that the bypassing of both the East and West Piers could be measured. After running the tests for a number of unrealistically long sequences of waves, it was found that little or no material bypassed the existing West Pier.

During the later tests to assess the performance of the proposed layout the material was extracted and weighed after each test sequence but not replaced. This difference in experimental procedure suggests that if the material was removed from the model during Test series 2, more material would have bypassed the East Pier as previously the accreting material formed a barrier.

Assuming that the beach will build up in the shelter of the East Breakwater extension, it is likely that shingle will be able to pass through the permeable structure. Observations on the model suggest that material passes through the breakwater as waves run down the beach. This material has been shown to accumulate in the harbour entrance for waves from both 220°N and 160°N. As the majority of the shingle passed through the extension, the accretion in the entrance for waves from 220°N could be significantly reduced by constructing the extension with an impermeable core. The rubble mound breakwater could be constructed with a finer rock core or a piled section. If an impermeable core can be used to reduce the permeability of the structure, less material will be able to pass through the extension, which will reduce the requirement for long term maintenance dredging.

7.5 Wave induced currents

Typical flows observed using the dye tracing technique are shown in Figures 32-35, the dye patterns shown are typical for all the conditions tested, although the currents increase with the increasing severity of the storm. The range of current speed measured at each location is also shown.

7.5.1 Existing Layout

Current patterns for the existing layout are shown in Figures 32 and 33 for the 160°N and 220°N wave directions respectively.

Currents adjacent to the coastline passed toward the west for waves from 160°N. The piers form an obstruction to the flow which caused a clockwise gyre in the lee of the West Pier. Virtually no water movement was detected close to the root of the West Pier. The currents increase as the water is squeezed passed the entrance, water velocities were generally higher at this location, currents of between 0.2-0.7m/s were measured. To either side of the structures, currents ranged between 0.1 and 0.3m/s.

For waves from 220°N the wave induced current appears to split close to the harbour entrance, currents flow to the west along the western frontage and to the east along the eastern beach. Currents ranging between 0.6 and 0.8m/s and 0.1and 0.3m/s were measured close to the entrance and in front of the eastern beach respectively.

The current patterns within the harbour were very similar for waves from both directions. A very slow clockwise gyre was noted to the west of the inner harbour basin, while the area to the east was relatively static. As the severity of the storms from 220°N increased, overtopping of the western frontage flowed into the western part of the inner harbour. The increased head within the harbour tended to create currents out through the entrance. Occasional slow re-circulating cells were noted at the west side of the inner harbour.

7.5.2 Preferred layout

Current patterns for the proposed layout are shown in Figures 34 and 35 for the 160°N and 220°N wave directions respectively.

The current patterns seaward of the harbour entrance for the preferred layout at 160°N, were similar to those of the existing harbour. Currents moved westward past the new entrance with little sign of dye passing into the harbour entrance. There was again a static area in the lee of the West Breakwater but the reverse flow seen in the existing layout, was not in evidence for the preferred scheme. The rates of flow were also similar to the existing layout.

The currents developed off the entrance for waves from 220°N were similar for both the existing and preferred schemes. There was a split in current direction off the harbour entrance. The split for the preferred scheme occurred further west, off the south-west corner of the West Breakwater. In front of the East Beach velocities were again similar to the existing layout, however across the entrance velocities were slightly lower, in the order of 0.3 – 0.5m/s. The more severe wave conditions caused larger currents at the entrance and maximum flows of 0.6 – 0.7m/s were recorded. Flow directions and speeds on the west side were also similar to the existing layout.

Within the inner harbour the currents were lower for the preferred layout. Increased currents were recorded at the entrance to the inner harbour between the East Pier and the stub of the old West Pier. Here a reversing flow was noted on tests from both wave directions. With waves from 160°N the measured velocity was 0.05m/s but for waves from 220°N the maximum measured flow was 0.7m/s.

During severe wave attack less water overtopped the root of the West Breakwater, and therefore the build up of water in the harbour experienced with the existing layout, did not occur. Consequently, the occurrence of rapid flows through the entrance was reduced for the preferred layout.

River flows were not modelled during this part of the study. River flows will therefore augment the wave induced currents out of the harbour entrance during flood conditions.

7.6 Currents induced by sluicing

Details of the layout for the sluicing tests are given in Section 4.2.

Four tests were run on the existing and preferred layouts. The sluice gates were opened to allow the ponded river water to flow in to the inner harbour. Dye was used in each case to follow the paths of the currents and overhead photographs were taken to track the velocities. For both the existing case and the preferred scheme, tests were carried out at two water levels of 0.0mODN and –1.65mODN. A representative plot of the currents for the –1.65mODN water level are shown in Figures 36 and 37 for the existing and proposed layouts respectively. No waves were running on any of the fluvial discharge tests.

In both cases the current patterns within the inner harbour are similar. This is not surprising as the inner harbour layout for the existing and proposed scheme are identical. Two circular paths were formed in each arm of the inner harbour, the largest confined to the western part of the harbour (Figures 36 and 37). The main flow from the gate towards the harbour entrance was of the order 1.8 – 2.0m/s. The circular paths were slower with speed ranging between 0.6 – 1.2m/s. For the existing layout the main current flowed through the area between inner and outer harbours and out through the entrance. For the preferred scheme the current tended to remain close to the East Pier and then splits as the water reaches the harbour entrance. Part of the current passed out of the harbour, the other part is diverted into the outer harbour up the inside face of the West Breakwater. The speed of this return current ranged between 0.5 – 0.2m/s.

A similar pattern of flows was observed on the tests with the higher water level of 0.0m ODN.

As wave conditions within the proposed harbour are less severe than for the existing layout, fine sediment may be deposited in the sheltered areas, where sluicing has little affect.

7.7 Armour stability

The stability of the proposed West Breakwater armour was monitored during Test Series 5 and 6, and also during the construction series, Test Series 7. Before and after these tests the structures were photographed to allow a detailed assessment of the armour movements to be made.

7.7.1 Test Series 5

Armour stability was assessed at the roundhead and three other locations along the trunk of the new West Breakwater. There was evidence of some minor settlement on the slope at the SWL at all four locations. At one of the locations there was also noticeable movement of a group of 3-4 rocks adjacent to the crest. At the inshore location, a group of 3-4 toe rocks were moved during the course of testing. However at no location were any armour units removed from the slope and all locations indicated a stable slope at the end of testing (Plates 22A and 22B).

Following Test Series 5, it was decided to reduce the size of the rock armour towards the root of the breakwater as the wave height reduces in shallower water.

7.7.2 Test Series 6

The preferred scheme was assessed during Test Series 6, a full overlay analysis of the rock armour was carried out. The stability of the following structures were analysed.

1. West Breakwater trunk, section P1 (transition between 3-6 and 6-10t rock)
2. West Breakwater trunk, section P2 (6-10t rock armour)
3. West Breakwater roundhead, sections P3 and P4 (10-15t rock armour)
4. East Pier rock armoured extension, section P5 (10-15t rock armour)
5. East Pier rock armoured extension, section P6 (3-6t rock armour at crest).

Details of the structure assessed during Test Series 6 can be found in Figures 5-7 and Plates 12 and 13. Results of the armour movement in terms of the percentage number of rocks contained within the two layers of the test area are described in Tables 18-21.

The first section towards the root of the West Breakwater covered the transition between the 3-6t and 6-10t rock armour sections. Previous physical model tests suggest that transition sections may be less stable than standard trunk sections. Only one 3-6t rock and three 6-10t rocks were extracted during the whole series of tests. The movement of the 6-10t rock armour was also assessed to ensure its stability in deeper water where we would expect the incident wave climate was more severe. Only one 6-10t rock was extracted during Test 6a, and there was little category 1 (0.5-1.0D_{n50}) movements.

The outer trunk and roundhead is located in relatively deep water and therefore the larger 10-15t rock armour was used to protect this part of the structure. The outer trunk was relatively stable, 10 rocks were extracted by the end of Test series 6. Fourteen rocks were extracted on the roundhead (Section P4), this amounts to 2.3% of the rock within the section. It is essential that the stepped structure should support the rock armour units at the tip of the roundhead. If the units are not supported waves washing over the roundhead are likely to push these susceptible units into the harbour entrance.

Two camera positions were also used to assess the armour movement on the East Pier extension. The 10-15t rocks on the harbour entrance side of the breakwater were stable, only 1 unit was extracted during the test series. The smaller 3-6t armour units protecting the crest of the East Pier extension were too light to eliminate smaller movements. Approximately 4%of the units moved between 0.5 and 1.0 D_{n50}. The smaller movements were not associated with increased extractions. Only 3 units were extracted during the test series, one during Test 6c and two more during Test 6e.

The greatest armour movement during Test Series 6 occurred on the seaward end of the West Breakwater roundhead where 2.3% of the rock units were extracted. Comparing this figure with the tolerable armour movement described in Section 5.2, the movement represents "Little - Moderate" damage. The armour movement during Test Series 6 was therefore acceptable (Plates 23a and 23b).

7.7.3 Test Series 7 (Construction phase tests)

During Test Series 7, tests were carried out to assess the performance of each of the two layouts during the 1:10 year storm event. The layouts represent two stages of the new West Breakwater construction, combined with the removal of the old West Pier. The layouts are shown in Figure17 and Plates 14-17.

For analysis purposes the outer end of each structure was divided into two areas.

1. The roundhead from the toe to the top of the slope and
2. The remaining length of trunk armoured with the same size of rock as the roundhead.

The shorter structure, Test 7a, and the longer structure, Test 7b, were armoured with 6-10t and 10-15t rock respectively. Details of the rock armour stability analysis are given in Tables 22 and 23. In terms of extractions (rocks moved by more than $1D_{n50}$) the roundheads in each test suffered less than 5% damage. This is acceptable because of the temporary nature of the roundhead structure. Less movement was seen on the trunk sections where the maximum percentage of extracted units occurred on the shorter breakwater section armoured with 6-10t rock units. In all the tests, the extracted units remained on the structure slope and continued to provide protection to the structure.

8. CONCLUSIONS

1. **Armour Stability**

   The 3-6t, 6-10t and 10-15t rock armouring to the West Breakwater and East Pier extension were stable under all conditions. Some initial settlement should be expected in the prototype.

   The 1:3 slope of the inner harbour revetment was protected by 1-3t armour. This was placed in two layers over a core of 0.3 – 1t. The rock was stable for all conditions tested.

   It is interesting to note that during each of the model test parts the water level was held constant. The stability tests are therefore conservative as in reality any particular level on the structure will only be exposed to direct wave attack for relatively brief periods of time, for example, approximately 1 hour over high water.

2. **Navigation**

   Navigation into the harbour has been improved by widening the entrance (to 20m) and protecting the entrance from the most severe waves from the south west. Wave conditions within the harbour have also been significantly reduced improving navigation within the harbour.

3. **Wave disturbance**

   Results show that the proposed 20m entrance significantly reduces wave conditions within the harbour. For waves from 160°N, wave heights within the harbour were reduced by between 0.25 and 0.45m (a reduction of approximately 50-55% for both the 1:1 year and 1:10 year condition). Similar reductions of between 0.35 and 0.6m (a reduction of 57% for both the 1:1 and 1:10 year events) were observed for waves from 220°N.

   Tests were carried out to assess whether additional structures within the outer harbour could be used to reduce the wave disturbance in the inner and outer harbours. These structures were shown to have little effect. In fact, structures placed on the harbour side of the Western Breakwater reflected wave energy into the inner harbour and therefore should not be considered.

4. **Slipway location**

   The preferred location for a slipway based on hydraulic considerations would be close to the root of the old West Pier, where the wave conditions are less severe. However, access to the slipway would be difficult and the slipway would direct vessels directly into the fairway of the inner harbour. Therefore, the preferred location is 10m away from the new West Breakwater.

5. **Wave overtopping**

   The proposed breakwater and western beach recharge schemes significantly reduce the volume of water overtopping the walls surrounding the harbour.

   The recharged West Beach and the root of the proposed breakwater provide significant shelter to the frontage behind. Overtopping of the area protected by the root of the breakwater was eliminated during the 1:10 year condition and significantly reduced for larger return period events.

   It should be noted that significant amounts of water overtop the proposed Western Breakwater for return events greater than 1:1 for wave from 220°N. This overtopping will not affect the integrity of the structures and is considered appropriate to the design criteria set for the new breakwater. It will be important that boats should not be allowed to moor along the inside of the breakwaters under these conditions and access to the crest by the public should be restricted.

6. **Protection to East Beach**

The new Western Breakwater and East Pier extension provide additional shelter to East Beach. The increased length of the East Pier allows East Beach to accumulate against East Pier during periods of south-easterly waves. The wider shingle beach then provides a flood defence with a higher standard of service than at present and the breach probability effectively reduces to zero. Waves from the south west push beach material in an easterly direction thus reducing the requirement for beach recycling to manageable levels.

The angle of wave attack remained constant during the physical model tests. In the prototype, wave directions vary over relatively short periods of time and therefore the beach profile changes measured within the model are exaggerated. For this reason the long term planshape response of both West and East Beach was assessed in the numerical model BEACHPLAN. The BEACHPLAN model is able to predict shoreline change by taking into account changing wave direction, and the percentage of time over which waves come from a particular direction and is therefore the appropriate tool to assess long term change.

7. **Wave induced currents in the inner and outer harbour**

Currents in the inner and outer harbours of the proposed layout are small. However, reversing currents of up to 0.7m/s were measured within the harbour channel during several of the tests. This is significantly larger than those noted for the existing layout.

8. **Flow induced currents in the inner and outer harbour**

Sluicing tests indicate a similar current pattern in the inner harbour for both the existing and preferred layouts at both the high and low water levels tested. With the preferred scheme a circulatory flow in the outer harbour was created at both water level conditions. This was of the order of 0.2-0.5m/s. Some siltation of fine sediments can be expected within the outer harbour.

9. **Stability of temporary works**

Two construction phase layouts were assessed. The scar ends were stable for both lengths of Western Breakwater tested. Wave conditions within the harbour for both layouts were less than for the existing layout. As expected, the longer breakwater tested during Test 7b provided the best protection.

At the harbour entrance the wave heights recorded during the short breakwater test, Test 7a, were greater than for the existing layout. This is probably due to reflections from the unprotected end caisson of the Western Breakwater. Wave conditions in the harbour entrance are not significantly different to those experienced at present. Therefore demolition of West Pier could start as the West Breakwater reaches this point.

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Tables

Table 1 Test wave conditions and water levels

Wave direction (deg)	Return period (years)	Calibration at –8.0mODN Contour			
		H_s (m)	T_m (s)	T_p (s)	Water level (mODN)
220	10 (Low Water)	5.3	8.6	10.4	0.00
220	0.1	2.7	6.1	7.4	1.85
220	1	4.0	7.5	9.0	2.00
220	10	4.0	7.5	9.0	2.50
220	100	5.0	8.3	10.1	2.70
220	2000	5.0	8.3	10.1	3.00
220	Swell (1 year)	2.3	16.0	19.3	1.85
220	Swell (100 year)	4.1	16.0	19.3	1.85
160	0.1	1.4	4.4	5.3	2.00
160	0.5	1.8	5.0	6.0	2.30
160	1.00	2.0	5.3	6.4	2.60
160	10	2.5	5.9	7.1	2.80
160	100	3.0	6.5	7.8	3.00

Table 2 Test programme

Summary of Test Conditions

Test Series (-)	Wave Direction (deg)	Structure (-)	Test Part (-)	Comment	Return Period (Years)	Hs (m)	Prototype Tm (s)	Water Level (mODN)	Wave Disturbance	Overtopping Rates	Beach Profile	Photograph Beach Plan	Re-Profile Beach	Shingle Intrusion	Fine Sediment Tracers (Fluvial)	West Beach Tracers (Wave)	Harbour Tracers (Wave)	East Beach Tracers (Wave)	Armour Stability (Front)	Armour Stability (Back)	Fluvial Pump Sluices
1	220	Existing	a		10 (Low Water)	5.3	8.6	0.00	✓ (measured)	✓ (measured)*			✓			✓ (visual)	✓ (visual)	✓ (visual)			
1	220	Existing	b		0.1	2.7	6.1	1.85	✓ (measured)	✓ (measured)*			✓			✓ (visual)	✓ (visual)	✓ (visual)			
1	220	Existing	c		1.0	4.0	7.5	2.00	✓ (measured)	✓ (measured)*			✓			✓ (visual)	✓ (visual)	✓ (visual)			
1	220	Existing	d		10	4.0	7.5	2.50	✓ (measured)	✓ (measured)*			✓			✓ (visual)	✓ (visual)	✓ (visual)			
1	220	Existing	e		100	5.0	8.3	2.66	✓ (measured)	✓ (measured)*	✓ (measured)	✓ (photo)	✓			✓ (visual)	✓ (visual)	✓ (visual)			
1	220	Existing	f		2000	5.0	8.3	3.00	✓ (measured)	✓ (measured)*	✓ (measured)	✓ (photo)	✓			✓ (visual)	✓ (visual)	✓ (visual)			
1	220	Existing	g		Swell (1 Year)	2.3	16.0	1.85	✓ (measured)	✓ (measured)*			✓			✓ (visual)	✓ (visual)	✓ (visual)			
1	220	Existing	h		Swell (100 Year)	4.1	16.0	1.85	✓ (measured)	✓ (measured)*			✓			✓ (visual)	✓ (visual)	✓ (visual)			
Calibrate longshore movement in physical model (infinitely Long Groyne & Beach Plan)																					
2	160	Existing	a	Fluvial (16m3/s)	-	-	-	-1.65							✓ (photo)						✓ (On)
2	160	Existing	b	Fluvial (16m3/s)	-	-	-	0.00							✓ (photo)						✓ (On)
2	160	Existing	c		0.1	1.4	4.4	2.00	✓ (measured)	✓ (measured)*	✓ (measured)	✓ (photo)	✓	✓ (measured)**		✓ (visual)	✓ (visual)	✓ (visual)			
2	160	Existing	d		0.5	1.8	5.0	2.30	✓ (measured)	✓ (measured)*	✓ (measured)	✓ (photo)	✓	✓ (measured)**		✓ (visual)	✓ (visual)	✓ (visual)			
2	160	Existing	e		1.0	2.0	5.3	2.60	✓ (measured)	✓ (measured)*	✓ (measured)	✓ (photo)	✓	✓ (measured)**		✓ (visual)	✓ (visual)	✓ (visual)			
2	160	Existing	f		10	2.5	5.9	2.80	✓ (measured)	✓ (measured)*			✓			✓ (visual)	✓ (visual)	✓ (visual)			
2	160	Existing	g		100	3.0	6.5	3.00	✓ (measured)	✓ (measured)*			✓			✓ (visual)	✓ (visual)	✓ (visual)			
Demolish West Pier and construct the new Breakwater																					
3	160	Scheme	a	Layout of outer harbour structures	1.0	2.0	5.3	2.6	✓ (measured)												
3	160	Scheme	b	Layout of outer harbour structures	1.0	2.0	5.3	2.6	✓ (measured)												
3	160	Scheme	c	Layout of outer harbour structures	1.0	2.0	5.3	2.6	✓ (measured)												
3	160	Scheme	d	Layout of outer harbour structures	1.0	2.0	5.3	2.6	✓ (measured)												
3	160	Scheme	e	Layout of outer harbour structures	1.0	2.0	5.3	2.6	✓ (measured)												
3	160	Scheme	f	Layout of outer harbour structures	1.0	2.0	5.3	2.6	✓ (measured)												
3	160	Scheme	g	Layout of outer harbour structures	1.0	2.0	5.3	2.6	✓ (measured)												
3	160	Scheme	h	Layout of outer harbour structures	1.0	2.0	5.3	2.6	✓ (measured)												
Beach Plan Runs																					
4	160	Scheme	a	East Pier Extension	0.1	1.4	4.4	2.00	✓ (measured)	✓ (measured)	✓ (measured)	✓ (photo)	✓	✓ (measured)		✓ (visual)	✓ (visual)	✓ (visual)	✓ (photo)	✓ (photo)	
4	160	Scheme	b	East Pier Extension	0.5	1.8	5.0	2.30	✓ (measured)	✓ (measured)	✓ (measured)	✓ (photo)	✓	✓ (measured)		✓ (visual)	✓ (visual)	✓ (visual)	✓ (photo)	✓ (photo)	
4	160	Scheme	c	East Pier Extension	1.0	2.0	5.3	2.60	✓ (measured)	✓ (measured)	✓ (measured)	✓ (photo)	✓	✓ (measured)		✓ (visual)	✓ (visual)	✓ (visual)	✓ (photo)	✓ (photo)	
4	160	Scheme	d	East Pier Extension	10	2.5	5.9	2.80	✓ (measured)	✓ (measured)						✓ (visual)	✓ (visual)	✓ (visual)	✓ (photo)	✓ (photo)	
4	160	Scheme	e	East Pier Extension	100	3.0	6.5	3.00	✓ (measured)	✓ (measured)						✓ (visual)	✓ (visual)	✓ (visual)	✓ (photo)	✓ (photo)	
5	220	Scheme	a	Fluvial (16m3/s)	-	-	-	-1.65							✓ (photo)						✓ (On)
5	220	Scheme	b	Fluvial (16m3/s)	-	-	-	0.00							✓ (photo)						✓ (On)
5	220	Scheme	c		10 (Low Water)	5.3	8.6	0.00	✓ (measured)				✓	✓ (measured)		✓ (visual)	✓ (visual)	✓ (visual)	✓ (photo)	✓ (photo)	
5	220	Scheme	d		0.1	2.7	6.1	1.85	✓ (measured)				✓	✓ (measured)		✓ (visual)	✓ (visual)	✓ (visual)	✓ (photo)	✓ (photo)	
5	220	Scheme	e		1.0	4.0	7.5	2.00	✓ (measured)				✓	✓ (measured)		✓ (visual)	✓ (visual)	✓ (visual)	✓ (photo)	✓ (photo)	
5	220	Scheme	f		10	4.0	7.5	2.50	✓ (measured)				✓	✓ (measured)		✓ (visual)	✓ (visual)	✓ (visual)	✓ (photo)	✓ (photo)	
5	220	Scheme	g		100	5.0	8.3	2.70	✓ (measured)	✓ (measured)	✓ (measured)	✓ (photo)	✓	✓ (measured)		✓ (visual)	✓ (visual)	✓ (visual)	✓ (photo)	✓ (photo)	
5	220	Scheme	h		2000	5.0	8.3	3.00	✓ (measured)	✓ (measured)	✓ (measured)	✓ (photo)	✓	✓ (measured)		✓ (visual)	✓ (visual)	✓ (visual)	✓ (photo)	✓ (photo)	
5	220	Scheme	i		Swell (1 Year)	2.3	16.0	1.85	✓ (measured)	✓ (measured)*			✓	✓ (measured)		✓ (visual)	✓ (visual)	✓ (visual)	✓ (photo)	✓ (photo)	
5	220	Scheme	j		Swell (100 Year)	4.1	16.0	1.85	✓ (measured)	✓ (measured)*			✓	✓ (measured)		✓ (visual)	✓ (visual)	✓ (visual)	✓ (photo)	✓ (photo)	
Go to MAFF for 'Approval in Principle'																					
6	220	Preferred Scheme	a		10 (Low water)	5.3	8.6	0.00	✓ (measured)	✓ (measured)			✓	✓ (measured)					✓ (photo)	✓ (photo)	
6	220	Preferred Scheme	b		0.1	2.7	6.1	1.85	✓ (measured)	✓ (measured)			✓	✓ (measured)							
6	220	Preferred Scheme	c		1.0	4.0	7.5	2.00	✓ (measured)	✓ (measured)			✓	✓ (measured)					✓ (photo)	✓ (photo)	
6	220	Preferred Scheme	d		10	4.0	7.5	2.50	✓ (measured)	✓ (measured)			✓	✓ (measured)					✓ (photo)	✓ (photo)	
6	220	Preferred Scheme	e		100	5.0	8.3	2.70	✓ (measured)	✓ (measured)	✓ (measured)	✓ (photo)	✓	✓ (measured)					✓ (photo)	✓ (photo)	
6	220	Preferred Scheme	f		2000	5.0	8.3	3.00	✓ (measured)	✓ (measured)	✓ (measured)	✓ (photo)	✓	✓ (measured)					✓ (photo)	✓ (photo)	
7	220	Temporary Works	a		1.0	4.0	7.5	2.50	✓ (measured)	✓ (measured)									✓ (visual)	✓ (visual)	
7	220	Temporary Works	b		1.0	4.0	7.5	2.50	✓ (measured)										✓ (visual)	✓ (visual)	

* Overtopping on East beach with a 50m breach at +5.5mODN and rest at +7.5mODN. If waves visually overtop the +7.5mODN then beach to be reformed with constant level of +7.5mODN and test re-run for overtopping.
** Shingle intrusion measured three times during each test part

Table 3 Wave calibrations for waves from 160°N

Return Period (years)	SWL (mODN)	H_s (m)	T_m (s)	Deep water Spectral results		Calibration point. Spectral results		Deep water Statistical results		Calibration point. Statistical results			
				H_s (m)	T_m (s)	H_s (m)	T_m (s)	H_s (m)	T_m (s)	H_s (m)	T_m (s)	H_{max} (m)	H_{10} (m)
0.1	2.00	1.4	4.4	1.32	4.2	1.37	4.3	1.32	4.4	1.41	4.4	2.68	1.81
0.5	2.30	1.8	5.0	1.67	5.0	1.77	4.9	1.71	5.1	1.81	5.0	3.16	2.30
1	2.60	2.0	5.3	1.84	5.3	1.92	5.2	1.86	5.4	2.00	5.4	3.44	2.53
10	2.80	2.5	5.9	2.27	5.9	2.41	5.9	2.34	6.1	2.52	6.0	4.73	3.23
100	3.00	3.0	6.5	2.79	6.4	2.98	6.2	2.83	6.7	3.10	6.6	5.11	3.87

Spectral results are band 31 data

Table 4 Wave calibrations for waves from 220°N

Return Period (years)	SWL (mODN)	H_s (m)	T_m (s)	Deep water Spectral results		Calibration point. Spectral results		Deep water Statistical results		Calibration point. Statistical results			
				H_s (m)	T_m (s)	H_s (m)	T_m (s)	H_s (m)	T_m (s)	H_s (m)	T_m (s)	H_{max} (m)	H_{10} (m)
10 (low water)	0.0	5.3	8.6	5.20	7.7	4.77	6.7	5.4	8.7	5.5	8.9	7.6	6.3
0.1	1.85	2.7	6.1	2.78	6.2	2.67	5.9	2.84	6.3	2.71	6.3	5.15	3.48
1	2.00	4.0	7.5	4.17	7.27	3.93	7.0	4.23	7.5	4.17	7.9	7.38	5.36
10	2.50	4.0	7.5	4.13	7.3	3.92	7.1	4.17	7.5	4.25	7.9	8.52	5.42
100	2.70	5.0	8.3	4.97	7.8	4.68	7.1	5.14	8.4	5.29	8.6	9.09	6.54
2000	3.00	5.0	8.3	5.01	7.8	4.67	7.1	5.2	8.4	5.15	8.7	8.30	6.44
Swell (1 year)	1.85	2.3	16.0	1.84	15.0	2.29	14.1	1.83	14.0	2.39	14.6	4.87	3.04
Swell (100 year)	1.85	4.1	16.0	3.13	14.9	3.8	11.8	3.13	15.0	4.30	14.9	8.75	5.65

