Appendix 7B

Water Level and Wave Modelling Study

Department of the Marine and Natural Resources
Roinn na Mara agus Acmhainní Nádúrtha

Rossaveel Harbour Development

Final Design Report

Volume 2 – Water Level and Wave Modelling Study

January 2002

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1 Introduction

1.1 Background

This document is Volume 2 of the Design Report for the Rossaveel Harbour Development and summarises the water level and wave modelling studies carried out to assist with the development of port layouts for the project. The modelling studies also assess the acceptability, or otherwise, of likely operating conditions at the deep water quay and pontoon ferry berths.

The primary focus of the document is the proposed deep water quay – this is because in relative terms it is more exposed – but consideration is also given to wave and still water conditions at the proposed ferry berths, as appropriate.

The Design Report comprises four volumes. Volume 1 provides the main text and associated appendices and in particular an assessment of the port development options considered. The third volume, Volume 3, summarises the DIVAST mass water movement studies carried out and reports on these and associated issues. For ease of reference, all figures in Volume 1 have been presented at A4 size and for greater clarity, Volume 4, presents these as a complete folio in A3 format. An Executive Summary is presented as a separate document.

1.2 Port Developments Options Considered

1.2.1 Preferred Deep Water Quay Layout

The deep water quay layout illustrated in this document is the "L" shaped configuration shown in Figure 9.5 of Volume 1[1.1]. This layout provides an approach causeway at the southern end of the layout, which provides shelter to an inside berthing basin. For ease of reference, a copy of this figure is included in Appendix A to this volume.

1.2.2 Preferred Ferry Berth Layout

The ferry berth layout considered in this document is the triple pontoon finger layout shown in Figure 11.5 of Volume 1. For ease of reference, a copy of this figure is included in Appendix A to this volume.

1.3 Organisation of the Report

Excluding this introductory chapter, this report comprises six chapters and these are supported by several figures. The principal content of the chapters is summarised in the following paragraphs.

Chapter 2 develops extreme positive and negative still water levels primarily for use in the subsequent MIKE 21 wave modelling analyses but also for consideration of reclamation levels, deck levels and under keel clearances.

Chapter 3 assesses the extent to which the sheltering effects of the Aran Islands may be taken into consideration in developing wave inputs for the MIKE 21 wave modelling analyses.

Chapter 4 develops wind generated wave conditions in the North Sound, using a hindcasting approach, for use in the MIKE 21 wave modelling analyses.

Chapter 5 develops on from Chapters 2 to 4 inclusive and describes the development of a MIKE 21 wave model for Rossaveel.

Chapter 6 discusses the results of the MIKE 21 analyses in the context of commonly accepted limiting wave criteria for the types of vessel which are expected to use the deep water quay.

Chapter 7 describes the use of a hindcasting approach to estimate locally generated wind wave heights in the immediate vicinity of the proposed pontoon ferry berths.

1.4 Reference Datum

As adopted for Volume 1, all levels and depths in this volume relate to Chart Datum.

1.5 References

(1.1) Mott MacDonald EPO, Rossaveel Harbour Development Design Report, Volume 1, November 2001

2 Development of Design Still Water Levels

2.1 Introduction

This chapter summarises the principal elements of the process of developing extreme still water levels for use in the design of the deep water quay and the pontoon ferry berths.

2.2 Astronomical Tides

2.2.1 Admiralty Tide Table Predictions

Tide level predictions for Standard and Secondary Ports are given in the United Kingdom Admiralty Tide Tables(2.1). Rossaveel does not appear in the tables as either a standard or a secondary port and accordingly no direct tidal predictions are available for Rossaveel.

The nearest Standard Port is Galway and the nearest Secondary Ports are Kilkieran Cove and Killeany Bay (Figure 2.1). Table 2.1 summarises tidal predictions for these locations for the principal spring and neap tide levels.

Table 2.1: Predicted Tide Levels at Galway, Kilkieran Cove and Killeany Bay

Tidal Prediction	Level in Galway	Level in Kilkieran Cove	Level in Killeany Bay
Highest Astronomical Tide	+ 5.6		
Mean High Water Springs	+ 5.1	+ 4.8	+ 4.7
Mean High Water Neaps	+ 3.9	+ 3.7	+ 3.6
Mean Sea Level	+ 2.9		
Mean Low Water Neaps	+ 2.0	+ 1.9	+ 1.8
Mean Low Water Springs	+ 0.6	+ 0.6	+ 0.5
Lowest Astronomical Tide	- 0.2		

Source: United Kingdom Admiralty Tide Tables, Volume 1

The above levels would be expected to be reasonably representative of predicted tide levels at Rossaveel. However, more refined tidal data is required for detailed design and the acquisition and analysis of this is described below.

2.2.2 Tidal Predictions for Rossaveel

Additional tidal data was requested from the Proudman Oceanographic Laboratory (POL), who carried out an analysis of a year's tidal data recorded in Rossaveel during the period 4 May 1973 to 3 May 1974 inclusive. Harmonic constants were obtained from this analysis and used to compute the high and low water predictions for the full nodal cycle (1983 to 2001 inclusive). From these, the Highest Astronomical Tide (HAT) and Lowest Astronomical Tide (LAT) values were extracted and the mean spring and neap tides were estimated. The estimated astronomic tide levels for Rossaveel are presented in Table 2.2:

Table 2.2: Predicted Tide Levels

Tidal Prediction	Level (m)
Highest Astronomical Tide (HAT)	+ 5.6
Mean High Water Springs (MHWS)	+ 4.9
Mean High Water (MHW)	+ 4.4
Mean High Water Neaps (MHWN)	+ 3.8
Mean Sea Level (MSL)	+ 2.8
Mean Low Water Neaps (MLWN)	+ 1.9
Mean Low Water Springs (MLWS)	+ 0.7
Lowest Astronomical Tide (LAT)	+ 0.0

Source: POL

The table shows that the predicted tide levels developed for Rossaveel by POL are in good agreement with those summarised in Table 2.1, giving confidence that the levels given in Table 2.2 may be used for planning and design purposes, as appropriate.

2.3 Extreme Surge Elevations

A direct estimate of storm surge elevations usually requires a statistical analysis of a long time series of good quality tide gauge data. POL considered that it would not be appropriate to predict extreme surge return period levels from the year's Rossaveel tidal data. Accordingly, extreme surge elevations were predicted using the data set of surge residuals obtained from the Continental Shelf (CSX) numerical model. The nearest model grid point at 53.17° North and 9.75° West was taken (Figure 2.1), for the years 1955 to 2000 inclusive.

A comparison of computed hourly Rossaveel surge residuals with the model grid point data shows a reasonable correlation, but the model significantly underestimates the two large positive surges recorded on 11th and 27th January 1974 by the Rossaveel temporary tide gauge. This may be due to a combination of the complicated local bathymetry, the distance to the harbour from the grid point and the shape of the adjacent coastline.

The extreme surge return period levels were estimated by POL using the “r’ largest” method on the surge series, using the 10 highest independent levels per year. Extreme positive surges resulting from this analysis are presented in Table 2.3.

Table 2.3: Extreme Positive Surge Values

Return Period (Years)	Height (m)
1	+ 0.6
2	+ 0.8
5	+ 1.0
10	+ 1.1
20	+ 1.3
50	+ 1.4
100	+ 1.6
250	+ 1.7

Source: POL

Extreme negative surges resulting from this analysis are presented in Table 2.4.

Table 2.4: Extreme Negative Surge Values

Return Period (Years)	Height (m)
1	- 0.5
2	- 0.6
5	- 0.7
10	- 0.7
20	- 0.8
50	- 0.9
100	- 0.9
250	- 1.0

Source: POL

2.4 Extreme Still Water Levels

2.4.1 Extreme Value Analyses

The observed still water level can be considered as the level arising from the combination of astronomical and surge components. Several methods are available for determining extreme still water levels by analysing available tidal data. These include the Generalised Extreme Value method (GEV), using annual maxima, or, a more refined, GEV technique known as Spatial Revised Joint Probability Method (SRJPM).

The GEV method cannot be applied to Rossaveel as no annual maxima data exists. Accordingly, POL used the SRJPM to produce extreme statistics in terms of the probabilities of exceeding high levels and of falling below low levels. These were then converted into return periods by taking account of the sampling intervals.

As there was only one year's hourly recorded data available for Rossaveel, estimates of tide and surge from the CSX model were also used to validate the observed data and to extend the surge population.

2.4.2 Resulting Positive Still Water Levels

Estimates of return periods of positive still water levels, assuming independence of astronomic tide and surge components, are given in Table 2.5.

Table 2.5: Extreme Positive Still Water Levels

Return Period (Years)	Level (m CD) 1973/74 Rossaveel records	Level (m CD) CSX Model	CSX Model Underestimate
1	+ 5.7	+ 5.5	- 0.2
2	+ 5.8	+ 5.6	- 0.2
5	+ 6.0	+ 5.8	- 0.2
10	+ 6.1	+ 5.9	- 0.2
20	+ 6.3	+ 5.9	- 0.4
50	+ 6.4	+ 6.0	- 0.4
100	+ 6.5	+ 6.1	- 0.4
250	+ 6.6	+ 6.2	- 0.4

Source: POL

The table shows that the CSX model underestimates the levels derived from the 1973/1974 recordings by approximately 0.2m for return periods of up to 10 years and by approximately 0.4m for return periods of between 20 and 250 years (shown shaded).

POL has indicated that two large positive surges of 1.2m and 1.4m recorded at Rossaveel on 11th and 27th January 1974 respectively dominate the surge distribution used in the SRJPM. Therefore the frequency of occurrence of these two surges is possibly greater than that from a much longer period of data and hence the still water level return periods may be overestimated. POL added that the CSX model gave a better estimate of the longer period surge distribution and hence better estimates for the return periods of still water levels.

It is evident from above that POL consider the still water level values derived from Rossaveel observed data may be an overestimate, whilst those derived from the synthesised model data are somewhat underestimated.

Accordingly, it is recommended that the mean of the values obtained from the two different data sets is adopted for design purposes and the resulting levels are presented in Table 2.6.

Table 2.6: Mean Extreme Positive Still Water Levels

Return Period (Years)	Level (m CD), Year 2000
1	+ 5.6
2	+ 5.7
5	+ 5.9
10	+ 6.0
20	+ 6.1
50	+ 6.2
100	+ 6.3
250	+ 6.4

Source: Derived from POL data

2.4.3 Adjustment of Positive Levels for Sea Level Rise

The still water levels given in Table 2.6 do not include any allowance for secular trends in mean sea level resulting from local and global long term oceanographic, atmospheric or geological changes.

However, sea levels are predicted to continue to rise at a faster rate in the future due to global warming and land level adjustments. According to the Intergovernmental Panel on Climate Change (IPCC 2001), predictions of mean sea level rise based on various numerical models range from 110mm to 770mm for the 20 year period from the year 1990 to 2010. This is equivalent to an annual rate of rise ranging from approximately 6mm to 38mm, or, a total rise, over a 50 years project life, of 0.3m to 1.9m.

The Marine Institute's 1999 environmental assessment of Ireland's coastline includes an estimate of a sea level rise of 0.3m between 1990 and 2030 [2.2].

The United Kingdom Department of Environment, Food and Rural Affairs (DEFRA)[2.3] have recommended values of 6mm per year and 4mm per year for planning and design of coastal defence schemes in the south and north west of England respectively. Both the Marine Institute and DEFRA values are at the lower end of the IPCC range described above.

In the absence of specific data, it is suggested that the DEFRA recommended sea level rise figure of 6mm per year for the south coast of England should be applied over the full design life (50 years) for the estimation of extreme still water levels at the site. This results in the still water levels summarised in Table 2.6 being increased by 0.3m – equal to the 40 years estimate included in the Marine Institute document.

2.4.4 Extreme Negative Still Water Levels

Estimates of return periods of negative still water levels assuming independence of tide and surge are given in Table 2.7.

Table 2.7: Extreme Negative Still Water Levels

Return Period (years)	Level (m CD) 1973/74 Rossaveel records	Level (m CD) CSX Model	CSX Model Underestimate(-) or Overestimate (+)
1	+ 0.0	- 0.0	0
2	- 0.0	- 0.1	- 0.1
5	- 0.1	- 0.1	0
10	- 0.2	- 0.2	0
20	- 0.2	- 0.3	+ 0.1
50	- 0.3	- 0.3	0
100	- 0.3	- 0.4	+ 0.1
250	- 0.3	- 0.4	+ 0.1

Source: POL

Table 2.7 indicates that in more extreme instances, for example in the 1 in 100 years event, the still water level could fall below Lowest Astronomic Tide by approximately 300 to 400mm, depending upon the analysis selected. This has implications for the planning and design of both the deep water quay and the pontoon ferry berths.

For the deep water quay, the principal issue is that a significant depression in still water level below LAT may result in a vessel alongside the quay grounding, albeit temporarily. Such a grounding may lead to damage to the vessel or pollution, or a combination of both, particularly as the seabed at the berth is rock. In order to minimise the risk of such an event it will be necessary to specify a reasonably generous under keel clearance for the berth. For practical purposes, a minimum under keel clearance of 1m is considered satisfactory.

For the ferry berths two issues require consideration. The first issue is that a minimum under keel clearance of 1m should be provided, in a similar manner to that specified for the deep water quay. The second issue is that the combined pontoon / linkspan system should continue to operate during and after the occurrence of, for example, the 1 in 100 years negative still water event. This is likely to be a detailed design issue.

2.5 Summary of Selected Design Still Water Levels

2.5.1 Positive Still Water Levels

The combination of the positive extreme still water levels given in Table 2.6, corrected to year 2050 using DEFRA's recommendations of 6mm/year for the rate of sea level rise, are presented in Table 2.8.

Table 2.8: Design Positive Extreme Still Water Levels

Return Period (Years)	Level (m CD), Year 2050
1	+5.9
2	+6.0
5	+6.2
10	+6.3
20	+6.4
50	+6.5
100	+6.6
200	+6.7

2.5.2 Negative Still Water Levels

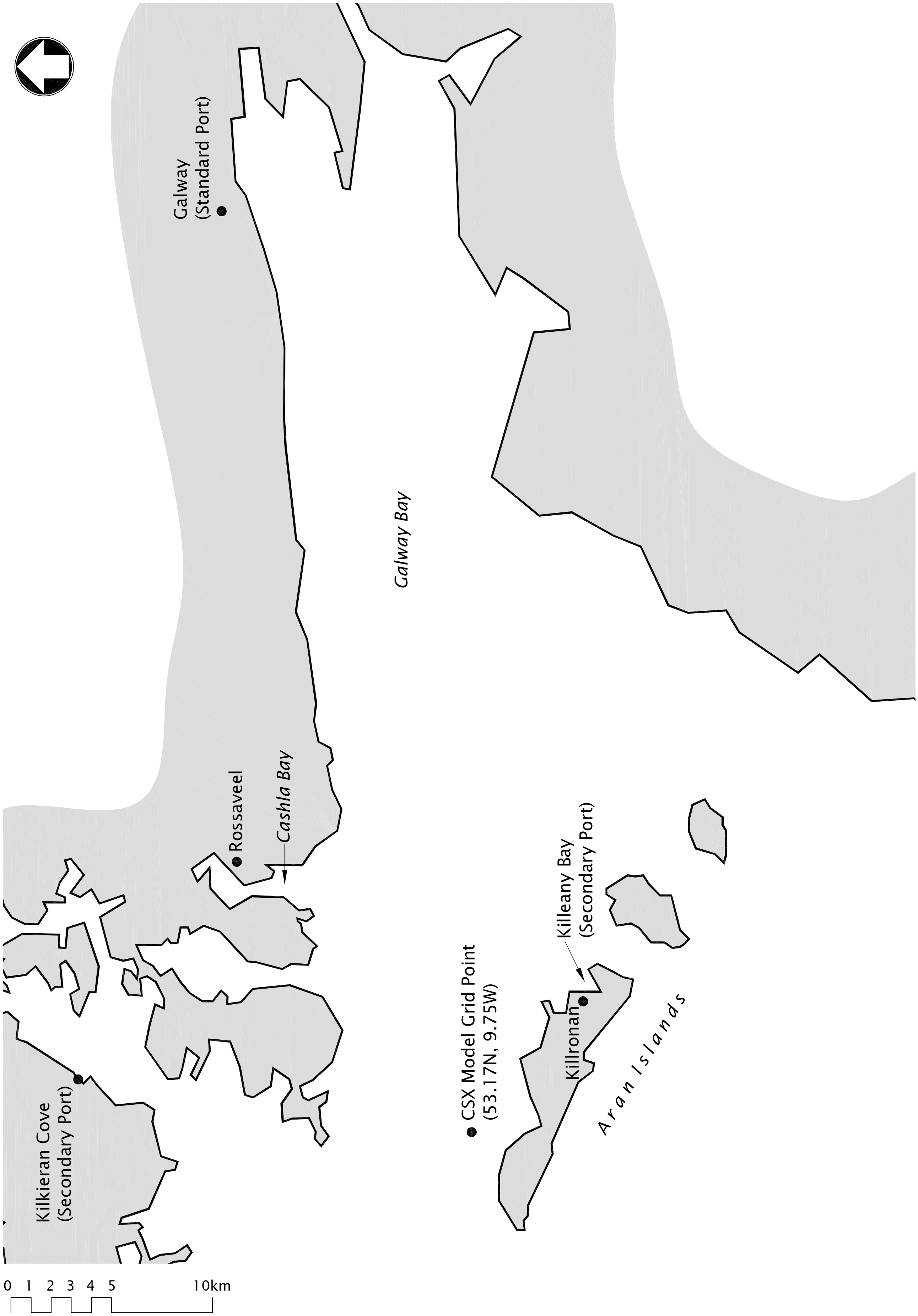
For the purposes of this report, a 1 in 100 years still water level of -0.4m CD, as predicted by the CSX model, has been adopted for planning and design purposes and in particular as a criterion for specifying under keel clearances.

2.6 References

(2.1) Admiralty Tide Tables, "*United Kingdom and Ireland (including European Channel Ports)",* Published by the Hydrographer of the Navy, Volume 1, 2001.

(2.2) Chapter 2, page 20 in *Ireland's Marine and Coastal Areas and Adjacent Seas: An Environmental Assessment*, The Marine Institute, Foras na Mara, March 1999.

(2.3) Flood and Coastal Defence Project Appraisal Guidance (Economic Appraisal - FCDPAG3: Table 4.4), DEFRA



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Location of CSX Model Grid Point
Figure 2.1

3 Assessment of Aran Islands Sheltering Effects

3.1 Introduction

This chapter assesses the extent to which the MIKE 21 wave modelling studies for the deep water quay, described in later chapters of this volume, should take into account the sheltering effects of the Aran Islands. In particular, the chapter considers what residual wave penetration, if any, should be input into the MIKE 21 analyses. The three main islands, Inishmore, Inishmaan and Inisheer form a natural barrier to incident Atlantic wind and swell waves and extend from approximately west north west to south east across the entrance to Galway Bay.

3.2 Exposure Conditions West of Aran Islands

The west and south facing shores of the Aran Islands are fully exposed to a severe wind and swell wave climate extending over a large sector from Slyne Head on the Conemara coastline to Loop Head on the County Clare coastline (Figure 3.1). However, in the context of a deep water quay at Rossaveel, enjoying the benefit of the shelter of the islands, the exposure sector is significantly reduced.

3.3 Open Sea Exposure Conditions at the Deep Water Quay

The seaward end of the proposed deep water quay is located at the head of outer Cashla Bay, just inside Lion Point, and because the outer bay takes the form of a relatively narrow funnel it is reasonably well sheltered from the open sea. In particular, fetch rays extending out from the seaward end of the quay define a narrow, 20^o, sector between approximately 170^o to 190^o. The 170^o and 190^o rays extend to Inisheer and the County Clare coastline respectively and the whole of the island of Inishmaan is captured within the limits defined by the rays.

Accordingly, the quay is sheltered from the full Atlantic fetch and in particular from the south west and westerly directions from which the most severe wave conditions would be expected. The quay location derives further shelter from Cannon Rock which is located within the 20^o sector described above.

The above paragraphs indicate that, in principle, the combination of the shape of Outer Cashla Bay and its spatial relationship with the Aran Islands - Inishmaan and Inisheer in particular - provides good shelter to the proposed deep water quay. The principal issue, as far as the open sea conditions considered in this chapter are concerned, is the extent to which Atlantic wind and/or swell waves may penetrate through the several channels between and around the Aran Islands into Outer Cashla Bay.

3.4 Orientation of the Aran Islands Channels

3.4.1 Overview

United Kingdom Admiralty Chart 3339 *Approaches to Galway Bay Including the Aran Islands* provides an appreciation of the geography of the islands' locations within the context of the Outer

Galway Bay coastline. In particular, the chart shows the spatial relationship between the islands and Outer Cashla Bay.

The chart shows that, although the islands provide an efficient barrier to Atlantic swell (Figure 3.1), they do not provide complete protection as there are four relatively wide and deep sounds, or channels, separating the islands from each other or the County Galway or County Clare coastlines, as appropriate. The channels extend from approximately west north west to south east in the sequence: North Sound, Gregory Sound, Foul Sound and finally, South Sound (Figure 3.2).

North Sound separates the largest island, Inishmore, from the Conemara coastline and South Sound separates Inisheer from the County Clare coastline. Gregory Sound, the most westerly of the two inner channels, separates Inishmore from Inishmaan and Foul Sound separates Inishmaan from Inisheer.

3.4.2 Commentary

It can be seen from Figure 3.2 that although the North Sound is wide and its east-west orientation permits the penetration of severe wave conditions, it is approximately normal to the axis of Outer Cashla Bay. Accordingly, no significant Atlantic wind or swell wave penetration would be expected from this direction and it is not considered further.

The South Sound is more likely to allow swell wave penetration which may subsequently reach Cashla Bay. It is considered, however, that due to a combination of the South Sound's distance from Cashla Bay and the deviation that swell waves would have to undergo to reach the Bay, their end effects would be negligible. Accordingly, swell penetration through the South Sound is not considered further.

Both Gregory and Foul Sounds are reasonably closely aligned with the axis of Outer Cashla Bay. Gregory Sound is likely to be of most concern, because it is slightly nearer to Outer Cashla Bay and is more closely orientated towards outer the Outer Cashla Bay axis. Swell penetration through Foul Sound is less likely to be an issue but requires consideration for completeness.

3.5 Wind Wave Penetration Through Gregory and Foul Sounds

Any wind waves generated in the Atlantic will be characterised by large directional and spreading parameters, and are therefore unlikely to penetrate with significant energy through the sounds. They may therefore be discounted and are not considered further.

3.6 Swell Wave Penetration Through Gregory and Foul Sounds

3.6.1 Extreme Atlantic Swell Wave Heights and Periods

Atlantic swell wave data has been obtained from the nearest United Kingdom Meteorological Office European Wave Model grid point at 53.0° North, 10.1° West (Figure 3.2). The period of data was from January 1990 to September 2001 inclusive, and the information provided included monthly and annual frequency, directional distribution of swell heights and periods.

The data also included an extreme value analysis of the swell wave heights for the two most critical sectors of 196° to 225° inclusive and 226° to 255° inclusive. The resulting extreme swell wave heights for the two sectors are presented in Table 3.1.

Incident Atlantic swell waves are predominantly from the south west and westerly directions, with approximately 50% of all swell waves originating from the 248° to 292° data sector.

Table 3.1: Extreme Atlantic Swell Wave Heights

Return Period (Years)	226° - 255° H_s (m)	196° - 225° H_s (m)
1	5.6	3.4
5	6.7	4.2
10	7.2	4.5
20	7.7	4.8
50	8.3	5.2
100	8.8	5.5
200	9.3	5.8

Source: United Kingdom Meteorological Office

The table shows that swell wave heights in the west south west sector are significantly higher than those in the south south west sector. For example, the 1 in 100 years swell wave height in the 226° to 255° sector is 60% greater than the corresponding height in the 196° to 225° sector.

By inspection of United Kingdom Meteorological Office frequency data, a wave period of 15 seconds was chosen and used for swell waves throughout the analysis.

3.6.2 Diffraction Analysis

Consideration was given to using a mathematical model to assess the sheltering effects of the Aran Islands. However, a preliminary desk study indicated that the use of such a model was not required and it was considered sufficient to evaluate the sheltering effects using a typical desk based diffraction analysis.

Accordingly, a desk study diffraction analysis was carried out for the incident swell waves given in Table 3.1 using the method described in Goda[3.1].

The geography of the Aran Islands was simplified by considering the relevant pair of islands to act as breakwaters, with the relevant sound being considered as an opening through which the swell waves enter and diffract around the heads of the breakwater.

Swell waves have been assumed to approach the sounds at angles of incidence 226° and 196°. This is to ensure that the most onerous residual swell wave height may be identified. Table 3.2 summarises the relevant angles of deviation for Gregory Sound.

Table 3.2: Swell Penetration Angles of Deviation for Gregory Sound

Incident Swell Wave Direction	Bearing of Gregory Sound from Cashla Bay	Deviation
196^o	192^o	4^o
226^o	192^o	34^o

The table shows that waves from the more onerous 226^o direction undergo significantly more deviation than the less onerous waves from the 196^o direction.

Table 3.3 summarises the relevant angles of deviation for Foul Sound.

Table 3.3: Swell Penetration Angles of Deviation for Foul Sound

Incident Swell Wave Direction	Bearing of Foul Sound from Cashla Bay	Deviation
196^o	176^o	20^o
226^o	176^o	50^o

The table shows that waves penetrating through Foul Sound would need to undergo significantly more deviation to reach the Cashla Point area than those through Gregory Sound and accordingly they are not considered further.

Residual swell wave heights have been translated from Gregory Sound to a representative point off Cashla Point, at the entrance to Outer Cashla Bay.

3.6.3 Swell Penetration Through Gregory Sound

Table 3.4 summarises residual swell wave heights off Cashla Point as derived from the 1 in 1 year and the 1 in 100 years incident waves, originating from the more onerous 226° to 255° sector, penetrating Gregory Sound from the 226° direction.

Table 3.4: Residual H_s for Swell from 226^o Penetrating Gregory Sound

Return Period (Years)	Incident Swell Wave Height	Residual Swell Wave Height at Cashla Point
1	5.6	0.9
100	8.8	1.4

The table shows the significant reduction in wave height which occurs after the swell passes through Gregory Sound.

Table 3.5 summarises the residual swell wave heights off Cashla Point resulting from the 1 in 1 year and the 1 in 100 years incident waves, originating from the 196° to 225° sector, penetrating Gregory Sound from the 196° direction.

Table 3.5: Residual H_s for Swell from 196° Penetrating Gregory Sound

Return Period (Years)	Incident Swell Wave Height	Residual Swell Wave Height at Cashla Point
1	3.4	1.4
100	5.8	2.3

The table shows that although the swell waves from the 196° to 225° are significantly smaller than those from the 226° to 255° sector, they result in higher residual wave heights off Cashla Point. This is because their angle of deviation towards Outer Cashla Bay is significantly smaller than that associated with the more onerous 226° to 255° sector.

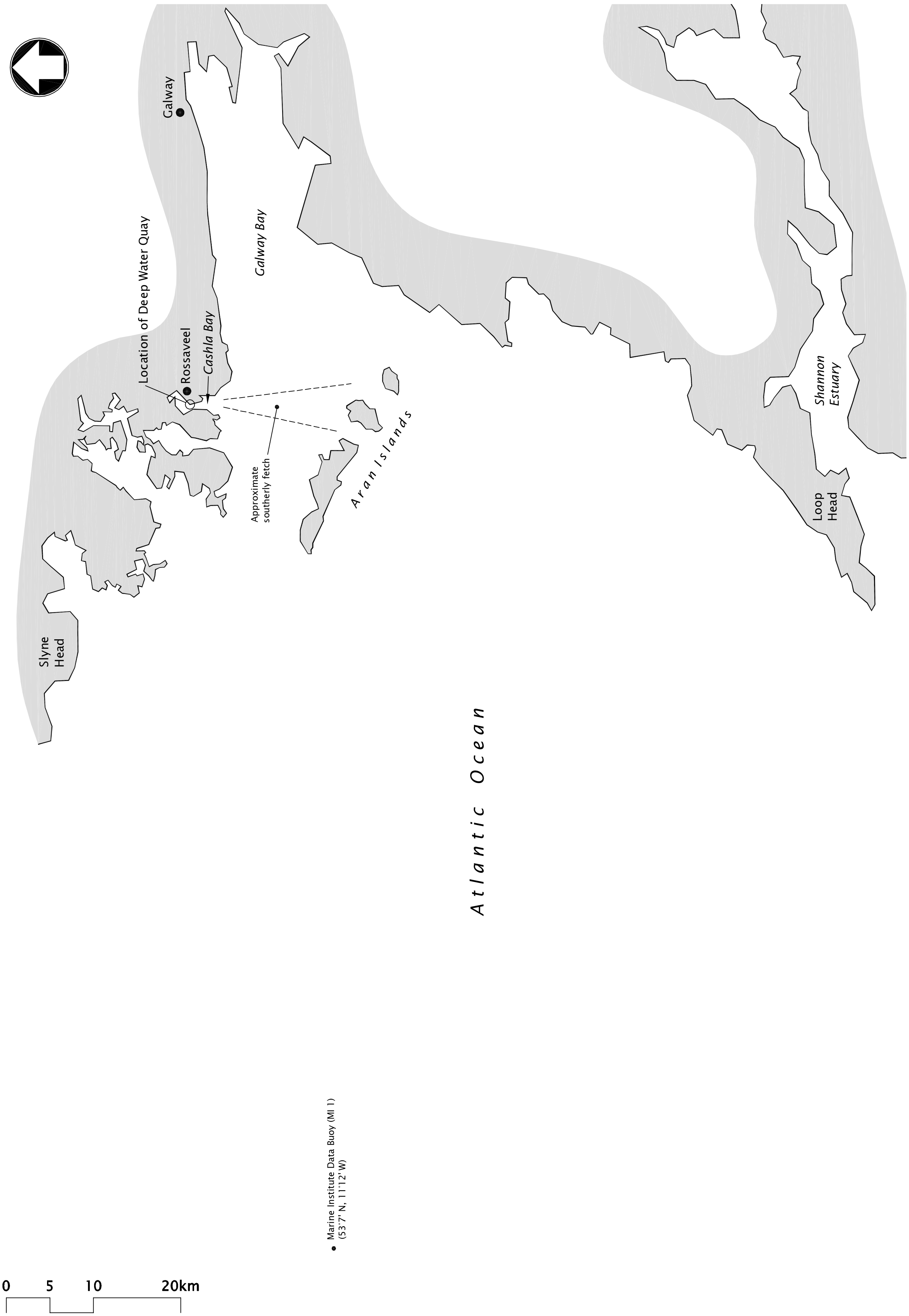
3.7 Summary

This chapter has presented an assessment of the shelter likely to be provided by the Aran Islands. The chapter concludes that in practical terms the islands may be considered to provide full shelter against wind waves, but consideration should be given to swell penetration through Gregory and Foul Sounds.

A desk study based diffraction analysis was carried out to investigate swell penetration through Gregory and Foul Sounds and the residual wave heights summarised in Table 3.5 have been selected as input data to the relevant MIKE 21 model runs described in Chapter 5.

3.8 References

(3.1) Goda Y., *"Random Seas and Design of Maritime Structures"*, Advanced Series on Ocean Engineering – Volume 15, Chapter 3, World Scientific Publishing, 2000.



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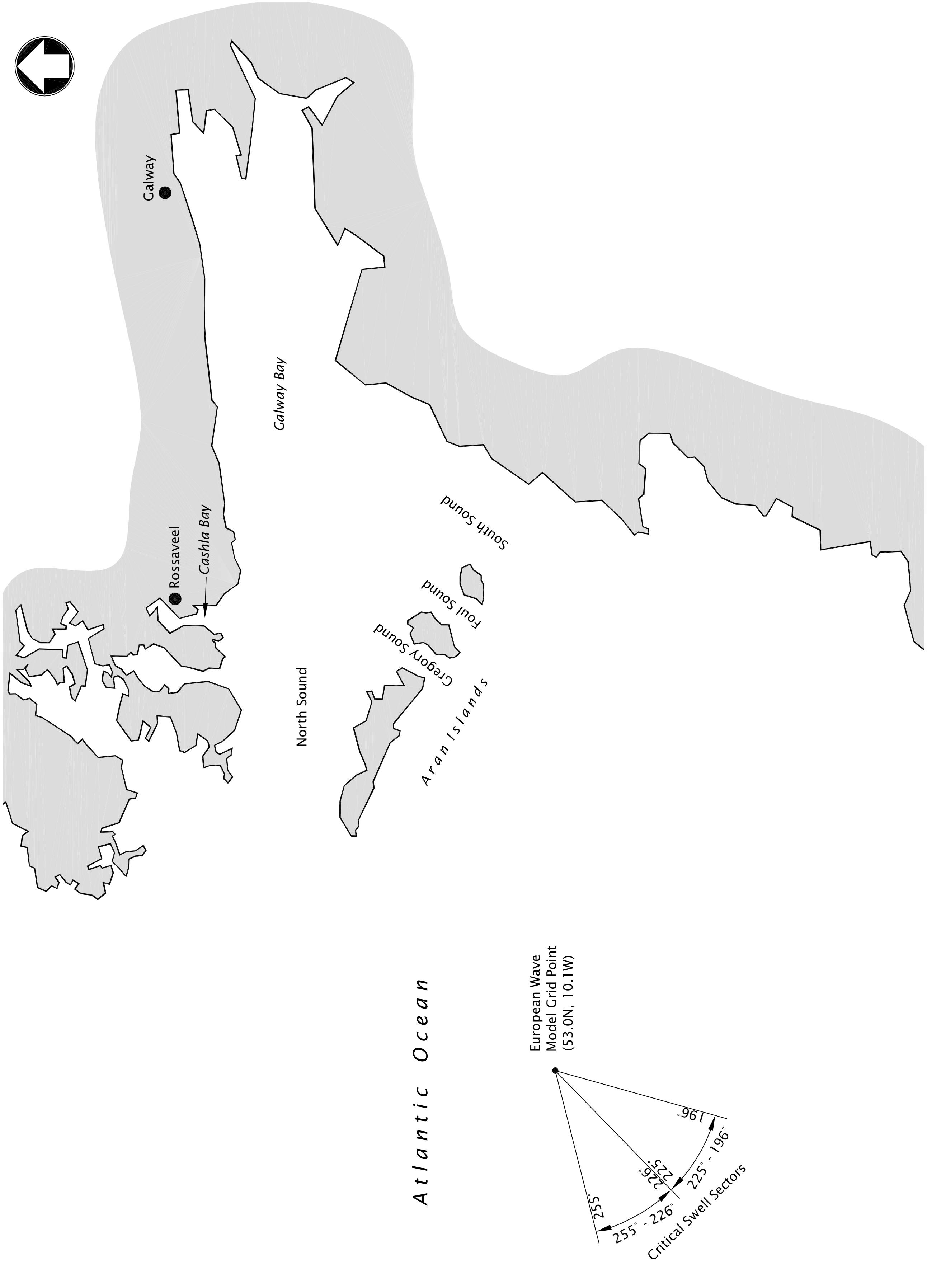
Location Plan
Figure 3.1