Spectral results are band 31 data

Table 5 Measured wave conditions, waves from 220°N, existing layout

Return Period	10 Low water		0.1		1		10		100		2000		Swell (1 year)		Swell (100 year)	
Design H_s (m) offshore	5.3		2.7		4.0		4.0		5.0		5.0		2.3		4.1	
Design T_m (s) offshore	8.6		6.1		7.5		7.5		8.3		8.3		16.0		16.0	
Design SWL (mODN)	0.0		1.85		2.0		2.5		2.7		3.0		1.85		1.85	
Probe location	H_{sl} (m)	H_{ss} (m)	H_{sl} (m)	H_{ss} (m)	H_{sl} (m)	H_{ss} (m)	H_{sl} (m)	H_{ss} (m)	H_{sl} (m)	H_{ss} (m)	H_{sl} (m)	H_{ss} (m)	H_{sl} (m)	H_{ss} (m)	H_{sl} (m)	H_{ss} (m)
0	0.73	5.25	0.25	2.80	0.51	4.13	0.49	4.14	0.67	5.07	0.72	5.19	0.27	1.92	0.48	3.16
1	0.95	4.59	0.28	2.66	0.64	3.95	0.59	3.88	0.79	4.68	0.80	4.77	0.29	2.37	0.52	3.79
2	0.90	2.36	0.49	2.48	0.98	3.60	0.96	3.69	1.07	4.13	1.12	4.34	0.47	2.80	0.68	3.21
3	1.06	2.38	0.55	2.82	0.93	3.53	0.92	3.75	1.14	4.19	1.09	4.36	0.36	1.98	0.77	3.01
4	0.96	2.30	0.60	2.98	1.02	3.18	1.01	3.35	1.17	3.53	1.15	3.48	0.46	2.62	0.77	2.80
5	1.20	1.82	0.54	1.93	1.04	2.65	1.05	2.87	1.26	3.07	1.29	3.26	0.54	1.80	0.99	2.59
6	0.92	1.38	0.37	0.96	0.83	2.25	0.85	2.31	1.06	3.05	1.09	3.24	0.41	1.54	0.89	2.85
7	1.03	2.43	0.79	2.97	1.18	3.79	1.13	3.92	1.29	4.25	1.31	4.65	0.57	3.29	0.71	3.37
8	1.12	1.92	0.99	2.62	1.42	3.09	1.36	3.21	1.43	3.33	1.39	3.43	0.60	3.12	0.80	3.20
9	0.48	0.50	0.46	0.97	0.62	1.12	0.61	1.15	0.76	1.33	0.65	1.11	0.33	1.24	0.41	1.28
10	0.97	2.50	0.52	2.44	0.92	3.31	0.90	3.57	1.02	3.90	1.05	3.96	0.38	2.37	0.60	3.00
11	0.38	0.38	0.29	1.01	0.43	1.04	0.46	1.06	0.52	0.93	0.52	0.78	0.32	1.06	0.43	1.10
12	0.42	0.34	0.39	0.99	0.58	1.07	0.59	1.04	0.64	0.93	0.64	0.84	0.33	1.21	0.43	1.22
13	0.35	0.31	0.34	0.77	0.49	0.87	0.48	0.81	0.54	0.72	0.54	0.57	0.29	0.94	0.36	0.96
14	0.40	0.28	0.39	0.71	0.50	0.80	0.50	0.67	0.51	0.56	0.51	0.39	0.31	0.97	0.43	0.96
15	0.57	0.24	0.52	0.72	0.64	0.74	0.66	0.63	0.73	0.44	0.69	0.27	0.45	0.81	0.65	0.80

Table 6 Overtopping discharges, waves from 220°N, existing layout

Test	Return Period (years)	Nominal Offshore Wave Conditions H_s (m)	T_m (m)	Discharge Tank 1 (l/s)	Discharge Tank 2 (l/s)	Discharge Tank 3 (l/s)	Discharge Tank 4 (l/s)	Discharge Tank 5 (l/s)	Discharge Tank 6 (l/s)	Discharge Tank 7 (l/s)	Discharge Tank 8 (l/s)	Discharge Tank 9 (l/s)
1c	1	4.0	7.5	0.0	0.0	0.0	0.0	0.0	37	0.0	0.0	0.6
1d	10	4.0	7.5	0.0	0.0	0.0	0.0	26	317	0.0	0.0	55
1e	100	5.0	8.3	0.0	0.0	29	33	115	873	133	5.1	508
1e *	100	5.0	8.3	0.0	0.0	29	33	115	**349**	133	5.1	508
1f *	2000	5.0	8.3	0.0	0.0	0.0	523	464	**897**	479	280	889
1g *	1 (swell)	2.3	16.0	0.0	0.0	0.0	0.0	2	**19**	1.3	0.0	96
1g	1 (swell)	2.3	16.0	0.0	0.0	0.0	0.0	2	34.5	1.3	0.0	96
1h	100 (swell)	4.1	16.0	0.0	0.0	0.0	0.0	0.5	138	191	108	474
1h *	100 (swell)	4.1	16.0	0.0	0.0	0.0	0.0	0.5	**127**	191	108	474

North wall gates closed

Table 7 Measured wave conditions, waves from 160°N, existing layout

Return Period	0.1		0.5		1		10		100	
Design H_s (m) offshore	1.4		1.8		2.0		2.5		3.0	
Design T_m (s) offshore	4.4		5.0		5.3		5.9		6.5	
Design SWL (m ODN)	2.0		2.3		2.6		2.8		3.0	
Probe location	H_{sl} (m)	H_{ss} (m)	H_sl (m)	H_{ss} (m)	H_{sl} (m)	H_{ss} (m)	H_{sl} (m)	H_{ss} (m)	H_{sl} (m)	H_{ss} (m)
0	0.05	1.34	0.07	1.69	0.08	1.87	0.13	2.33	0.15	2.81
1	0.06	1.32	0.09	1.77	0.14	1.94	0.19	2.51	0.25	3.12
2	0.10	1.24	0.17	1.71	0.22	2.01	0.35	2.49	0.54	3.17
3	0.10	1.29	0.17	1.65	0.21	1.87	0.38	2.42	0.48	2.89
4	0.13	1.22	0.23	1.40	0.28	1.64	0.46	2.12	0.61	2.82
5	0.15	1.35	0.30	1.72	0.37	2.05	0.58	2.48	0.88	2.87
6	0.16	1.35	0.30	1.96	0.35	2.00	0.55	2.50	0.87	2.88
7	0.17	1.21	0.27	1.49	0.33	1.88	1.49	2.31	0.75	3.28
8	0.21	1.01	0.35	1.31	0.44	1.64	0.64	2.11	0.90	2.82
9	0.11	0.40	0.18	0.60	0.22	0.78	0.29	0.86	0.37	0.95
10	0.12	1.26	0.20	1.68	0.26	2.05	0.40	2.59	0.56	3.17
11	0.09	0.47	0.14	0.68	0.16	0.84	0.21	1.00	0.28	0.99
12	0.10	0.52	0.16	0.73	0.19	0.84	0.25	0.85	0.33	0.84
13	0.09	0.38	0.14	0.50	0.15	0.58	0.20	0.61	0.23	0.57
14	0.10	0.46	0.17	0.53	0.20	0.48	0.27	0.42	0.32	0.36
15	0.12	0.38	0.22	0.47	0.30	0.51	0.40	0.47	0.53	0.36

Table 8 Overtopping discharges, waves from 160°N, existing layout

Test	Return Period (years)	Nominal Offshore Wave Conditions H_s (m)	T_m (m)	Discharge Tank 1 (l/s)	Discharge Tank 2 (l/s)	Discharge Tank 3 (l/s)	Discharge Tank 4 (l/s)	Discharge Tank 5 (l/s)	Discharge Tank 6 (l/s)	Discharge Tank 7 (l/s)	Discharge Tank 8 (l/s)	Discharge Tank 9 (l/s)
2c	0.1	1.4	4.4	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
2d	0.5	1.8	5.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
2e	1	2.0	5.3	0.0	0.0	0.0	0.0	0.0	55	0.0	0.0	0.0
2f	10	2.5	5.9	0.0	0.0	0.0	0.0	0.8	246	0.0	0.0	4.5
2g*	100	3.0	6.5	0.0	0.0	0.0	26	95	**76**	0.0	0.0	110
2g	100	3.0	6.5	0.0	0.0	0.0	26	95	716	0.0	0.0	0.0

2g* North wall gates closed

Table 9 Measured wave conditions, 1:1 year condition, waves from 160°N, tests to assess the performance of a number of entrance layouts

Return Period	Test 3A		Test 3B		Test 3C		Test 3D		Test 3E		Test 3F		Test 3G		Test 3H	
Design H_s (m) offshore	2.0		2.0		2.0		2.0		2.0		2.0		2.0		2.0	
Design T_m (s) offshore	5.3		5.3		5.3		5.3		5.3		5.3		5.3		5.3	
Design SWL (m ODN)	2.6		2.6		2.6		2.6		2.6		2.6		2.6		2.6	
Probe location	H_{sl} (m)	H_{ss} (m)	H_{sl} (m)	H_{ss} (m)	H_{sl} (m)	H_{ss} (m)	H_{sl} (m)	H_{ss} (m)	H_{sl} (m)	H_{ss} (m)	H_{sl} (m)	H_{ss} (m)	H_{sl} (m)	H_{ss} (m)	H_{sl} (m)	H_{ss} (m)
0	0.08	1.85	0.09	1.86	0.09	1.86	0.08	1.86	0.09	1.86	0.08	1.86	0.08	1.85	0.08	1.84
1	0.12	1.93	0.12	1.93	0.12	1.94	0.12	1.93	0.12	1.94	0.11	1.94	0.11	1.89	0.11	1.92
2	0.29	0.63	0.29	0.70	0.28	0.56	0.30	0.69	0.29	0.49	0.29	0.66	0.30	0.62	0.27	0.43
3	0.30	0.91	0.36	0.41	0.32	0.50	0.32	0.79	0.31	0.57	0.31	1.26	0.33	1.39	0.28	1.09
4	0.30	1.04	0.31	0.86	0.30	0.30	0.31	0.59	0.30	0.43	0.30	1.06	0.32	1.19	0.28	0.91
5	0.36	0.67	0.34	0.71	0.33	0.48	0.35	0.59	0.35	0.45	0.35	0.68	0.35	0.73	0.32	0.52
6	0.36	0.89	0.38	0.51	0.35	0.45	0.36	0.75	0.35	0.53	0.35	1.11	0.35	1.22	0.31	0.97
7	0.34	1.49	0.31	1.73	0.30	1.68	0.30	1.61	0.32	1.58	0.30	1.21	0.29	1.53	0.28	1.49
8	0.29	1.07	0.28	0.83	0.27	1.06	0.29	0.87	0.28	1.17	0.27	1.04	0.28	1.17	0.24	0.80
9	0.17	0.47	0.17	0.51	0.16	0.57	0.17	0.50	0.16	0.57	0.16	0.51	0.15	0.53	0.15	0.34
10	0.35	1.04	0.36	0.56	0.34	0.32	0.35	0.66	0.34	0.43	0.34	1.08	0.34	1.21	0.31	0.94
11	0.16	0.47	0.15	0.48	0.16	0.53	0.16	0.52	0.16	0.58	0.16	0.49	0.15	0.50	0.14	0.32
12	0.19	0.46	0.18	0.49	0.18	0.56	0.19	0.51	0.18	0.57	0.18	0.53	0.17	0.55	0.18	0.34
13	0.16	0.29	0.16	0.31	0.15	0.34	0.16	0.32	0.16	0.36	0.16	0.32	0.16	0.34	0.15	0.23
14	0.20	0.31	0.19	0.36	0.19	0.35	0.20	0.34	0.19	0.37	0.19	0.37	0.19	0.41	0.18	0.28
15	0.24	0.29	0.24	0.34	0.22	0.34	0.24	0.33	0.24	0.36	0.24	0.33	0.22	0.36	0.21	0.24

Table 10 Measured wave conditions, waves from 160°N, Test Series 4

Return Period	0.1		0.5		1		10		100	
Design H_s (m) offshore	1.4		1.8		2.0		2.5		3.0	
Design T_m (s) offshore	4.4		5.0		5.3		5.9		6.5	
Design SWL (m ODN)	2.0		2.3		2.6		2.8		3.0	
Probe location	H_{sl} (m)	H_{ss} (m)	H_{sl} (m)	H_{ss} (m)	H_{sl} (m)	H_{ss} (m)	H_{sl} (m)	H_{ss} (m)	H_{sl} (m)	H_{ss} (m)
0	0.04	1.30	0.07	1.69	0.09	1.85	0.12	2.30	0.15	2.79
1	0.05	1.29	0.08	1.72	0.12	1.91	0.18	2.46	0.26	2.97
2	0.09	1.28	0.17	1.70	0.20	1.95	0.34	2.55	0.49	3.04
3	0.10	1.25	0.17	1.65	0.21	1.86	0.38	2.35	0.50	2.71
4	0.11	1.14	0.21	1.51	0.24	1.60	0.43	2.19	0.60	2.95
5	0.13	1.19	0.23	1.68	0.30	1.83	0.44	2.15	0.65	2.49
6	0.20	1.40	0.30	1.82	0.36	2.04	0.53	2.42	0.75	2.69
7	0.15	1.09	0.24	1.34	0.28	1.61	0.53	1.95	0.80	2.25
8	0.11	0.60	0.22	0.85	0.25	0.78	0.47	0.88	0.61	1.02
9	0.08	0.23	0.14	0.40	0.16	0.41	0.23	0.38	0.26	0.45
10	0.11	0.50	0.21	0.70	0.24	0.72	0.43	0.98	0.55	1.21
11	0.07	0.23	0.14	0.37	0.14	0.39	0.21	0.39	0.25	0.39
12	0.08	0.27	0.15	0.38	0.18	0.38	0.27	0.37	0.31	0.33
13	0.08	0.21	0.14	0.27	0.15	0.27	0.24	0.27	0.28	0.26
14	0.09	0.20	0.16	0.31	0.19	0.30	0.28	0.21	0.34	0.18
15	0.11	0.20	0.19	0.26	0.23	0.27	0.33	0.21	0.41	0.14

Table 11 Overtopping discharges, waves from 160°N, Test Series 4

Test	Return Period (years)	Nominal Offshore Wave Conditions H_s (m)	T_m (m)	Discharge Tank 1 (l/s)	Discharge Tank 2 (l/s)	Discharge Tank 3 (l/s)	Discharge Tank 4 (l/s)	Discharge Tank 5 (l/s)	Discharge Tank 6 (l/s)	Discharge Tank 7 (l/s)	Discharge Tank 8 (l/s)	Discharge Tank 9 (l/s)
4a	0.1	1.4	4.4	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
4b	0.5	1.8	5.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
4c	1	2.0	5.3	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
4d	10	2.5	5.9	0.0	0.0	0.0	0.0	0.0	70	0.0	0.0	0.0
4e*	100	3.0	6.5	0.0	0.0	0.0	0.0	11	**61**	0.0	0.0	0.0
4e	100	3.0	6.5	0.0	0.0	0.0	0.0	11	503	0.0	0.0	0.0

4e* North wall gates closed

Table 12 Measured wave conditions, waves from 220°N, Test Series 5

Return Period	10 Low water		0.1		1		10		100		2000		Swell (1 year)		Swell (100 year)	
Design H_s (m) offshore	5.3		2.7		4.0		4.0		5.0		5.0		2.3		4.1	
Design T_m (s) offshore	8.6		6.1		7.5		7.5		8.3		8.3		16		16	
Design SWL (m ODN)	0.0		1.85		2.0		2.5		2.7		3.0		1.85		1.85	
Probe location	H_{sl} (m)	H_{ss} (m)	H_{sl} (m)	H_{ss} (m)	H_{sl} (m)	H_{ss} (m)	H_{sl} (m)	H_{ss} (m)	H_{sl} (m)	H_{ss} (m)	H_{sl} (m)	H_{ss} (m)	H_{sl} (m)	H_{ss} (m)	H_{sl} (m)	H_{ss} (m)
0	0.52	4.87	0.18	2.67	0.34	0.33	3.84	3.81	0.45	4.69	0.43	4.67	0.18	1.87	0.34	3.08
1	0.91	4.54	0.26	2.59	0.57	0.54	4.05	4.00	0.72	4.69	0.72	4.83	0.23	2.35	0.48	3.81
2	0.87	2.72	0.53	2.70	0.92	0.96	3.80	3.71	1.05	4.26	1.06	4.30	0.45	2.46	0.65	2.89
3	1.00	2.44	0.59	2.73	0.94	0.88	3.61	3.59	1.02	4.05	1.02	4.09	0.36	1.91	0.67	2.79
4	1.02	2.21	0.58	2.71	0.92	0.88	3.37	3.26	1.04	3.39	1.09	3.62	0.42	2.55	0.70	2.81
5	1.29	1.89	0.64	1.87	1.17	1.15	2.70	2.65	1.34	2.97	1.32	3.06	0.57	2.16	1.06	2.85
6	-	-	0.55	0.73	0.97	0.97	1.88	1.69	1.17	2.68	1.14	2.60	0.50	1.00	0.82	2.09
7	0.85	1.02	0.52	1.22	0.95	1.06	1.59	1.52	1.29	1.78	1.27	1.97	0.39	1.65	0.54	1.77
8	0.82	0.71	0.42	0.69	0.63	0.68	0.96	0.95	0.87	1.22	0.91	1.26	0.31	1.31	0.42	1.34
9	0.33	0.33	0.28	0.35	0.37	0.41	0.53	0.53	0.47	0.63	0.44	0.57	0.20	0.70	0.32	0.72
10	0.74	0.59	0.39	0.56	0.51	0.58	0.91	0.90	0.76	0.99	0.84	1.02	0.33	0.90	0.43	0.92
11	0.37	0.25	0.25	0.42	0.35	0.40	0.51	0.52	0.49	0.56	0.45	0.47	0.22	0.59	0.38	0.60
12	0.38	0.20	0.31	0.32	0.45	0.48	0.44	0.45	0.58	0.52	0.54	0.44	0.25	0.74	0.41	0.75
13	0.38	0.20	0.25	0.26	0.37	0.41	0.38	0.39	0.54	0.44	0.51	0.35	0.25	0.55	0.40	0.55
14	0.45	0.19	0.32	0.26	0.41	0.49	0.35	0.37	0.59	0.42	0.57	0.30	0.27	0.71	0.43	0.68
15	0.56	0.15	0.38	0.26	0.46	0.56	0.30	0.32	0.69	0.31	0.65	0.18	0.31	0.46	0.49	0.47

Table 13 Overtopping discharges, waves from 220°N, Test Series 5

Test	Return Period (years)	Nominal Offshore Wave Conditions H_s (m)	T_m (m)	Discharge Tank 1 (l/s)	Discharge Tank 2 (l/s)	Discharge Tank 3 (l/s)	Discharge Tank 4 (l/s)	Discharge Tank 5 (l/s)	Discharge Tank 6 (l/s)	Discharge Tank 7 (l/s)	Discharge Tank 8 (l/s)	Discharge Tank 9 (l/s)
5e	1	4.0	7.5	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
5f	10	4.0	7.5	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	25
5g	100	5.0	8.3	0.0	0.0	0.0	0.0	16	169	92	0.0	237
5ga*	100	5.0	8.3	0.0	0.0	0.0	0.0	16	**26**	92	0.0	237
5h	2000	5.0	8.3	0.0	0.0	0.0	51	129	0.0	216	135.0	520
5i	1 (swell)	2.3	16.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	19
5j	100 (swell)	4.1	16.0	0.0	0.0	0.0	0.0	0.0	10.3	38.5	1.2	168

5ga* North wall gates closed

Table 14 Measured wave conditions waves from 220°N, Test Series 6

Return Period	10 Low water		0.1		1		10		100		2000	
Design H_s (m) offshore	5.3		2.7		4.0		4.0		5.0		5.0	
Design T_m (s) offshore	8.6		6.1		7.5		7.5		8.3		8.3	
Design SWL (m ODN)	0.0		1.85		2.0		2.5		2.7		3.0	
Probe location	H_{sl} (m)	H_{ss} (m)	H_{sl} (m)	H_{ss} (m)	H_{sl} (m)	H_{ss} (m)	H_{sl} (m)	H_{ss} (m)	H_{sl} (m)	H_{ss} (m)	H_{sl} (m)	H_{ss} (m)
0	0.49	5.11	0.18	2.77	0.36	3.82	0.35	3.84	0.43	4.79	0.44	4.67
1	0.93	5.00	0.26	2.83	0.59	4.29	0.56	4.34	0.66	4.73	0.70	4.85
2	1.04	2.70	0.59	2.67	1.04	3.68	1.03	3.82	1.20	4.42	1.23	4.22
3	0.87	2.37	0.54	2.56	0.88	3.49	0.87	3.64	1.04	4.23	1.02	4.20
4	0.91	2.08	0.53	2.37	0.89	2.94	0.88	3.12	1.01	3.33	1.02	3.46
5	1.28	1.73	0.60	1.89	1.12	2.59	1.07	2.77	1.28	3.06	1.25	3.02
6	1.52	0.84	0.87	0.55	1.63	1.52	1.56	1.42	1.83	2.04	1.82	2.08
7	0.93	0.85	0.49 0.83	0.92 2.34	0.95	1.23	0.96	1.31	1.11	1.46	1.12	1.59
8	0.70	0.62	0.41 0.69	0.47	0.64	0.68	0.63	0.74	0.66	0.95	0.76	1.02
9	0.37	0.30	0.30 0.5	0.28	0.46	0.46	0.46	0.48	0.64	0.58	0.52	0.52
10	0.68	0.52	0.39	0.49	0.59	0.71	0.61	0.79	0.65	0.92	0.77	0.94
11	0.40	0.21	0.24	0.33	0.38	0.41	0.39	0.44	0.51	0.48	0.47	0.42
12	0.41	0.20	0.30	0.30	0.49	0.39	0.49	0.40	0.64	0.45	0.57	0.39
13	0.38	0.17	0.26	0.22	0.42	0.34	0.44	0.34	0.56	0.39	0.53	0.30
14	0.39	0.14	0.27	0.19	0.39	0.29	0.41	0.28	0.56	0.33	0.55	0.24
15	0.54	0.13	0.38	0.21	0.54	0.29	0.56	0.28	0.71	0.25	0.66	0.15

Table 15 Overtopping discharges, waves from 220°N, Test Series 6

Test	Return Period (years)	Nominal Offshore Wave Conditions H_s (m)	T_m (m)	Discharge Tank 1 (l/s)	Discharge Tank 2 (l/s)	Discharge Tank 3 (l/s)	Discharge Tank 4 (l/s)	Discharge Tank 5 (l/s)	Discharge Tank 6 (l/s)	Discharge Tank 7 (l/s)	Discharge Tank 8 (l/s)	Discharge Tank 9 (l/s)
6a	10 (low water)	5.3	8.6	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
6b	0.1	2.7	6.1	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
6c	1	4.0	7.5	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
6d	10	4.0	7.5	0.0	0.0	0.0	0.0	0.0	10.0	0.0	0.0	32
6e	100	5.0	8.3	0.0	0.0	0.0	0.0	9	141	100	0.0	264
6f	2000	5.0	8.3	0.0	0.0	0.0	95	117	0.0	216	88.0	508

Table 16 Measured wave conditions waves from 220°N, Test Series 7a, b

Return Period	10 (Test 7a)		10 (Test 7b)	
Design H_s (m) offshore	4.0		4.0	
Design T_m (s) offshore	7.5		7.5	
Design SWL (m ODN)	2.5		2.5	
Probe location	H_{sl} (m)	H_{ss} (m)	H_{sl} (m)	H_{ss} (m)
0	0.38	4.08	0.39	3.96
1	0.56	4.27	0.55	4.17
2	0.99	3.96	0.96	3.90
3	1.15	3.49	1.04	1.70
4	1.00	3.78	0.98	3.63
5	0.99	3.83	1.19	3.77
6	1.22	3.21	1.32	3.10
7	1.14	4.76	0.98	2.77
8	1.31	3.49	1.06	2.31
9	0.37	0.98	0.37	0.86
10	1.09	1.84	1.01	0.97
11	0.44	0.98	0.43	0.92
12	0.52	1.02	0.50	0.92
13	0.42	0.78	0.40	0.75
14	0.40	0.60	0.42	0.61
15	0.53	0.55	0.53	0.57

Table 17 Beach material deposited within the harbour entrance compared with the position of the measured beach profile at the still water level

Test	Cumulative Bypassing volume (m^3)	Distance to swl (m)	Cumulative sediment scaled time (Hours)	Test	Cumulative bypassing volume (m^3)	Distance to SWL (m)	Cumulative sediment scaled time (Hours)
Test 2c				**Test 4a**			
a	0	80	24.1	a	0	95	83.2
b	0	81	36.1	b	0	no profile	107.2
c	0	84	48.1	c	0	np profile	131.3
d	0	85	60.2	d	291	103	155.4
e	0	88	72.2	e	607	104	179.4
f	0	89	84.2	f	908	109	203.5
g	0	91	96.2	g	1227	no profile	227.5
h	0	no profile	120.3				
I	449	96	144.3				
j	988	97	156.4				
k	2115	95	168.4				
Test 2d				**Test 4b**			
a	0	no profile	27.3	a	0	no profile	29.9
b	34	88	54.5	b	0	no profile	43.6
c	370	92	68.1	c	304	95	70.8
d	1499	90	81.8	d	753	105	98.1
e	2016	91	95.4	e	1807	110	125.3
				f	3921	115	152.6
				g	6775	115	179.8
Test 2e				**Test 4c**			
a	15	77	14.6	a	0	no profile	31.8
b	42	81	29.2	b	595	94	61.0
c	794	85	43.7	c	2758	100	90.2
d	1999	88	58.3	d	4598	103	104.7
				e	6496	105	119.3
				f	8665	109	133.9
				g	11171	no profile	148.5

100m bypass started approx.

In terms of Beach Plan Base line add + 90m to get results in BP no's.

150-160 m

∴ bypass starts at approx ~~180-190m~~

150 m safety case. or when need to start reaction

108 m 2m WL

119 m

Table 18 Armour stability, West Breakwater section P1, transition between 3-6t and 6-10t rock armour

Test No.	Water Level (mOD)	Design H_s (m) offshore	Design T_m (sec) offshore	Rock armour 3-6t Percentage of (3-6t) rocks displaced		Rock armour 6-10t Percentage of (6-10t) rocks displaced	
				0.5-1 Dn_{50}	>1 Dn_{50}	0.5-1 Dn_{50}	>1 Dn_{50}
6a	0.0	5.3	8.6	0.0 (0.0)	0.0(0.0)	0.4(0.4)	0.4 (0.4)
6b	1.85	2.7	6.1	0.0 (0.0)	0.0(0.0)	0.0(0.4)	0.0 (0.4)
6c	2.0	4.0	7.5	0.0 (0.0)	0.0(0.0)	0.0(0.4)	0.0 (0.4)
6d	2.5	4.0	7.5	0.0 (0.0)	0.0(0.0)	0.0(0.4)	0.4 (0.9)
6e	2.66	5.0	8.3	0.0 (0.0)	0.0(0.0)	0.0(0.4)	0.0 (0.9)
6f	3.0	5.0	8.3	0.3 (0.3)	0.0(0.3)	0.0(0.4)	0.0 (0.9)

Note:
1. The numbers in brackets refers to the cumulative percentage of rocks displaced.
2. The total number of armour rocks were 390 (3-6t) and 462 (6-10t).