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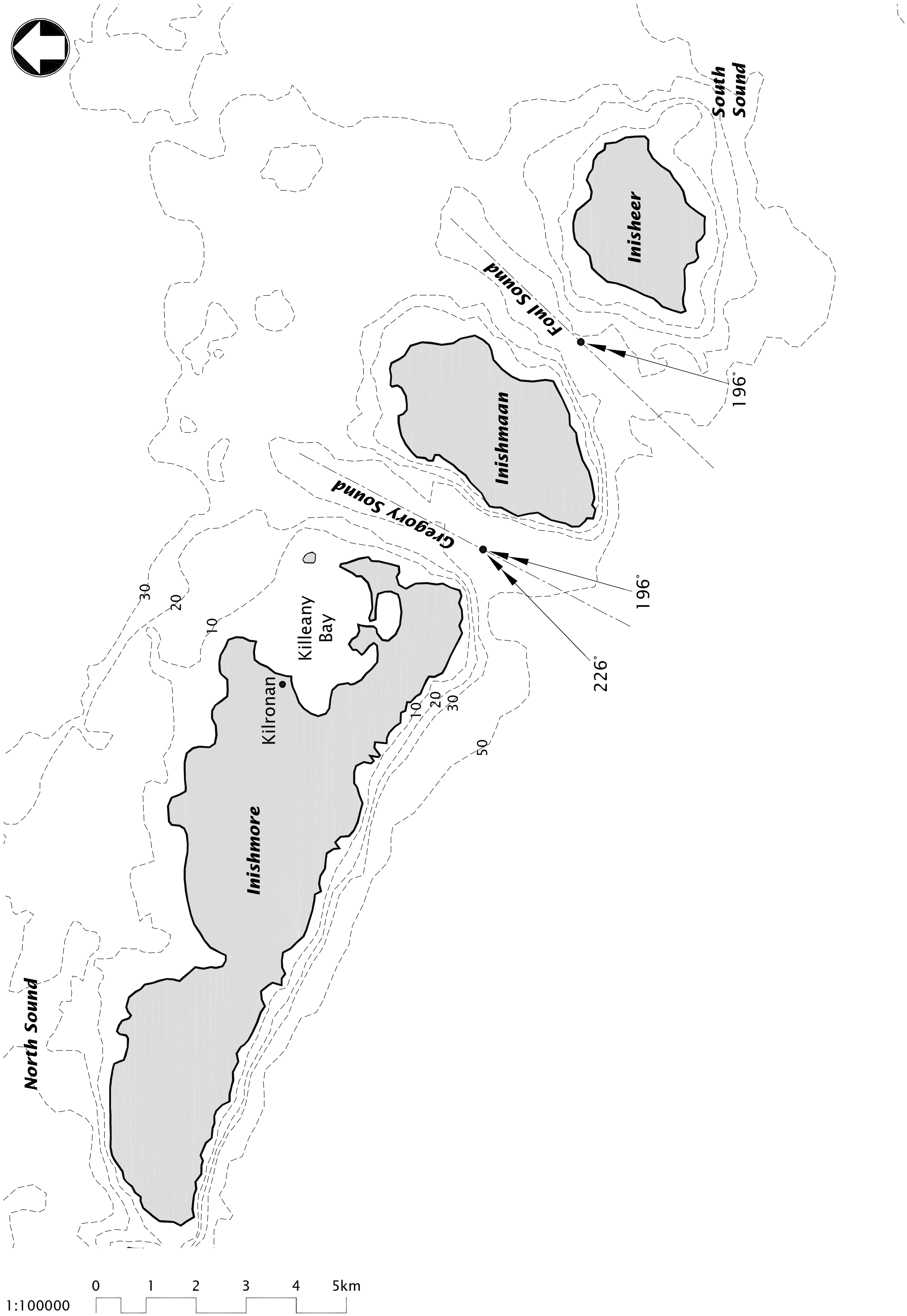
Location of European Wave Model
Swell Data Grid Point
Figure 3.2



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Orientation of Gregory and Foul Sounds
Figure 3.3

4 Wind Wave Generation in North Sound

4.1 Introduction

This chapter summarises the process of developing wind generated, extreme wave heights in the North Sound – the sea area between the north shores of the Aran Islands and the entrance to Outer Cashla Bay. The wave heights are primarily required as input data for the MIKE 21 model runs in order to assess the suitability or otherwise of the residual wave climate at the deep water quay.

In the absence of immediately available wave height measurements in this sea area, wave heights have been estimated using a hindcasting technique based on extreme winds speeds generated from the nearest European Wave Model grid point.

4.2 Selection of Wind Directional Sector

Chapter 3 indicated that the deep water quay is reasonably well sheltered and is exposed only to the North Sound over a relatively narrow sector extending between 170^{o} and 190^{o}. In order to obtain a reasonable extreme wind climate for the hindcasting analysis, the directional sector of 120° to 225° was chosen as that encompassing the potentially most critical winds.

4.3 Extreme Wind Speeds Data

Wind data was then obtained from the United Kingdom Meteorological Office from the nearest European Wave Model grid point, located at 53.3° North, 9.7° West, (Figure 4.1), for the period January 1990 to September 2001 inclusive. The data included monthly and annual 3 hourly wind speeds and a directional frequency distribution.

The Meteorological Office used the above data to produce an extreme value analysis for the selected directional sector, the results of this are presented in Table 4.1.

Table 4.1: Extreme Wind Speeds for Sector 120° to 225°

Return Period (years)	Wind Speed (m/s)
1	25
5	28
10	28
20	29
50	30
100	31
200	32

Source: United Kingdom Meteorological Office

4.4 Wind Wave Hindcasting

Hindcast techniques were used to derive the wind generated wave climate within the North Sound, using the extreme wind speeds summarised in Table 4.1.

Within the selected sector, several fetch rays were extended out from an approximately central point in the entrance to Outer Cashla Bay into the North Sound and Galway Bay. It was considered that with the highest winds occurring from the south west, the two most representative rays were likley to be those lying between Outer Cashla Bay and Inishmore (225°), and between Outer Cashla Bay and Inishmaan (182°).

Using the significant wave height prediction chart given in BS6349[4.1], wave heights for both one year and 100 year return periods were calculated at the entrance to Outer Cashla Bay. The results of the analysis are presented in Table 4.2.

Table 4.2: Wind Wave Heights at Entrance to Outer Cashla Bay

Return Period (Years)	Significant Wave Height (m)	Approximate Significant Period (s)
1	2.2	6
100	2.8	6

These results were verified using the wind and wave growth chart given in the *Shore Protection Manual*[4.2], from which similar results were obtained.

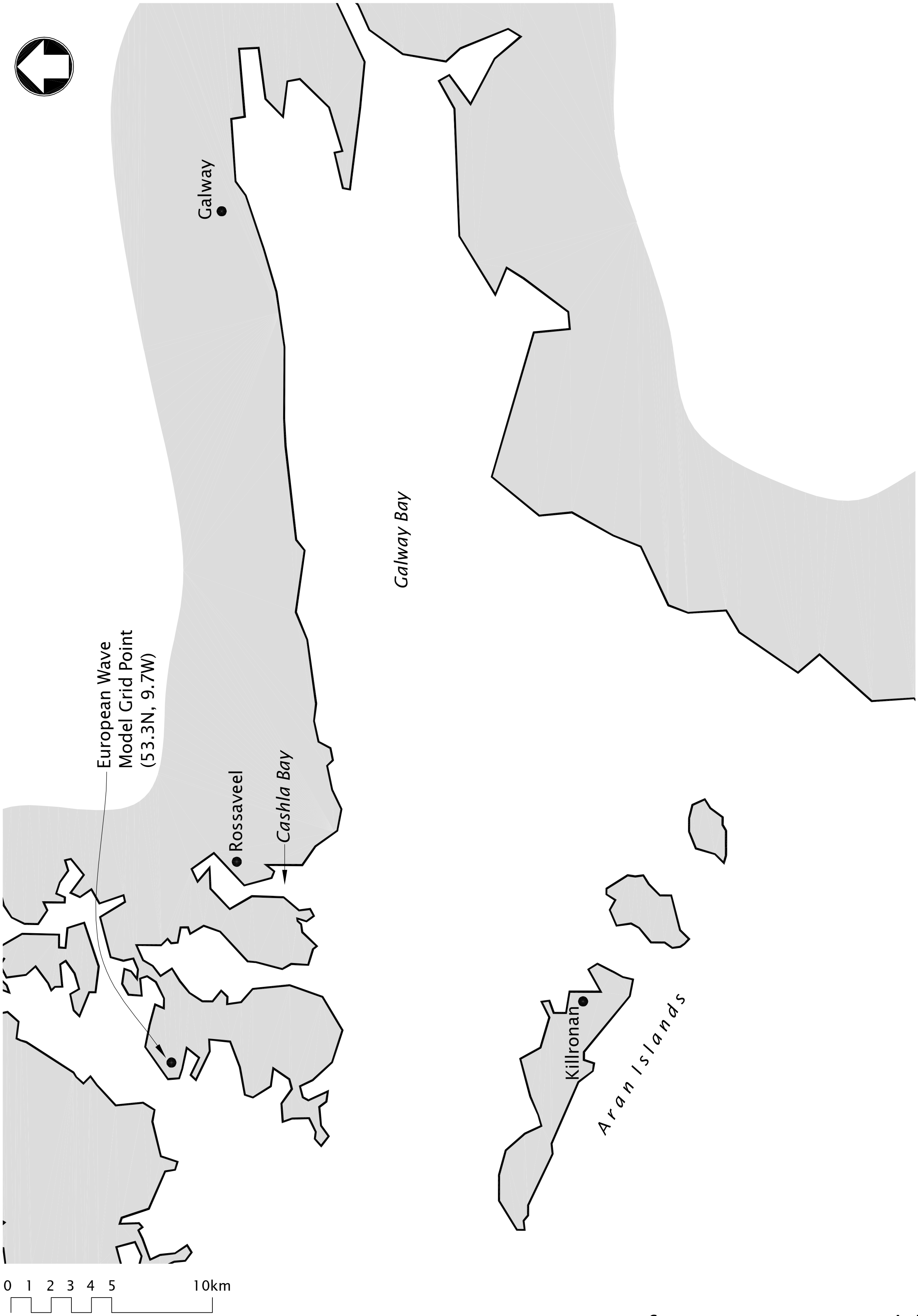
4.5 Summary

This chapter has summarised the process of estimating wave heights arising from wind generation in the North Sound. The relevant wave heights are summarised in Table 4.2 and form the basis of the wind generated wave condition inputs into the MIKE 21 model runs described in Chapter 5.

4.6 References

(4.1) BS 6349 - 1:2000 British Standard *"Maritime Structures – Part 1: Code of practice for general criteria"*.

(4.2) Department of the Army, *"Shore Protection Manual"*, Waterways Experiment Station, Corps of Engineers, Coastal Engineering Research Centre, Volume 1, Chapter 2, US Government Printing Office, Fourth Edition, 1984.



Location of European Wave Model
Wind Data Grid Point
Figure 4.1

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063107\drawings\figures\Vol2\Fig 4-1
Project: 200046

5 Development of a MIKE 21 Wave Model for Rossaveel

5.1 Introduction

This chapter describes the development of a MIKE 21 wave model for translating wind and swell waves from the North Sound into Inner Cashla Bay. The primary objective of the modelling is to assess whether residual wave conditions at the deep water quay are within acceptable limits. Limits are commonly specified for the types of vessels which will be moored alongside a quay and the cargo handling or other operations associated with these vessels.

5.2 MIKE 21 EMS Wave Disturbance Modelling

The approach adopted for this project is based on the use of the MIKE 21 EMS module. The **E**lliptic **M**ild-**S**lope module MIKE 21 (EMS) is part of the MIKE 21 wave modelling software suite developed by the Danish Hydraulics Institute. The full suite includes several modules relating to the hydraulic modelling of lakes, bays, coastal areas and seas where stratification may be neglected. The MIKE 21 EMS module was used to study wave disturbance for the various deep water quay options.

The EMS module simulates the propagation of linear time harmonic water waves on a gently sloping bathymetry with arbitrary water depth. EMS is based on the numerical solution of the Elliptic Mild-Slope equation and is capable of reproducing the combined effects of shoaling, refraction, diffraction and back-scattering.

5.3 Development of the Model

5.3.1 Overview

The model parameters in EMS include physical input data (bathymetry, wave information and surface elevation), as well as parameters controlling the dissipation processes (partial reflection from structures, wave breaking and bottom friction).

5.3.2 Bathymetry

Bathymetry was generated from the contours on the United Kingdom Admiralty Chart 2096 *Cashla Bay to Kilkieran Bay* and bathymetric survey drawings prepared by Hydrographic Surveys Ltd[5.1].

Chart 2096 was used for Outer Cashla Bay and the Hydrographic Surveys drawings were used for Inner Cashla Bay.

5.3.3 Grid Spacing

An orthogonal grid of spacing 5m was used, which is considered sufficiently fine both to resolve the wave lengths anticipated and to give an accurate account of wave conditions at the target area.

5.3.4 Reflection Coefficients

The coastline within Cashla Bay is rocky and is likely to be significantly reflective. The structures within the inner harbour bay range from vertical faced quays to sloping rock armoured revetments. To simulate the reflective nature of such structures, porous layers were specified according to structure type. Table 5.1 summarises the reflection coefficients that were considered for each type of structure and the coastline. In the model, friction coefficients of a specified thickness were used with a porosity selected to reproduce the required reflection characteristics of the structure. Reflection coefficients for the structures were estimated using guidelines in Thompson *et al*[5.2].

Table 5.1: Reflection Coefficients

Type of Structure	Reflection Coefficient
Coastline	0.8
Vertical caisson walls	1.0
Sloping rock armour	0.4

Source: Thompson *et al*

5.3.5 Sponge Layers

So called sponge layers are introduced into MIKE 21 to absorb wave energy. They are required for a number of reasons, most significantly to prevent any wave energy reflecting back from open sea boundaries into the model area, which may then interfere with results.

They are also often applied to geographical areas which can be neglected from the model, such as inlets or bays, where no wave analysis is required. This has the effect of simplifying the model and significantly reducing processing time. For the Rossaveel model, a sponge layer has been introduced across the entrance to the tidal inlet which extends north towards Rossaveel Lough from Tonacrick Point. This is in addition to open boundary sponge layers at the north and south model boundaries.

5.3.6 Bed Friction

The effects of bed friction on the wave climate were assessed and the comparative results did not show significant sensitivity to bed friction. Therefore, the effects of bed friction within the model were ignored.

5.3.7 Wave Breaking

The effects of wave breaking were included within the model.

5.4 Overall Modelling Approach

5.4.1 Overview

The modelling has been carried out in two stages. The first investigated residual wave conditions within the existing harbour, prior to the introduction of alternative deep water quay layouts, for a selection of wind and swell wave inputs. The primary focus of attention was the residual wave conditions in Inner Cashla Bay, in particular at the site of the proposed deep water quay berthing line.

The second stage of the modelling investigated residual wave conditions at the deep water quay for the two alternative quay layouts considered.

5.4.2 Model Runs to Assess Existing Harbour

The model was initially set up to establish the wave conditions at the entrance to Inner Cashla Bay (the Harbour) for the various swell and wind wave inputs at the entrance to Outer Cashla Bay, described in Chapters 3 and 4. The runs were carried out to provide information on the resulting wave conditions at the entrance to the harbour from waves entering the Outer Cashla Bay and propagating across the Outer Bay towards the entrance to the Inner Bay (bathymetry shown in Figure 5.1).

Subsequent runs were focussed on more critical combinations of wave heights and still water levels. These runs encompassed Outer and Inner Cashla Bay (bathymetry shown in Figure 5.2, with increased level of detail).

5.4.3 Model Runs to Assess Deep Water Quay Options

Runs were then carried out to determine the residual wave heights at the location of the proposed deep water quay, under the critical wave conditions previously established. Model runs were carried out for Deep Water Quay Option DWQ 5, for both ‘South Causeway’ and ‘North Causeway with Breakwater’ configurations.

5.5 Existing Harbour

Table 5.2 summarises the principal wave height and still water level combinations adopted for the wave modelling analyses.

Table 5.2: Initial Model Runs for Existing Harbour Assessment

Input Wave Condition	Wave Ht H_s (m)	Period T_z (s)	Still Water Level (m)		Figure No.
1 in 100 years swell wave through Gregory Sound	2.3	15	+5.2	MHWS	Figure 5.3
1 in 100 years wind wave from North Sound	2.8	6			Figure 5.4
1 in 1 year swell wave through Gregory Sound	1.4	15	+6.6	1 in 100 years	Figure 5.5
1 in 1 year wind wave from North Sound	2.2	6			Figure 5.6
1 in 1 year swell wave through Gregory Sound	1.4	15	+4.7	MHW	Figure 5.7
1 in 1 year wind wave from North Sound	2.2	6			Figure 5.8

From the results of these initial runs, combinations of wave heights and still water levels which produced the most adverse conditions at the entrance to Inner Cashla Bay were chosen. These are shown in the shaded areas of the table, and represent the input data for the next stage of the wave modelling study.

5.6 Addition of Deep Water Quay

The model runs were then repeated for the wave inputs shown in Table 5.2 but this time with the addition of the deep water quay options summarised below:

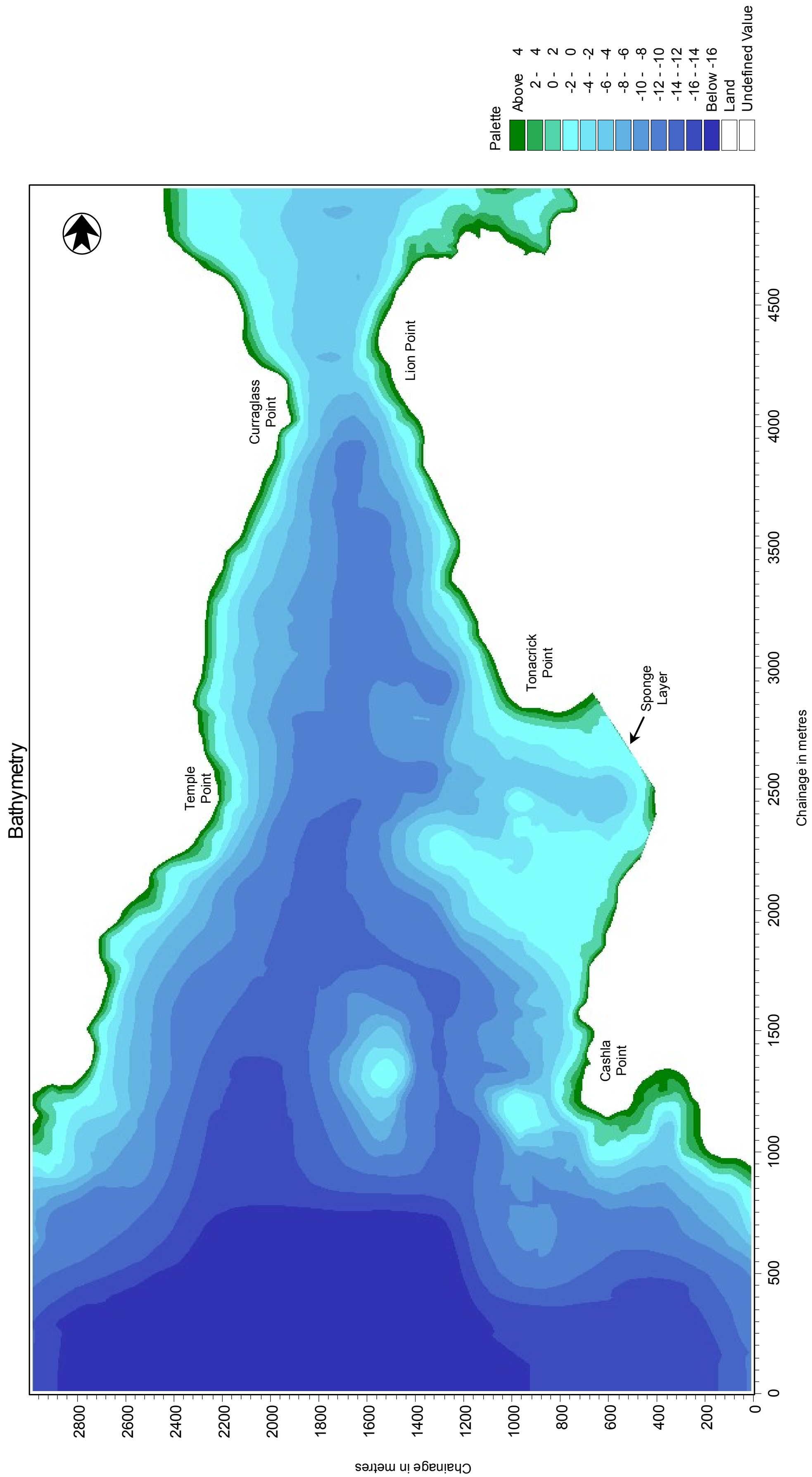
(a) Deep Water Quay Option DWQ5 with a South Approach Causeway Configuration

(b) Deep Water Quay Option DWQ5 with a North Approach Causeway Configuration and a breakwater to protect the inner basin.

5.7 References

(5.1) Hydrographic Surveys Ltd drawings HS:21/01(16/02/01) and HS:29/01(21-23/03/01 and 17-20/02/01).

(5.2) Thompson, E. F., Chen, H. S. and Hadley, L.L., *"Validation of Numerical Model for Wind Waves and Swell in Harbours"*, Journal of Waterway, Port Coastal and Ocean Engineering, Volume 122 no. 5, 1996.



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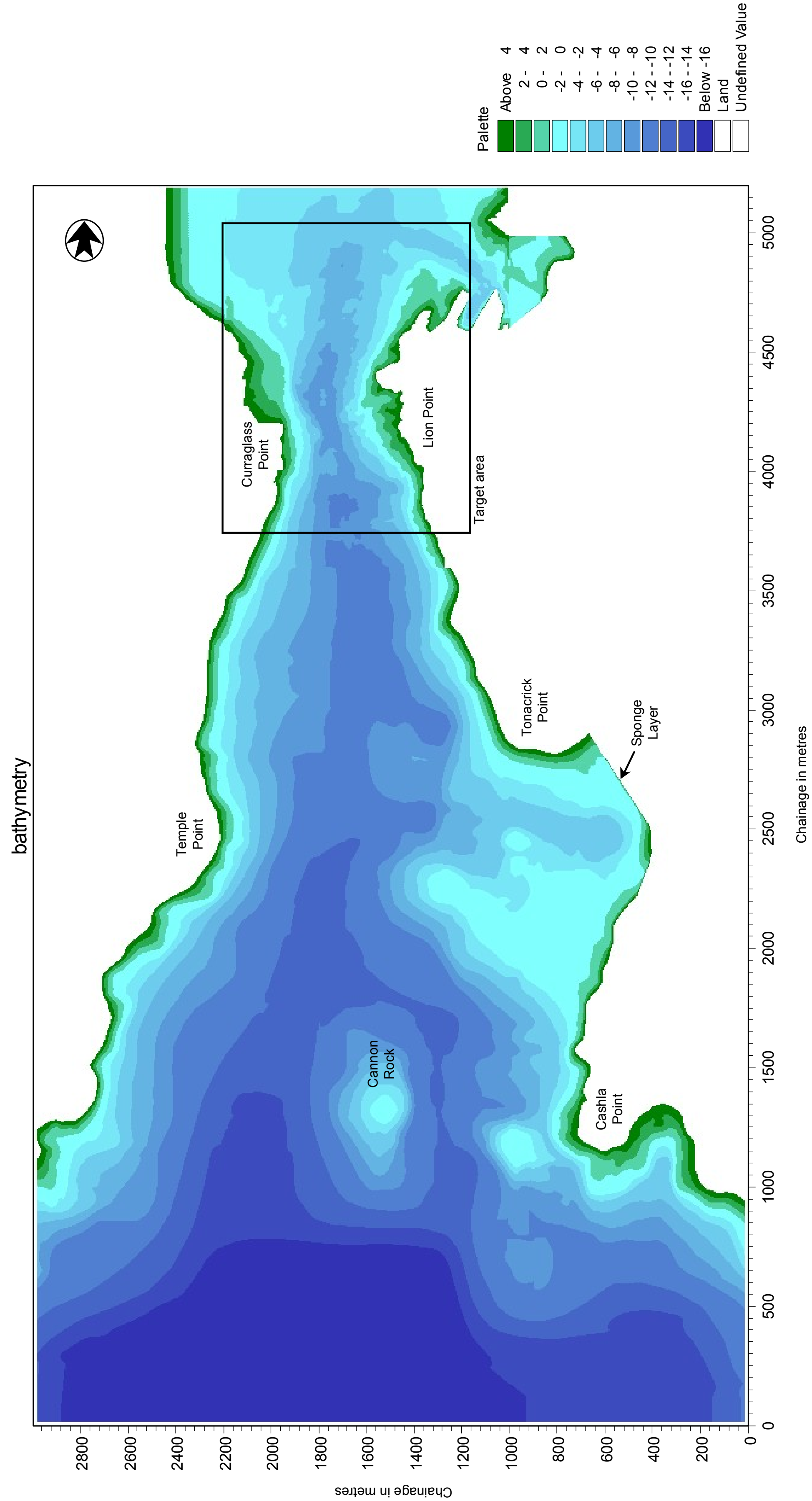
Model Boundaries
Outer Cashla Bay Detail
Figure 5.1



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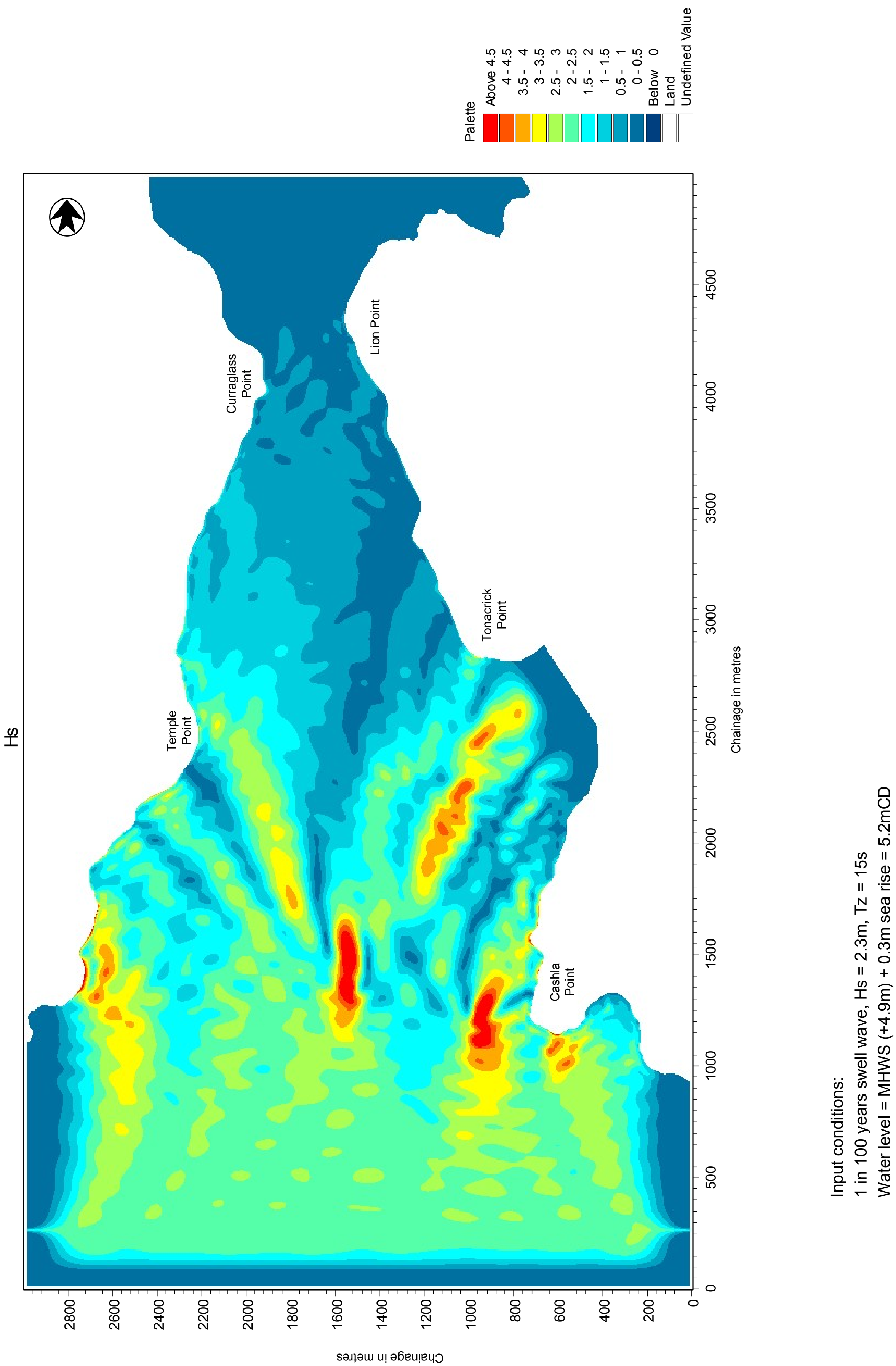
Model Boundaries
Outer and Inner Cashla Bay Detail
Figure 5.2



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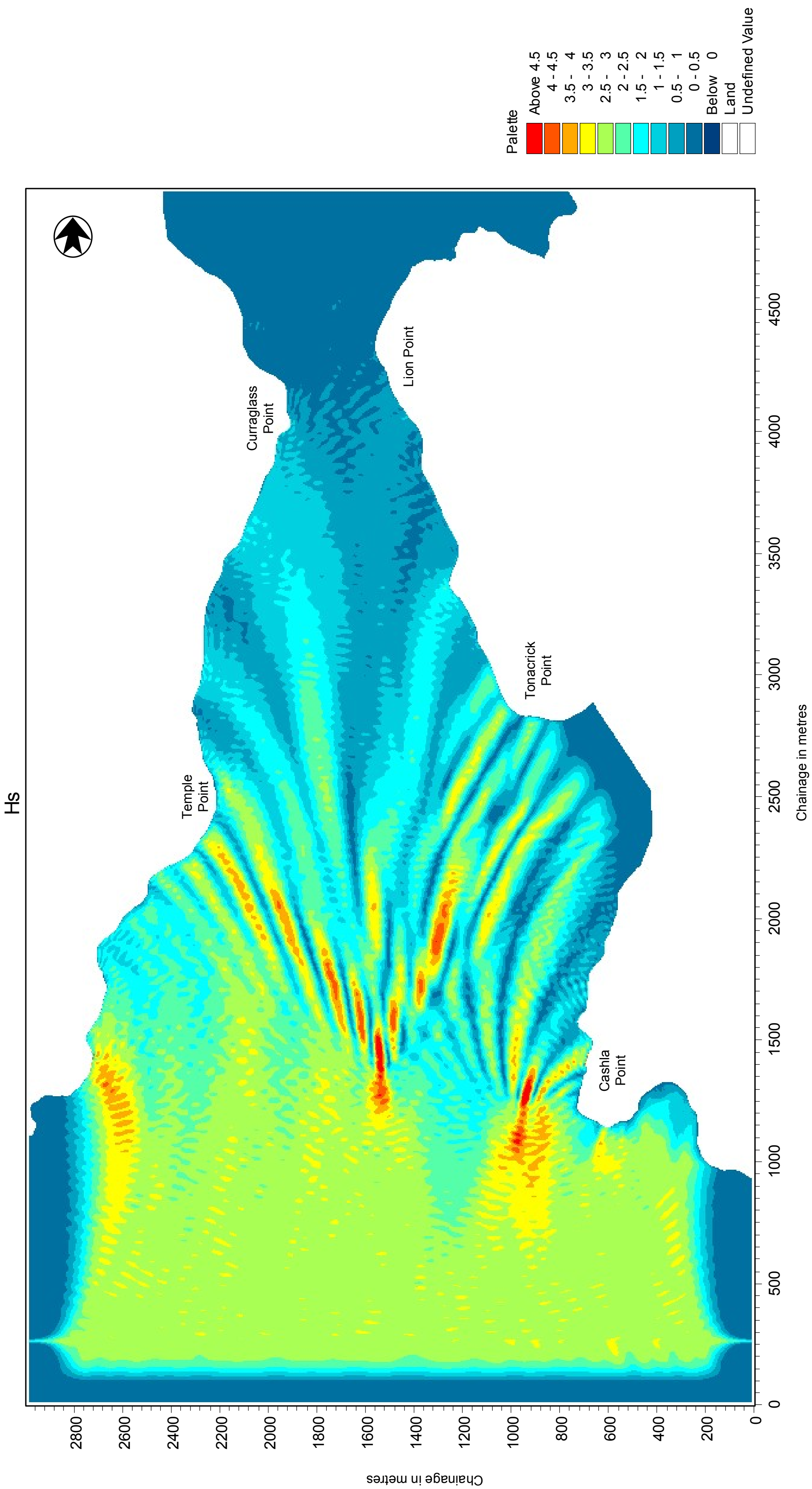
Existing Harbour
1 in 100 Years Swell Wave + MHWS
Figure 5.3



Input conditions:
1 in 100 years wind wave, Hs = 2.8m, Tz = 6s
Water level = MHWS (+4.9m) + 0.3m sea rise = 5.2mCD

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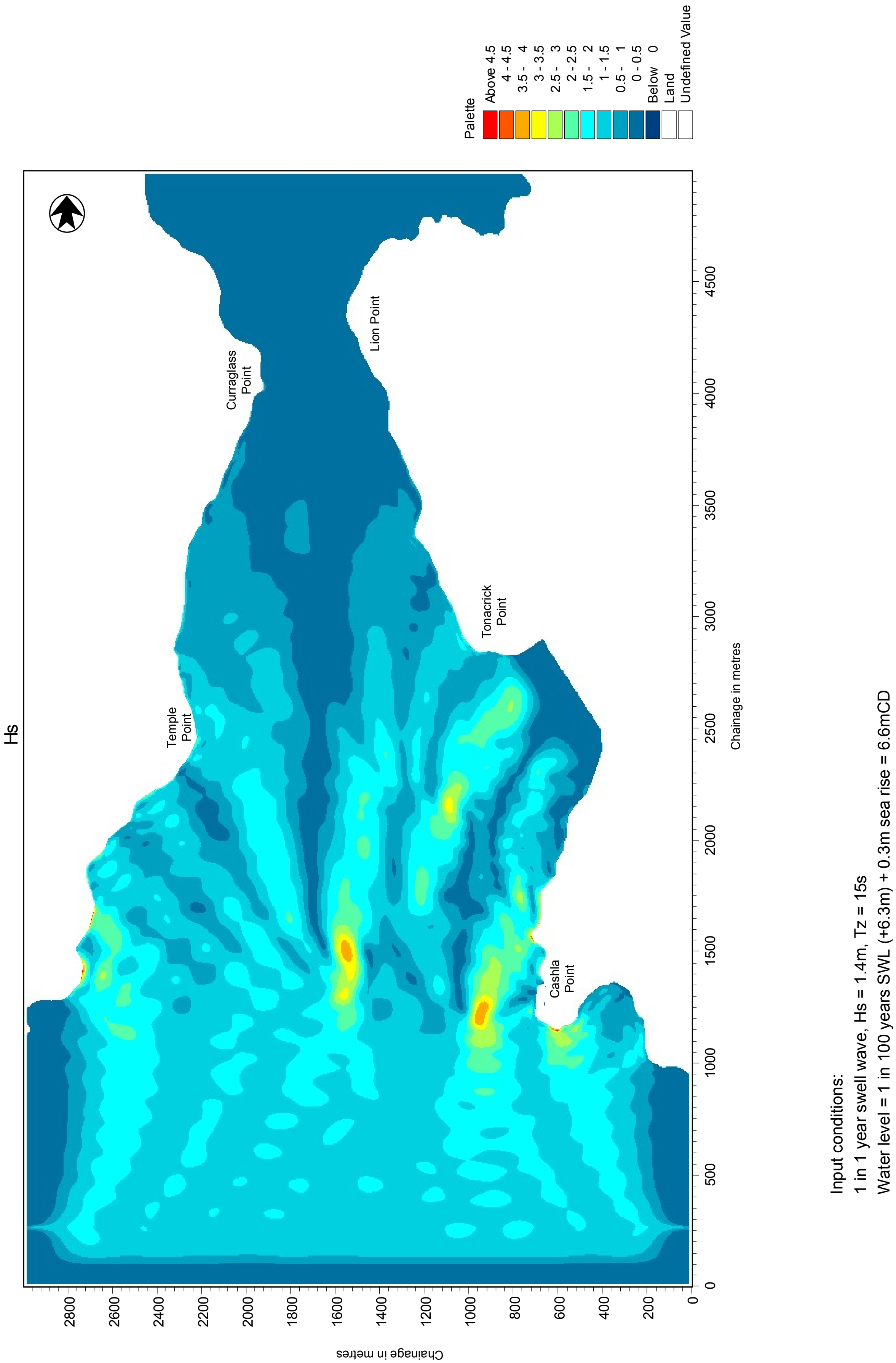
Existing Harbour
1 in 100 Years Wind Wave + MHWS
Figure 5.4