Table 19 Armour stability, West Breakwater section P2, 6-10t deeper water section

Test No.	Water Level (mOD)	Design H_s (m) offshore	Design T_m (sec) offshore	Rock armour 6-10t Percentage of (6-10t) rocks displaced	
				0.5-1 Dn_{50}	>1 Dn_{50}
6a	0.0	5.3	8.6	0.5 (0.5)	0.3 (0.3)
6b	1.85	2.7	6.1	0.0 (0.5)	0.0 (0.3)
6c	2.0	4.0	7.5	0.0 (0.5)	0.0 (0.3)
6d	2.5	4.0	7.5	0.3 (0.8)	0.0 (0.3)
6e	2.66	5.0	8.3	0.3 (1.0)	0.0 (0.3)
6f	3.0	5.0	8.3	0.0 (1.0)	0.0 (0.3)

Note:
1. The numbers in brackets refers to the cumulative percentage of rocks displaced.
2. The total number of armour rocks was 384.

Table 20 Armour stability, West Breakwater Roundhead sections P3 and P4, 10-15t rock armour

Test No.	Water Level (mOD)	Design H_s (m) offshore	Design T_m (sec) offshore	Section P3 (adjacent to breakwater trunk) Percentage of (10-15t) rocks displaced		Section P4 (adjacent to harbour entrance) Percentage of (10-15t) rocks displaced	
				0.5-1 Dn_{50}	>1 Dn_{50}	0.5-1 Dn_{50}	>1 Dn_{50}
6a	0.0	5.3	8.6	0.3 (0.3)	0.0 (0.0)	0.2 (0.2)	0.2 (0.2)
6b	1.85	2.7	6.1	0.0 (0.3)	0.2 (0.2)	0.0 (0.2)	0.2 (0.3)
6c	2.0	4.0	7.5	0.5 (0.8)	0.2 (0.4)	0.3 (0.5)	0.3 (0.7)
6d	2.5	4.0	7.5	0.2 (1.0)	0.5 (0.8)	0.7 (1.2)	0.3 (1.0)
6e	2.66	5.0	8.3	0.5 (1.5)	0.7 (1.5)	1.3 (2.5)	0.7 (1.7)
6f	3.0	5.0	8.3	0.0 (1.5)	0.2 (1.7)	0.0 (2.5)	0.7 (2.3)

Note:
1. The numbers in brackets refers to the cumulative percentage of rocks displaced.
2. The total number of armour rocks were 598 at P3 and 598 at P4.

Table 21 Armour stability, Eastern Breakwater extension, Sections P5 and P6

Test No.	Water Level (mOD)	Design Hs (m) offshore	Design Tm (sec) offshore	Section P5 (adjacent to harbour entrance) Percentage of (10-15t) rocks displaced		Section P6 (adjacent to beach) Percentage of (3-6t) rocks displaced at crest	
				0.5-1 Dn50	>1 Dn50	0.5-1 Dn50	>1 Dn50
6a	0.0	5.3	8.6	0.0 (0.0)	0.0 (0.0)	0.0 (0.0)	0.0 (0.0)
6b	1.85	2.7	6.1	0.0 (0.0)	0.4 (0.4)	0.0 (0.0)	0.0 (0.0)
6c	2.0	4.0	7.5	0.0 (0.0)	0.0 (0.4)	0.8 (0.8)	0.4 (0.4)
6d	2.5	4.0	7.5	0.0 (0.0)	0.0 (0.4)	1.6 (2.4)	0.0 (0.4)
6e	2.66	5.0	8.3	0.0 (0.0)	0.0 (0.4)	0.8 (3.2)	0.8 (1.2)
6f	3.0	5.0	8.3	0.0 (0.0)	0.0 (0.4)	0.8 (4.0)	0.0 (1.2)

Note: 1. The numbers in brackets refers to the cumulative percentage of rocks displaced.
2. The total number of armour rocks were 260 at P5 (10-15t) and 252 at P6 (3-6t).

Table 22 Armour stability, West Breakwater under construction, Test 7a, short length of new West Breakwater installed

Test No.	Water Level (mOD)	Design H_s (m) offshore	Design T_m (sec) offshore	Roundhead Percentage of (6-10t) rocks displaced		Breakwater Trunk Percentage of (6-10t) rocks displaced	
				0.5-1 Dn_{50}	>1 Dn_{50}	0.5-1 Dn_{50}	>1 Dn_{50}
7a	2.50	4.0	7.5	(1.9)	(1.9)	(0.6)	(1.7)

Note: 1. The numbers in brackets refers to the cumulative percentage of rocks displaced.
2. The total number of armour rocks in the roundhead section were 160 and 181 in the trunk section.

Table 23 Armour stability, West Breakwater under construction, Test 7a, long length of new West Breakwater installed

Test No.	Water Level (mOD)	Design H_s (m) offshore	Design T_m (sec) offshore	Roundhead Percentage of (10-15t) rocks displaced		Breakwater Trunk Percentage of (10-15t) rocks displaced	
				0.5-1 Dn_{50}	>1 Dn_{50}	0.5-1 Dn_{50}	>1 Dn_{50}
7b	2.50	4.0	7.5	(2.1)	(5.0)	(3.4)	(0.7)

Note: 1. The numbers in brackets refers to the cumulative percentage of rocks displaced.
2. The total number of armour rocks in the roundhead section were 141 and 147 in the trunk sections.

Figures

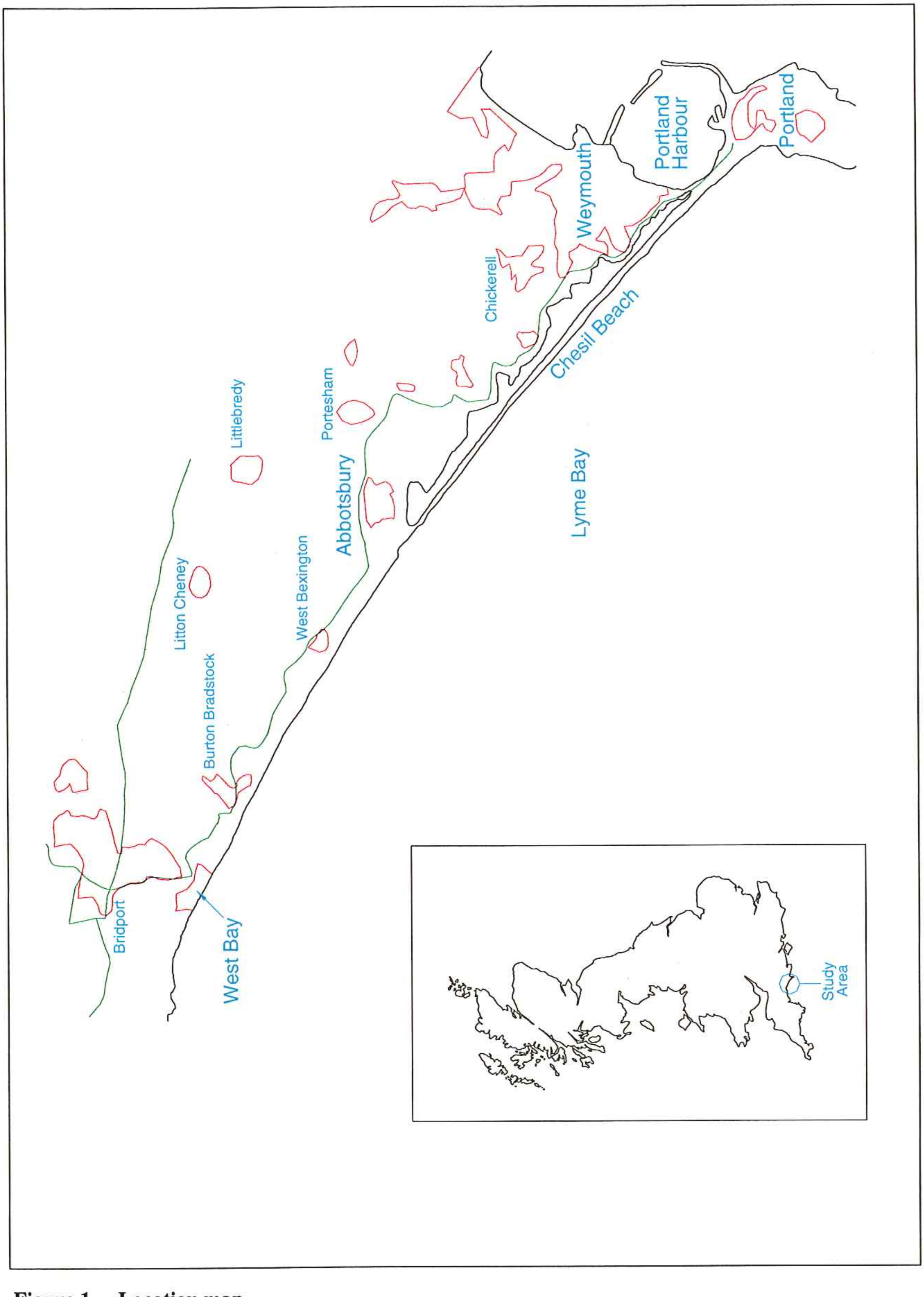

Figure 1 Location map

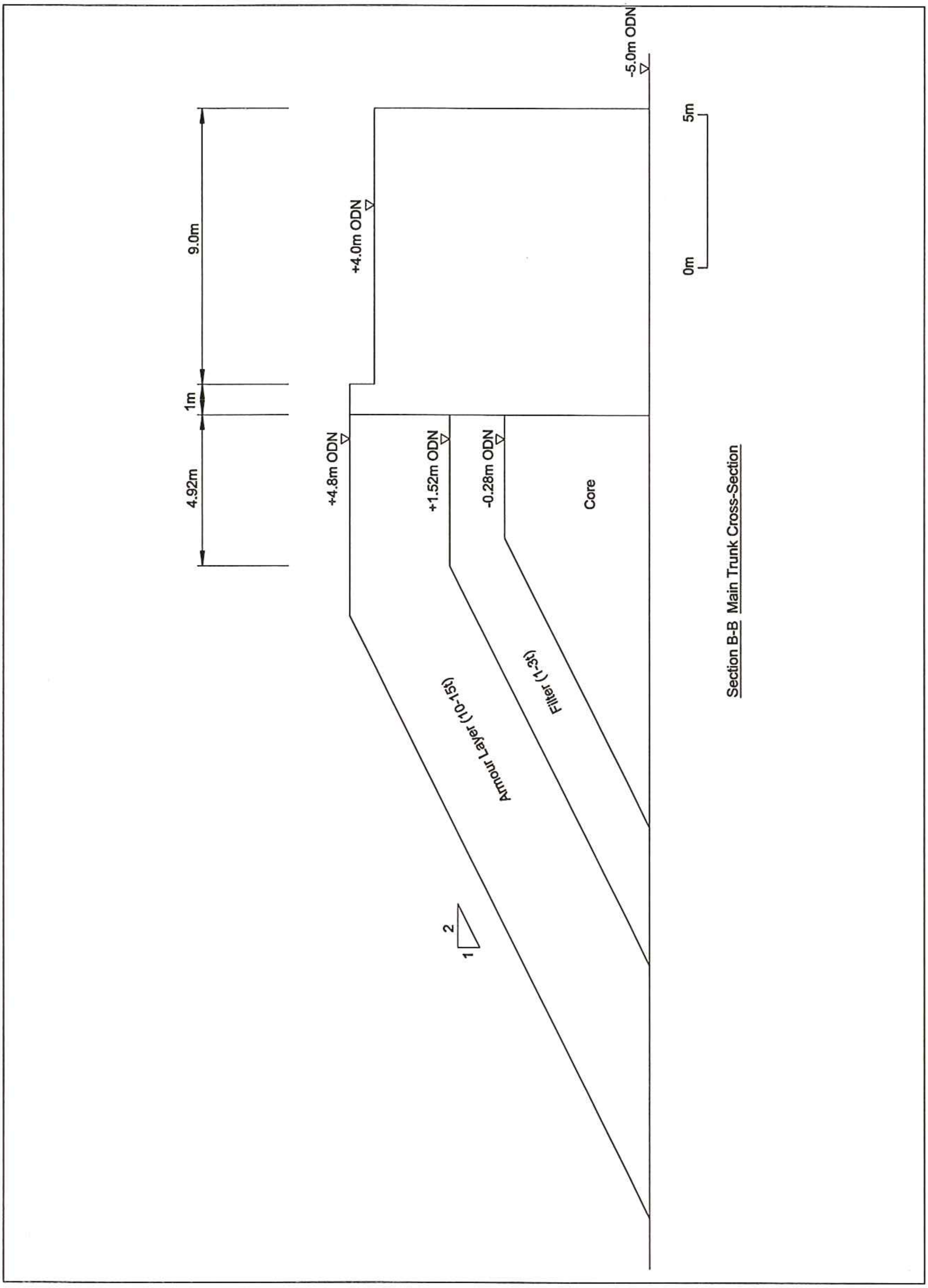

Figure 2 Typical sections through the proposed West Breakwater trunk, Test Series 3, 4 and 5

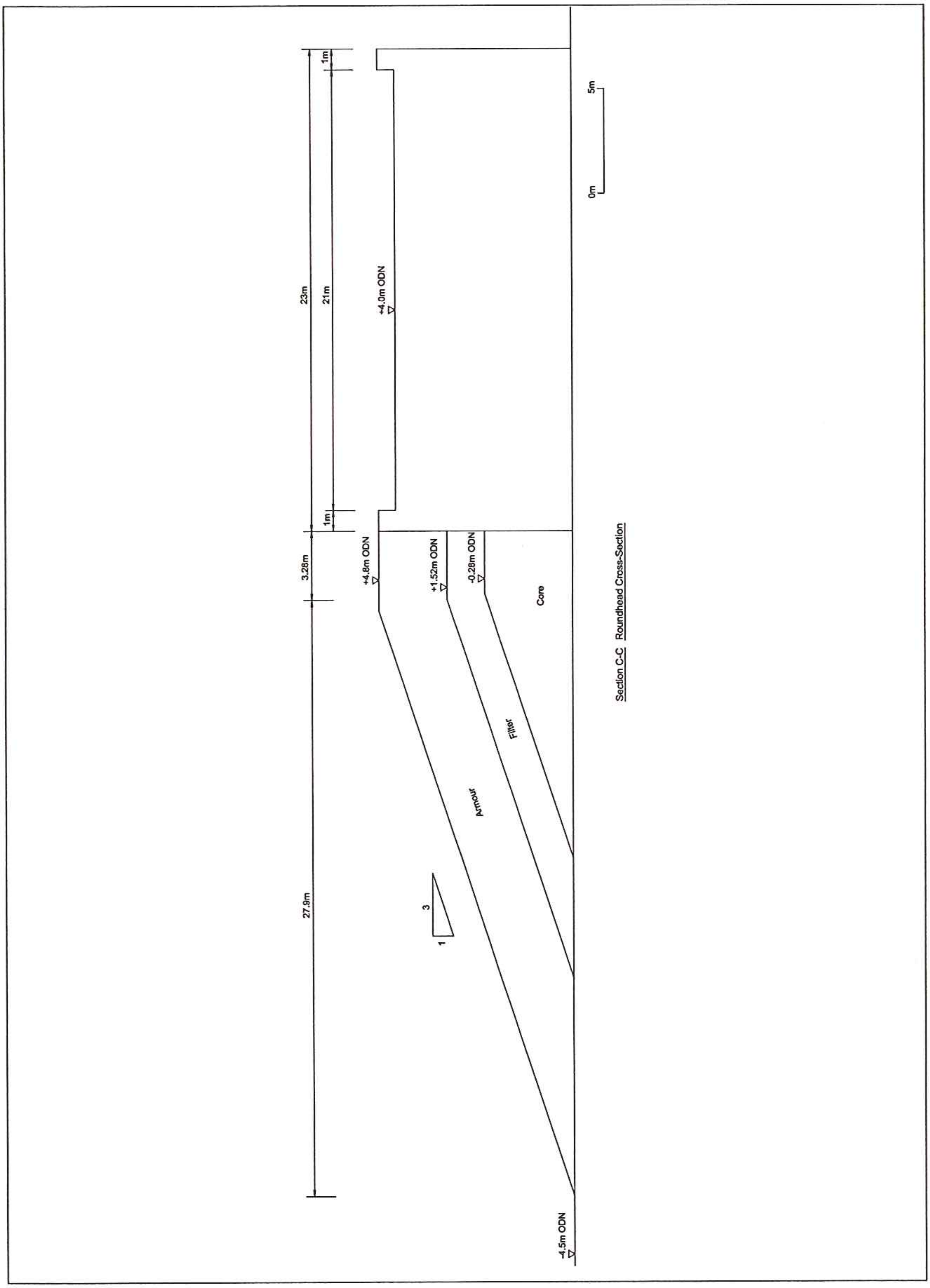

Figure 3 Typical sections through the proposed West Breakwater roundhead, Test Series 3, 4 and 5

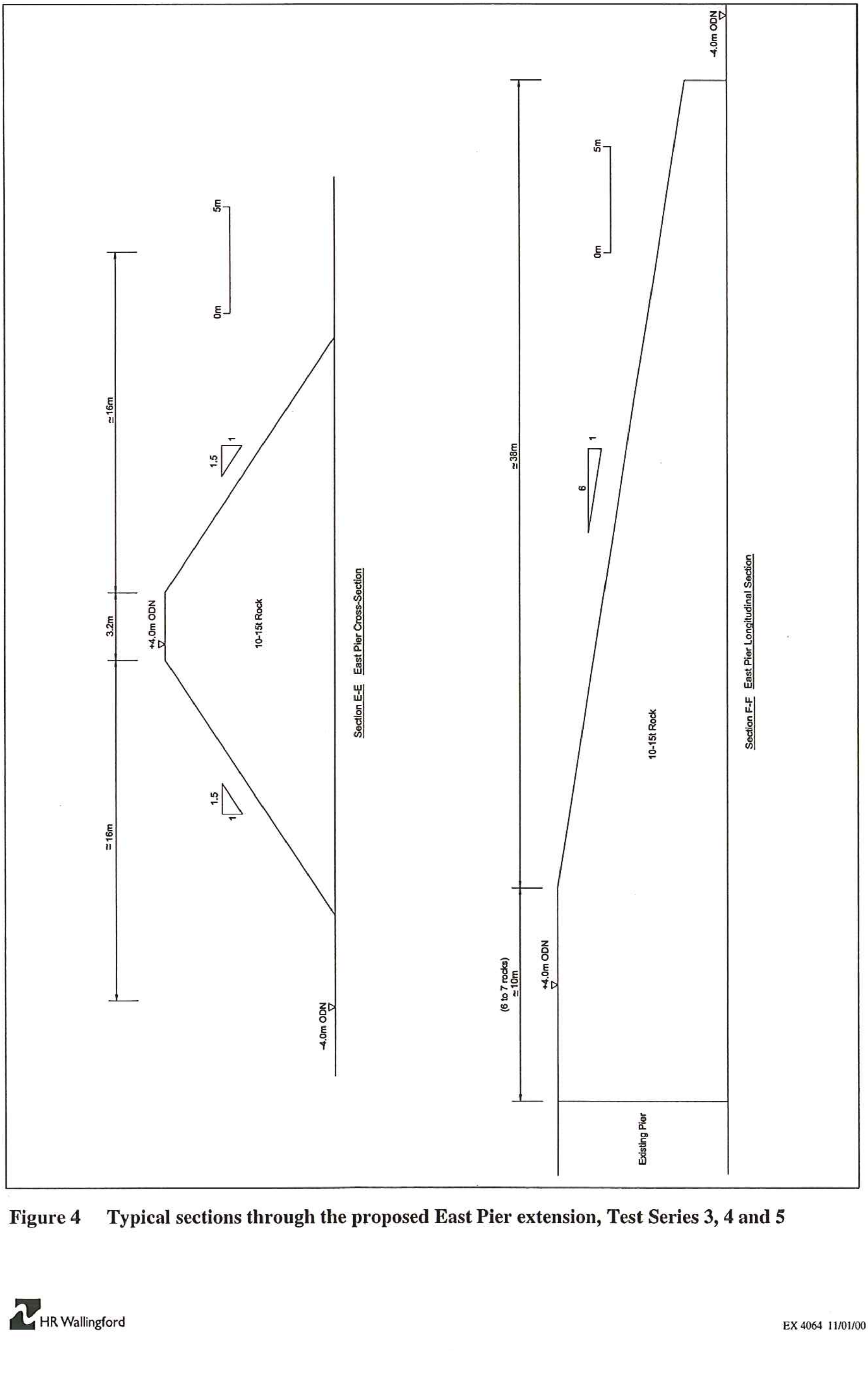

Figure 4 Typical sections through the proposed East Pier extension, Test Series 3, 4 and 5

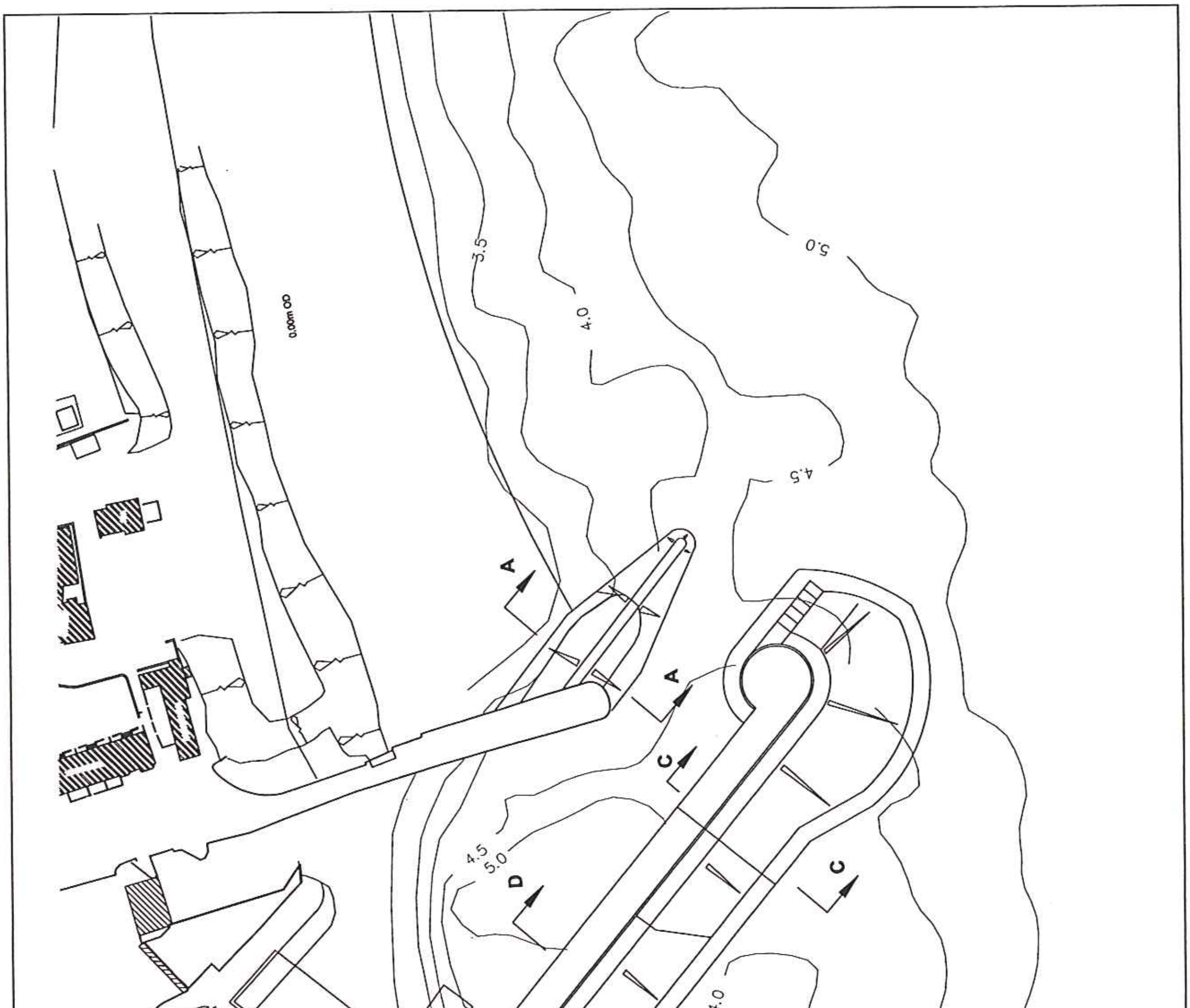

Figure 5 **Plan of the preferred West Breakwater alignment**

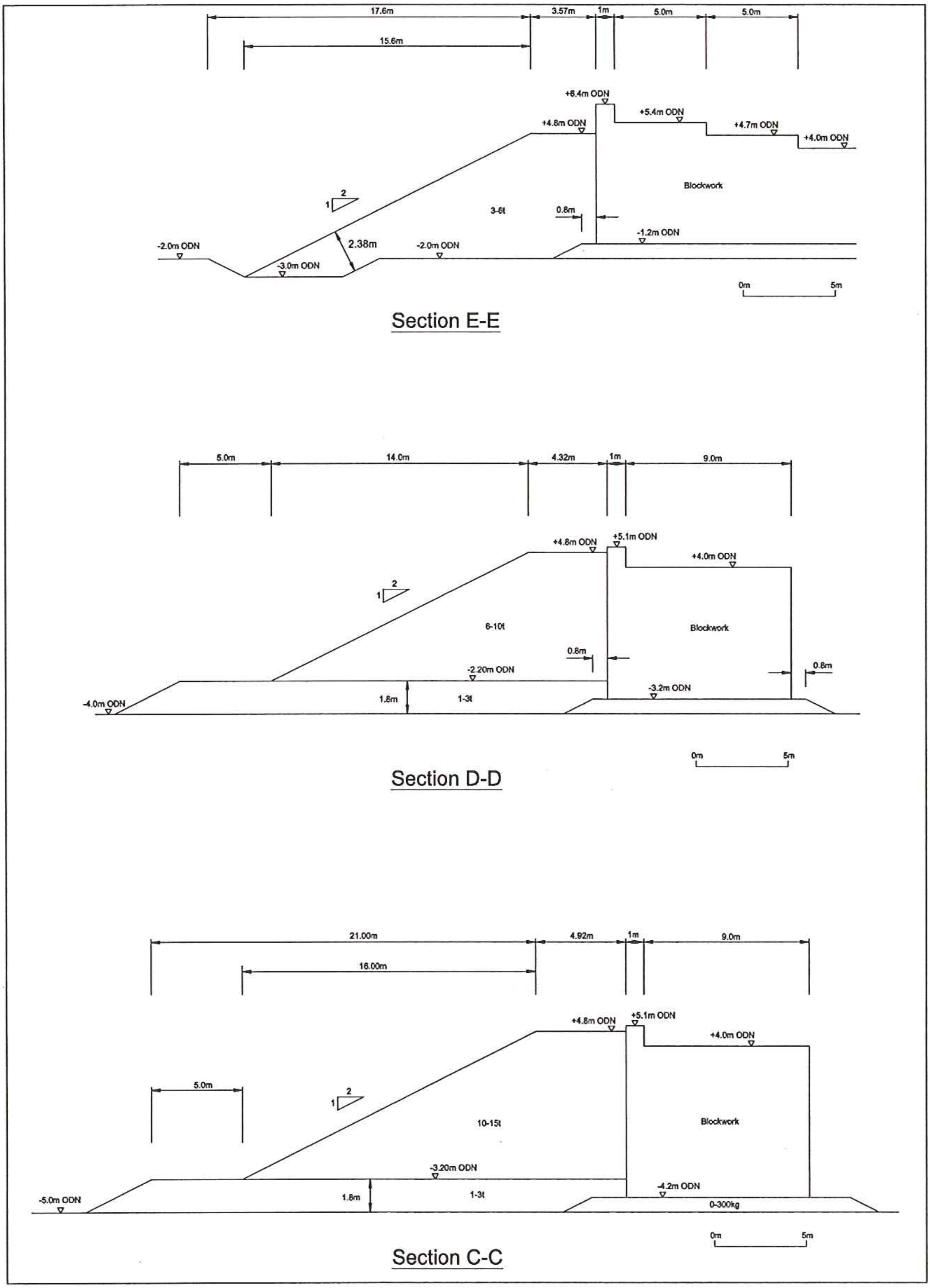

Figure 6 Typical sections through the proposed West Breakwater trunk, Test Series 6