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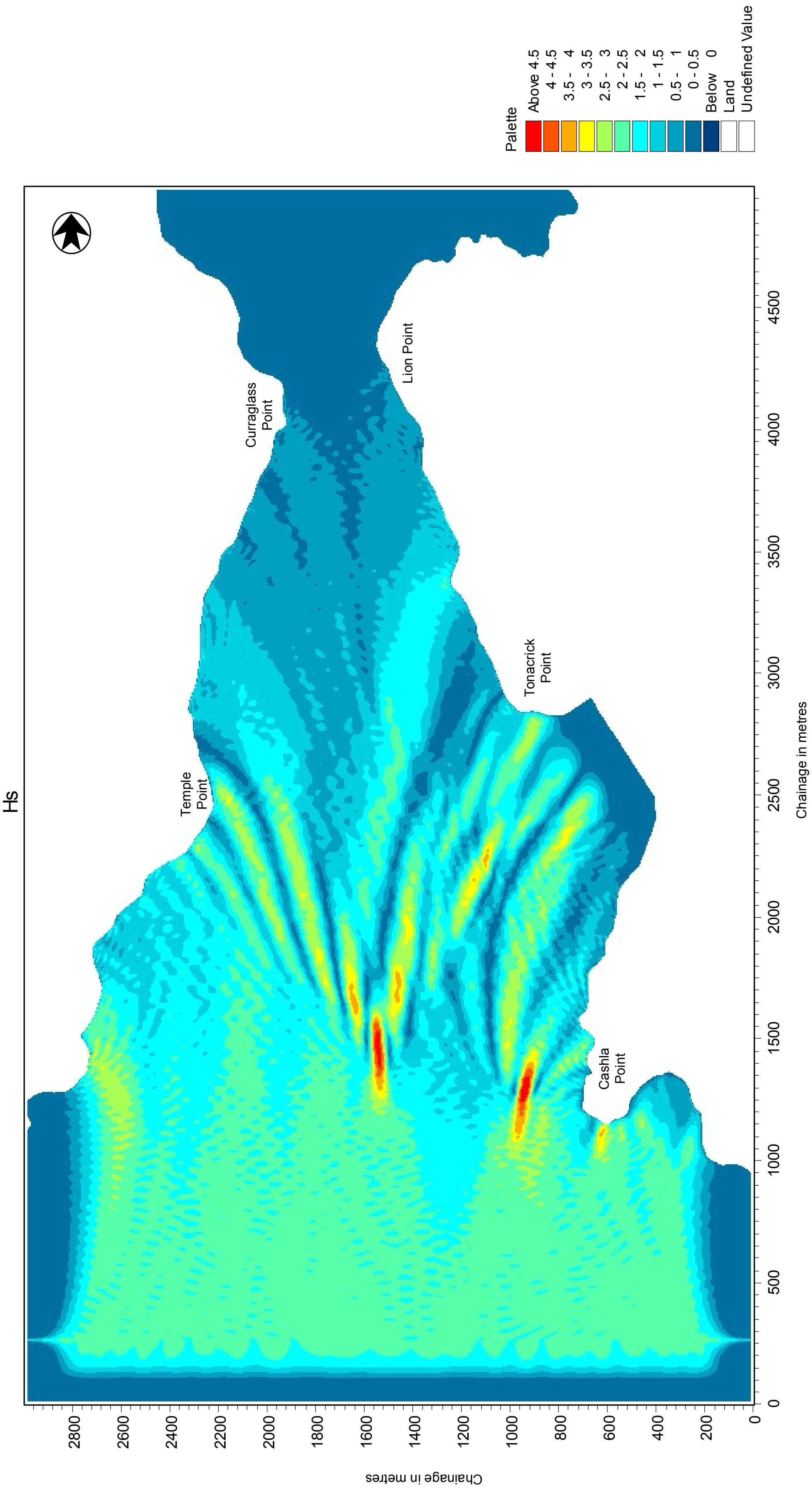
Existing Harbour
1 in 1 Year Swell Wave + 1 in 100 Years SWL
Figure 5.5



Input conditions:
1 in 1 year wind wave, H_s = 2.2m, T_z = 6s
Water level = 1 in 100 years SWL (+6.3m) + 0.3m sea rise = 6.6mCD

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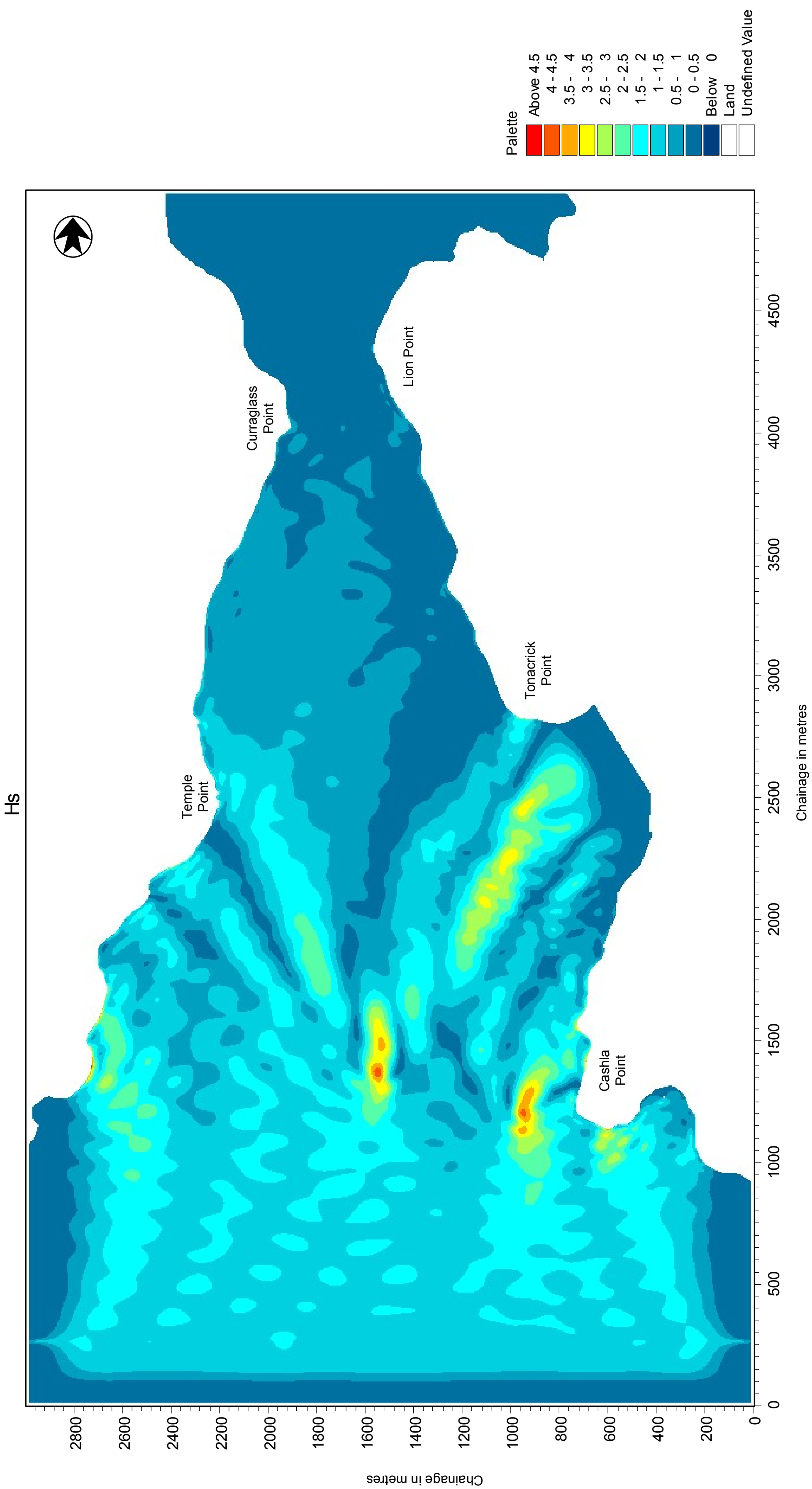
Existing Harbour
1 in 1 Year Wind Wave + 1 in 100 Years SWL
Figure 5.6



Input conditions:
1 in 1 year swell wave, Hs = 1.4m, Tz = 15s
Water level = MHW (+4.4) + 0.3m sea rise = 4.7mCD

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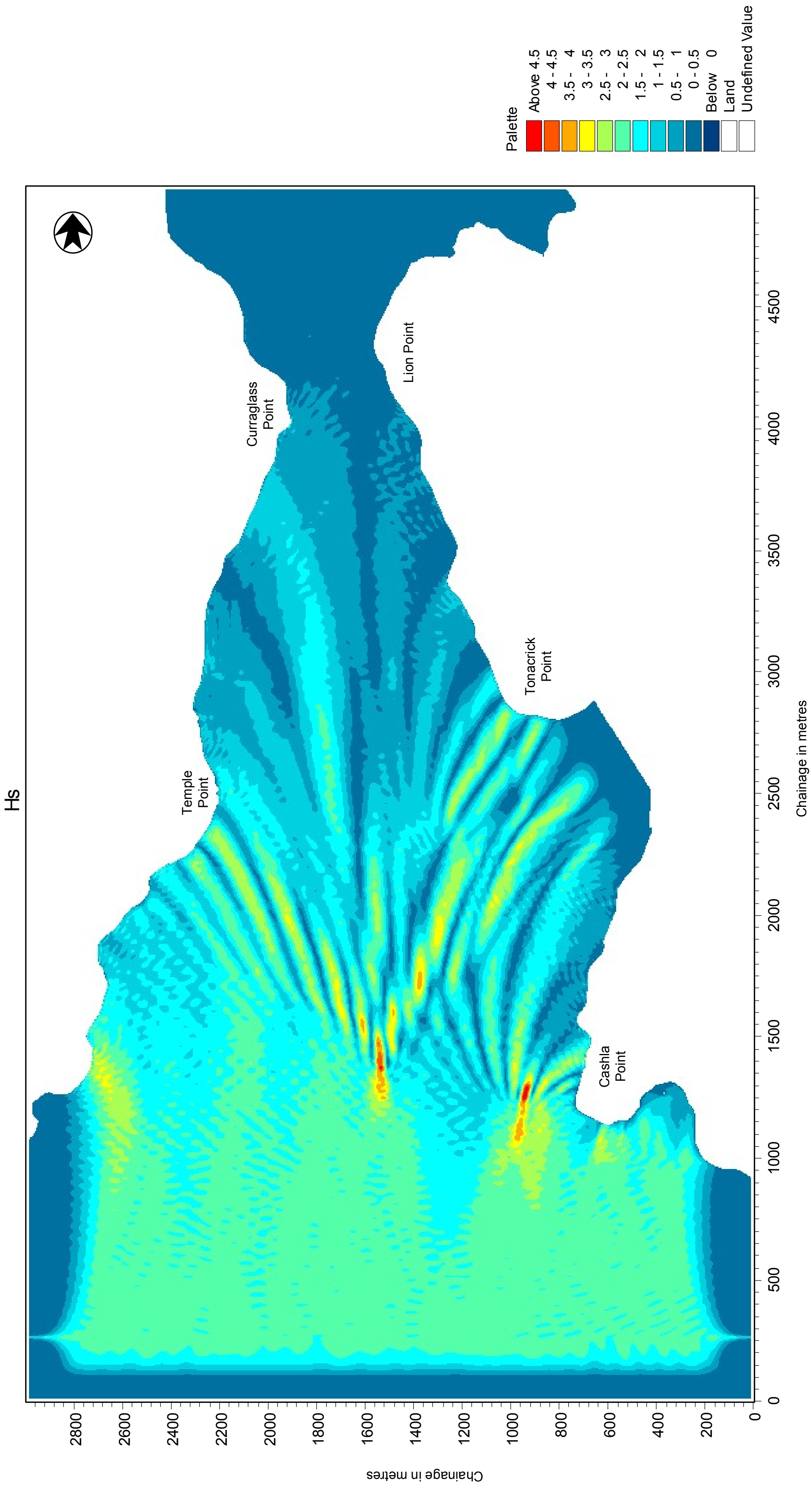
Existing Harbour
1 in 1 Year Swell Wave + MHW
Figure 5.7



Input conditions:
1 in 1 year wind wave, Hs = 2.2m, Tz = 6s
Water level = MHW (+4.4) + 0.3m sea rise = 4.7mCD

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Existing Harbour
1 in 1 Year Wind Wave + MHW
Figure 5.8

6 Summary and Discussion of Results

6.1 Introduction

This chapter summarises the results of the MIKE 21 runs in the context of commonly specified limiting wave conditions for the types of vessels and cargo handling operations for which the deep water quay is to be designed.

6.2 Limiting Wave Conditions for Vessels on the Deep Water Quay

Table 6.1 summarises the limiting wave conditions which have been adopted for assessing whether vessels are able to moor in safety alongside the two berthing faces of the deep water quay.

Table 6.1: Limiting Wave Conditions for Deep Water Quay Vessel Operations

Vessel Type	Limiting Wave Height, H_s (m) (Head Sea)	Applicable Wave Period (s)
Fishing vessels	0.4	Up to 10
General cargo vessels (up to 30,000 DWT)	0.7	Up to 10
Bulk carrier (up to 30,000 DWT)	0.8	Up to 10
Container vessels	0.5	7 to 12
Ro Ro vessels	0.5	7 to 12
Passenger vessels	0.7	Up to 10
Tankers (up to 30,000 DWT)	0.7	Up to 10

Source: Thoresen, C.A

The table shows that limiting wave heights vary between a minimum of 0.4m for fishing vessels to a maximum of 0.8m for a bulk carrier.

6.3 Existing Harbour

Table 6.2 summarises the results of the principal runs carried out to assess wave conditions at the site of the proposed deep water quay prior to its construction.

Table 6.2: Results of Principal Runs for Existing Harbour

Input Wave Condition	Still Water Level Condition	Residual Wave Height at Outside Berthing Line H_s (m)	Figure No.
1 in 100 years swell wave	MHWS	Less than 0.3	Figure 6.1
1 in 100 years wind wave	MHWS	0.3	Figure 6.2
1 in 1 year swell wave	MHW	Less than 0.3	Figure 6.3
1 in 1 year wind wave	MHW	0.3	Figure 6.4

The results summarised in Table 6.2 show that all wave heights are within the limiting values given in Table 6.1 for all types of vessels.

6.4 Option DWQ5 with South Causeway Configuration

Table 6.3 summarises the results of the principal runs carried out to assess wave conditions at the outside berthing line of the proposed deep water quay prior for layout option DWQ5 with South Causeway.

Table 6.3: Results of Principal Runs for Option DWQ5, South Causeway

Input Wave Condition	Still Water Level	Residual Wave Height at Outside Berthing Line H_s (m)	Figure No.
1 in 100 years swell wave	MHWS	Less than 0.3	Figure 6.5
1 in 100 years wind wave	MHWS	Less than 0.3	Figure 6.6
1 in 1 year swell wave	MHW	Less than 0.3	Figure 6.7
1 in 1 year wind wave	MHW	Less than 0.3	Figure 6.8

The results summarised in Table 6.3 show that all wave heights are within the limiting values given in Table 6.1 for all types of vessels.

6.5 Option DWQ5 with North Causeway and Breakwater

Table 6.4 summarises the results of the principal runs carried out to assess wave conditions at the outside berthing line of the proposed deep water quay prior for layout option DWQ5 with North Causeway and Breakwater.

Table 6.4: Results of Runs for Option DWQ5, North Causeway and Breakwater

Input Wave Condition	Still Water Level	Residual Wave Height at Outside Berthing Line H_s (m)	Figure No.
1 in 100 years swell wave	MHWS	Less than 0.3	Figure 6.9
1 in 100 years wind wave	MHWS	Less than 0.3	Figure 6.10

The results summarised in Table 6.4 show that all wave heights are within the limiting values given in Table 6.1 for all types of vessels.

6.6 Summary

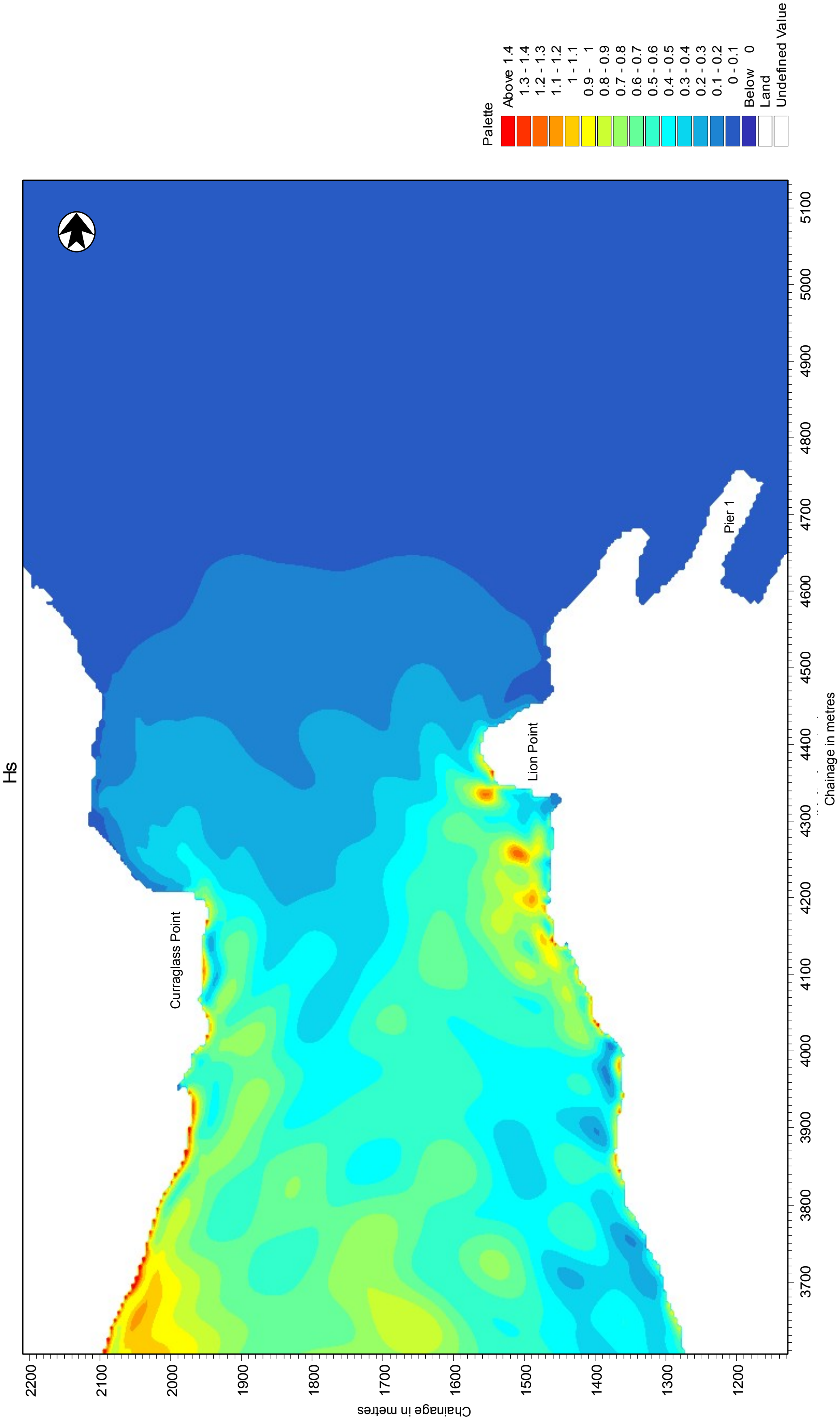
Wave conditions are only one of several which require consideration in assessing potential down time for the deep water quay. Wind, mist, fog or channel blockages may all contribute to berth down time during a year.

However, notwithstanding the above, the results presented in Tables 6.2 to 6.4 indicate that wave conditions at the outside berthing line are expected to be well within the limits specified in Table 6.1. Accordingly, it would be expected that the operational availability of the deep water quay is potentially high – as far as waves are concerned.

More detailed wave modelling and related studies are required at the detailed design stage to address issues such as wave overtopping of the quay and any wave reflection effects on passing vessels such as the Aran Islands ferries. These are not expected to be significant issues.

6.7 References

(6.1) Thoresen, C. A., *"Port Design, Guidelines and Recommendations"*, Tapir Publishers, 1988.



Input conditions:
1 in 100 years swell wave, Hs = 2.3m, Tz = 15s
Water level = MHWS (+4.9m) + 0.3m sea rise = 5.2mCD

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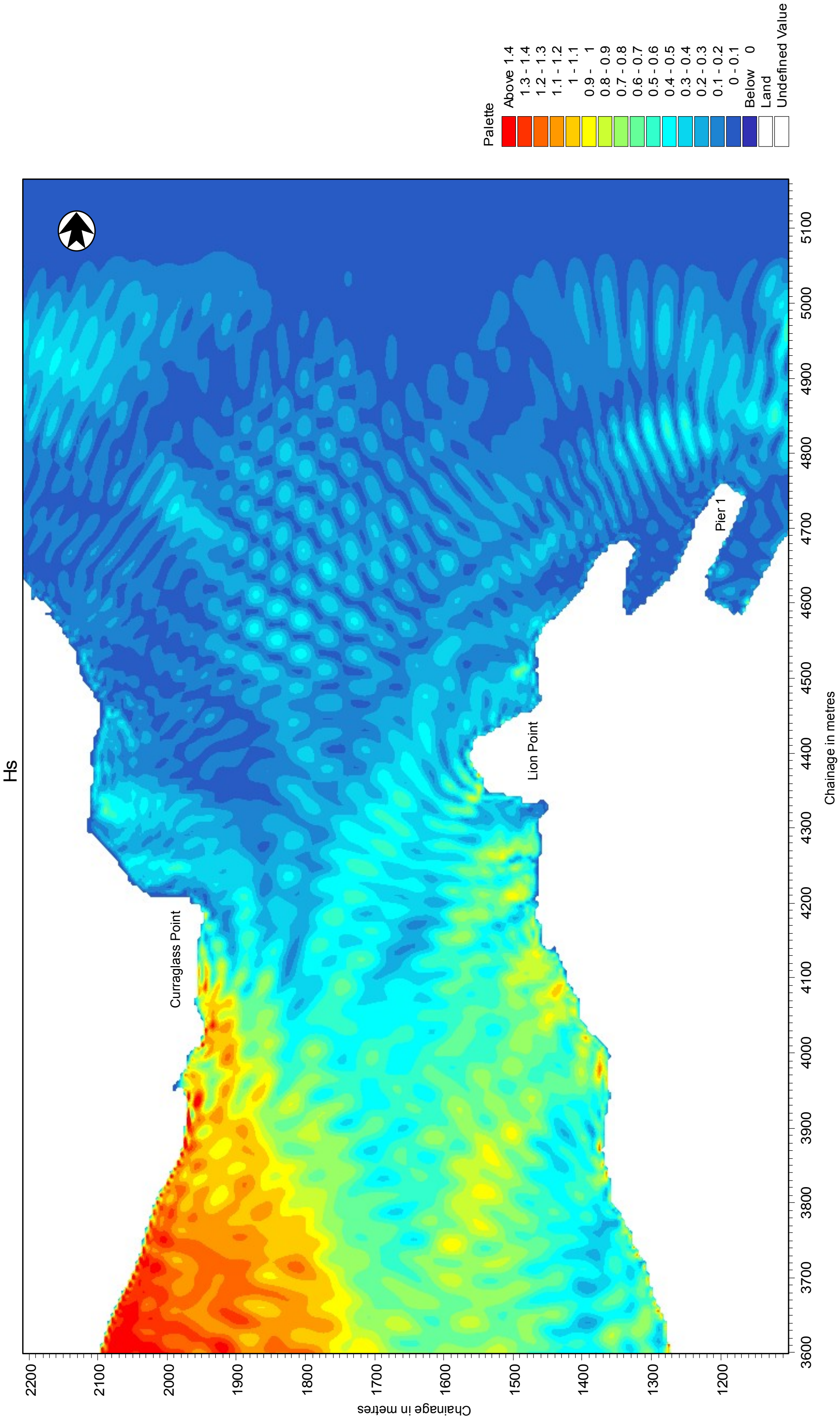
Existing Harbour
1 in 100 Years Swell Wave + MHWS
Figure 6.1



Input conditions:
1 in 100 years wind wave, Hs = 2.8m, Tz = 6s
Water level = MHWS (+4.9m) + 0.3m sea rise = 5.2mCD

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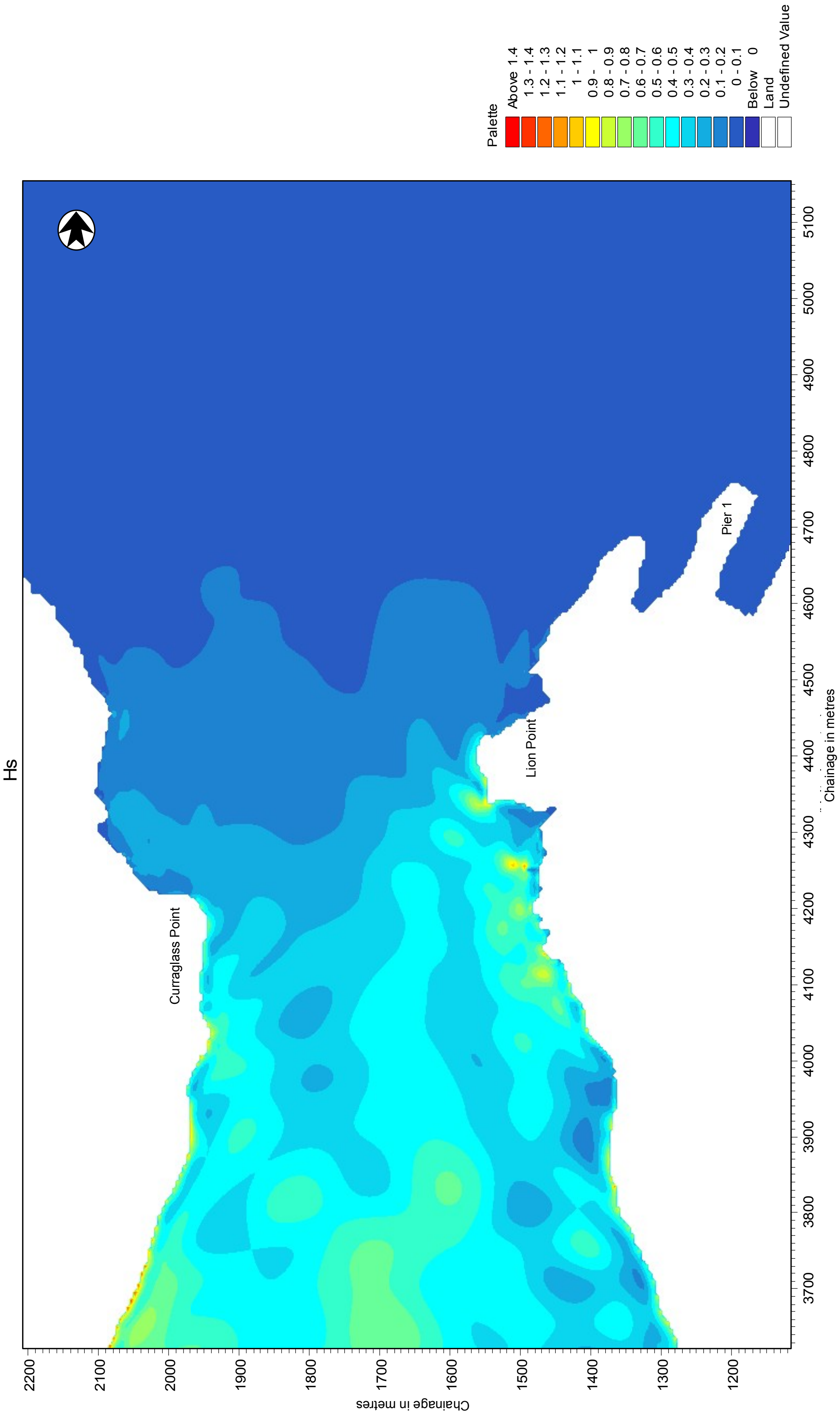
Existing Harbour
1 in 100 Years Wind Wave + MHWS
Figure 6.2



Input conditions:
1 in 1 year swell wave, Hs = 1.4m, Tz = 15s
Water level = MHW (+4.4) + 0.3m sea rise = 4.7mCD

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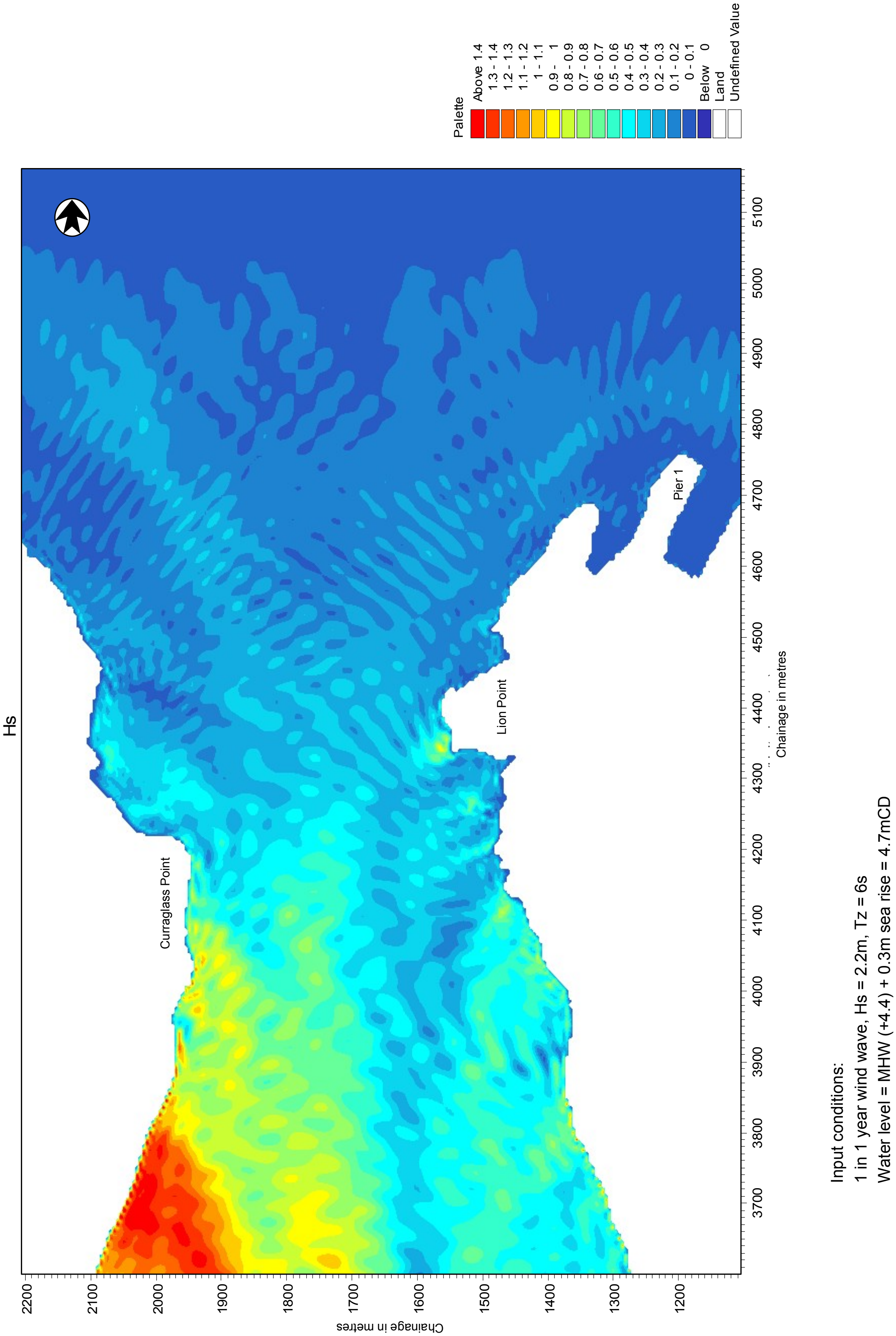
Existing Harbour
1 in 1 Year Swell Wave + MHW
Figure 6.3



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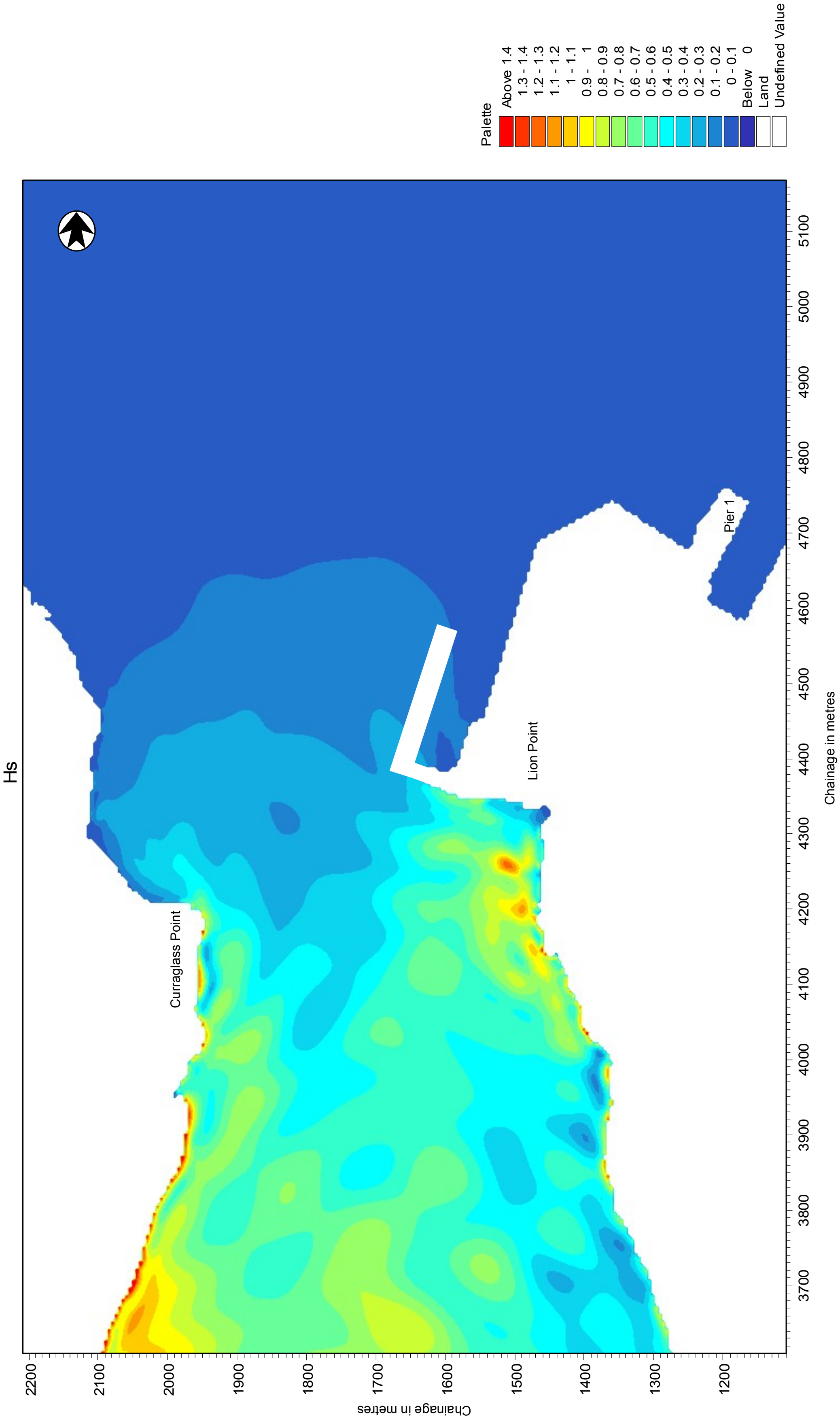
Existing Harbour
1 in 1 Year Wind Wave + MHW
Figure 6.4