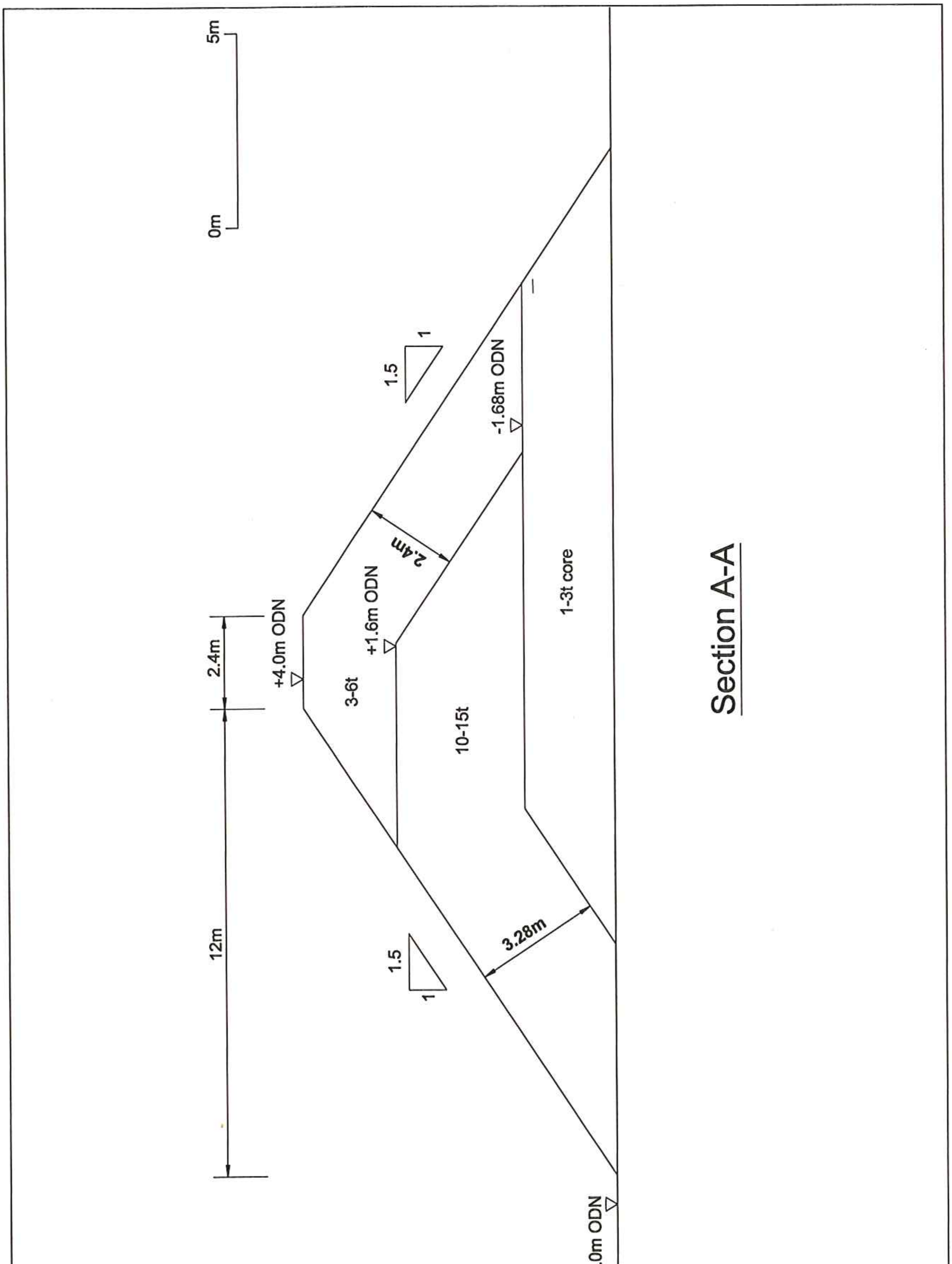

Figure 7 East Pier extension, Test Series 6

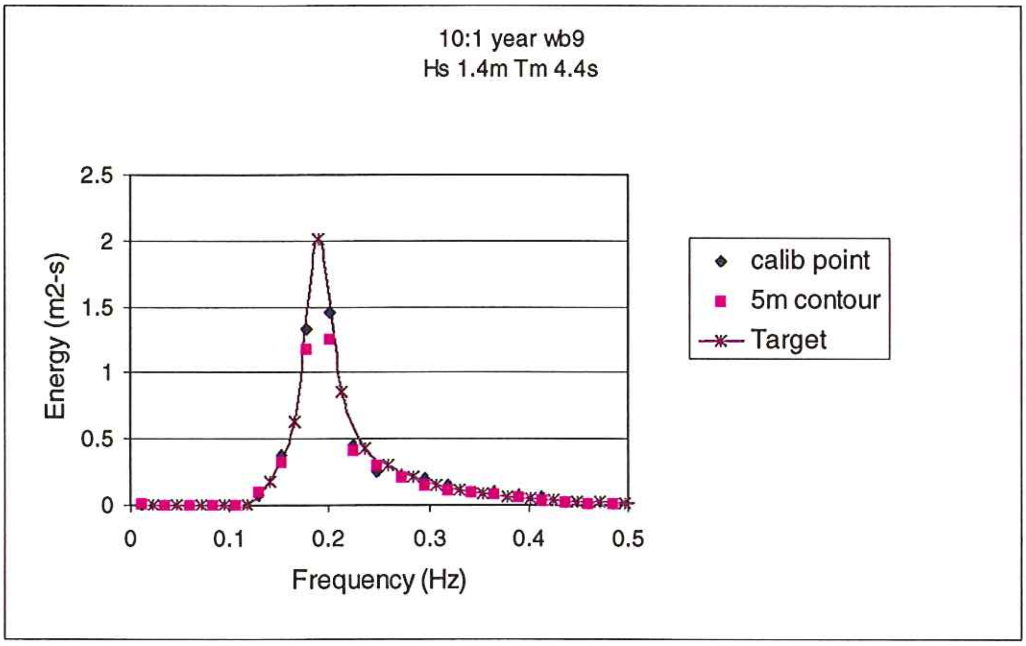

Figure 8 Typical wave spectra, waves from 160°N, 10:1 year return period

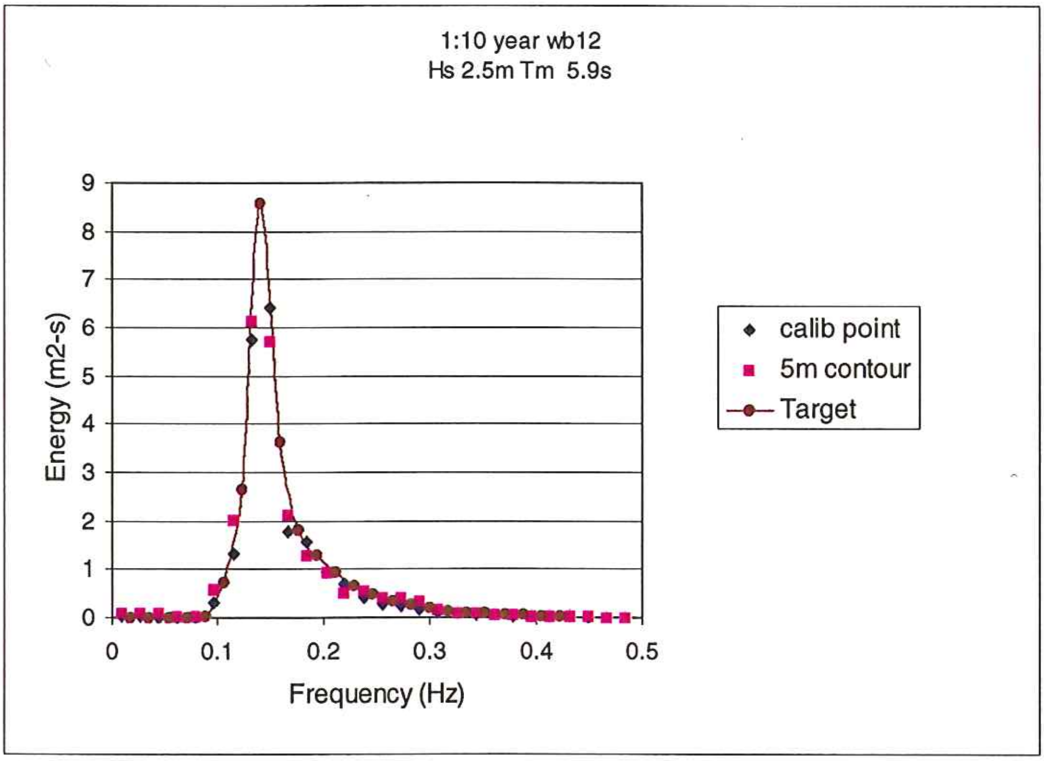

Figure 9 Typical wave spectra, waves from 160°N, 1:10 year return period

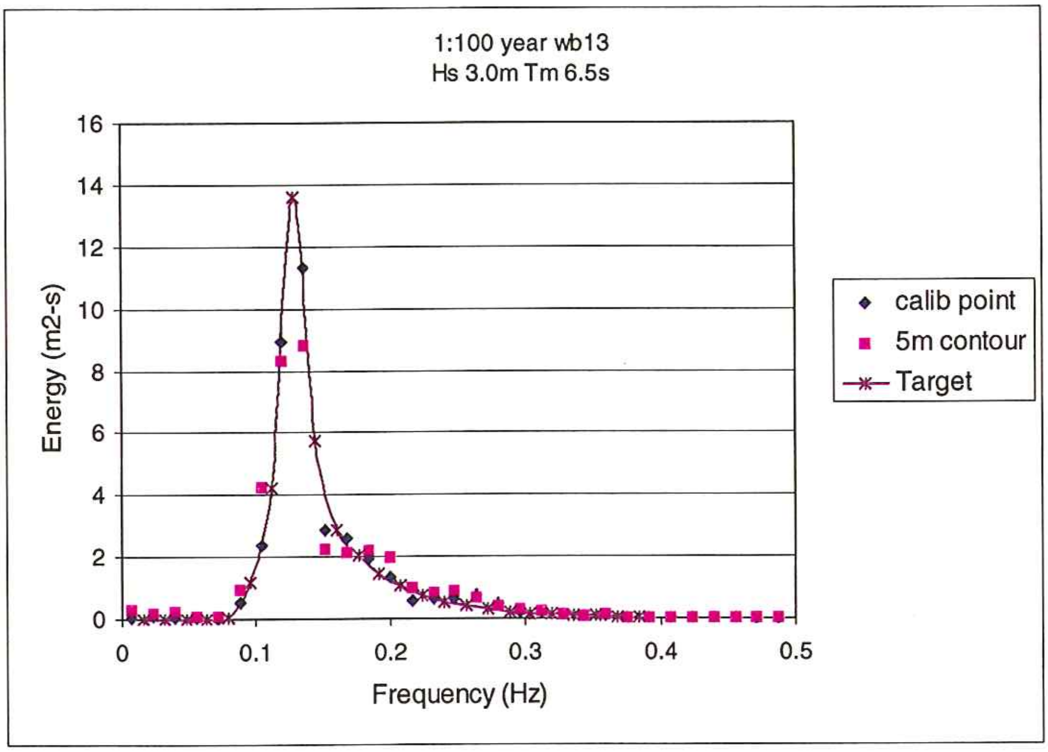

Figure 10 Typical wave spectra, waves from 160°N, 1:100 year return period

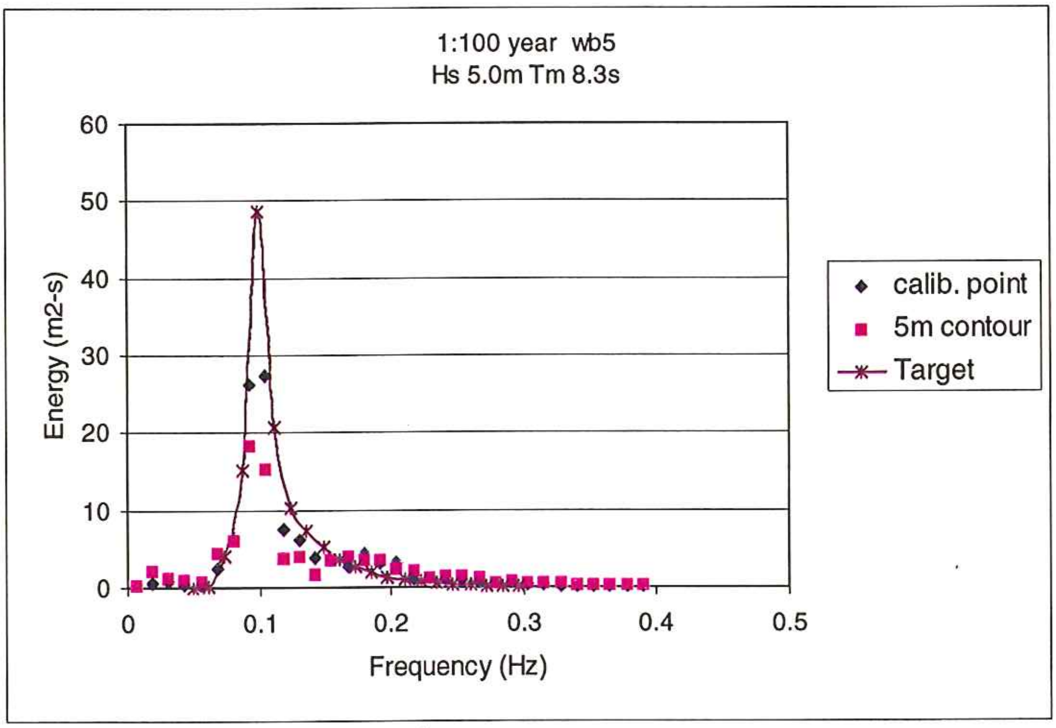

Figure 11 Typical wave spectra, waves from 220°N, 1:100 year return period

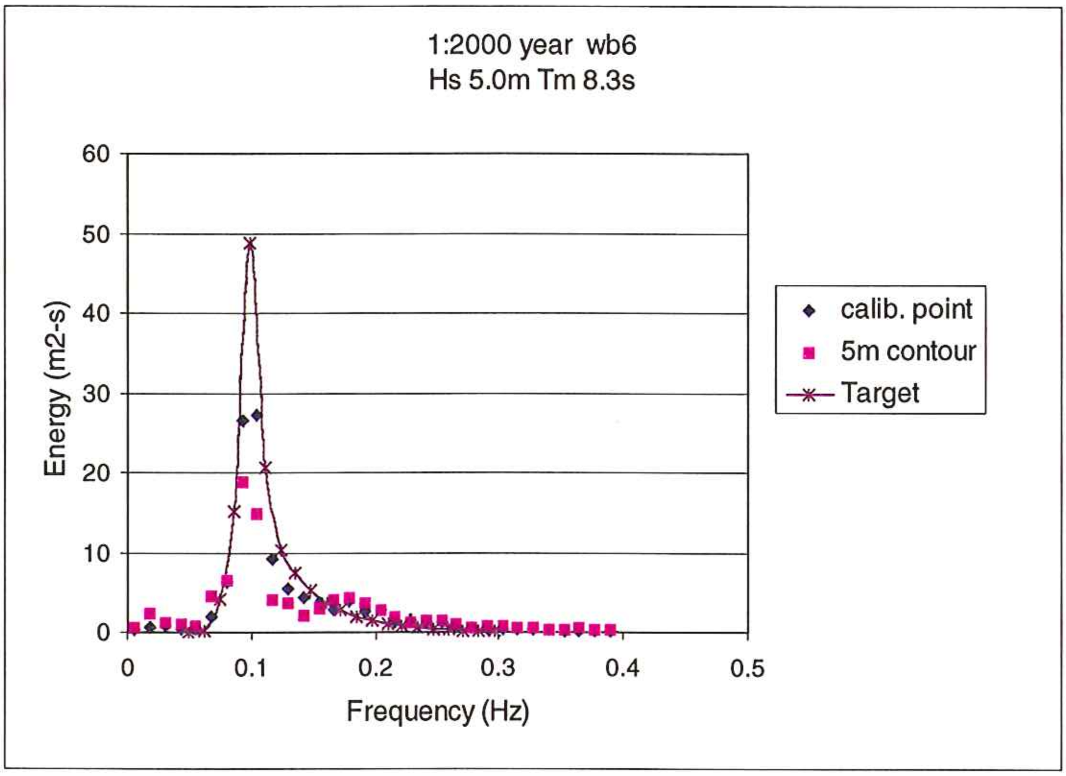

Figure 12 Typical wave spectra, waves from 220°N, 1:2000 year return period

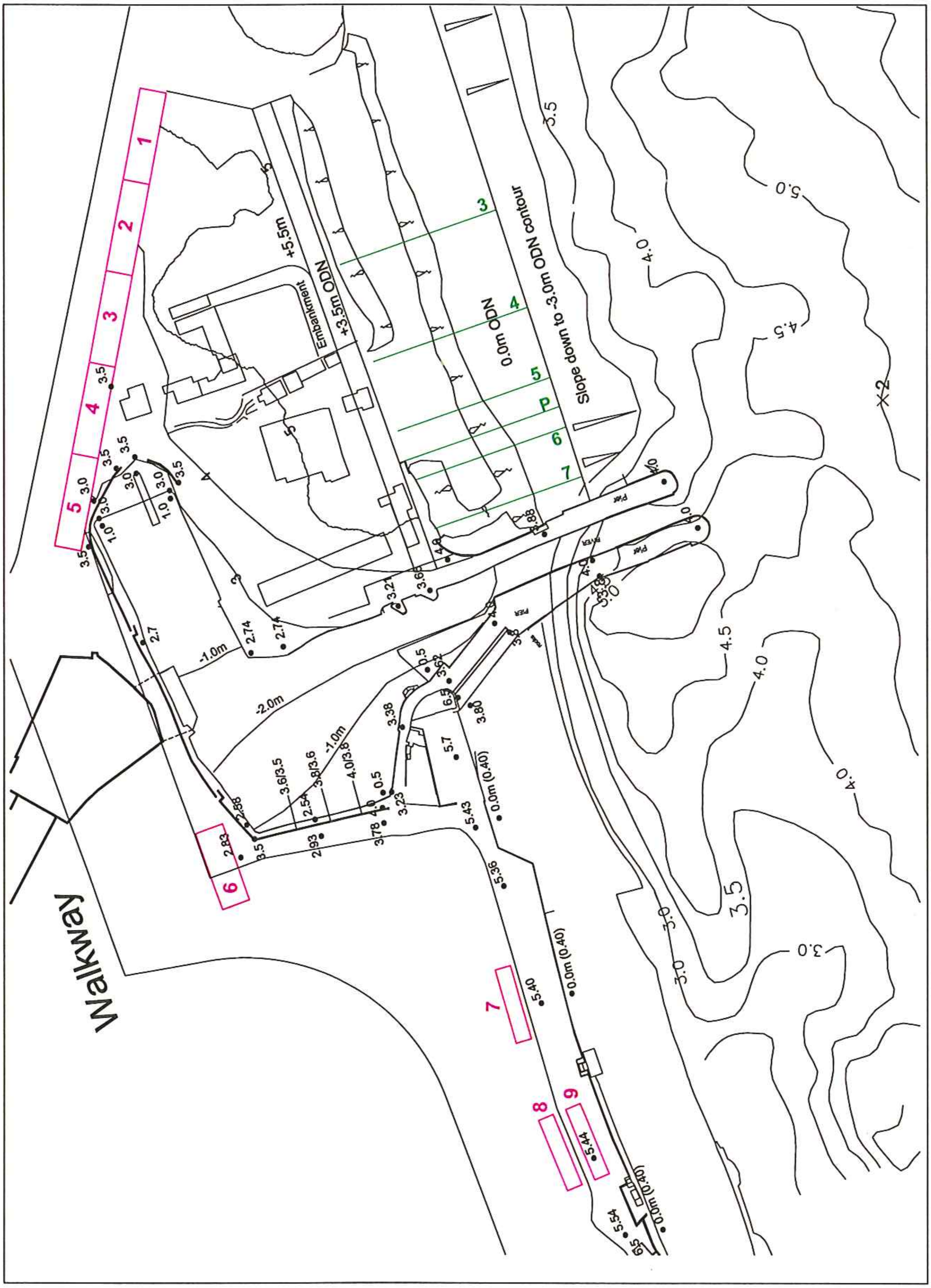

Figure 13 Overtopping tank and beach profile locations

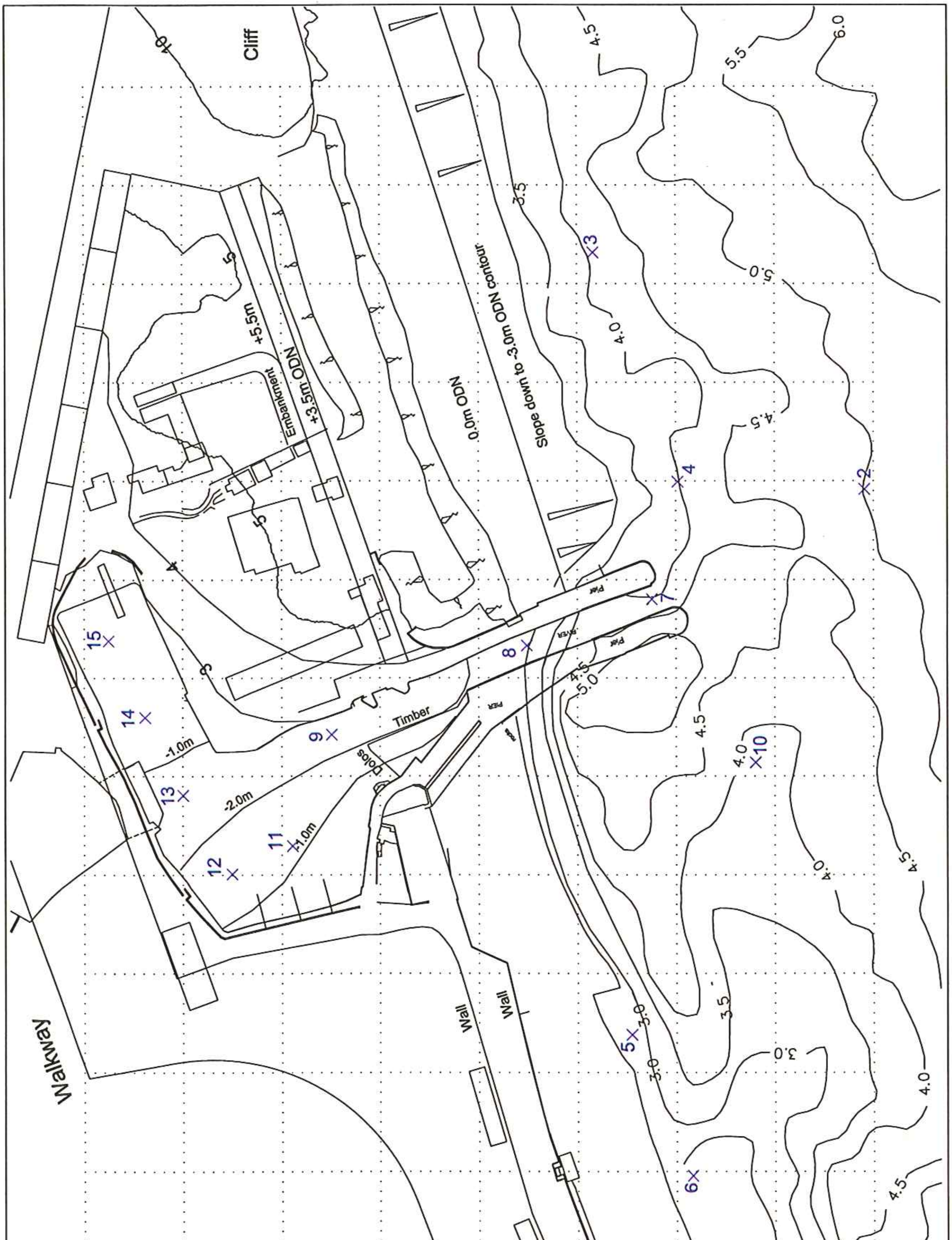

Figure 14 Wave probe locations, existing layout

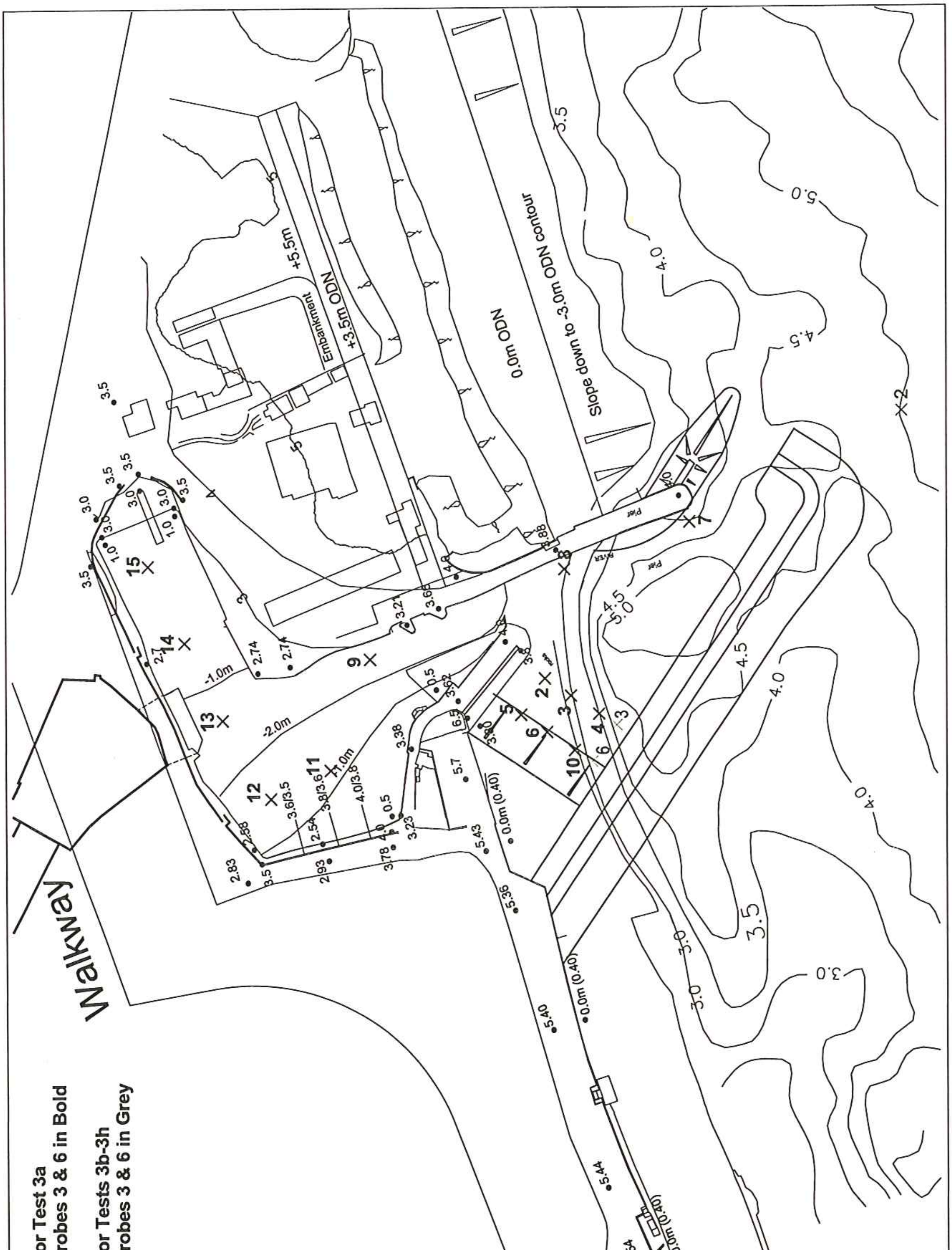

Figure 15 Wave probe locations, modifications to harbour entrance

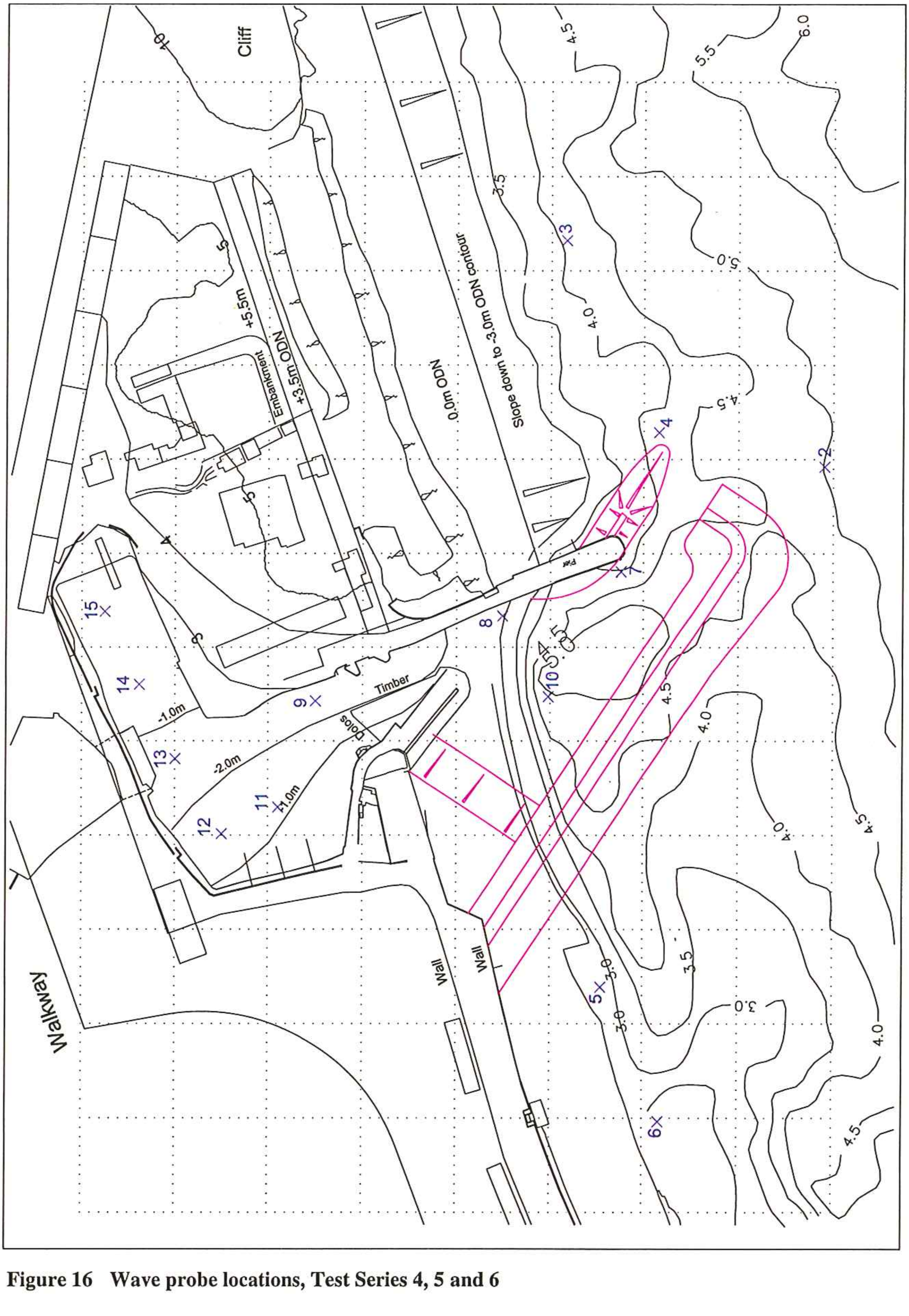

Figure 16 Wave probe locations, Test Series 4, 5 and 6

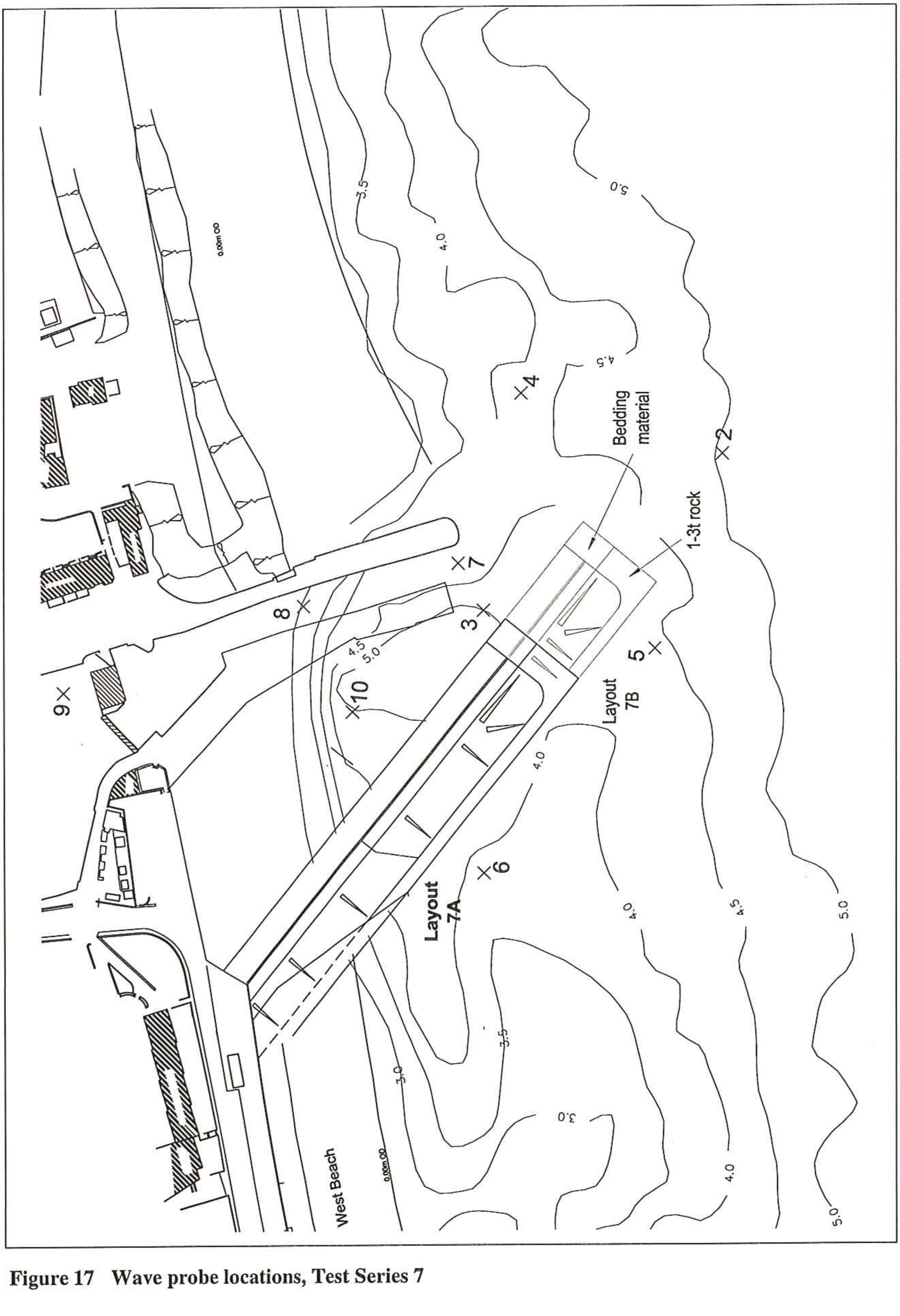

Figure 17 Wave probe locations, Test Series 7

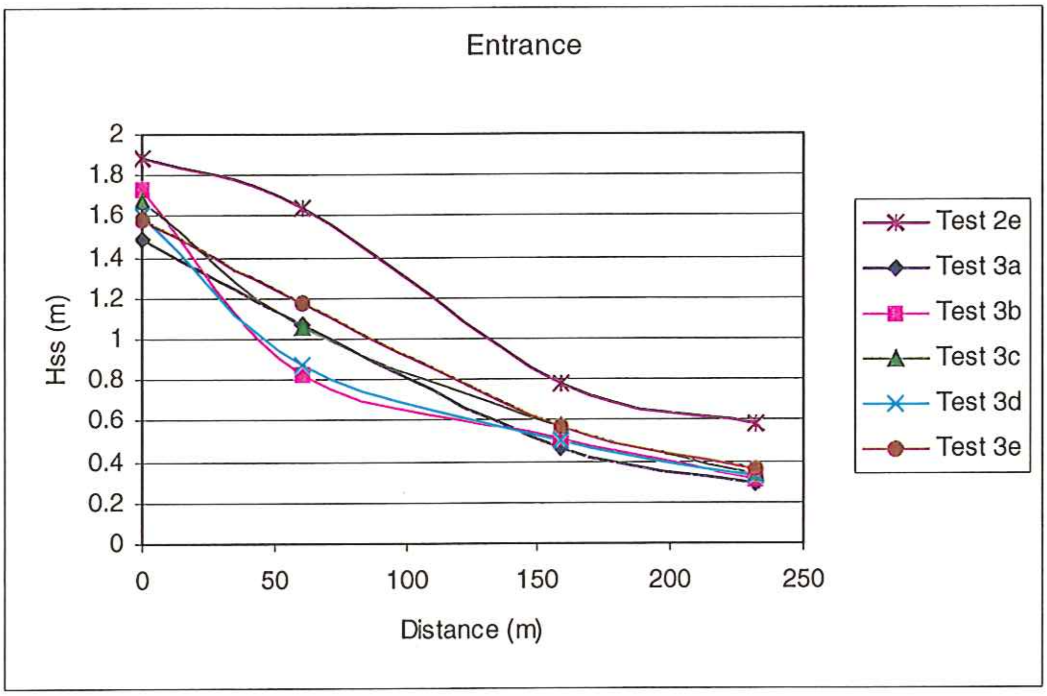

Figure 18 **A comparison of wave heights along the harbour entrance channel, 1:1 year condition, waves from 160°N, existing and inner harbour modifications**

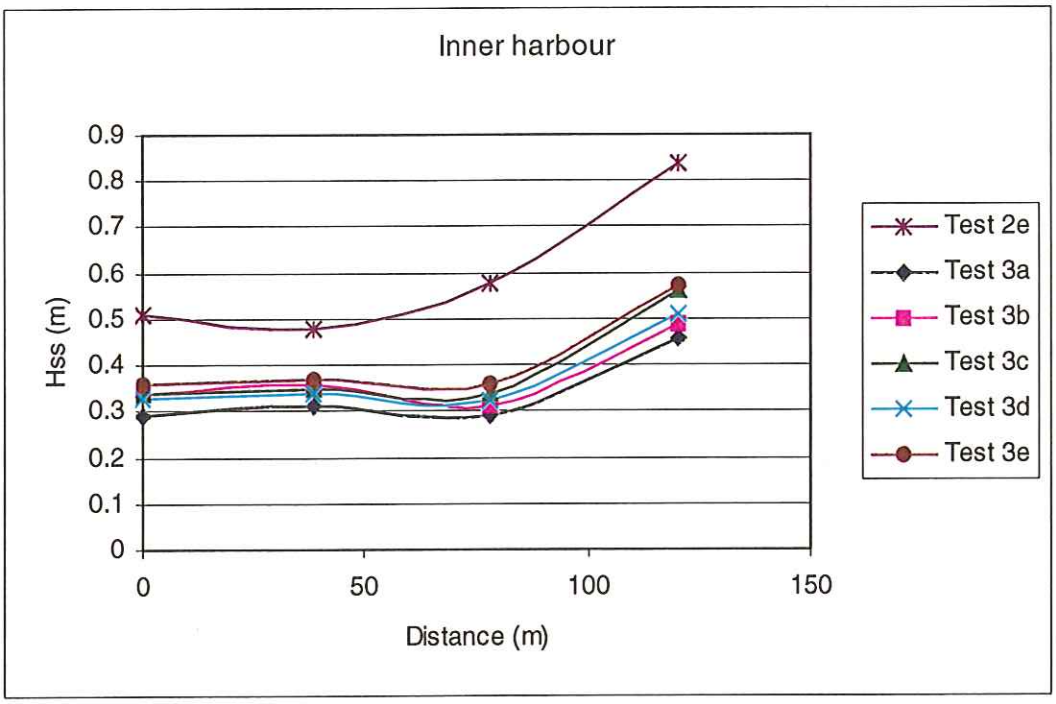

Figure 19 **A comparison of wave heights within the inner harbour, 1:1 year condition year condition, waves from 160°N, existing and inner harbour modifications**

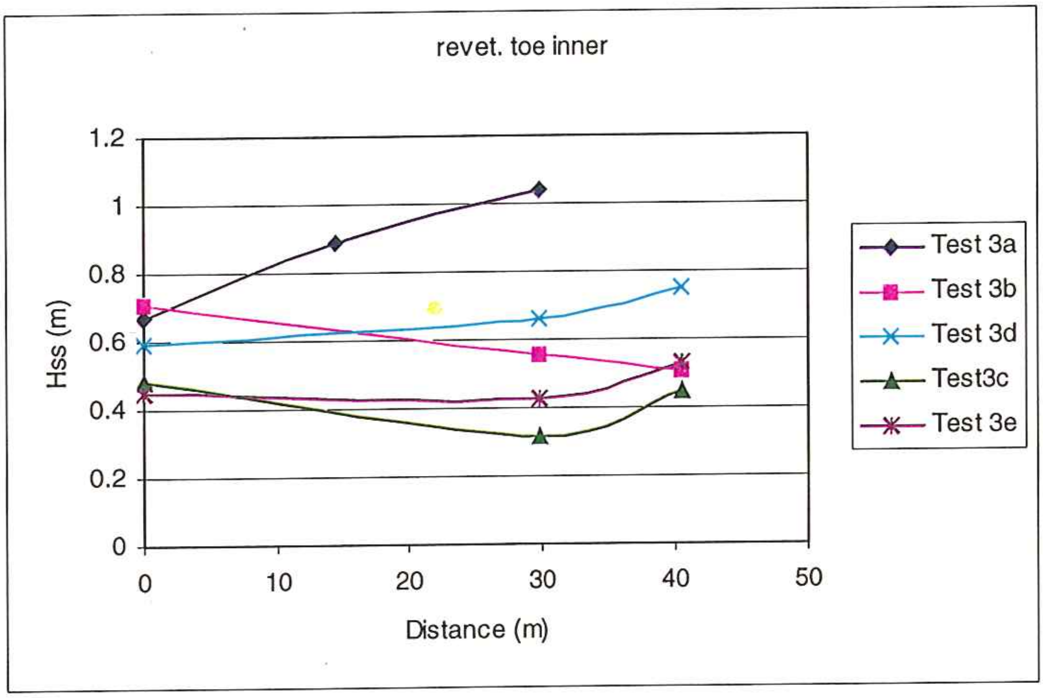

Figure 20 A comparison of wave heights in the outer harbour, 1:1 year condition, waves from 160°N, inner harbour modifications

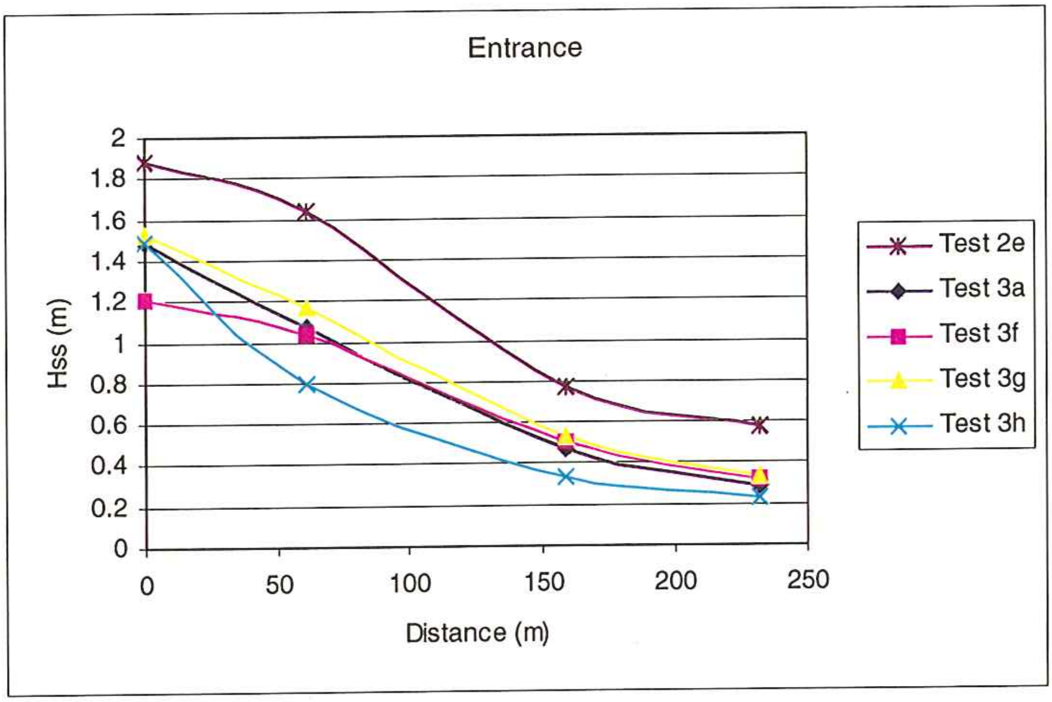

Figure 21 A comparison of wave heights along the harbour entrance channel, 1:1 year condition, waves from 160°N, existing and entrance modifications

Figure 22 A comparison of wave heights within the harbour, 1:1 year condition, waves from 160°N, existing and entrance modifications

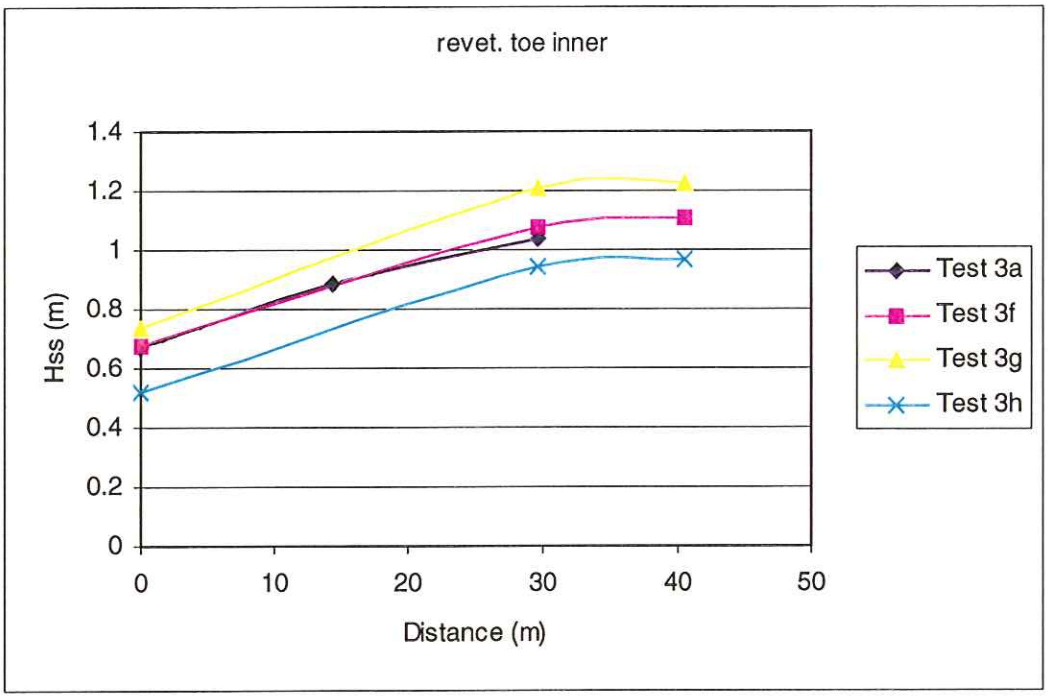

Figure 23 A comparison of wave heights in front of the outer harbour revetment, 1:1 year condition, waves from 160°N, entrance modifications

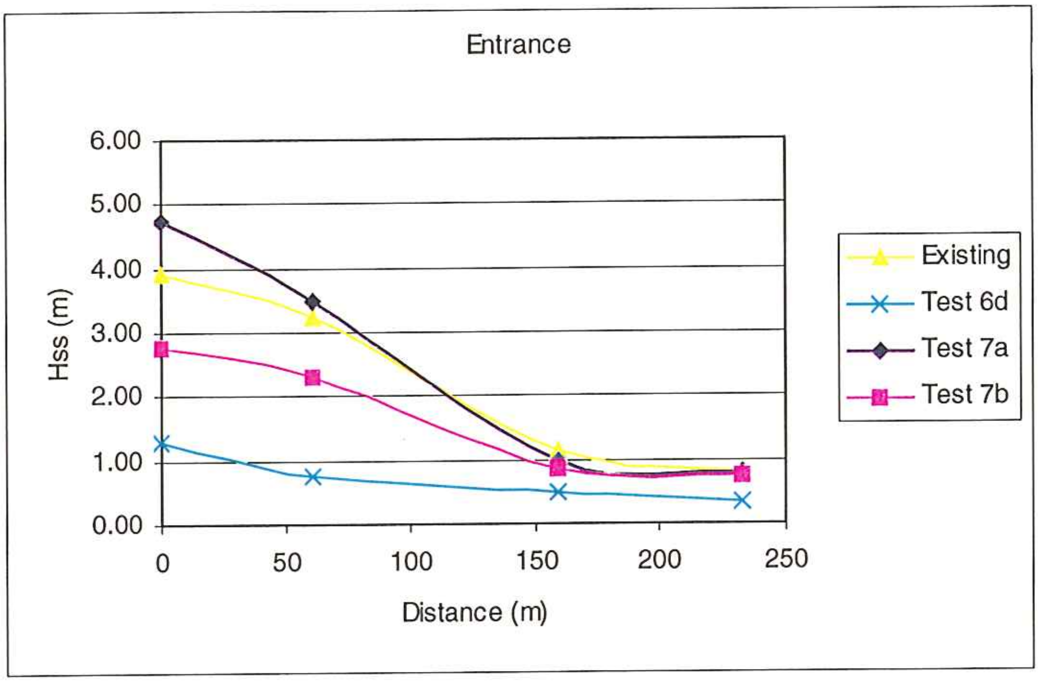

Figure 24 **A comparison of wave heights along the entrance channel, 1:10 year condition, waves from 220°N, existing and temporary works**

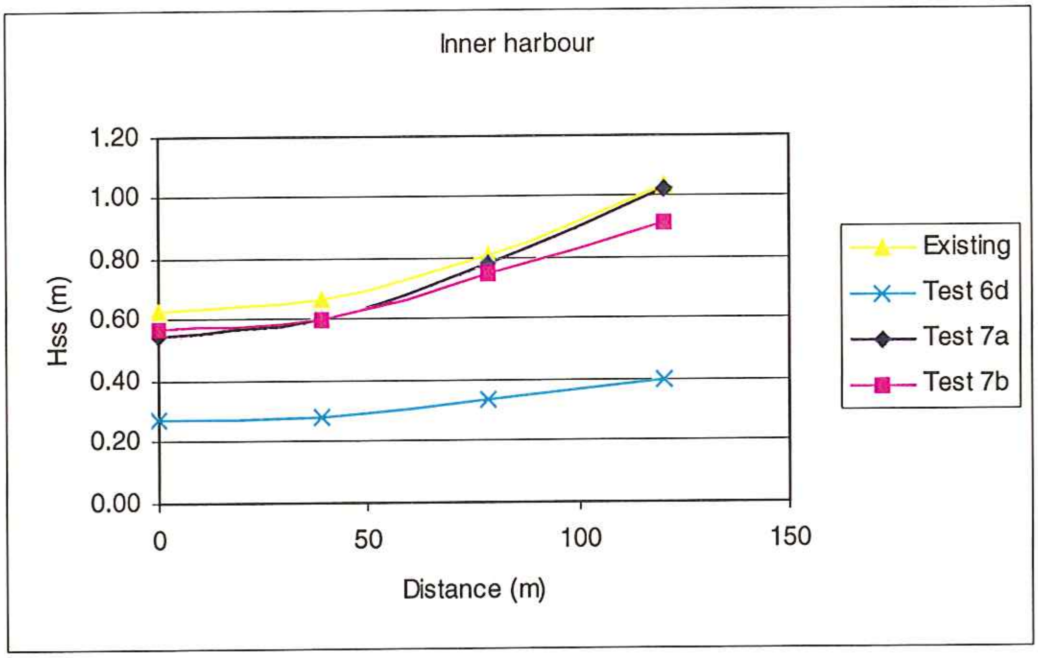

Figure 25 **A comparison of wave heights within the harbour, 1:10 year condition, waves from 220°N, existing and temporary works**