Input conditions:
1 in 100 years swell wave, Hs = 2.3m, Tz = 15s
Water level = MHWS (+4.9m) + 0.3m sea rise = 5.2mCD

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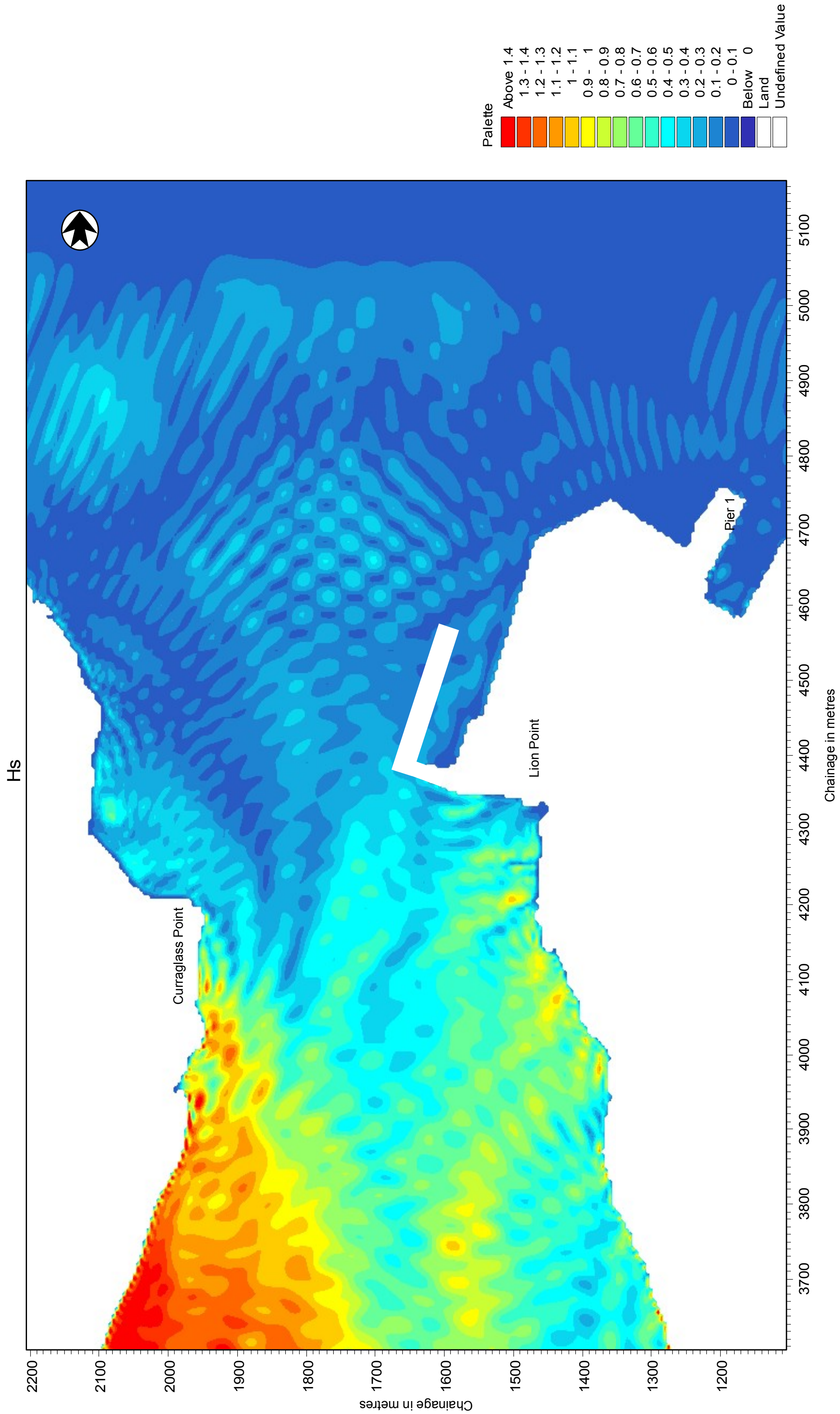
Addition of DWQ5, South Causeway
1 in 100 Years Swell Wave + MHWS
Figure 6.5



Input conditions:
1 in 100 years wind wave, Hs = 2.8m, Tz = 6s
Water level = MHWS (+4.9m) + 0.3m sea rise = 5.2mCD

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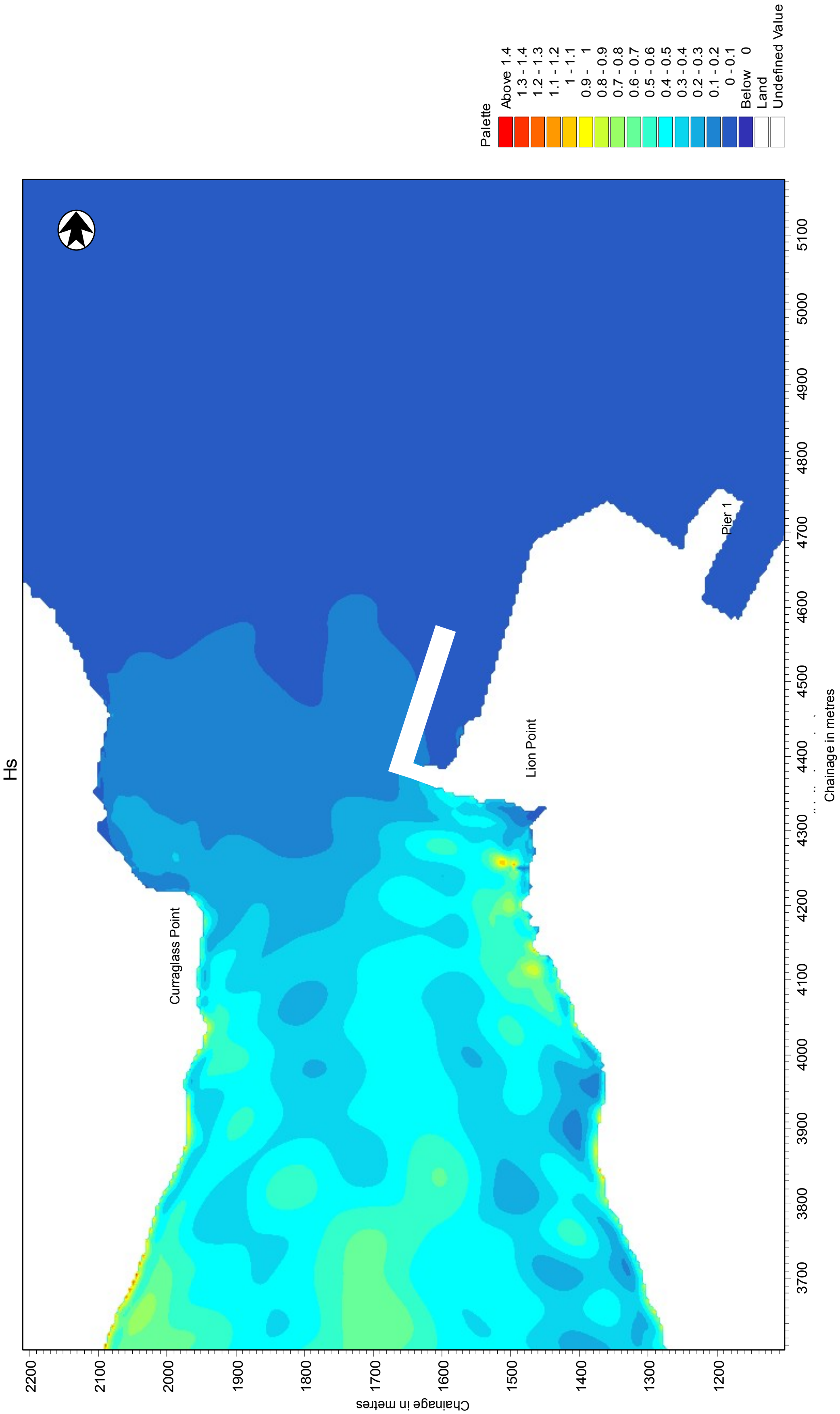
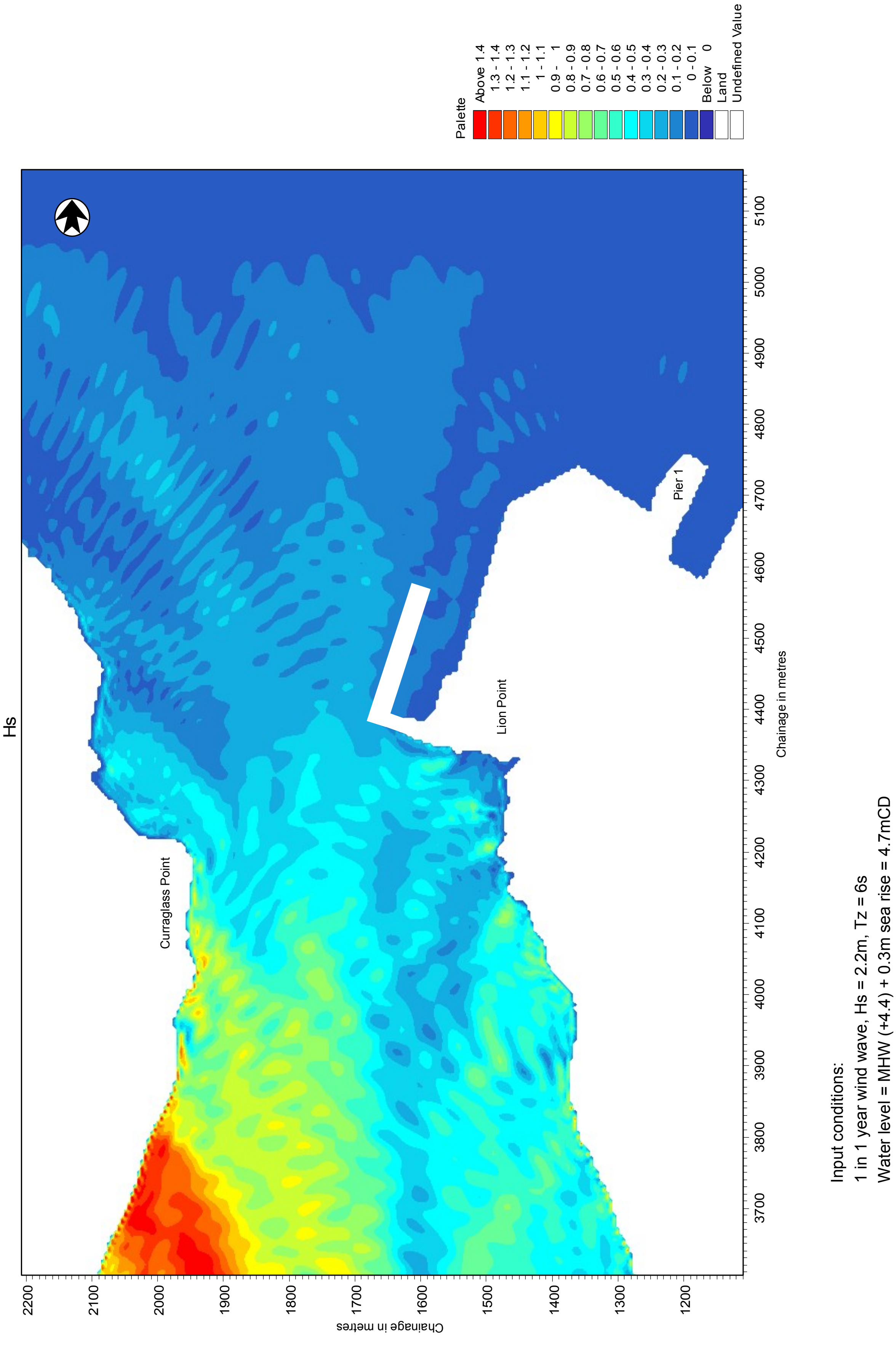
Addition of DWQ5, South Causeway
1 in 100 Years Wind Wave + MHWS
Figure 6.6



Input conditions:
1 in 1 year swell wave, Hs = 1.4m, Tz = 15s
Water level = MHW (+4.4) + 0.3m sea rise = 4.7mCD

Addition of DWQ5, South Causeway
1 in 1 Year Swell Wave + MHW
Figure 6.7



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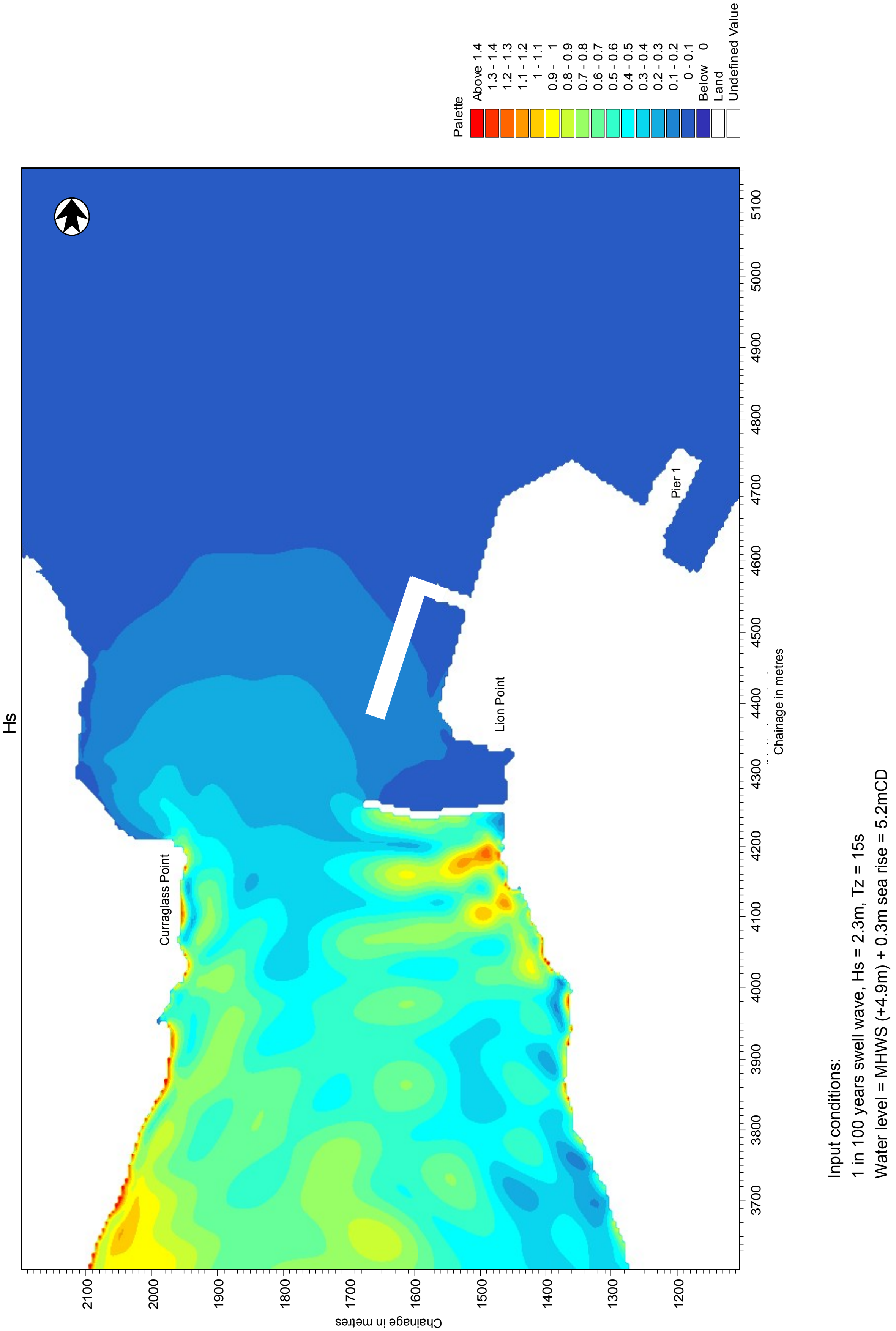
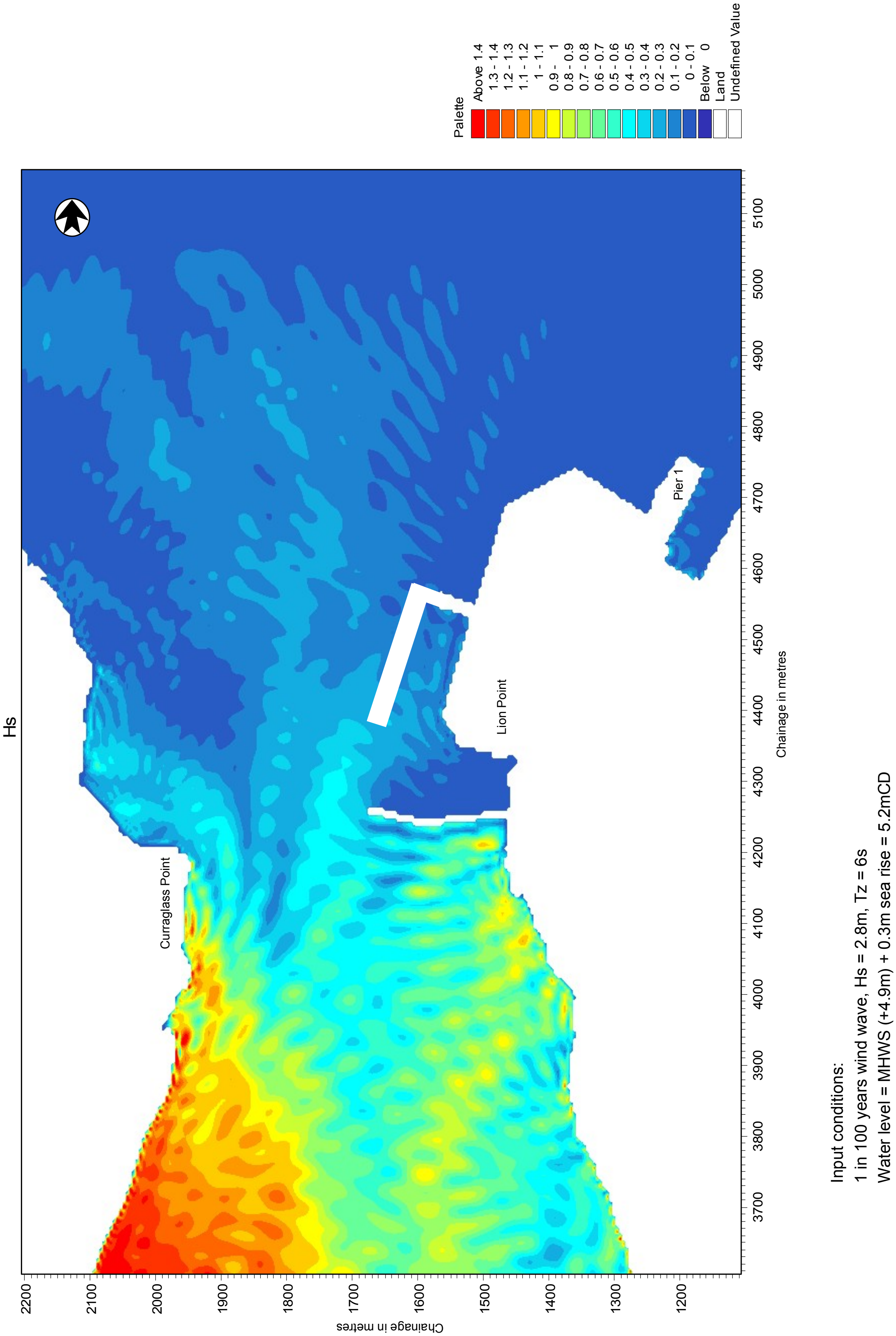
Addition of DWQ5, South Causeway
1 in 1 Year Wind Wave + MHW
Figure 6.8