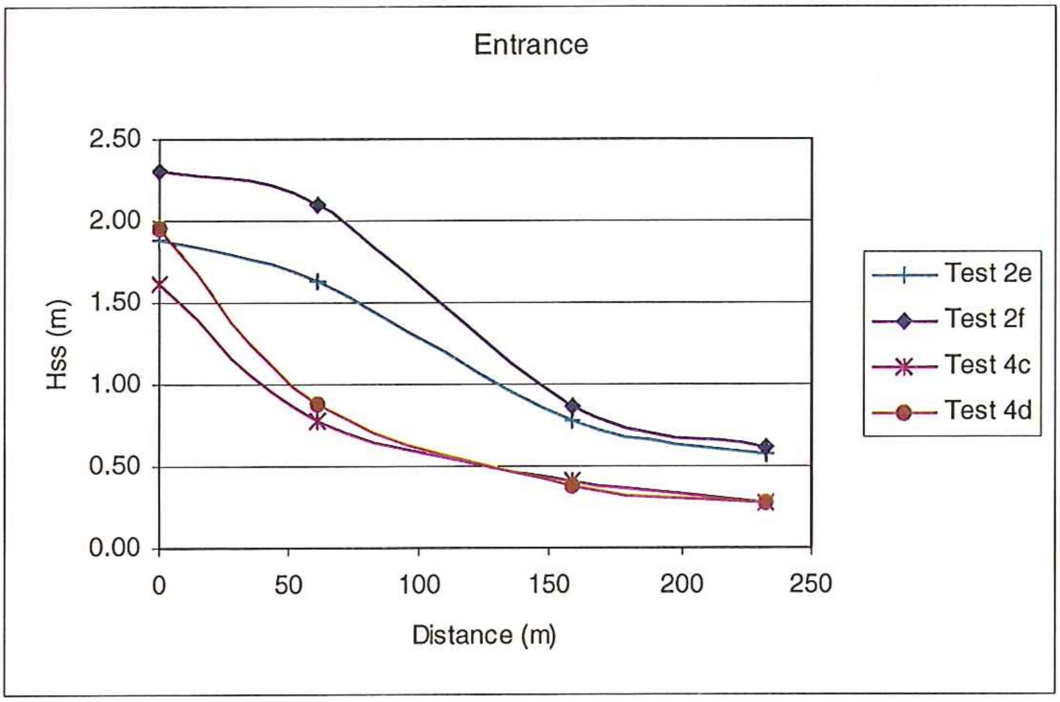

Figure 26 **A comparison of wave heights along the harbour entrance channel, 1:1 and 1:10 year conditions, waves form 160°N, existing and layout assessed duringTest Series 4**

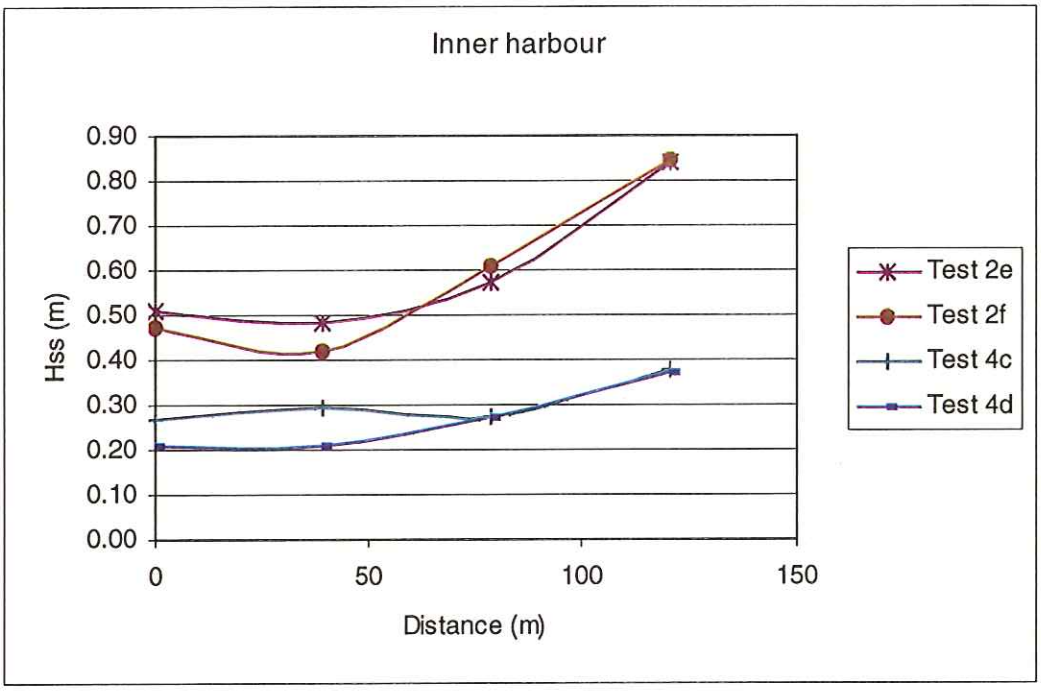

Figure 27 **A comparison of wave heights within the harbour, 1:1 and 1:10 year condition, waves from 160°N, existing and layout assessed during Test Series 4**

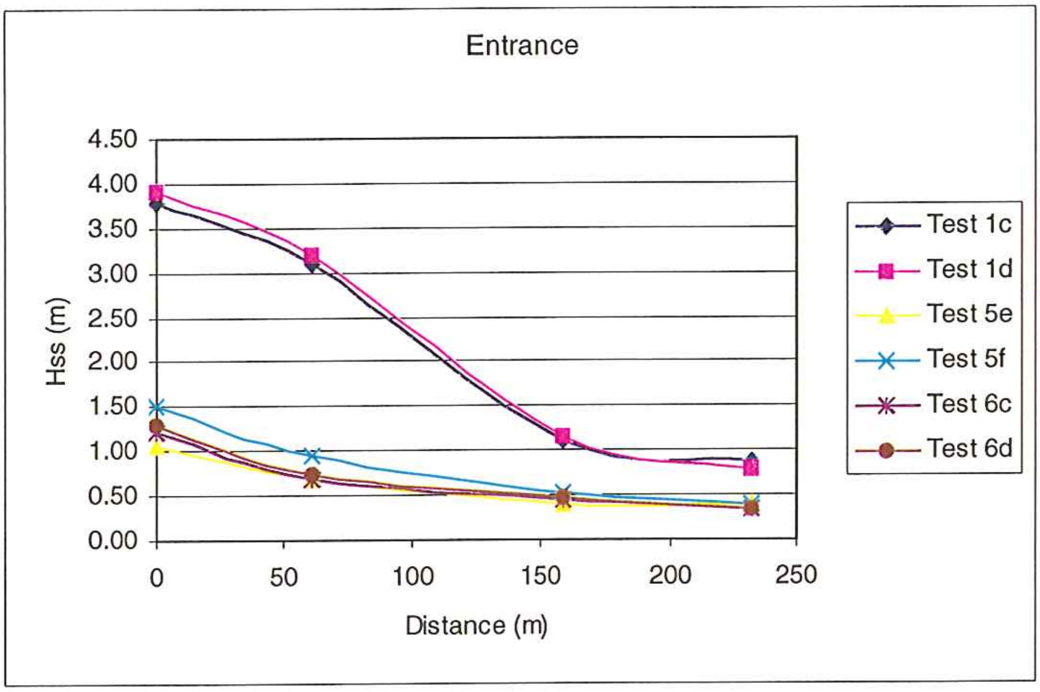

Figure 28 A comparison of wave heights along the harbour entrance channel, 1:1 and 1:10 year condition, waves from 220°N, existing and layout assessed during Test Series 5 and 6

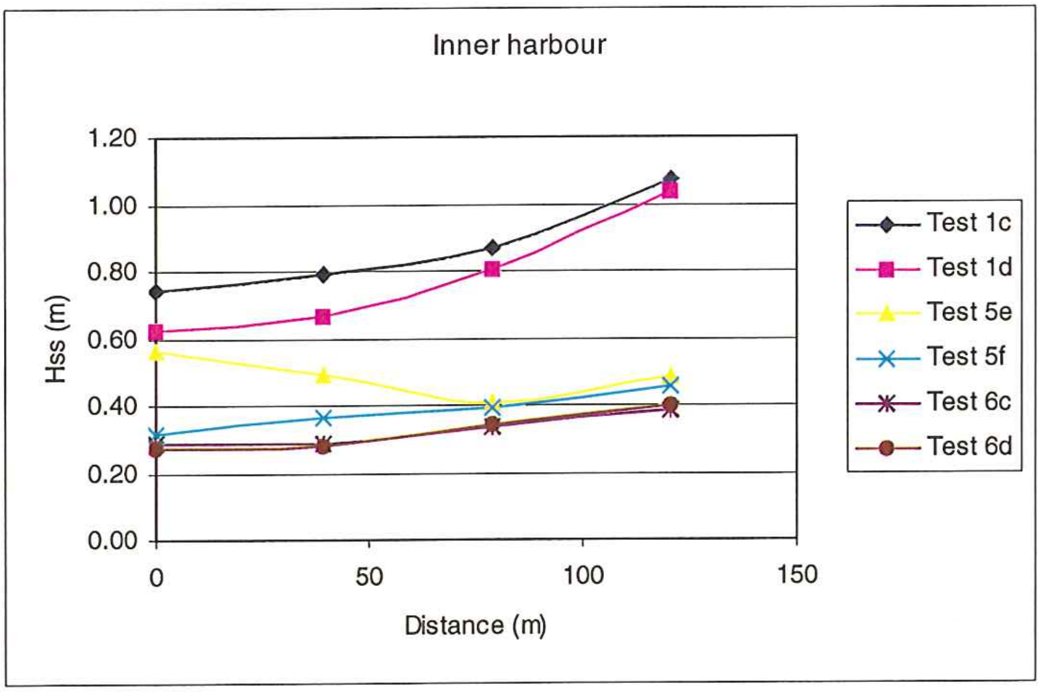

Figure 29 A comparison of wave heights within the harbour, 1:1 and 1:10 year condition, waves from 220°N, existing and proposed layouts

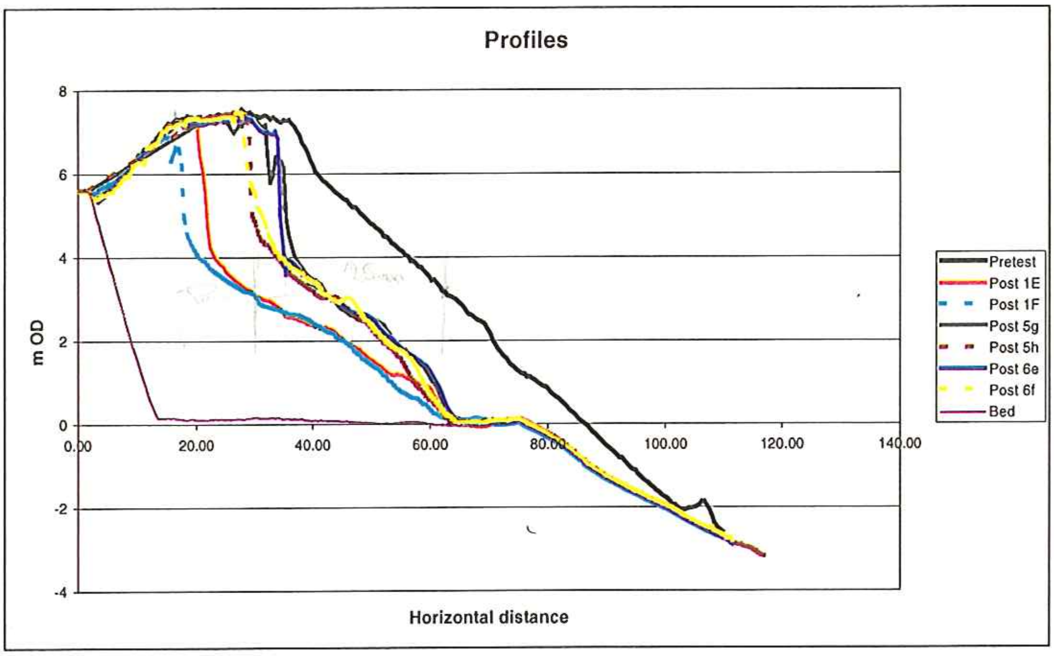

Figure 30 **A comparison of beach profiles for the 1:100 and 1:2000 year conditions for waves from 220°N, existing and proposed layouts** after 3hrs 185.7 mins

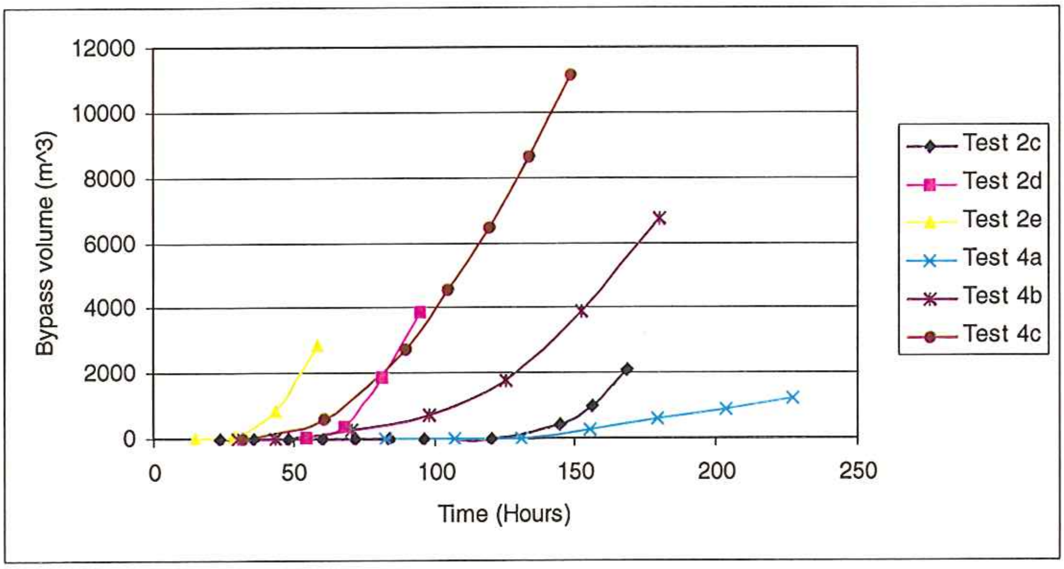

Figure 31 **Beach material deposited within the harbour entrance for waves from 160°N, existing and proposed layouts**

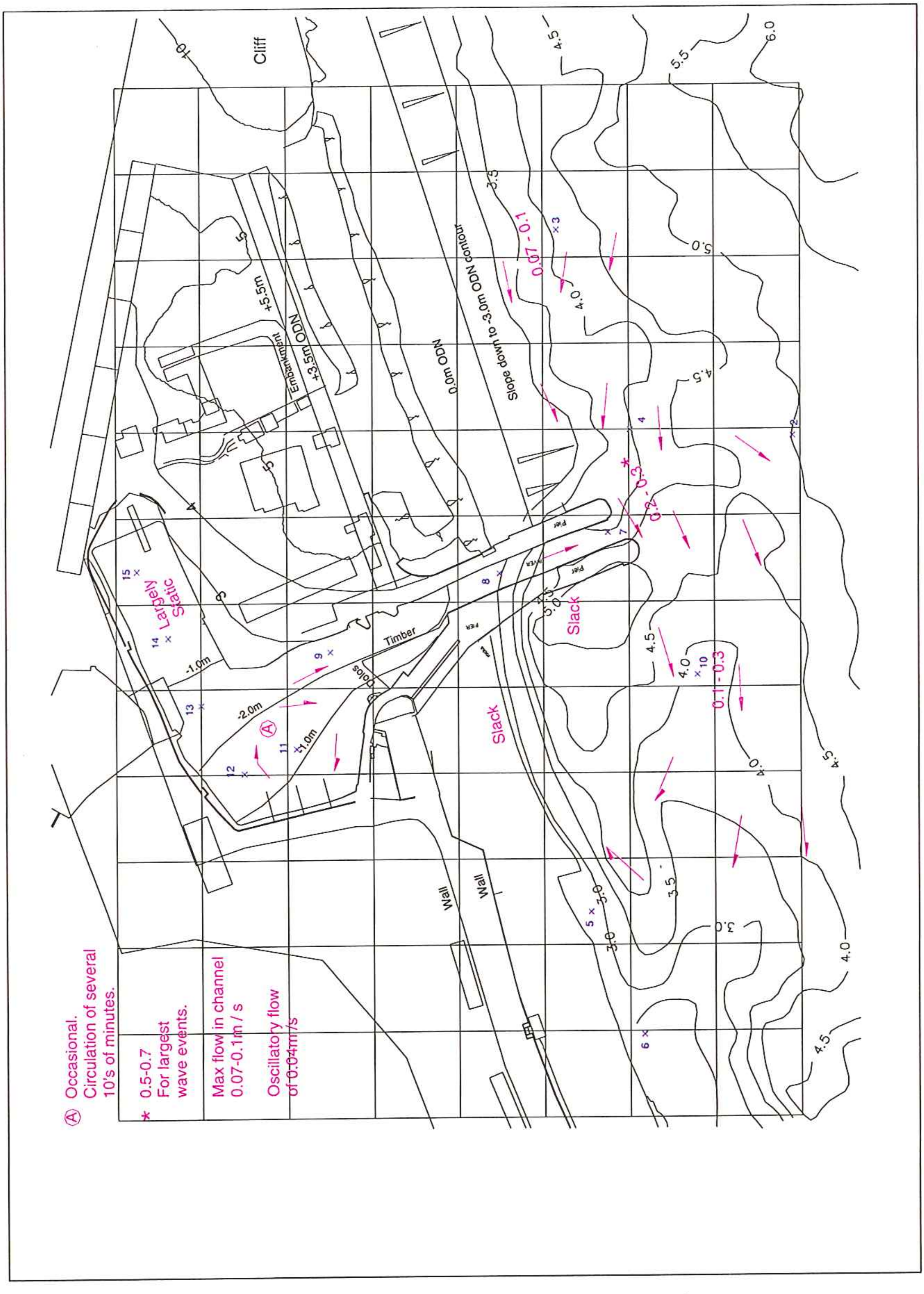

Figure 32 Wave induced currents for the existing layout (waves from 160° N)

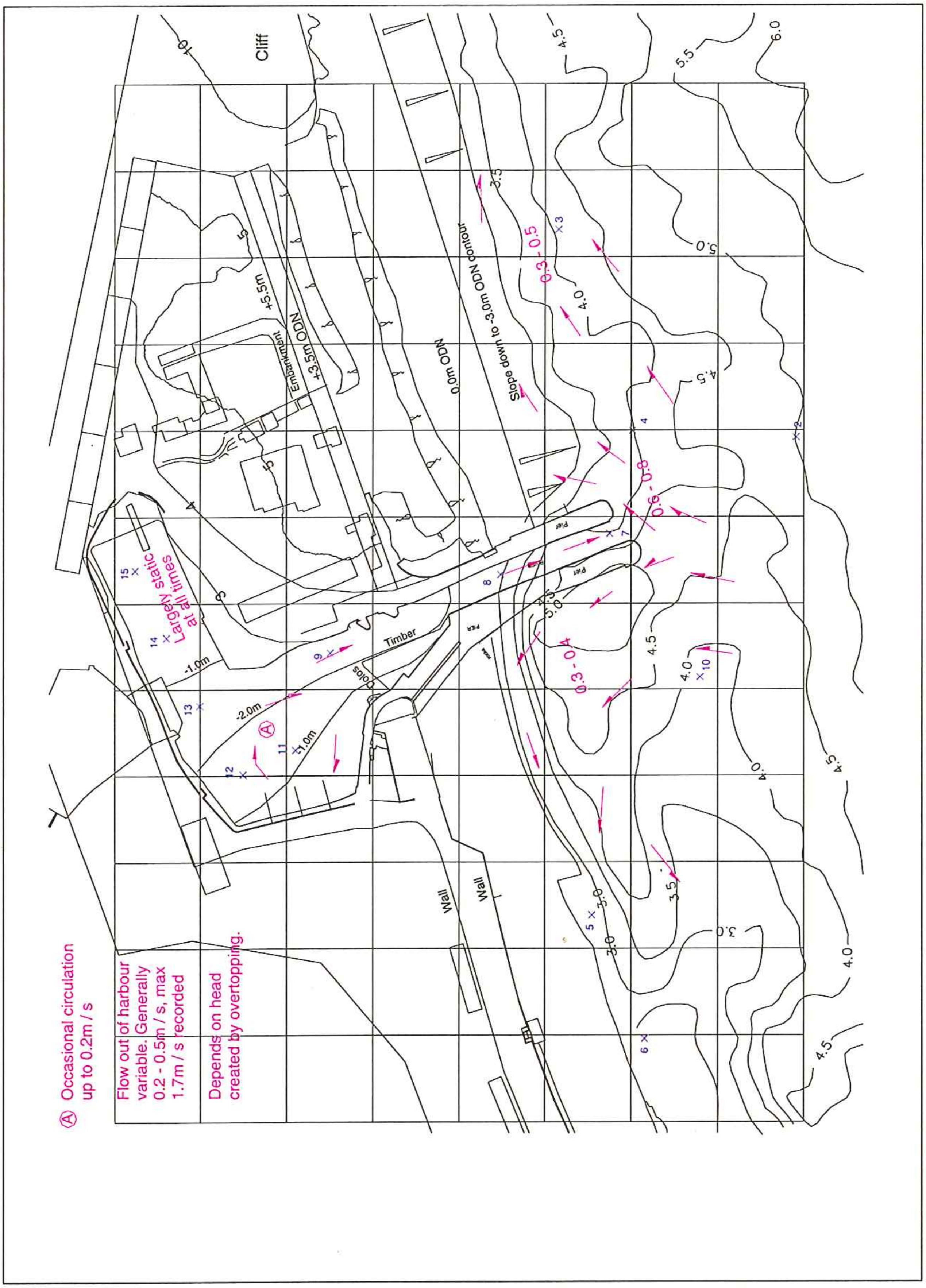

Figure 33 Wave induced currents for the existing layout (waves from 220°N)

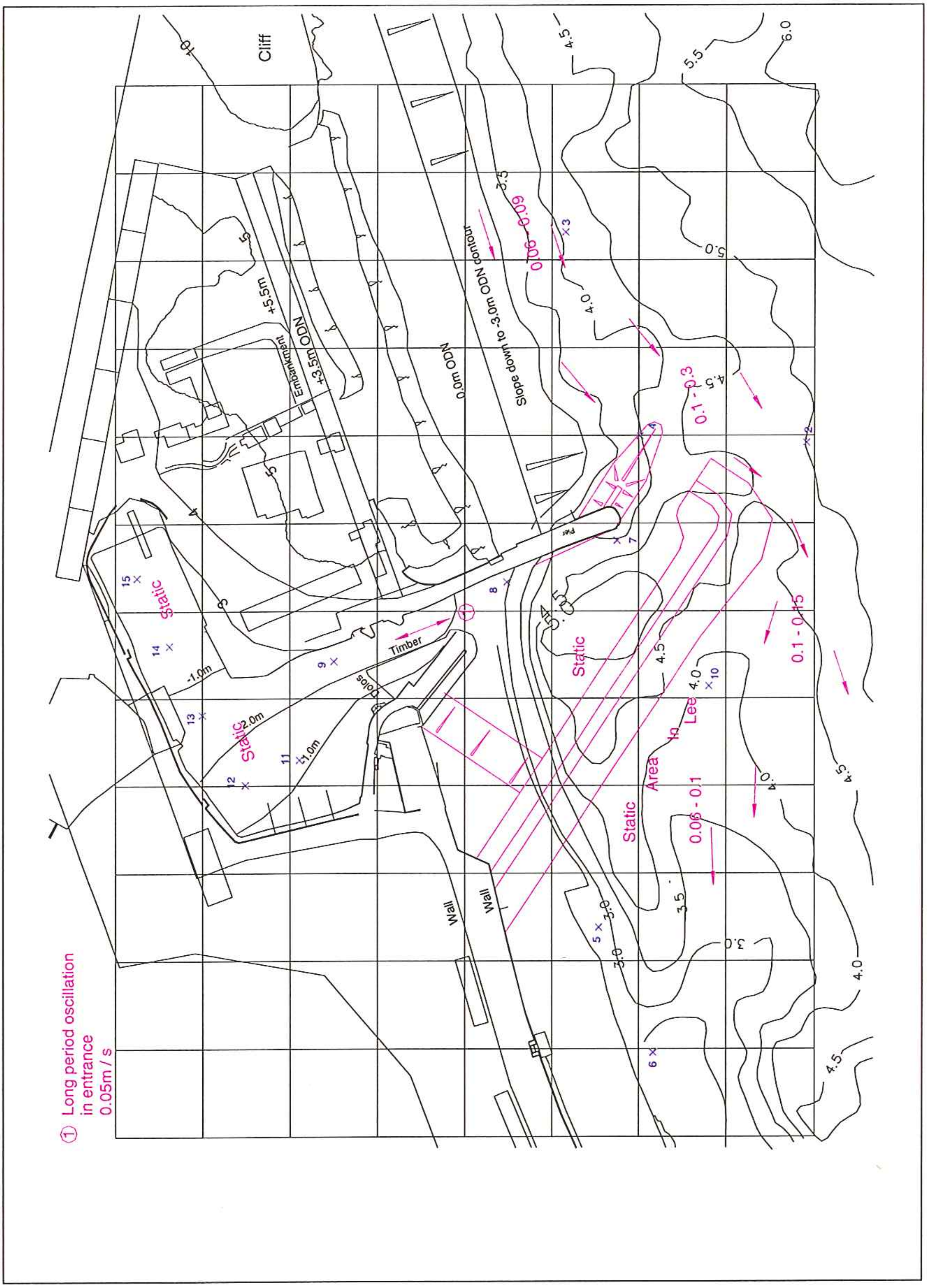

Figure 34 Wave induced currents for the preferred layout (waves from 160°N)

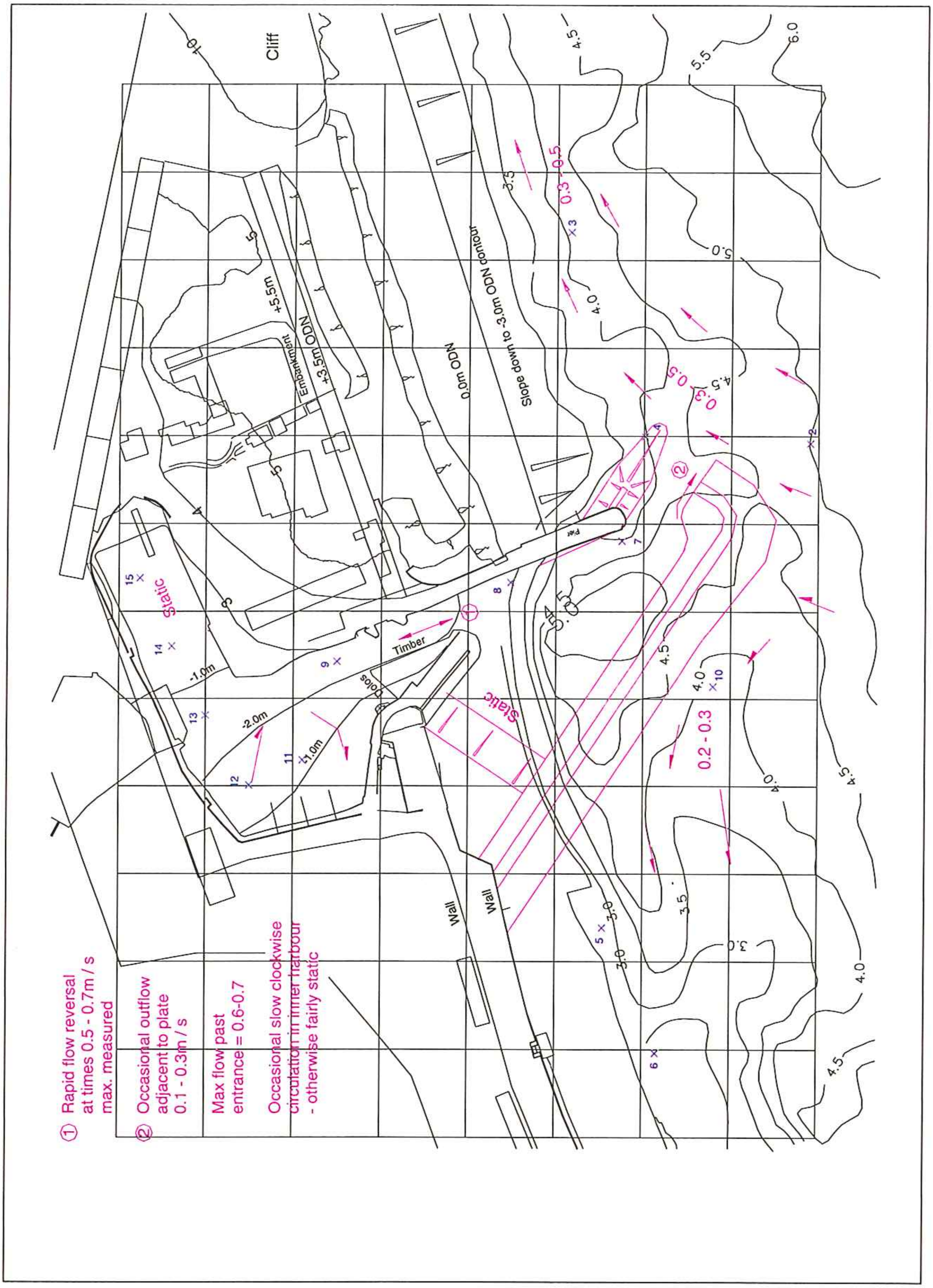

Figure 35 Wave induced currents for the preferred layout (waves from 220°N)

Figure 36 Currents induced during sluicing, existing layout Test 2a

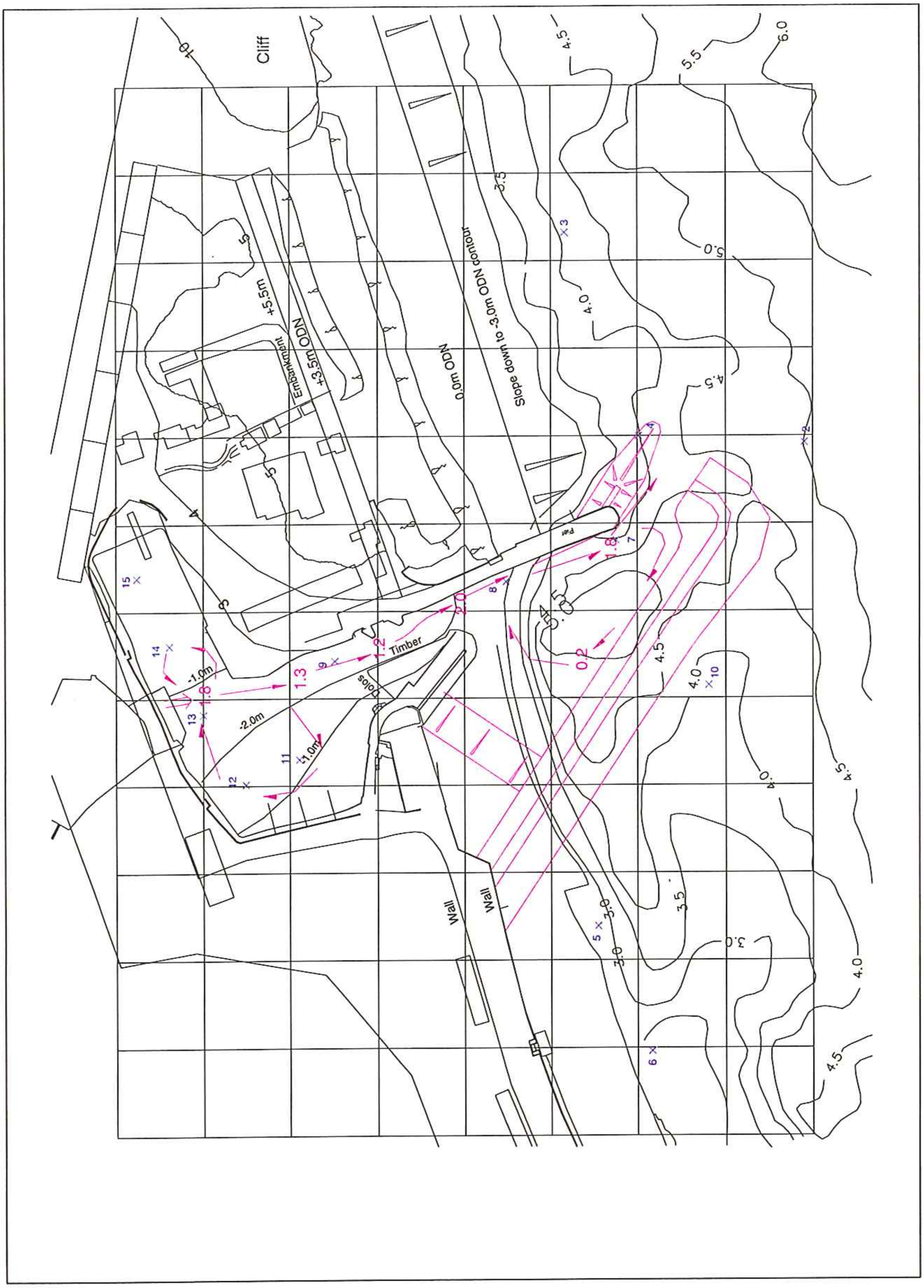

Figure 37 Currents induced during sluicing, preferred layout Test 5a

Plates

Plate 1 West Bay Harbour

Plate 2 View of the physical model, existing layout

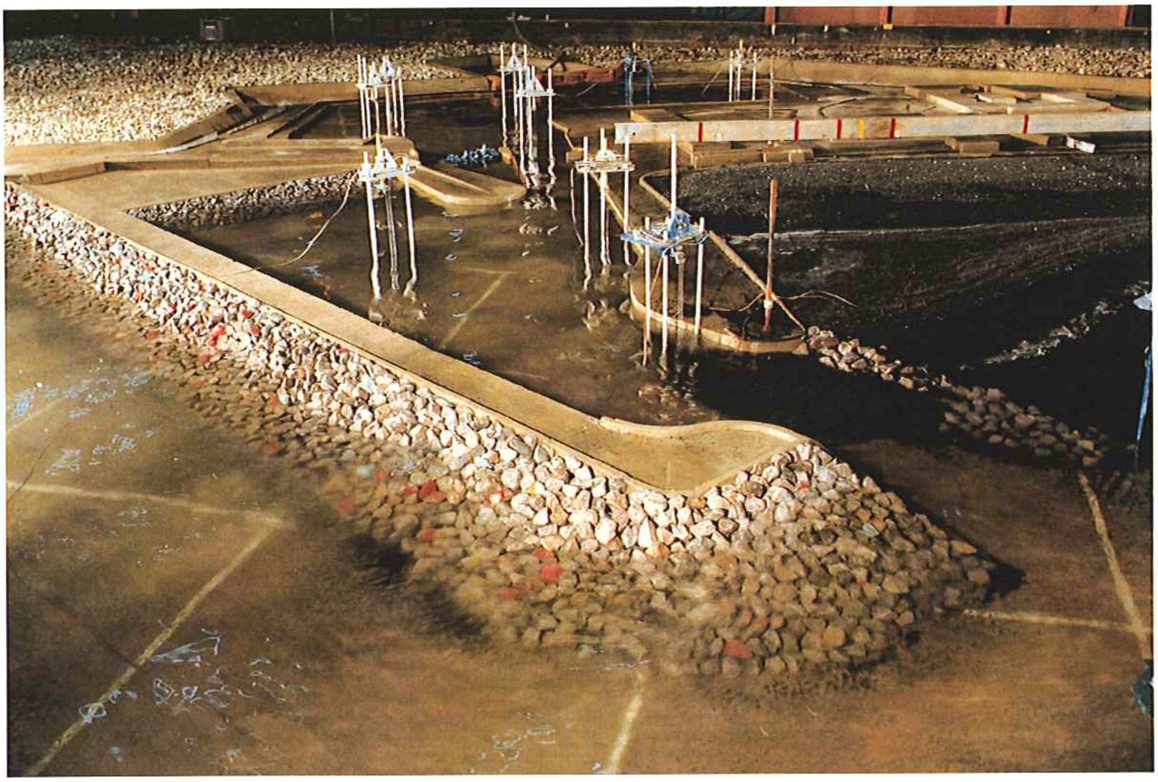

Plate 3 View of the physical model, Test Series 4 and 5

Plate 4 Rock armoured roundhead, Test 3a

Plate 5 Outer harbour modification, Test 3b

Plate 6 Outer harbour modification, Test 3c

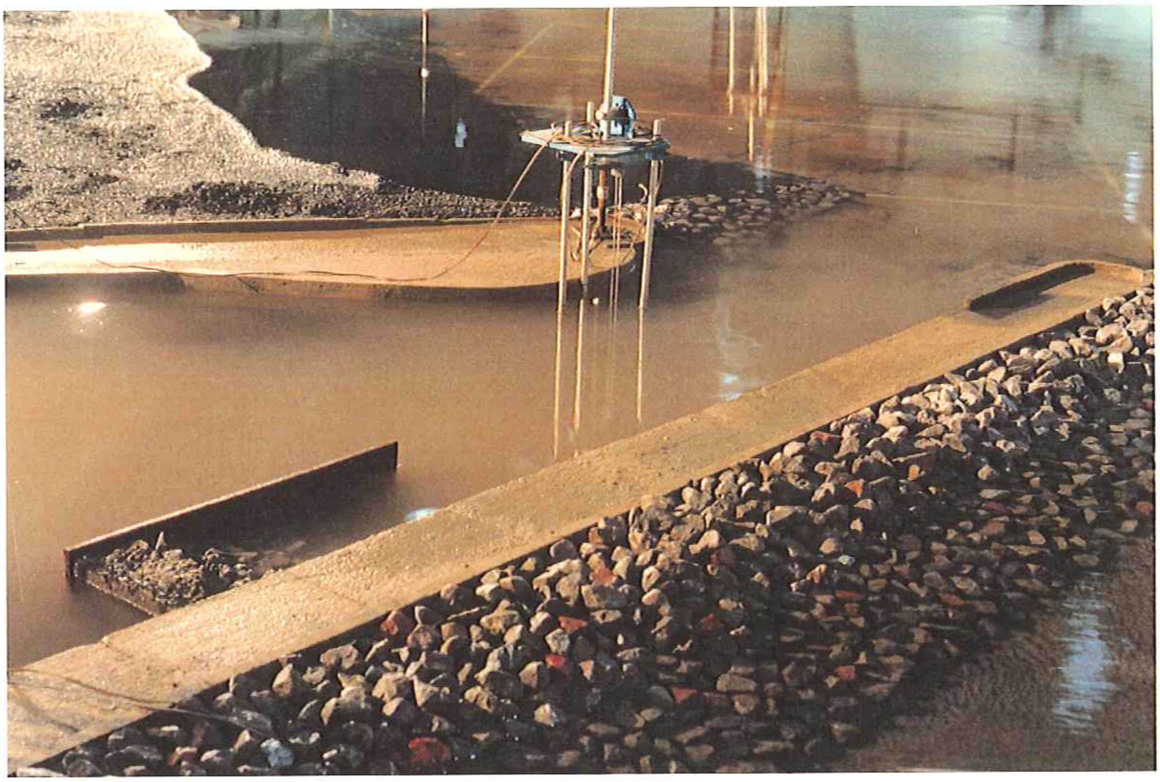

Plate 7 Outer harbour modification, Test 3d

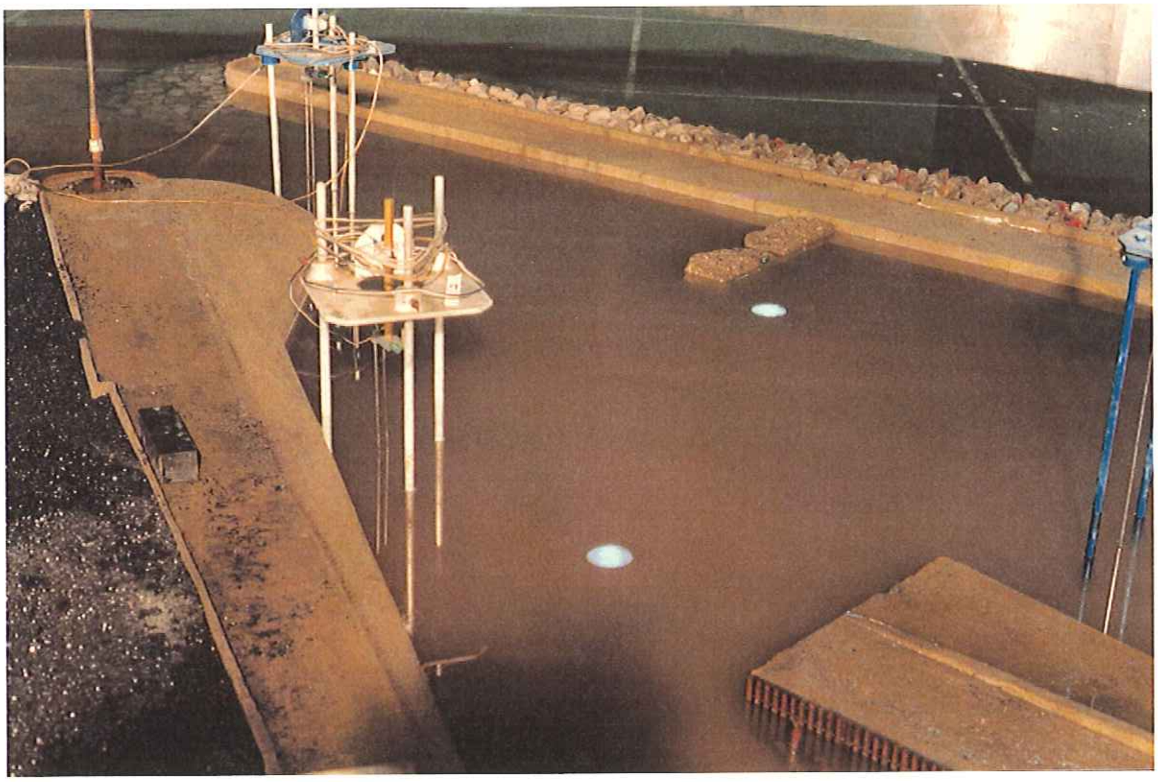

Plate 8 Outer harbour modification, Test 3e

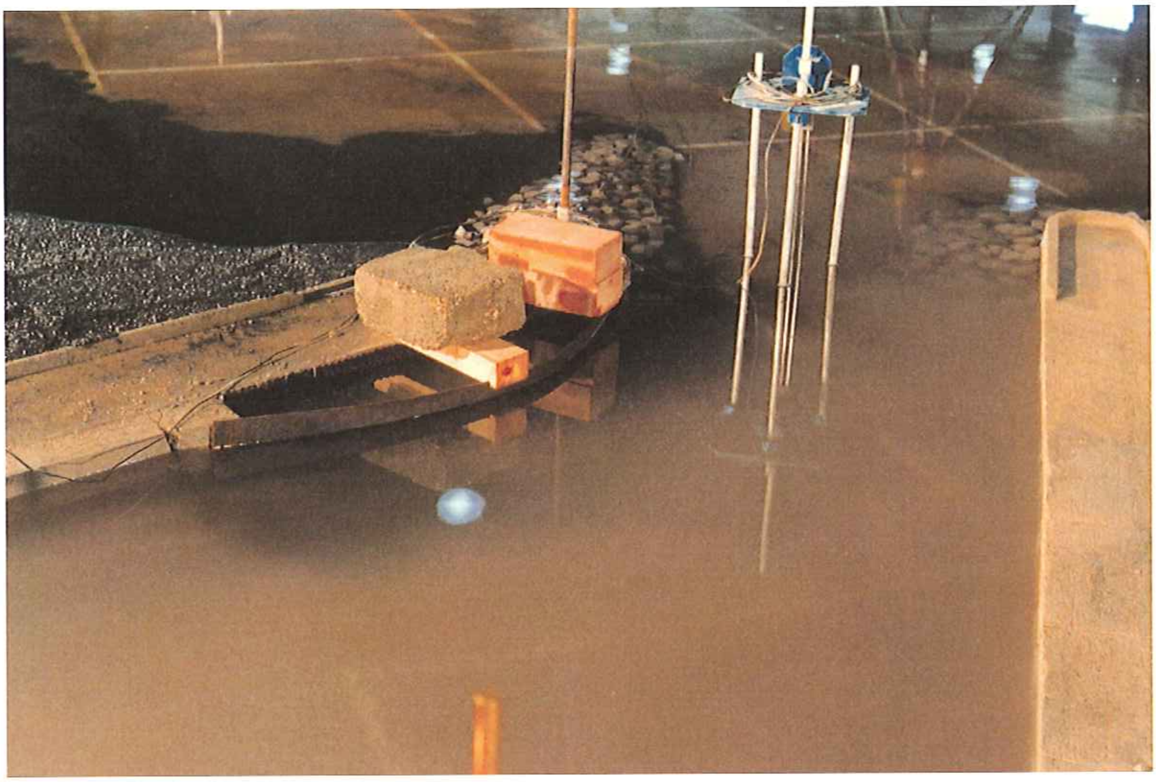

Plate 9 **Rock armour roundhead, with re-aligned curvature of East Pier, Test 3f**

Plate 10 **Vertical wall, entrance channel width 30m, Test 3g**

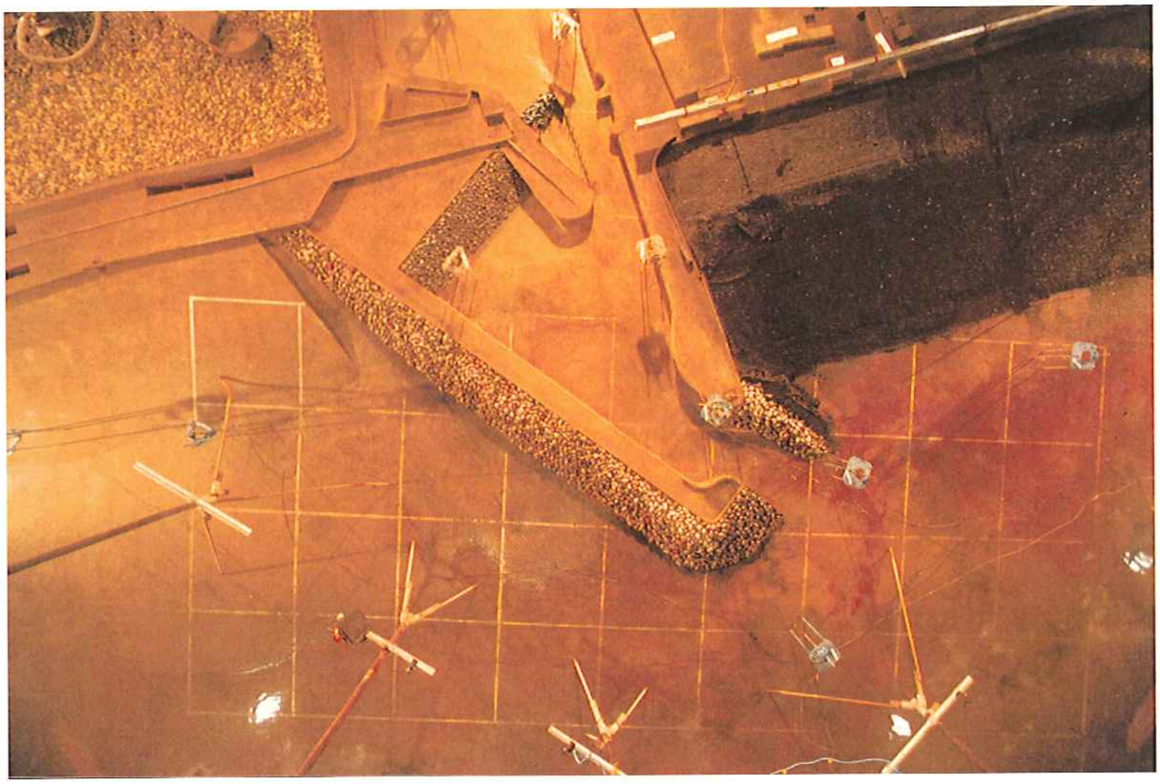

Plate 11 Vertical wall, entrance channel width 20m, Test 3h

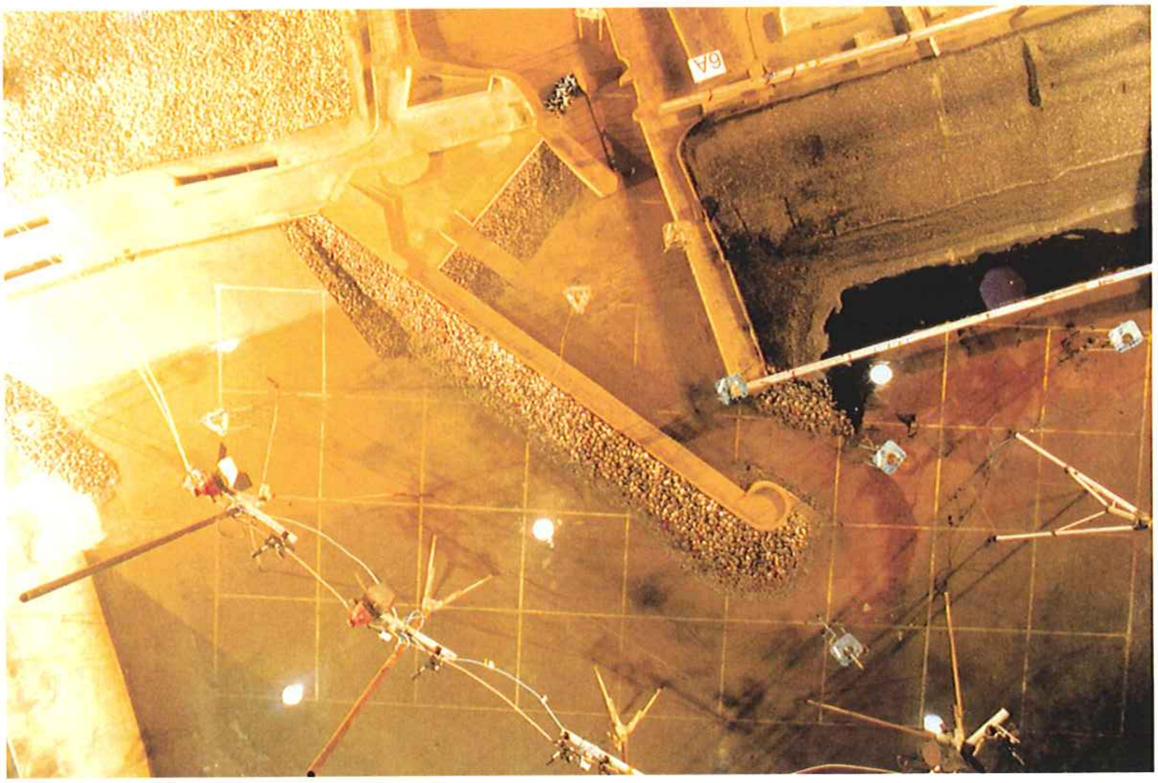

Plate 12 Harbour layout, Test Series 6

Plate 13a Roundhead detail after Test Series 6

Plate 13b Roundhead and East Pier extension detail after Test Series 6

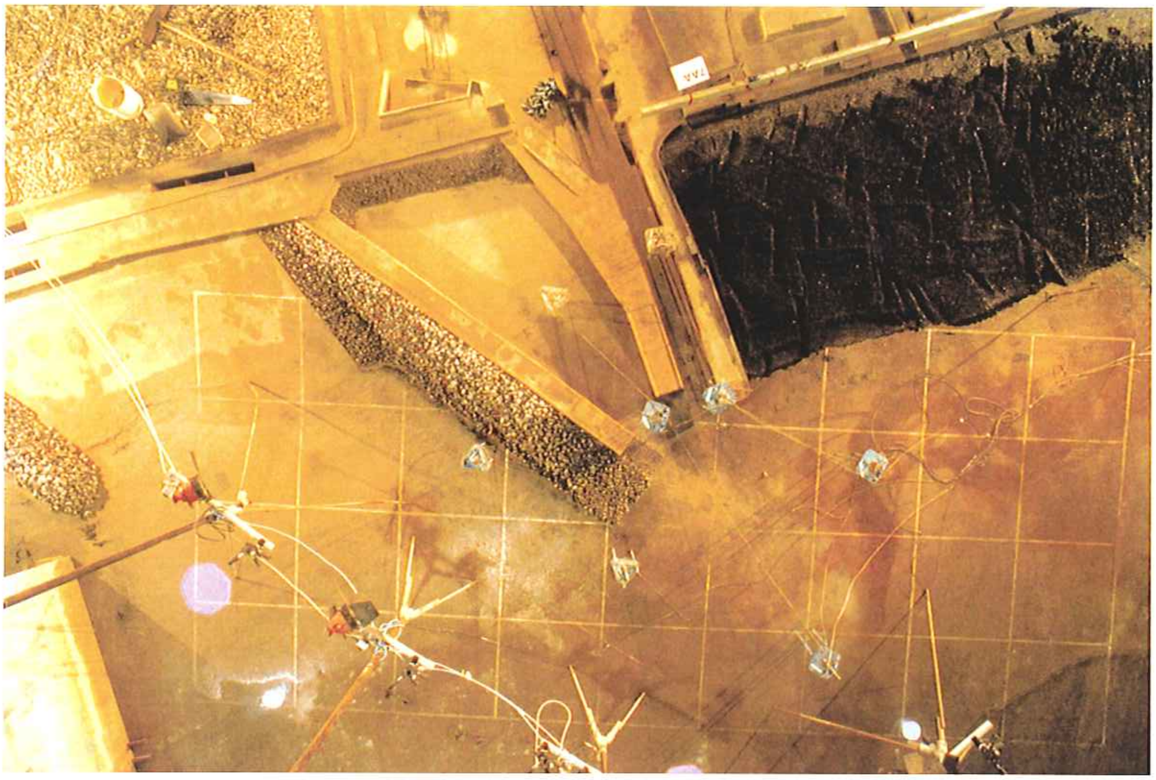

Plate 14 Construction phase tests, Test Series 7a

Plate 15 Construction phase tests, close-up of temporary works, Test Series 7a

Appendices

Plate 16 Construction phase tests, Test Series 7b

Plate 17 Construction phase tests, close-up of temporary works, Test Series 7b

Plate 18a Accretion of the beach at the existing East Pier, waves from 160°N, pre-test

Plate 18b Accretion of the beach at the existing East Pier, waves from 160°N, prototype test duration 14.6 hours

Plate 18c Accretion of the beach at the existing East Pier, waves from 160°N, prototype test duration 43.7 hours

Plate 18d Accretion of the beach at the existing East Pier, waves from 160°N, prototype test duration 58.3 hours

Plate 19a Accretion of the beach at the extended East Pier, waves from 160°N, prototype test duration 31.8 hours