Addition of DWQ5, North Causeway & Breakwater
1 in 100 Years Swell Wave + MHWS
Figure 6.9



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Addition of DWQ5, North Causeway & Breakwater
1 in 100 Years Wind Wave + MHWS
Figure 6.10

7 Local Wind Waves at Ferry Pontoon Berths and Boat Harbour

7.1 Introduction

This chapter estimates an appropriate locally generated wind wave height for use in the planning and design of the ferry pontoon berths and the associated pile restraint system.

The ferry terminal is well sheltered (Figure 7.1) and it is not expected that wave conditions are likely to be a significant design issue for this area. Accordingly, a pragmatic, hindcasting approach has been adopted to estimate a reasonable wave height.

7.2 Wind Wave Hindcasting

The ferry berths are well sheltered not only by Outer Cashla Bay, but by also the two quays lying to its south west and by Illaunawehichy Rock to its north. However, there is a narrow fetch extending approximately along the line of the leading lights at 296° (116°).

A wave height has been estimated using the method described in the *Shore Protection Manual*[7.1]. This involved constructing nine radials from the point of interest at 3° intervals, and extending each of these radials until they intersect the shoreline (Figure 7.1). The fetch length is then derived from the average of the lengths of these radials.

The 296° central fetch ray falls within the 293° to 337° wind data sector, supplied by the United Kingdom Meteorological Office. While an extreme value analysis for this sector is currently being prepared, over the 11 year recording period a single occurrence of a 26m/s wind was recorded. This single wind speed has therefore been used in the wind wave hindcasting until further data in made available.

Earlier in the report, BS6349[7.2] was used to predict wave heights due to wind. However, the *Shore Protection Manual*[7.1] takes greater account of water depths in a confined wave generation area, in this case up to 11m under maximum tidal conditions.

The *Shore Protection Manual*[7.1] was therefore used to calculate wave heights at the location of the proposed ferry terminal, giving the results presented in Table 7.1.

Table 7.1: Locally Generated Wave Heights at Ferry Berths

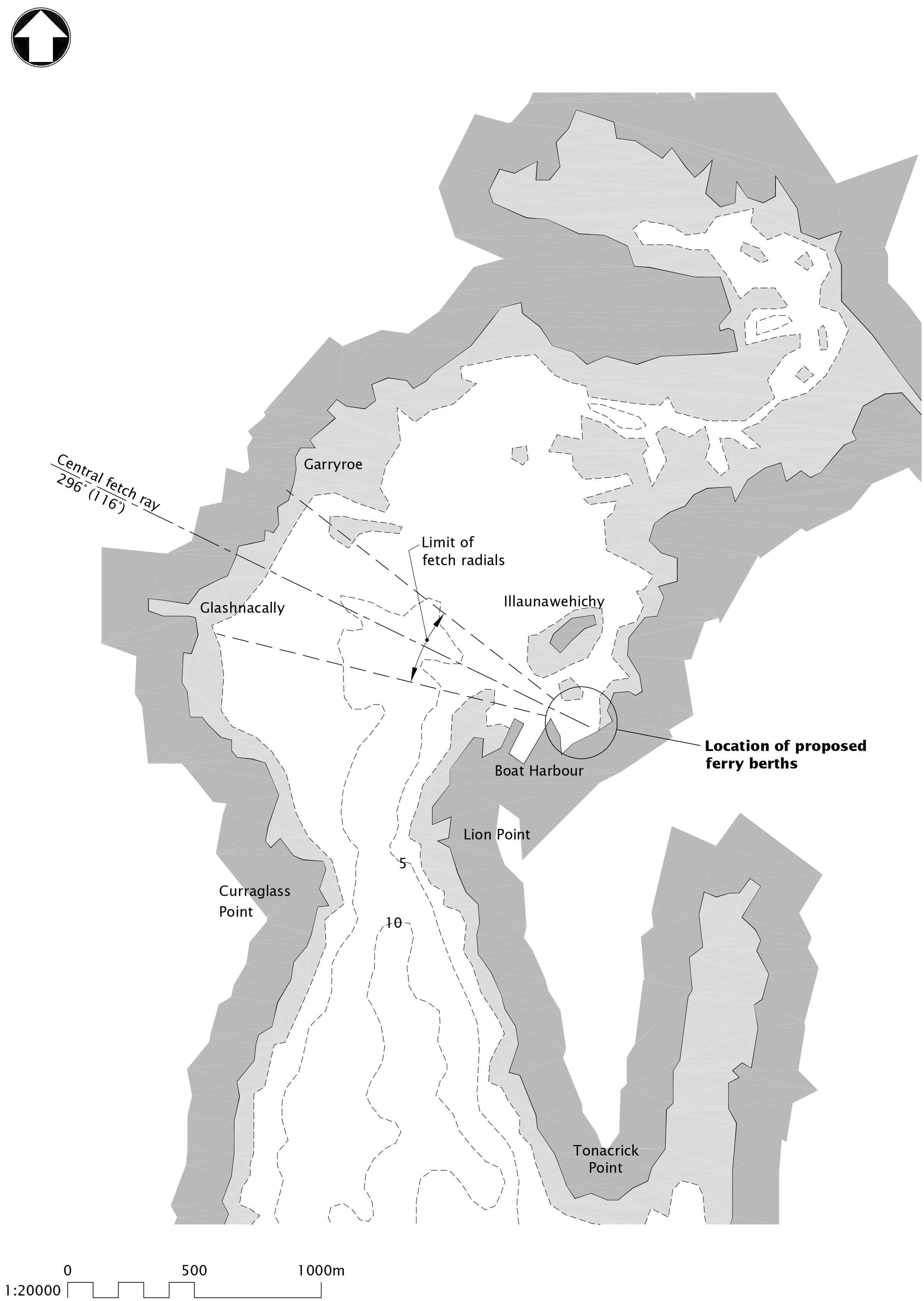
Approximate Return Period, in the absence of further data (Years)	Significant height (m)	Significant period (s)
50	0.5 to 0.75	2 to 3

The above value will be confirmed at the detailed design stage, but is expected to be a reasonable estimate for the planning and preliminary design of the pontoon ferry berths.

7.3 References

(7.1) Department of the Army, *"Shore Protection Manual"*, Waterways Experiment Station, Corps of Engineers, Coastal Engineering Research Centre, Volume 1, Chapter 2, US Government Printing Office, Fourth Edition, 1984.

(7.2) BS 6349 - 1:2000 British Standard *"Maritime Structures – Part 1: Code of practice for general criteria"*.



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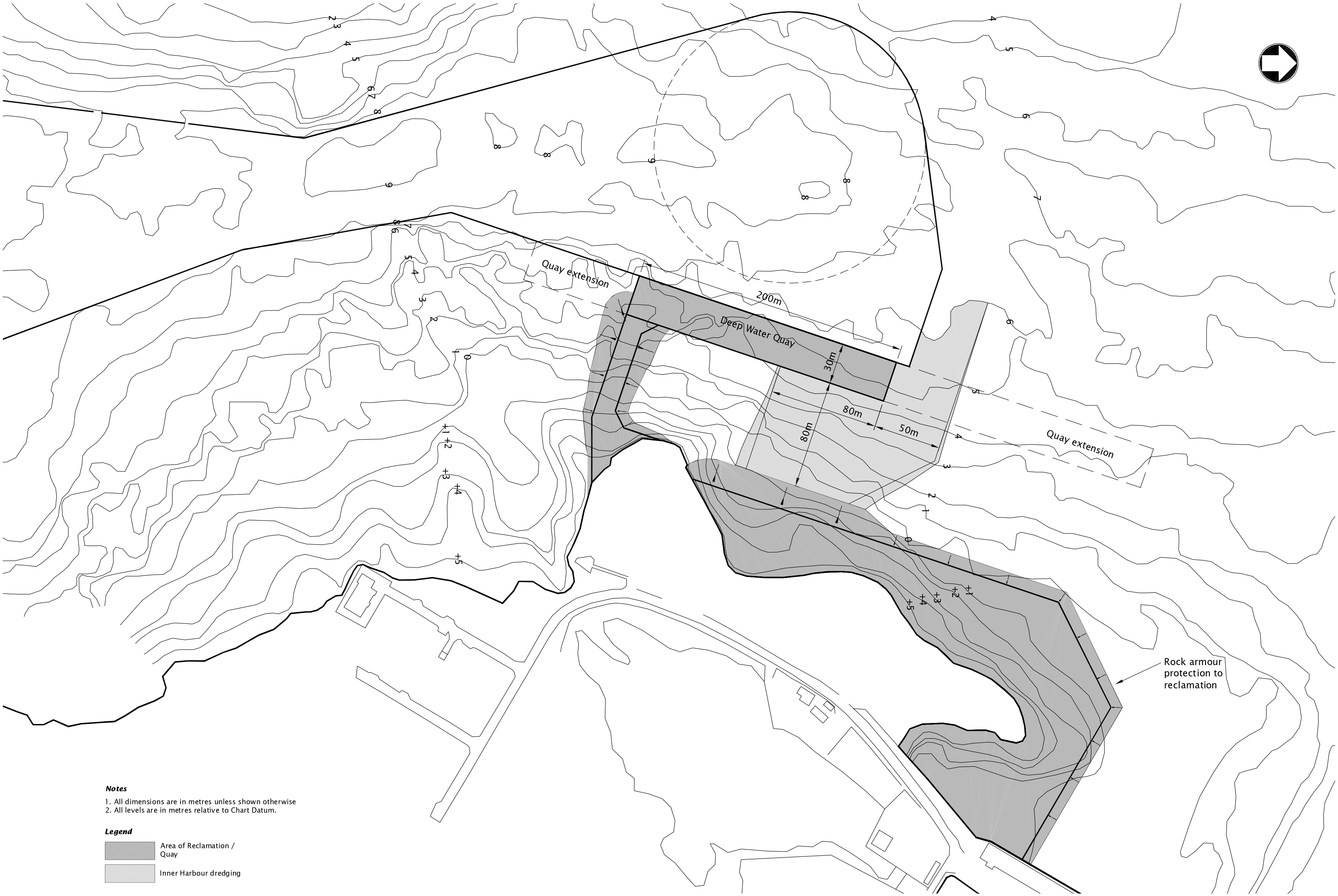
Wind Wave Fetch at Ferry Pontoon Berth
Figure 7.1

Appendix A Figures Included from Volume 1



Deep Water Quay - 'L south' Configuration

Figure 9.5

Notes

1. All dimensions are in metres unless shown otherwise
2. All levels are in metres relative to Chart Datum.

Legend

Area of Reclamation / Quay

Inner Harbour dredging

1:2500 @ A3 — 0, 50, 100, 150, 200, 250m

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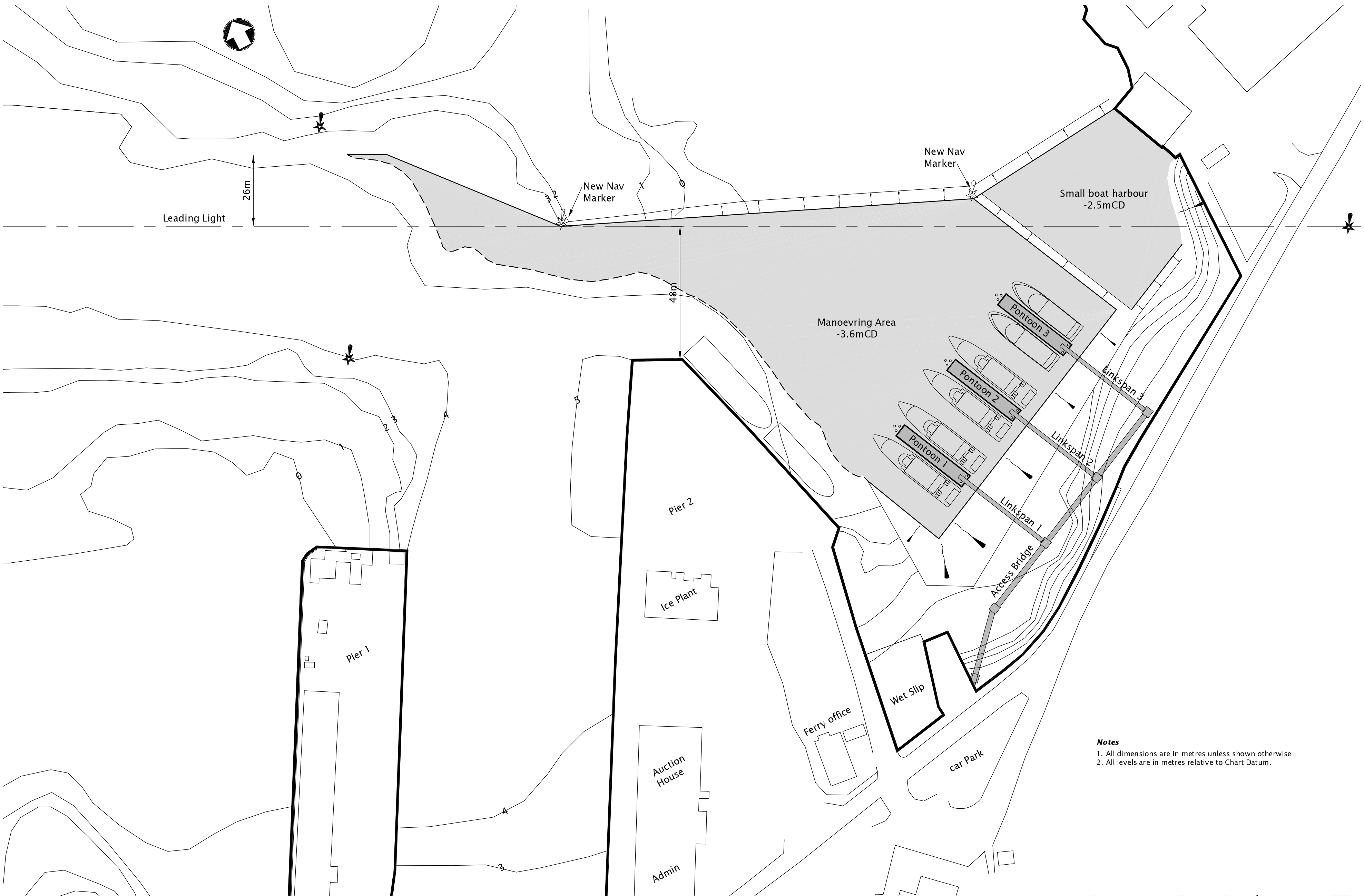

Notes
1. All dimensions are in metres unless shown otherwise
2. All levels are in metres relative to Chart Datum.

1:1250 @ A3 — 0 10 20 30 40 50 100m

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Passenger Ferry Berth Option FT4
Finger Piers, Full Development
Figure 11.5