Plate 19b Accretion of the beach at the extended East Pier, waves from 160°N, prototype test duration 70.0 hours

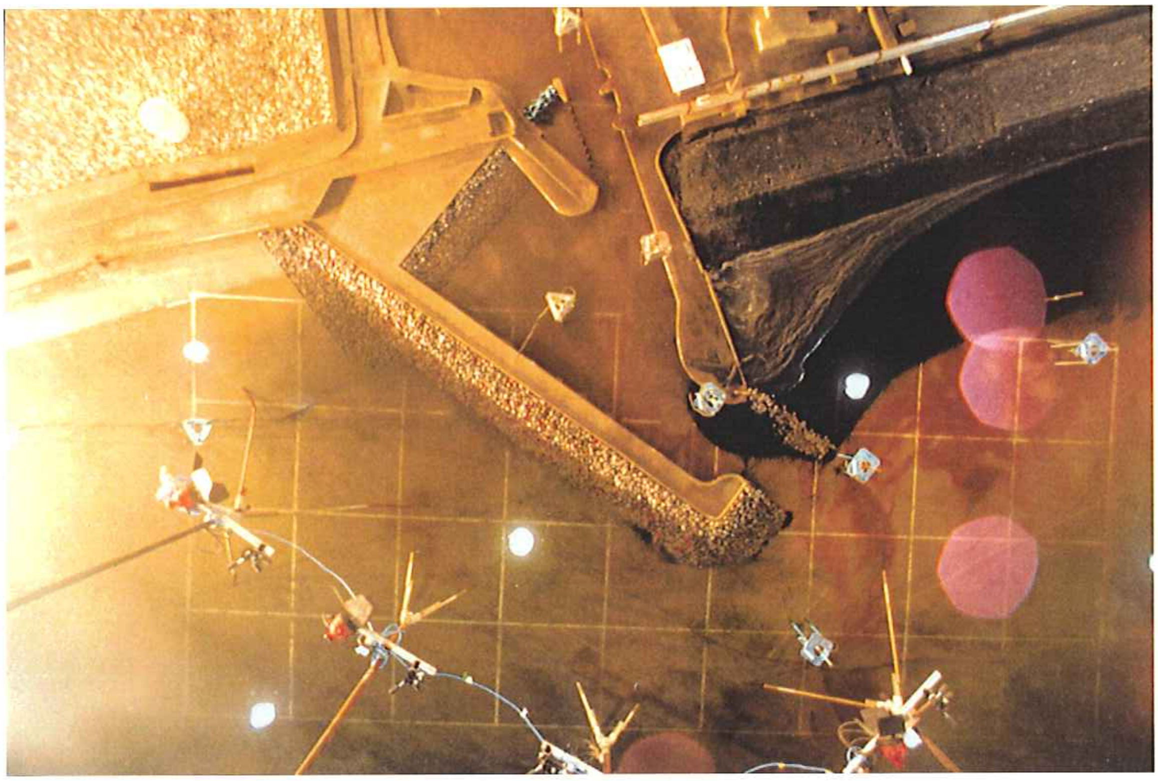

Plate 19c **Accretion of the beach at the extended East Pier, waves from 160°N, prototype test duration 90.2 hours**

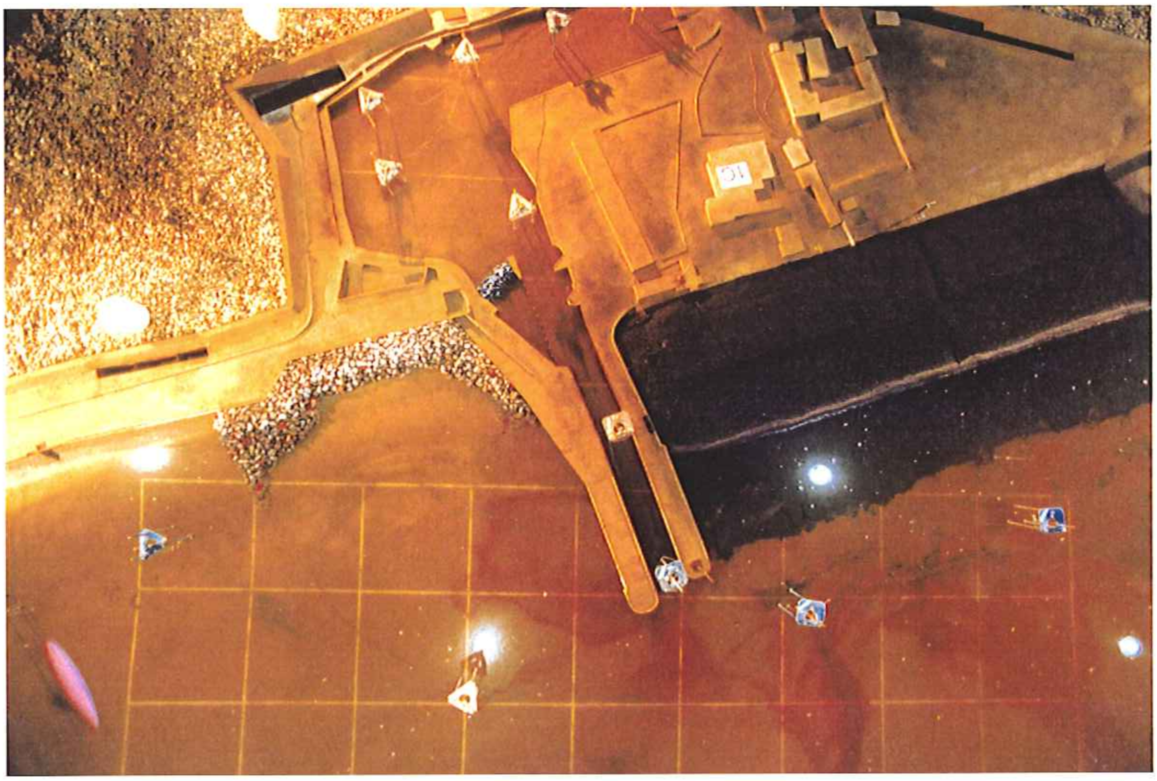

Plate 20a Erosion of the beach at the existing East Pier, waves from 220°N, pre-test

Plate 20b Erosion of the beach at the existing East Pier, waves from 220°N, prototype test duration 20.5 hours

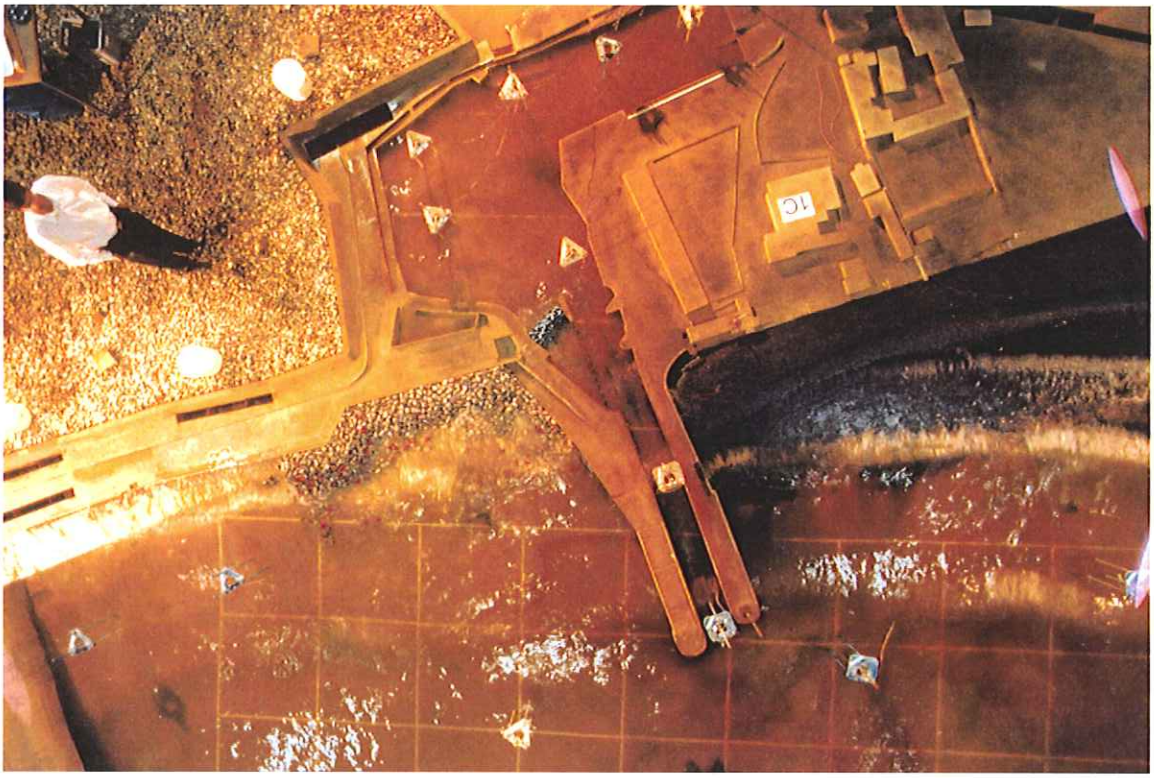

Plate 20c Erosion of the beach at the existing East Pier, waves from 220°N, prototype test duration 57.3 hours

Plate 20d Erosion of the beach at the existing East Pier, waves from 220°N, prototype test duration 89.4 hours

Plate 21a Erosion of the beach at the extended East Pier, waves from 220°N, pre-test

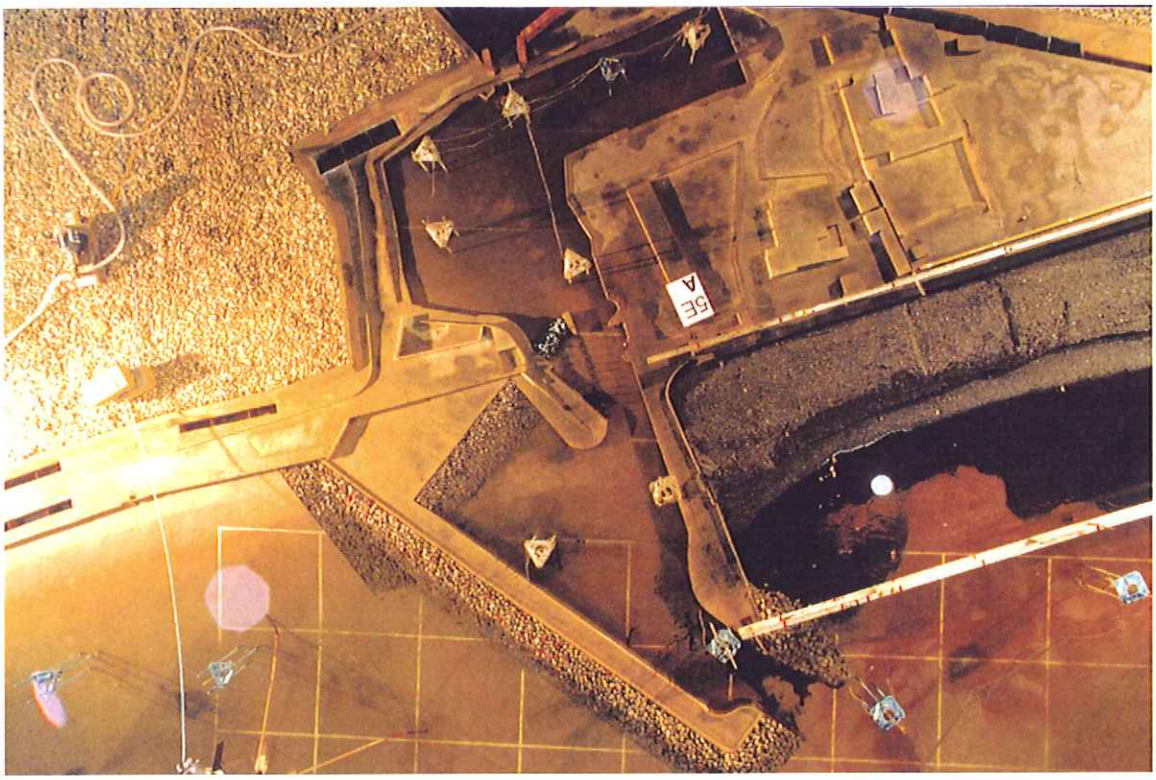

Plate 21b Erosion of the beach at the extended East Pier, waves from 220°N, prototype test duration 20.5 hours

Plate 21c **Erosion of the beach at the extended East Pier, waves from 220°N, prototype test duration 42.8 hours**

Plate 22a Roundhead before Test Series 5

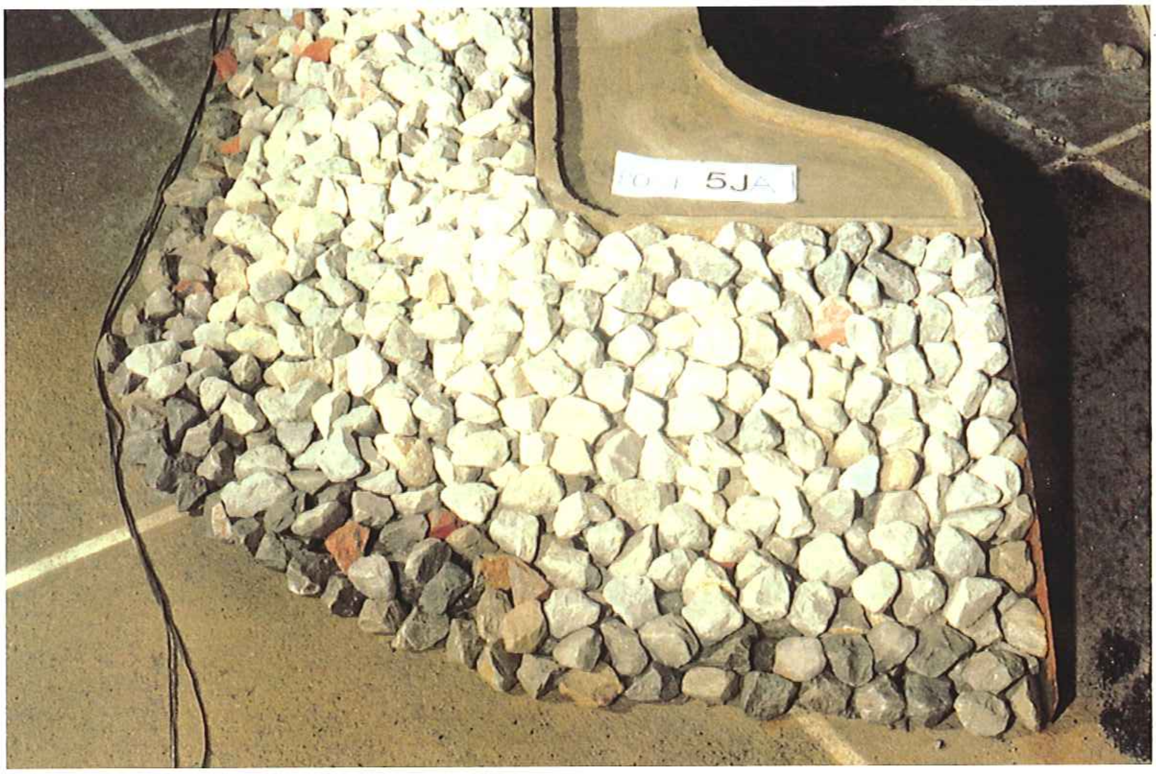

Plate 22b Roundhead after Test Series 5

Plate 23a Roundhead before Test Series 6

Plate 23b Roundhead after Test Series 6

Appendix 1

Design of model armour for correct stability

Appendix 1 Design of model armour for correct stability

The rock used in the physical model was a Carboniferous Limestone of density 2.71t/m^3. The fluid used in hydraulic model tests was fresh water of density 1.00t/m^3. In prototype, however, the sea water will have a density of about 1.025t/m^3 and the armour stone approximately 2.65t/m^3. This variation in densities means that without compensation, the rock in the model representing the toe rock would be more stable than in the prototype. Such a model would therefore underestimate movement and, hence, damage. It was therefore necessary to correct the size of rock to be used in the model, so that it exhibited the same stability characteristics as the prototype.

A correction factor for density may be derived by reference to the Hudson equation (Reference A1.1) which states:-

$$M \alpha \frac{\rho_s H_s^3}{(\rho_s / \rho_f - 1)^3 \cot \theta}$$

where	M	is the mass of the armour unit
	H_s	is the significant wave height
	θ	is the structure slope angle to the horizontal
	ρ_s	is the density of the armour
and	ρ_f	is the density of the displacing fluid.

The correction factor for the armour mass may thus be calculated from the following equation:-

$$\left(\frac{M_p}{\rho_{sp}}\right)^{0.33}\left(\frac{\rho_{sp}}{\rho_{fp}}-1\right)=\left(\frac{M_m}{\rho_{sm}}\right)^{0.33}\left(\frac{\rho_{sm}}{\rho_{fm}}-1\right)$$

where the subscripts p and m respectively refer to parameters in the prototype and model.

Reference

A1.1Coastal Engineering Research Centre. (1984) "Shore Protection Manual." CERC, US Government Printing Office, Washington, Vols 1 & 2.

Appendix 2

Design of model underlayer for correct permeability

Appendix 2 Design of model underylayer for correct permeability

For testing of overtopping performance the rocks used to construct the rubble mound breakwater were scaled to accurately reproduce the porosity of the prototype structure. This ensured that the hydro-dynamic processes which result in wave run-up and overtopping were modelled correctly

This scaling method was also used for the core and underlayer during the overtopping and the stability testing. This reflects the different physical processes which occurred within the structure as opposed to those due to direct wave action. Without allowing for this there may have been conditions where the flow through the model underlayer was not completely turbulent, since the model was at a reduced scale. Scale effects would have affected the flow of water through the underlayer and core.

The sizes of material scaled in this way were therefore adjusted to ensure that their permeability gave correctly-scaled flow conditions. Work by Jensen & Klinting (Reference A2.1) suggest a method of compensating for scale effects by applying a correction factor. The calculation of the correction factor uses a special Reynold's number, ξ_p, which is defined as the ratio of turbulent to laminar hydraulic gradients. This Reynold's number is defined as:

$$\xi_p = \frac{\beta_o}{\alpha_o} \frac{1}{n_r (1 - n_r)^2} \frac{U_p D}{\nu}$$

where α_o and β_o are empirical dimensionless coefficients, n_r is the porosity of the prototype rock mound, D is the diameter of the prototype rock (m), ν is the kinematic viscosity of water($m^2 s^{-1}$) and U_p is the maximum water particle velocity in the prototype rock mound ($m s^{-1}$).

The ratio of the rock size in prototype to model, K, is then given by:

$$K = \frac{\xi_p}{2\lambda^{1/2}} \left[\left(1 + 4\lambda^{3/2} \frac{1 + \xi_p}{\xi_p^2} \right)^{1/2} - 1 \right]$$

The porosity of model and prototype rock mounds will need to be the same to avoid changes in the potential storage volume.

Certain assumptions were made to enable the above equations to be used in calculating a correction factor. Experimental work by Engelund suggested values for the empirical coefficients of $\alpha_o = 1500$ and $\beta_o = 3.6$. The maximum prototype velocity in the mound was estimated at 0.5-1.0 m s^{-1} from some simple calculation of wave velocities and comparisons with velocities calculated by a simple mathematical model of flow in rubble. The porosity of the rock mound, n_r, was also estimated at 35-40%.

There is some scope for error in the calculation of the ratio of prototype to model rock size, K. A series of calculations were therefore completed to carry out sensitivity tests on the variables. These results of gave K values between 55-63 for the core material, between 72-76 for the filter layer, between 75-78 for the toe armour and 76-78 for the main rock armour. These values are always less than the geometric scale and were used for the preparation of the material for overtopping testing.

Reference

A2.1Jensen O J and Klinting P. (1983) "Evaluation of Scale Effects in Hydraulic Models by Analysis of Laminar and Turbulent Flows". Coastal Engineering, pp 319-329.

Appendix 3

Scaling of beach material

Appendix 3 Scaling of beach material

Introduction
In order that a mobile bed physical model may accurately reproduce natural beach processes, such as scour and accretion, it is necessary to ensure that the active sediment used in the model is representative of that occurring in nature.

Ideally the model sediment should satisfy three criteria:-

a) the beach permeability

b) the relative magnitude of the onshore and offshore motion

c) the threshold of motion.

The first of these governs the beach slope, the second determines whether the beach will erode or accrete under given wave conditions and the third establishes at what wave velocity motion will begin.

Permeability
In order to correctly reproduce permeability in the model, the percolation slope, J, must be identical in model and prototype. The percolation slope is defined as:-

$$J = \frac{K v^2}{g D_{10}} \tag{3.1}$$

where
- K is the permeability = $f\{Re_v\}$
- Re_v is the voids Reynolds number = vD_{10}/υ
- v is the velocity through the voids
- g is acceleration due to gravity
- D_{10} is the 10% undersize of the sediment

and υ is the kinematic viscosity of water.

For identical percolation slopes in model and prototype:-

$$\lambda_v^2 \lambda_k / \lambda_D = 1 \tag{3.2}$$

where λ is the prototype to model ratio of the relevant parameter.

Assuming the model is operated according to Froude's Law then $\lambda_v^2 = \lambda$, the geometric scale, so that:-

$$\lambda \lambda_k / \lambda_D = 1 \tag{3.3}$$

Now $\lambda_K = K_p / K_m$, where p and m respectively denote prototype and model, and hence:-

$$\lambda_D = \lambda K_p / K_m \tag{3.4}$$

Also:-

$$\log K = 3.17 - 1.134 \log Re_v + 0.155 (\log Re_v)^2 \quad (3.5)$$

within the range $1 < Re_v < 200$

Hence for a given percolation slope, J, the value of K_p can be found, by adopting an iterative method, using equations (3.1) and (3.5). Now:-

$$Re_{vm} = Re_{vp} / \lambda_D \lambda_v = Re_{vp} / \sqrt{\lambda}\, \lambda_D \quad (3.6)$$

A further iterative procedure can therefore be employed to calculate λ_D using equations (3.4), (3.5) and (3.6).

Direction of motion

Several authors have postulated that the relative tendency for sediments to move onshore or offshore depends on the dimensionless parameter H_b/wT, where H_b is the wave height at breaking, T is the wave period and w is the settling velocity of the sediment particles. Roughly speaking if $H_b/wT < 1$ then the sediment moves onshore, and if $H_b/wT > 1$ then offshore movement occurs (see for example Reference A3.1). In physical terms the parameter represents the ratio between the wave height and the distance which the sediment particle can settle during one wave period. For correct reproduction of the relative magnitudes of onshore and offshore movement the model scales must therefore be such that:-

$$\lambda_{H_b} / \lambda_w \lambda_T = 1 \quad (3.7)$$

With a Froudian model $\lambda_T = \lambda^{1/2}$, and assuming that the beach slope is correctly modelled then $\lambda_{H_b} = \lambda$, so that we have $\lambda_w = \lambda^{1/2}$.

In general, the settling velocity is given by:-

$$w = \left[\frac{4 gD(\rho_s - \rho_f)}{3 C_D \rho_f} \right]^{1/2} \quad (3.8)$$

where ρ_s and ρ_f are specific gravities of the sediment and fluid respectively and C_D is the drag coefficient for the settling particles.

For modelling purposes we therefore have:-

$$\lambda_w = \lambda_D^{1/2} \lambda_{\rho_{s'}}^{1/2} / \lambda_{C_D}^{1/2} = \lambda^{1/2}$$

or

$$\lambda_{\rho_{s'}} = \lambda \lambda_{C_D} / \lambda_D \quad (3.9)$$

where $\rho_{s'} = (\rho_s - \rho_f) / \rho_f$

Unfortunately C_D is also a non-linear function, in this case a function of the sediment particle Reynolds number $Re_s = wD/\upsilon$. Rouse has documented the variation in C_D with Re_s in the form of a curve. Using this curve in conjunction with equation (3.8) allows the prototype drag coefficient, C_{Dp}, and sediment particle Reynolds number, Re_{sp}, to be estimated. Now:-

$$Re_{sm} = Re_{sp} / \lambda_D \lambda_w = Re_{sp} / \sqrt{\lambda}\, \lambda_D \qquad (3.10)$$

Equation (3.10) allows Re_{sm} and hence C_{D_m} and λ_{C_D} to be calculated. Substituting λ_{C_D} in equation (3.9) then allows $\lambda_{\rho_{s'}}$ to be determined.

Threshold of motion

For oscillating flow Komar and Miller (Reference A3.2) proposed that for sediment sizes greater than 0.50mm, which is expected to be the case for both model and prototype sediments, the threshold of movement was defined by the expression:-

$$\frac{U_m^2}{\rho_{s'}\, gD} = 0.46\,\pi \left[\frac{d_o}{D}\right]^{1/4} \qquad (3.11)$$

where U_m is the peak value of the near-bed orbital velocity at the threshold of motion and $d_o$is the near-bed orbital diameter.

Since $U_m = \pi d_o/T$, this expression can be re-written:-

$$U_m^{7/4} / (\rho_{s'}\, D^{3/4}\, T^{1/4}) = 0.46\, \pi^{3/4}\, g \qquad (3.12)$$

To the first order, the maximum orbital velocity near the bed is given by:-

$$U_m = \frac{\pi H}{T\ \mathrm{Sinh}(2\pi\, h/L)} \qquad (3.13)$$

where L is the wavelength
H is the wave height
and h is the water depth.

Substituting this expression, and rearranging, gives the threshold in terms of wave height and period as:-

$$H^{7/4}\, A^{7/4} / (\rho_{s'}\, D^{3/4}\, T^2) = 0.46\, g/\pi \qquad (3.14)$$

where A is the depth attenuation factor $1/\sinh(2\pi h/L)$.

For correct modelling we therefore have:-

$$\lambda_H^{7/4}\, \lambda_A^{7/4} / (\lambda_{\rho_{s'}}\, \lambda_D^{3/4}\, \lambda_T^2) = 1$$

In a Froudian model $\lambda_H = \lambda_L = \lambda_h = \lambda$ and $\lambda_T = \lambda^{1/2}$

Therefore $\lambda_A = 1$. This gives:-

$$\lambda_{\rho_{s'}}\, \lambda_D^{3/4} = \lambda^{3/4} \qquad (3.15)$$

Hence the use of equation (3.15) and the value of λ_D derived previously allows $\lambda_{\rho_{s'}}$ to be calculated.

West Bay physical model study - choice of model sediment

Grading curves for the renourished beach material proposed for the West Bay frontage, had a D_{50} of approximately 7mm.

Using the above criteria a model material with a D_{50} of 2.0mm and a specific gravity of between 1.24 – 1.30 is required. However, the choice of specific gravities for the model sediment is in practice severely limited by the availability of suitable materials. The usual choice of material for shingle beach studies is crushed anthracite which has a specific gravity of 1.39.

For this study, the use of anthracite as the model material requires that the threshold of motion is relaxed. This was compensated for by calibrating the physical model against the BEACHPLAN numerical model. The permeabilty of the beach, important when assessing overtopping, is not affected by the change in density of the material so a D_{50} of 2.0mm was still required.

References

A3.1 `Shore protection manual' Published by the US Army Coastal Engineering Research Centre, Vicksburg, Mississippi, 1984

A3.2 Komar P D and Miller M C. `The threshold of movement under oscillatory water waves'. J Sediment Petrol. 43: 1101-1110, 1973.